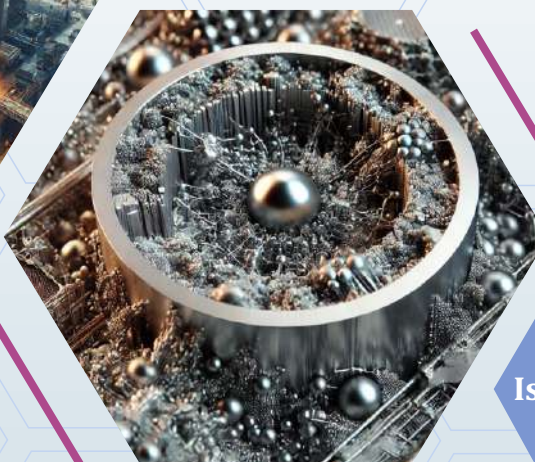




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UAV remote sensing-based road corridor monitoring with spatial cross validation and deep semantic segmentation

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Abstract

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UAV optical imagery provides the detail needed to map road surfaces, roadside vegetation, buildings and traffic participants in road corridors, but the accuracy of such maps is often estimated with random partitions of spatially and temporally autocorrelated frames. This study quantifies how much a random frame split inflates the accuracy of deep semantic segmentation compared with a sequence-based spatial cross validation that holds out complete UAV flight sequences. U-Net, DeepLabV3+ and SegFormer were trained under identical settings on all 42 labeled sequences (420 frames) of the UAVid benchmark of urban street scenes, together with a pixel-based Random Forest reference. Every frame was predicted once under each five-fold protocol, and the protocols were compared with paired sequence-level tests. The random split gave mIoU values of 72.9–74.0%, whereas sequence-based cross validation gave 68.6–69.9%. The difference of 4.1–4.3 percentage points was present in 41 or 42 of the 42 sequences for every model (Holm-adjusted Wilcoxon $p < 10^{-10}$), overall accuracy was inflated by 2.1–2.6 points, and the fold-to-fold variability of mIoU was two to three times larger under sequence-based validation. The inflation was largest for static cars (10.4–11.4 IoU points), moderate for road and low vegetation, small for buildings and trees and absent for pedestrians, while the ranking of the models did not change. Sequence-based validation therefore gives a more realistic estimate of the accuracy expected on new flights and is recommended for reporting UAV-based road corridor mapping results.

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1. Introduction

Road corridors are linear infrastructure systems whose condition depends on the pavement surface and on the land cover that surrounds it. Vegetation encroaching on the right-of-way (RoW), changes in adjacent built-up areas, parked and moving vehicles and the presence of pedestrians all influence operational safety and maintenance planning, and road agencies therefore need repeatable information on what occupies the corridor and how it changes over time [1]. Ground surveys provide this information at high thematic detail but are labor-intensive, expose field crews to traffic and cover only short sections per campaign. Satellite remote sensing offers systematic coverage of large areas, yet its revisit time, cloud contamination and spatial resolution limit the mapping of narrow corridor elements such as road verges, individual trees and vehicles [2–4].

Unmanned aerial vehicles (UAVs) fill this gap by acquiring optical imagery with centimeter-level ground sampling distance on demand and at low operational cost [5]. UAV image blocks are routinely processed with photogrammetric techniques into orthomosaics and digital surface

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models [6] and fused with terrestrial laser scanning point clouds for detailed 3D documentation [7], while the individual images and video frames are themselves a rich source for semantic interpretation. UAV studies have used such imagery to map vegetation communities, invasive species, land cover and hydrological variables with machine learning and deep learning [8–12], and to detect pavement damage along roads [13]. For road corridor monitoring, the classes of primary interest are the road surface, the adjacent buildings, roadside trees and low vegetation, and traffic participants such as vehicles and pedestrians. These classes are annotated in the UAVid benchmark, which provides densely labeled 4K frames of oblique UAV video over urban street scenes [14].

Deep semantic segmentation has become the standard approach for assigning a thematic label to every pixel of very high-resolution imagery. Fully convolutional networks [15] and encoder-decoder architectures such as U-Net [16] established dense prediction with skip connections, DeepLabV3+ combined atrous convolution and spatial pyramid pooling with a light decoder [17], and SegFormer introduced a hierarchical transformer encoder with a simple multilayer perceptron decoder [18]. These model families are widely applied to remote sensing imagery [19] and to UAV scenes in particular [10, 20], and deep networks generally outperform pixel-based classical learners when spatial context matters [21].

The accuracy figures reported for such models are, however, only as informative as the protocol used to estimate them. Remote sensing data are spatially autocorrelated, so that observations close to each other tend to be more similar than distant ones [22]. When training and test samples are drawn at random from the same scenes, the test set contains samples that are near duplicates of training samples, and the estimated accuracy describes interpolation within the sampled area instead of prediction for new locations [23, 24]. Studies in ecology and remote sensing have shown that random cross validation can substantially overestimate map accuracy compared with spatially blocked validation [25, 26], and that the effect also applies to convolutional neural networks trained on UAV imagery [27]. Spatial and group-based resampling schemes, including block cross validation and leave-location-out or leave-one-flight-out designs, are therefore increasingly recommended [9, 12, 28–30], although the choice of validation strategy has to match the intended use of the map [31, 32].

UAV video data are particularly exposed to this problem, because frames extracted from the same flight share viewing geometry, illumination and scene content and often cover largely overlapping ground areas, so a random partition of frames places almost identical views of the same street into the training and the test set. Benchmark splits defined by dataset authors avoid this by assigning complete sequences to training, validation and test sets [14], but when UAV frames or patches are pooled and split at random, as is common in applied studies with a single collection of images, the size of the resulting optimism for corridor classes is rarely quantified. It is also unclear whether the optimism depends on the segmentation architecture or on the class, which matters when small and dynamic objects such as vehicles and pedestrians are part of the monitoring task.

This study addresses these questions with a controlled experiment on UAV optical imagery of urban road corridors. Three segmentation models that represent the main architectural families, U-Net, DeepLabV3+ and SegFormer, are trained under identical conditions on all 42 labeled sequences of UAVid, and a pixel-based Random Forest classifier with RGB spectral indices and local texture features serves as a simple classical reference. Accuracy is estimated with two five-fold cross validation protocols that differ only in how frames are assigned to folds: a random frame-level partition and a sequence-based partition in which all frames of a flight sequence are held out together. Because every frame is predicted exactly once under each protocol, the two estimates can be compared in a paired design at sequence level. The experiments quantify how much random splitting inflates overall and class-wise accuracy for corridor classes in urban street scenes, whether the inflation changes the ranking of the models and which classes remain difficult when a model is applied to unseen flights.

2. Materials and Methods

As summarized in Fig. 1, UAV optical frames and their reference labels were checked and down sampled, partitioned into folds with two cross validation protocols, used to train three deep

segmentation models and one classical reference under fixed settings, and assessed with pixel-based accuracy measures and sequence-level statistical tests.

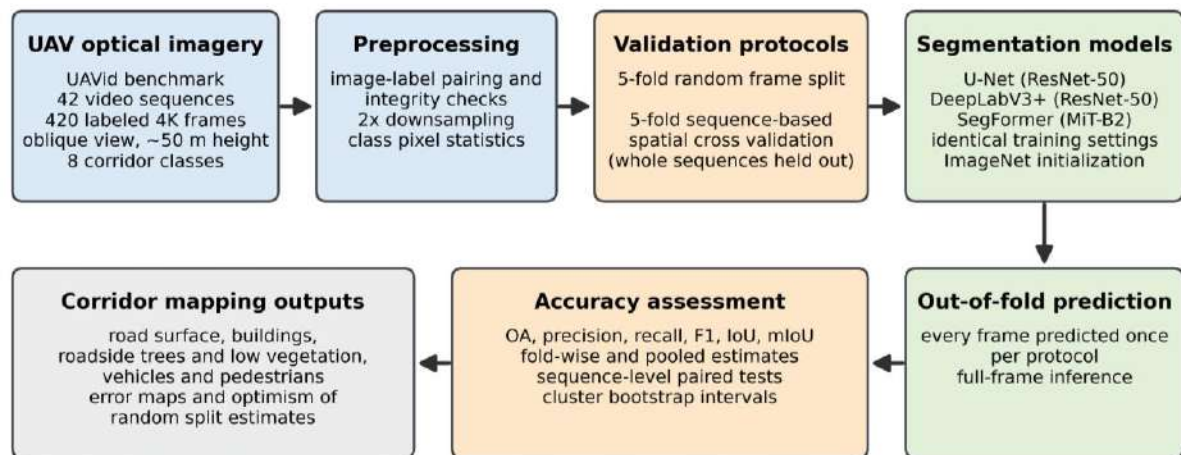


Fig. 1. Workflow of the study, from UAV optical imagery and validation design to accuracy assessment of corridor classes

2.1. Study Data and UAV Remote Sensing Context

The experiments use the UAVid semantic segmentation benchmark [14], which consists of oblique UAV video of urban and suburban street scenes (Table 1, Fig. 2). According to the dataset description, the imagery was recorded in 4K video mode at a flying height of approximately 50 m with the camera tilted by about 45° from the vertical, so that both the tops and the facades of objects are visible and the ground sampling distance increases from the near to the far part of each frame. Frame sizes are 3840 × 2160 or 4096 × 2160 pixels. Ten frames per sequence, taken at intervals of 100 video frames (about 5 s), were densely annotated with eight classes: building, road, tree, low vegetation, static car, moving car, human and background clutter. The original publication describes 30 sequences with 300 labeled images [14]; the dataset was later extended by 12 sequences, and the reference labels of the 15 test sequences were released by the dataset authors in 2023 through the UAVid website. All 42 sequences with 420 labeled frames were used in this study, and the original benchmark split was not used because both cross-validation protocols reassign all frames to folds.

Table 1. Summary of the data and validation designs used in this study

Item	Description
Dataset	UAVid, all 42 labeled sequences [14]
Platform and view	UAV video, oblique view (about 45°), flying height about 50 m
Labeled frames	420 (10 per sequence, about 5 s apart)
Original frame size (px)	3840 × 2160 (260 frames), 4096 × 2160 (160 frames)
Processed frame size (px)	1920 × 1080 or 2048 × 1080 (down sampling by 2)
Bands	RGB
Classes	building, road, tree, low vegetation, static car, moving car, human, clutter
Sources	RGB frames and train/val labels: DANS archive; test labels: third-party redistribution
Random split	5 folds of 84 frames assigned at random
Sequence-based CV	5 folds of 8 or 9 complete sequences (80 or 90 frames)

The data were obtained from two sources: the RGB frames of all 42 sequences and the color-coded reference labels of the 27 training and validation sequences were taken from the official UAVid archive in the DANS Data Station (doi:10.17026/dans-x9f-w9sa), and all 420 image files were verified against the SHA-1 checksums of this archive. The reference labels of the 15 test sequences are not contained in that archive; they were taken from a third-party redistribution on the Hugging

Face platform (dataset dronefreak/UAVid-2020), which provides the labels of all 42 sequences as class index images and states that the official labels were only reformatted. The index masks of this redistribution were used for all frames. For a random sample of 30 of the 270 training and validation frames they agreed in every pixel with the official color labels of the DANS archive. The test labels could not be compared with the official 2023 release, which is distributed through a separate file share that was not accessed in this study, and two anomalies found in them are described in Section 2.2. The dataset is distributed under a CC BY-NC-SA 4.0 license.

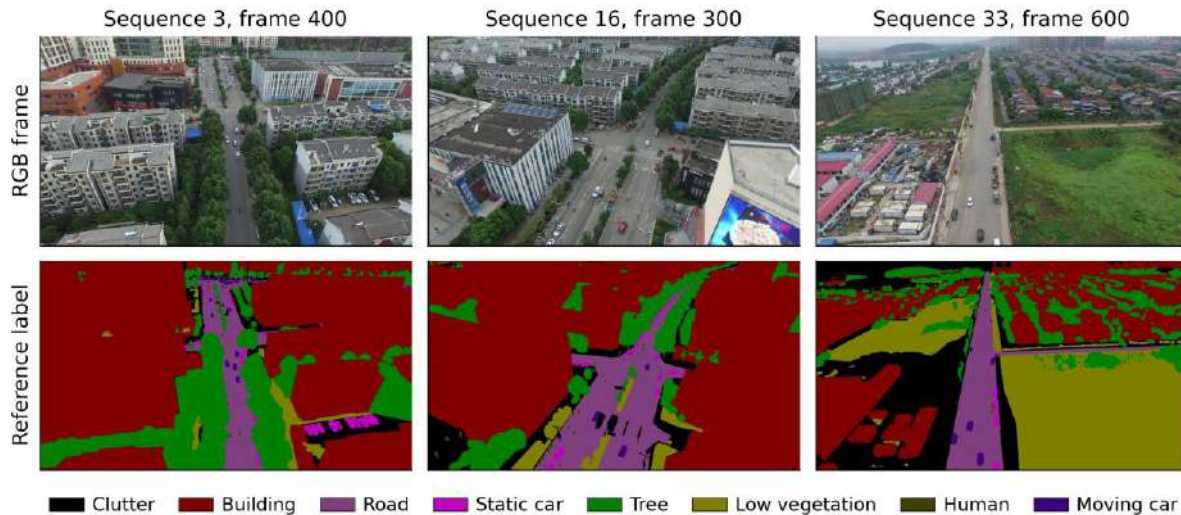


Fig. 2. Example UAVid frames from three sequences and the corresponding reference labels in the official color code

The UAVid classes cover the thematic content that is relevant for road corridor monitoring in urban areas. Road represents the carriageway and paved surfaces used by traffic, tree and low vegetation describe the roadside green that is subject to vegetation management within the RoW, building represents the adjacent built environment, static and moving cars represent parking and traffic, and human represents pedestrian presence. Background clutter collects all remaining surfaces and objects. The eight original classes were retained so that results remain comparable with the benchmark. The study is based entirely on RGB imagery; photogrammetric products such as orthomosaics or digital surface models were not derived, since the benchmark provides individual oblique frames without camera orientation data.

2.2. Data Preparation and Class Harmonization

Each image was paired with its label mask by sequence and frame index, and the pairs were checked for missing files, equal dimensions and label values outside the class range. One test frame (sequence 30, frame 600) contains 10,172 pixels (0.11% of the frame) without a valid class value; these pixels were excluded from training and evaluation. A plausibility check of the human class showed that the largest connected human region per frame is below 8,700 pixels in 99% of the frames, whereas two test frames (sequence 27, frame 300 and sequence 39, frame 300) contain human regions of 97,890 to 137,428 pixels covering lawn and paved areas that are labeled clutter, tree, low vegetation or road in the neighboring frames of the same sequences (Fig. A1). These regions (360,351 pixels in total) were treated as unlabeled in the evaluation. Because they were detected after training, they remained in the training data of the folds that contained the two frames, where they make up about 0.01% of the labeled pixels. The corrected evaluation is treated as the primary result, and the uncorrected evaluation is reported as a sensitivity analysis in Table A1; the masks of the excluded regions are available from the corresponding author upon reasonable request.

To make full-frame training and inference feasible on a 6 GB GPU, all frames were down sampled by a factor of two, using area interpolation for the RGB images and nearest neighbor interpolation for the masks. During training, square windows were sampled at random positions from the training frames at a random scale between 0.75 and 1.5 and resized to 512 × 512 pixels. Random

horizontal flipping and a random brightness and contrast change (gain 0.8 to 1.2, offset ± 20 gray values) were applied, and images were normalized with the ImageNet channel statistics. Vertical flips and rotations were not used, because they would produce viewing geometries that do not occur in oblique UAV imagery. Windows are drawn on the fly from the frames of the training folds only, so no patch is shared between training and test data.

2.3. Spatial Cross Validation Design

Two five-fold cross validation protocols were applied to the same 420 frames (Fig. 3). In the random protocol, frames were assigned to folds by a random permutation, independently of their sequence, which reproduces the common practice of splitting pooled UAV imagery at random. Because consecutive labeled frames of a sequence are only about 5 s apart and view largely overlapping parts of the same street, 386 of the 420 test frames (91.9%) in this protocol had at least one temporally adjacent frame of the same sequence in the training set, and on average 7.2 of the 9 other frames of their sequence were used for training.

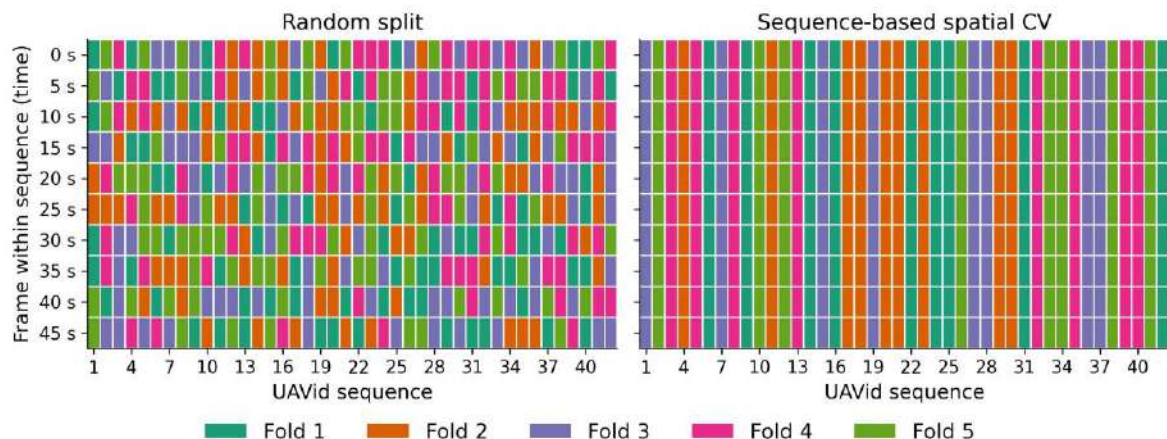


Fig. 3. Fold assignment of the 420 labeled frames under the random split (left) and the sequence-based spatial cross validation (right); columns are UAVid sequences and rows are the ten labeled frames of each sequence in temporal order

In the sequence-based protocol, the 42 sequences were randomly assigned to five groups of eight or nine sequences and all frames of a sequence were placed in the same fold, so that training and test data are sequence-disjoint. Since each sequence is a separate flight over a distinct street scene, this design approximates a leave-one-flight-out setting [23, 24, 28]. It does not guarantee geographic separation between folds, because location identifiers and flight campaign metadata are not provided with the dataset. The fold assignment of all frames is available from the corresponding author upon reasonable request.

In both protocols each fold was used once as the test set while the model was trained on the other four folds. No validation subset was used for early stopping or model selection within the cross validation: all models were trained for a fixed number of iterations and the final weights were evaluated. The only design decision informed by data of the study was the choice of the loss function, which followed short pilot runs on one sequence-based fold (Section 2.4). Every frame received exactly one out-of-fold prediction per model and protocol, which allows a paired comparison of the protocols.

2.4. Deep Semantic Segmentation Models

Three architectures that represent the main families of semantic segmentation networks were compared (Table 1). U-Net [16] is an encoder decoder network whose decoder recovers spatial detail by concatenating encoder features through skip connections; here it was built on a ResNet-50 encoder [33] with decoder stages of 256, 128, 64, 32 and 16 channels. DeepLabV3+ [17] applies atrous spatial pyramid pooling to encoder features at an output stride of 16 and fuses the result with low-level features in a light decoder; it was also built on ResNet-50. SegFormer [18] uses a hierarchical Mix Transformer (MiT) encoder with efficient self-attention and an all-MLP decoder

that aggregates features from four scales; the MiT-B2 encoder was used with a 256-channel decoder, because the 768-channel decoder of the original B2 configuration did not fit into GPU memory with the batch size used for the other models. The encoders were initialized with ImageNet pretrained weights [34] and the decoders randomly.

All models were trained with the same settings, and the loss was the sum of a class-weighted cross-entropy and a soft multiclass Dice loss [35]. Class weights were computed from the pixel frequencies f_c of the training folds as $w_c = (\text{median}(f)/f_c)^{1/2}$ and normalized to a mean of one, a damped form of median frequency balancing [36]. This loss was adopted as a limited preliminary choice to handle the class imbalance, after a pilot run on one sequence-based fold in which unweighted cross-entropy never predicted the human class. It was not the result of a systematic hyperparameter optimization: in short pilot runs of U-Net on the same fold, cross-entropy with Dice loss still failed to predict humans, whereas its class-weighted variant did, and this setting was retained without further tuning. The pilot runs are not part of the reported results, and the selected loss was applied unchanged to all models and both protocols. Because the pilot runs used data of the same study, the test folds cannot be regarded as entirely independent of this decision. Optimization used AdamW [37] with a learning rate of 1×10^{-4} and a weight decay of 0.01, a linear warm-up over 500 iterations followed by linear decay to zero, a batch size of eight crops and 6000 iterations. Mixed precision (16-bit floating point) arithmetic was used for training and inference. At test time each full frame was padded by reflection to a multiple of 32 pixels and predicted in one pass together with its horizontally flipped copy, whose logits were averaged. The random seed was fixed per fold, and the same seeds were used for both protocols.

Table 2. Segmentation models and computational cost; common settings are 512×512 crops, batch size 8, AdamW, learning rate 1×10^{-4} , weight decay 0.01, 6000 iterations and class-weighted cross-entropy plus Dice loss; training time is the median over the ten runs of each model and inference time the median over 24 full frames including flip averaging

Model	Encoder	Decoder	Parameters (M)	Training time per fold (min)	Inference (s per frame)
U-Net	ResNet-50	U-Net decoder (256–16 channels)	32.5	60	0.68
DeepLabV3+	ResNet-50, output stride 16	ASPP and decoder (256 channels)	26.7	48	0.55
SegFormer	MiT-B2	All-MLP decoder (256 channels)	24.7	74	2.08

As a simple classical reference, a pixel-based Random Forest classifier [38] with 100 trees and a minimum leaf size of five was trained on a limited set of features computed at a quarter of the original resolution: the RGB and HSV channels, the excess green index, the visible atmospherically resistant index and the normalized green red difference index, the local mean and standard deviation of brightness in 5×5 and 15×15 windows, the Sobel gradient magnitude and the relative row position in the frame. For each training frame up to 250 pixels per class were sampled, giving about 600,000 training pixels per fold. The reference was evaluated with the same folds and metrics as the deep models.

The experiments were implemented in Python with PyTorch 2.12, the Segmentation Models PyTorch library 0.5.0 and scikit-learn 1.8, and run on a laptop workstation with an NVIDIA GeForce RTX 2060 GPU (6 GB), an Intel Core i7-10750H CPU (2.60 GHz) and 32 GB RAM.

2.5. Evaluation Metrics

Accuracy was assessed from confusion matrices accumulated over all labeled pixels of the evaluated frames at the processed resolution [32, 39]. For class c , TP_c , FP_c and FN_c denote the numbers of true positive, false positive and false negative pixels, N is the total number of evaluated

pixels and $C = 8$ is the number of classes. Overall accuracy (OA), precision (P_c), recall (R_c), F1-score and intersection over union (IoU) are defined in Eqs. (1)–(5).

$$OA = \frac{\sum_{c=1}^C TP_c}{N} \quad (1)$$

$$P_c = \frac{TP_c}{TP_c + FP_c}, \quad R_c = \frac{TP_c}{TP_c + FN_c} \quad (2)$$

$$F1_c = \frac{2 P_c R_c}{P_c + R_c} \quad (3)$$

$$IoU_c = \frac{TP_c}{TP_c + FP_c + FN_c} \quad (4)$$

$$mIoU = \frac{1}{C} \sum_{c=1}^C IoU_c, \quad mF1 = \frac{1}{C} \sum_{c=1}^C F1_c \quad (5)$$

IoU is the Jaccard index used in the PASCAL VOC benchmark [40], and mIoU is the primary metric of the UAVid benchmark [14]. OA is reported for completeness, although it is dominated by the frequent classes. Two aggregation levels were used. Pooled estimates were computed from the sum of the confusion matrices of all 420 out-of-fold predictions, and fold-wise estimates were computed per test fold and summarized by their mean and standard deviation. In both cases every class occurs in the reference and in the predictions, so none of the denominators in Eqs. (2)–(4) is zero.

To test whether the protocols differ, the mIoU of each sequence was computed from the confusion matrix of its ten frames. Classes that occur neither in the reference nor in the prediction of a sequence have an undefined IoU and were omitted from the mean of that sequence. The paired sequence-level differences between the random and the sequence-based protocol were tested with the two-sided Wilcoxon signed-rank test [41], and 95% confidence intervals for the difference in pooled mIoU and OA were obtained with a cluster bootstrap that resamples the 42 sequences with replacement (10,000 replicates). The same sequence-level test was used to compare models within each protocol. The p -values were adjusted with the Holm procedure [42] within each family of comparisons.

3. Results

3.1. Dataset Characteristics and Class Distribution

The 420 frames contain 3.57×10^9 labeled pixels at the original resolution. Building (27.5%), tree (25.5%), clutter (18.9%), low vegetation (13.4%) and road (12.5%) account for 97.8% of the pixels, whereas static car (1.20%), moving car (0.89%) and human (0.12%) together account for 2.2% (Table 2, Fig. 4). The frequent classes are present in almost all frames and in all 42 sequences. Humans occur in 370 frames of 41 sequences, and moving cars in 334 frames of 39 sequences, so the rare classes are widespread but occupy small areas in each frame.

Table 3. Class distribution of the 420 labeled UAVid frames at the original resolution, before the correction of the two implausible human regions described in Section 2.2

Class	Pixels ($\times 10^6$)	Share (%)	Frames present	Sequences present
Building	982.8	27.51	419	42
Tree	911.2	25.51	420	42
Clutter	674.6	18.88	420	42
Low vegetation	479.4	13.42	420	42
Road	445.3	12.47	417	42
Static car	42.8	1.20	409	42
Moving car	31.9	0.89	334	39
Human	4.2	0.12	370	41

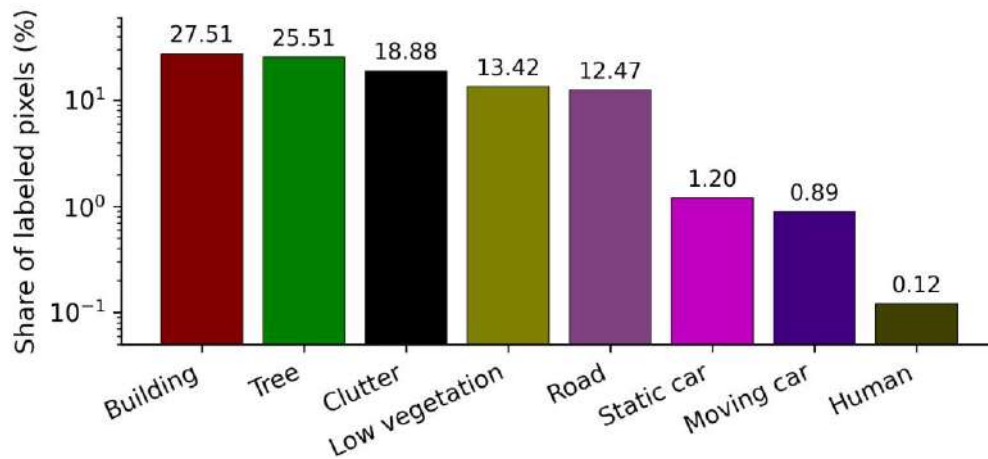


Fig. 4. Share of labeled pixels per class (logarithmic scale)

3.2. Model Performance under Random Split

Under the random frame split the three deep models reached mIoU values between 72.9% and 74.0% and OA values between 89.2% and 90.0% (Table 3, Fig. 5). U-Net obtained the highest mIoU (74.0%), followed by DeepLabV3+ (73.1%) and SegFormer (72.9%). The five folds gave very similar results, with standard deviations of the fold-wise mIoU between 0.7 and 0.9 points. The Random Forest reference reached 28.5% mIoU and 54.8% OA.

Table 4. Accuracy (%) under the five-fold random frame split; pooled values are computed from all 420 out-of-fold predictions

Model	OA	mIoU	mF1	Mean precision	Mean recall	Fold mIoU (mean $\pm$ SD)
U-Net	90.0	74.0	84.0	82.2	86.4	74.0 $\pm$ 0.9
DeepLabV3+	89.5	73.1	83.4	82.5	84.5	73.1 $\pm$ 0.7
SegFormer	89.2	72.9	83.4	81.9	85.4	72.9 $\pm$ 0.7
Random Forest	54.8	28.5	41.0	41.9	53.8	28.5 $\pm$ 0.5

The correction of the two implausible human regions (Section 2.2) raised the pooled mIoU of the deep models by 0.2 to 0.3 points and their human IoU by 1.8 to 2.4 points under both protocols, while the differences between the protocols changed by at most 0.1 point and all statistical conclusions remained unchanged (Table A1). All values reported below refer to the corrected evaluation.

3.3. Model Performance under Spatial Cross Validation

When complete sequences were held out, the mIoU of the deep models decreased to 68.6–69.9% and OA to 86.9–87.5% (Table 4). U-Net again obtained the highest mIoU (69.9%), followed by DeepLabV3+ (68.8%) and SegFormer (68.6%). The variation between folds increased markedly: the standard deviation of the fold-wise mIoU was 1.7 to 2.1 points, and fold-wise mIoU values ranged from 65.4% to 72.2%. The Random Forest reference reached 26.5% mIoU.

Table 5. Accuracy (%) under the five-fold sequence-based spatial cross validation

Model	OA	mIoU	mF1	Mean precision	Mean recall	Fold mIoU (mean $\pm$ SD)
U-Net	87.5	69.9	81.4	80.5	82.5	69.4 $\pm$ 2.1
DeepLabV3+	86.9	68.8	80.6	81.0	80.3	68.3 $\pm$ 2.1
SegFormer	87.1	68.6	80.5	79.7	81.5	68.2 $\pm$ 1.7
Random Forest	52.0	26.5	38.8	39.8	50.8	26.4 $\pm$ 1.0

The paired comparison of the two protocols is summarized in Table 5 and Fig. 5. For the deep models, the random split overestimated the pooled mIoU by 4.1 to 4.3 points, which corresponds to 5.5–5.9% of the random split estimate, and OA by 2.1 to 2.6 points. The 95% cluster bootstrap intervals of the mIoU difference did not include zero for any model. At sequence level, the random

split gave a higher mIoU for 41 of the 42 sequences with U-Net and for all 42 sequences with DeepLabV3+ and SegFormer; the median sequence-level difference was 3.3 to 4.0 points, and individual sequences showed differences of up to 16 points (Fig. 5c). The Random Forest reference showed a smaller absolute inflation of 2.0 mIoU points, but a similar relative inflation (7.2%).

Table 6. Difference between the random split (R) and the sequence-based spatial cross validation (S) in percentage points; confidence intervals from a cluster bootstrap over sequences and Holm-adjusted p-values from two-sided Wilcoxon signed-rank tests on sequence-level mIoU

Model	mIoU R	mIoU S	Δ mIoU (95% CI)	Relative Δ (%)	Δ OA (95% CI)	Sequences with lower mIoU	Adjusted p
U-Net	74.0	69.9	4.1 (3.0–5.3)	5.5	2.4 (2.0–2.9)	41/42	5.0×10^{-11}
DeepLabV3+	73.1	68.8	4.3 (3.4–5.3)	5.9	2.6 (2.1–3.1)	42/42	1.8×10^{-12}
SegFormer	72.9	68.6	4.3 (3.1–5.8)	5.9	2.1 (1.7–2.6)	42/42	1.8×10^{-12}
Random Forest	28.5	26.5	2.0 (1.5–2.6)	7.2	2.8 (2.2–3.6)	41/42	1.8×10^{-12}

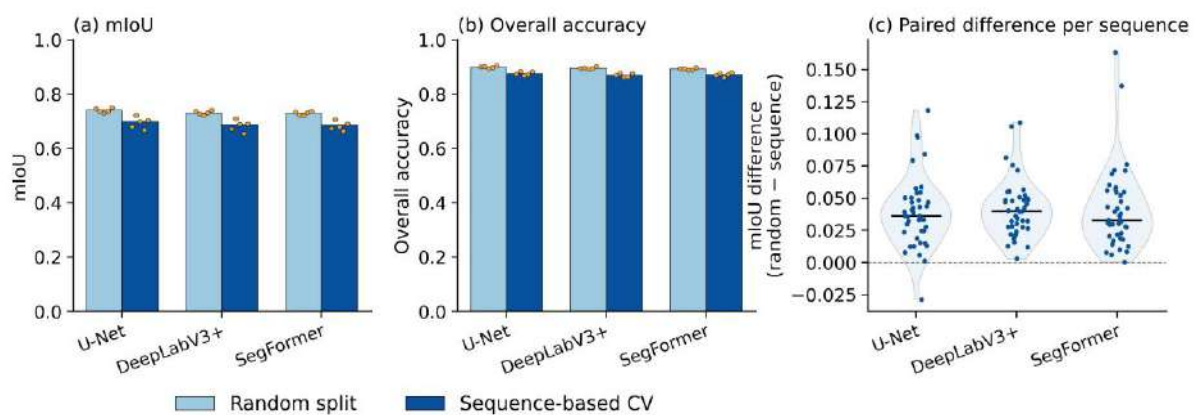


Fig. 5. (a) Pooled mIoU and (b) overall accuracy of the three models under both protocols, with dots showing the five fold-wise values; (c) paired difference in mIoU between the random split and the sequence-based cross validation for each of the 42 sequences, with horizontal lines marking the median

The ranking of the models was the same under both protocols, with U-Net achieving a higher sequence-level mIoU than DeepLabV3+ (39 of 42 sequences under both protocols) and SegFormer (37 and 34 of 42 sequences under the random and sequence-based protocols); the median differences were 1.0 to 1.7 points (Holm-adjusted $p < 0.001$). DeepLabV3+ and SegFormer did not differ significantly (adjusted $p = 0.20$ under the random split and $p = 0.80$ under the sequence-based validation). All deep models exceeded the Random Forest reference in every sequence, by at least 24 and in the median by 37 to 41 mIoU points.

3.4. Class Wise Mapping Accuracy and Qualitative Assessment

The class-wise IoU values in Table 6 and Fig. 6 show that, under sequence-based cross validation, buildings were mapped most accurately (IoU 89.3–90.7%), followed by road (80.1–80.8%) and tree (78.7–79.5%). Low vegetation (65.4–66.4%) and clutter (66.0–67.3%) reached intermediate values. Among the traffic classes, moving cars (68.1–72.4%) were mapped better than static cars (58.9–62.7%), and humans had the lowest IoU (38.8–39.8%).

The inflation caused by the random split differed strongly between classes. It was largest for static cars, with IoU differences of 10.4 to 11.4 points, and was driven by recall: the recall of static cars decreased from 86.3–88.6% under the random split to 72.2–75.4% under sequence-based validation, while precision changed by less than 3 points. Clutter (5.1–6.8 points), low vegetation (4.2–5.3 points), road (4.0–4.8 points) and moving cars (3.3–5.2 points) showed intermediate differences, and buildings (1.5–2.2 points) and trees (2.4–2.8 points) small differences. For

humans, the IoU under sequence-based validation was equal to or slightly higher than under the random split (−2.5 to +0.4 points).

Table 7. Class-wise IoU (%) under the random split (R) and the sequence-based spatial cross validation (S); mean Δ is the average difference R – S over the three deep models; class-wise F1-scores, precision and recall are available from the corresponding author upon reasonable request

Class	U-Net R	U-Net S	DeepLabV3+ R	DeepLabV3+ S	SegFormer R	SegFormer S	Mean Δ
Clutter	73.6	67.3	72.8	66.0	71.8	66.7	6.1
Building	92.3	90.7	91.5	89.3	91.2	89.8	1.7
Road	85.6	80.8	85.1	80.4	84.2	80.1	4.5
Static car	73.1	62.7	71.2	60.7	70.3	58.9	10.8
Tree	82.1	79.5	81.5	78.7	81.5	79.1	2.6
Low vegetation	71.4	66.1	70.6	65.4	70.6	66.4	4.9
Human	37.2	39.8	37.6	38.8	40.2	39.8	−1.1
Moving car	76.4	72.4	74.4	71.1	73.3	68.1	4.2

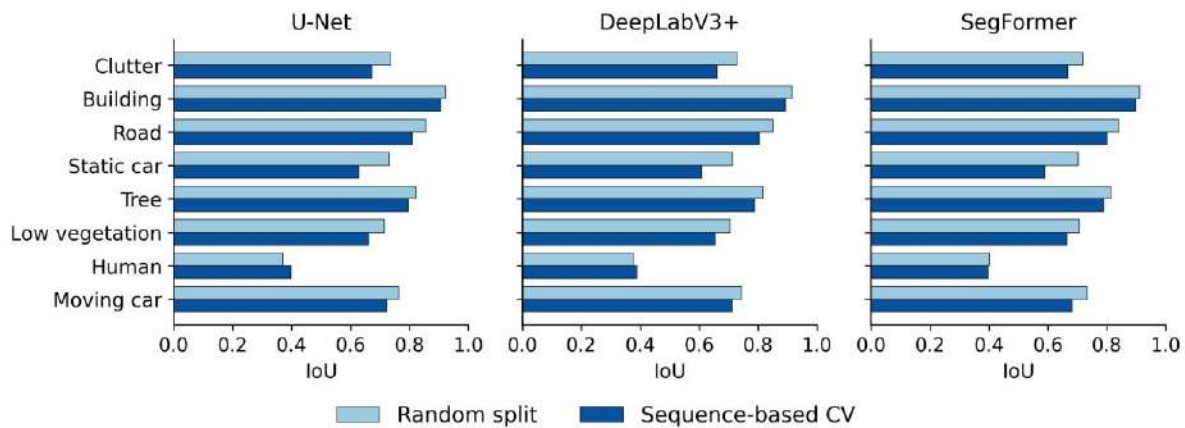


Fig. 6. Class-wise IoU of the three models under the random split and the sequence-based spatial cross validation

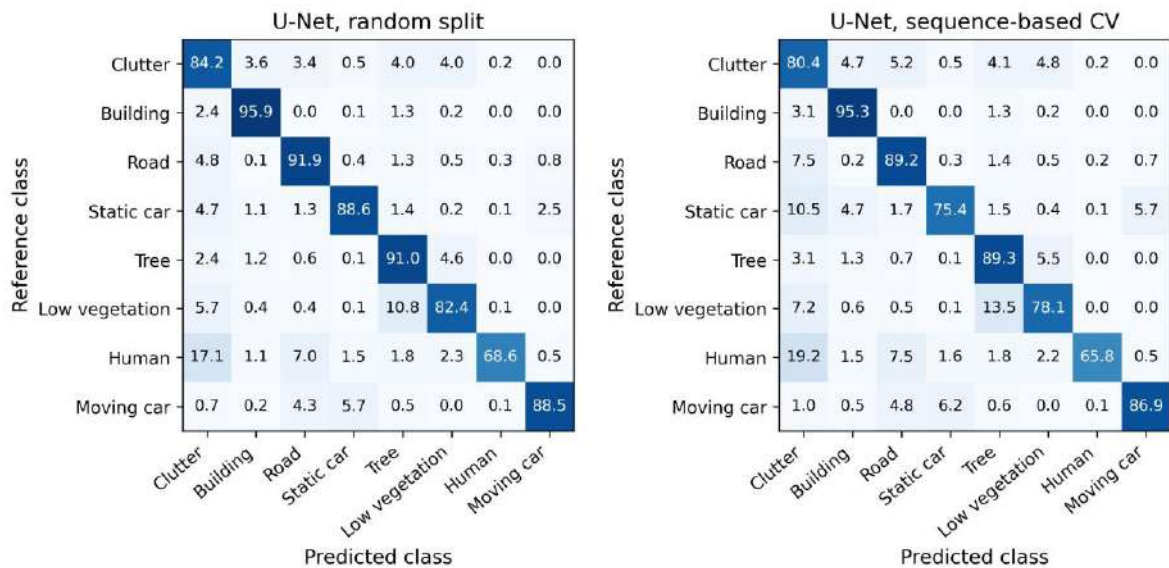


Fig. 7. Row-normalized confusion matrices (%) of U-Net under the random split and the sequence-based spatial cross validation

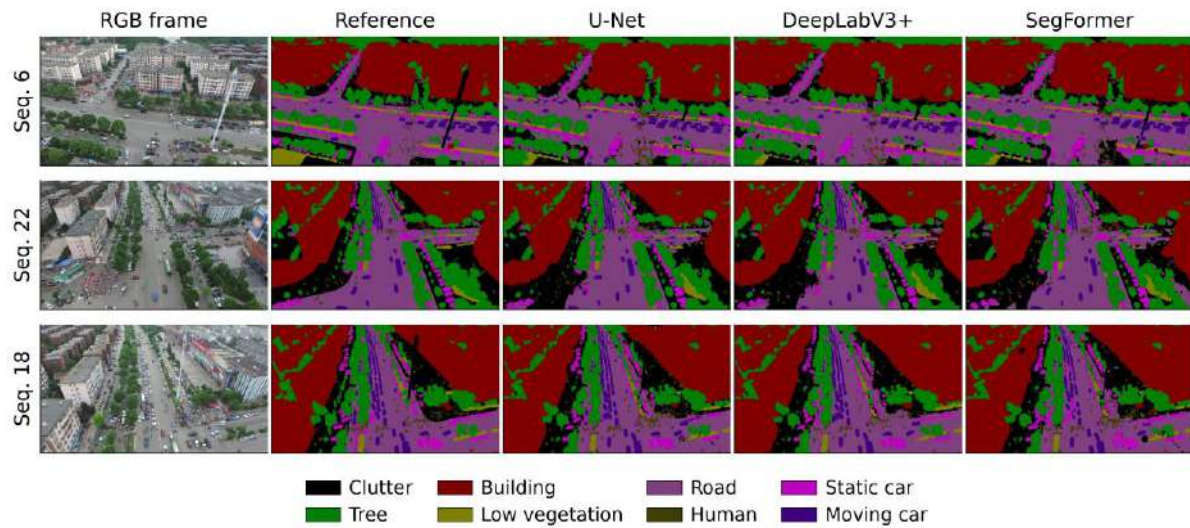


Fig. 8. Out-of-fold predictions under sequence-based spatial cross validation for frames of sequences 6, 22 and 18

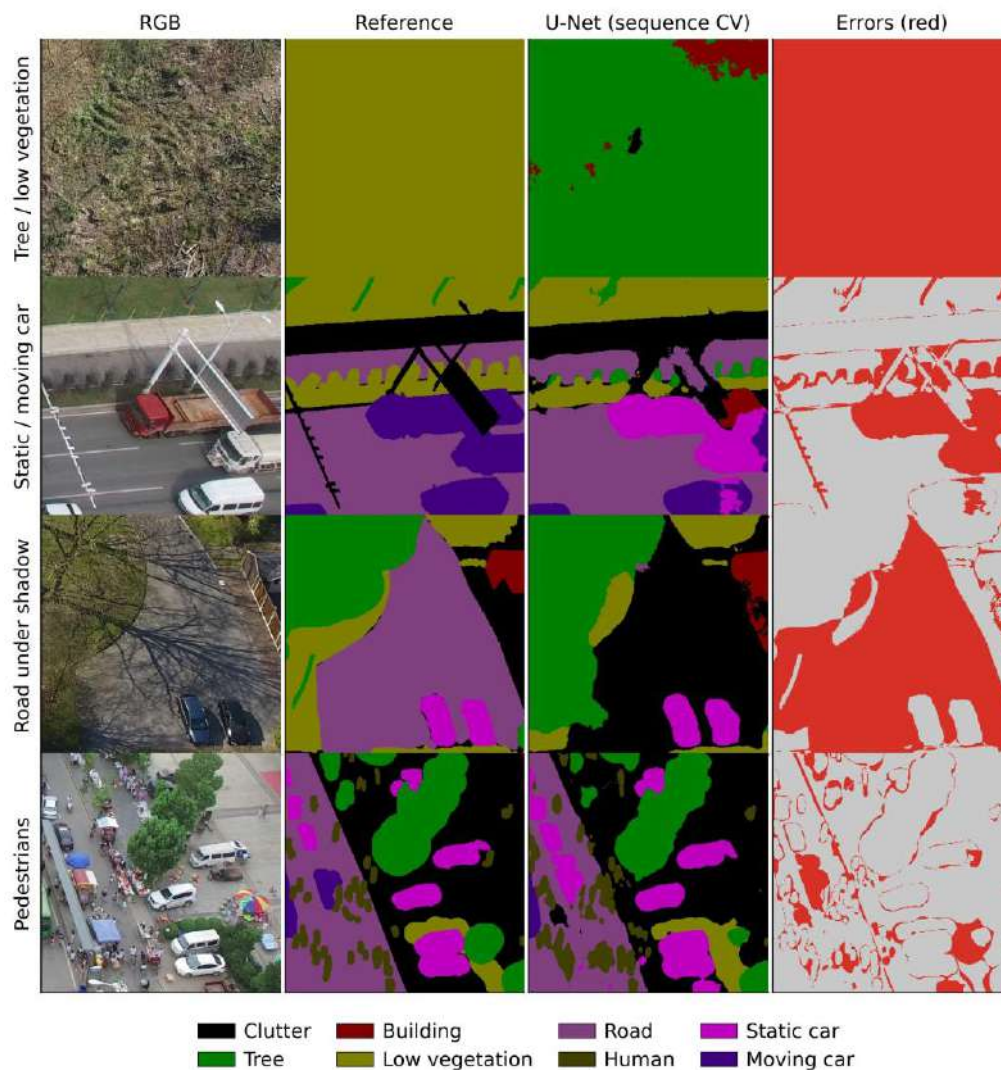


Fig. 9. Typical error situations of U-Net under sequence-based spatial cross validation: low vegetation labeled as tree, moving trucks labeled as static cars, road under tree shadow labeled as clutter and a crowded pedestrian area; crops of 360×360 pixels at the processed resolution, with red marking misclassified pixels

The confusion matrices of U-Net (Fig. 7) show where the additional errors on unseen sequences occur. Under sequence-based validation, 10.5% of the static car pixels were assigned to clutter and 5.7% to moving car, compared with 4.7% and 2.5% under the random split. Confusion between the two vegetation classes also increased: 13.5% of the low vegetation pixels were labeled as tree (10.8% under the random split), and 7.5% of the road pixels as clutter (4.8%). Humans were mainly confused with clutter (19.2%) and road (7.5%) under both protocols.

Fig. 8 shows out-of-fold predictions of the three models for frames of three sequences under sequence-based validation. All models reproduce the layout of the road network, building blocks, tree rows and parked vehicles, and their predictions differ mainly at object boundaries, in small vegetation patches and in the separation of static and moving cars. Typical error situations are shown in Fig. 9. Extended areas of low shrubs and cleared vegetation were classified as tree, large trucks on the carriageway were labeled as static instead of moving cars, road surfaces under strong tree shadows were assigned to clutter, and in crowded street scenes pedestrians were detected but with incomplete extents and confusion with clutter.

4. Discussion

4.1. Implications for Road Corridor Monitoring

The experiments indicate which corridor elements can be mapped reliably from RGB UAV imagery of urban streets when a model is applied to flights that were not used for training. Buildings, road surfaces and trees reached IoU values of about 80% or higher under sequence-based validation, which is sufficient for inventory tasks such as delineating the carriageway, identifying roadside tree rows or detecting changes in the adjacent built environment. Low vegetation and clutter were mapped with IoU values of about 66%, and static cars with about 60%, so maps of grass verges, shrubs and parked vehicles derived from new flights will contain a considerable share of misclassified pixels. Pedestrians remained the most difficult class. Their low IoU is consistent with the small number of pixels per person at a flying height of about 50 m, which was further reduced by the down sampling used in this study, and with the extreme class imbalance of the dataset (Table 2).

The sequence-based estimates are also of the same magnitude as benchmark results that were obtained on the sequence-disjoint official UAVid split. UNetFormer, for example, reported 67.8% mIoU on the UAVid test set [20], compared with 68.6–69.9% for the models of this study under sequence-based cross validation. Because that study used a different subset of sequences, image resolution and training schedule, the values are not directly comparable, but the agreement indicates that sequence-level separation yields accuracy estimates in the range known from benchmark evaluation, whereas the random split places the same models about four points higher. For practitioners who train models on their own corridor flights and evaluate them on randomly selected frames of the same flights, this optimism translates directly into overconfident expectations for subsequent survey campaigns.

4.2. Effect of Spatial Validation on Reported Performance

The consistent difference between the two protocols agrees with the literature on spatial autocorrelation in remote sensing and ecological mapping, which has repeatedly shown that random cross validation overestimates the accuracy of predictions for new locations [23, 25, 26]. Kattenborn et al. [27] reported the same effect for convolutional neural networks trained on UAV imagery of vegetation. The present results add to these findings in two respects. First, the inflation was observed in 41 or 42 of 42 sequences for every model and was of similar size for a CNN encoder decoder, an atrous pyramid network and a transformer, which suggests that it is a property of the data partition and not of a particular architecture. Second, random splitting also concealed the variability of the accuracy between groups of flights: the fold-wise standard deviation of mIoU increased from below one point to about two points under sequence-based validation, so a random split conveys an overly narrow picture of the uncertainty of the reported accuracy.

The class-wise pattern is consistent with the mechanism behind this optimism. In a random split, most test frames have neighboring frames from the same flight in the training set, taken about 5 s

earlier or later. Stationary objects such as parked cars, clutter objects and low vegetation patches appear in these neighboring frames at almost the same position and with the same appearance, so the model can in effect recognize objects it has already seen. This explains why static cars, clutter and low vegetation were the most inflated classes. Buildings and trees are large, frequent and visually distinctive, so they are mapped well even in unseen scenes and gained little from the leakage. Moving cars and pedestrians change position between frames, so the nearby training frames provide less direct information about their location, which is consistent with the smaller inflation for moving cars and the absence of inflation for humans. The magnitude of the overall effect is moderate compared with some large-area ecological mapping studies [25], which is plausible because all UAVid sequences were recorded with similar sensors, flying heights and urban scene types, so that unseen sequences are still similar to the training data in many respects.

The sequence-based design is not the only valid validation strategy, since spatial cross validation estimates the accuracy for new locations and can be pessimistic when the aim is to assess a map of the sampled area itself [31], and rigorous map accuracy assessment ideally relies on a probability sample of reference data [32]. For corridor monitoring, however, the typical use case is to apply a trained model to new flights, other road sections or later campaigns, and the sequence-based protocol matches this use case. The results therefore support grouping UAV frames by flight, sequence or location when accuracy is estimated, in line with the leave-location-out and leave-one-flight-out designs recommended for UAV mapping [9, 12] and with block cross validation in other spatial prediction tasks [28, 30].

4.3. Model Behavior across Corridor Classes

The three deep models behaved similarly across classes, and the choice of validation protocol did not alter their ranking. U-Net with a ResNet-50 encoder was about one point better than DeepLabV3+ and SegFormer, while the latter two did not differ significantly. This result should not be generalized to a statement on CNNs versus transformers: SegFormer was used with a reduced decoder width, all models were trained for the same fixed number of iterations on about 330 frames per fold, and larger transformer variants or longer schedules may benefit more from the data. The finding that the ranking was stable is nevertheless practically relevant, because it means that model comparisons made with a random split are not necessarily wrong, even though the absolute accuracies are inflated.

The main error sources were similar for all models, and the confusion between tree and low vegetation reflects the limited spectral separability of vegetation types in RGB imagery; in the oblique view, dense shrubs and low tree crowns have similar color and texture, and the distinction relies on height and context. The separation of static and moving cars is inherently difficult in single frames, as noted by the dataset authors [14], because the class depends on motion that is only observable over time; large trucks in traffic were typically labeled as static. Strong cast shadows on the road surface reduced the road recall, and pedestrians were missed or merged with clutter in crowded scenes. The pixel-based Random Forest reference, which used a limited set of RGB indices and local texture features at a quarter of the original resolution, reached less than 30% mIoU and almost failed for cars and humans. This shows that such a simple feature set is insufficient for oblique UAV street scenes, in line with earlier comparisons of deep and classical learners in remote sensing [8, 21], but it does not imply that classical classifiers with object-based or multi-scale features would perform equally poorly.

4.4. Limitations and Future Work

Several limitations should be considered when interpreting the results. The experiments are based on a single dataset of urban street scenes, and UAVid does not represent highway corridors with wider verges, embankments and cut slopes; the absolute accuracies and the size of the protocol effect may differ for rural and interurban road networks. The imagery is RGB only, so vegetation types and surface materials could only be separated by color and texture. Frames were down sampled by a factor of two, which reduces the detail available for small objects such as pedestrians, and all models were trained with a fixed budget of 6000 iterations and a common learning rate without model-specific tuning. Each protocol was evaluated with a single fold assignment; repeated

partitions with different random seeds were not computed, so the variability caused by the assignment itself is not quantified, although the consistency of the effect across 41 or 42 of 42 sequences indicates that the main result does not hinge on a particular assignment. The loss function was selected after short pilot runs on one sequence-based fold, so this fold was not entirely independent of the training design. The test labels were taken from a third-party redistribution, and two test frames contained implausible human labels, which were excluded from the evaluation but remained in the training data. Sequences were also treated as independent units, although some of them may have been recorded in neighboring parts of the same city; if so, the sequence-based estimates may still be slightly optimistic compared with a transfer to entirely new regions.

Future work should evaluate the transfer of models trained on UAVid to other UAV datasets with compatible classes, such as AeroScapes [43], and to corridor flights along highways, where validation by road section or flight campaign can be applied directly. Multispectral and thermal sensors could improve the separation of vegetation types and surface conditions, and temporal models that exploit consecutive video frames are a natural way to distinguish moving from static vehicles. Multi-temporal UAV and satellite observations would allow vegetation encroachment and land cover change along the RoW to be monitored over time [44], and combining corridor land cover maps with terrain information would support slope hazard assessment along road cuts and embankments [45]. For traffic participants, object detection with localization measures suited to tiny targets, such as the normalized Wasserstein distance [46, 47], could complement pixel-based segmentation. Estimating the area of applicability of a trained model [48] would further help to flag new flights whose conditions differ from the training data.

5. Conclusions

This study compared a random frame split with a sequence-based spatial cross validation for deep semantic segmentation of UAV optical imagery of urban road corridors. U-Net, DeepLabV3+ and SegFormer were trained under identical settings on all 42 labeled UAVid sequences, and every frame was predicted once under each protocol. Across the three models, the random split overestimated mIoU by 4.1 to 4.3 percentage points and overall accuracy by 2.1 to 2.6 points, and the optimism was present in 41 or 42 of the 42 sequences. It was concentrated in stationary and context-dependent classes such as static cars, clutter, low vegetation and road, whereas buildings and trees were affected little and pedestrians not at all. This class-wise pattern is consistent with information leakage between temporally adjacent frames of the same flight. Sequence-based validation also revealed a two to three times larger variability of accuracy between groups of flights, which a random split conceals.

On unseen flights, RGB-based deep segmentation mapped buildings, road surfaces and trees with IoU values of about 80% or higher, low vegetation and clutter with about 66%, vehicles with 59–72% and pedestrians with about 40%. These values indicate that the road surface, the adjacent built environment and roadside tree rows can be inventoried reliably from RGB UAV imagery of urban streets, whereas grass verges, shrubs, parked vehicles and pedestrians require further improvement before they can be mapped operationally. The relative ranking of the models was not affected by the validation protocol, with U-Net performing slightly better than DeepLabV3+ and SegFormer under the fixed training budget used here, and all deep models clearly outperformed a pixel-based Random Forest reference with a limited feature set.

For UAV-based road corridor monitoring, accuracy should be reported with validation schemes that hold out complete flights or sequences, because these estimates correspond to the intended application of trained models to new survey campaigns. Reporting class-wise metrics, the variability between folds and confidence intervals obtained at the level of flights or sequences, together with the fold assignment itself, makes such results easier to verify and compare. The transfer of these findings to highway corridors, to other sensors and to repeated partitions of the data remains to be tested in future studies.

Appendix A. Sensitivity to the Label Correction

Table A1. Main results with the uncorrected and the corrected evaluation of the two implausible human regions (%) under the random split (R) and the sequence-based spatial cross validation (S); Δ is R minus S mIoU with the 95% cluster bootstrap interval, sequences lower is the number of sequences with lower mIoU under S, and p is the unadjusted Wilcoxon signed-rank p-value

Evaluation	Model	mIoU R	mIoU S	Human IoU R	Human IoU S	Δ mIoU (95% CI)	Sequences lower	p
Uncorrected	U-Net	73.7	69.6	35.4	37.6	4.1 (3.1–5.3)	41/42	5.0×10^{-11}
Uncorrected	DeepLabV3+	72.8	68.5	35.6	36.4	4.3 (3.4–5.3)	42/42	4.5×10^{-13}
Uncorrected	SegFormer	72.6	68.3	38.1	37.7	4.3 (3.1–5.8)	42/42	4.5×10^{-13}
Uncorrected	Random Forest	28.5	26.5	1.3	1.2	2.0 (1.5–2.6)	41/42	9.1×10^{-13}
Corrected	U-Net	74.0	69.9	37.2	39.8	4.1 (3.0–5.3)	41/42	5.0×10^{-11}
Corrected	DeepLabV3+	73.1	68.8	37.6	38.8	4.3 (3.4–5.3)	42/42	4.5×10^{-13}
Corrected	SegFormer	72.9	68.6	40.2	39.8	4.3 (3.1–5.8)	42/42	4.5×10^{-13}
Corrected	Random Forest	28.5	26.5	1.3	1.2	2.0 (1.5–2.6)	41/42	9.1×10^{-13}

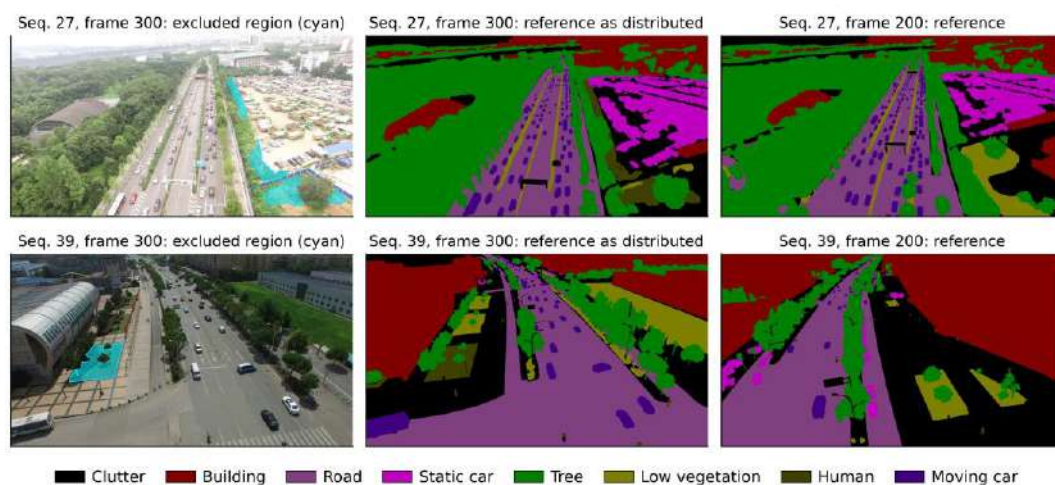


Fig. A1. Implausible human labels in two test frames: the excluded regions (cyan, left), the reference labels as distributed (middle) and the reference labels of the preceding labeled frame of the same sequence (right), in which the corresponding areas are labeled clutter, tree, low vegetation or road

Data Availability

The UAVid dataset is available from the UAVid website (<https://uavid.nl/>) and from the DANS Data Station (doi:10.17026/dans-x9f-w9sa) under a CC BY-NC-SA 4.0 license. The fold assignment, the corrected and uncorrected per-frame, per-sequence and class-wise metrics, the masks of the excluded label regions, the training and evaluation scripts and the software environment are available from the corresponding author upon reasonable request.

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Regional wind-energy suitability screening under uncertainty using GIS-MCDA in Kayseri, Türkiye

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Abstract

Regional wind energy planning requires spatial screening methods that account for modelling uncertainty before candidate areas advance to detailed investigation. This study presents an uncertainty-aware Geographic Information Systems–Multi-Criteria Decision Analysis framework for onshore wind-energy siting within a Kayseri-centered regional analytical frame in Türkiye. Seven continuous criteria were standardized using an empirical logistic fuzzy approach and evaluated under Equal, CRITIC, and Entropy weighting schemes across permissive, balanced, and strict planning assumptions. A 1000-iteration Monte Carlo simulation was used to examine spatial uncertainty. Within the regional analytical frame, the suitable area decreased from approximately 21,003.5 km² under permissive assumptions to 4,946.7 km² under strict assumptions. Within the tested perturbation ranges, planning-threshold variation produced a larger mean spatial effect than weight variation. Probability and connectivity rules yielded 31 candidate areas covering approximately 324.0 km², and all candidates retained at least 3 km² of connected area after LULC-based physical screening. Within the balanced eligible domain, mapped turbine locations showed higher modelled suitability than general eligible background locations (AUC = 0.961), providing an external spatial-consistency diagnostic rather than a measure of predictive accuracy. The resulting candidate areas are intended for detailed investigation and require subsequent legal, ecological, wind-resource, cadastral, engineering, and grid-capacity assessment.

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1. Introduction

The transition to renewable energy systems involves regional spatial planning to balance technical requirements with environmental and social constraints [1]. Onshore wind energy siting is a spatial decision problem where resource quality, topographical characteristics, infrastructure accessibility, settlement separation, and ecological protection statuses frequently interact [2,3]. To synthesize these heterogeneous spatial layers, GIS based multi criteria decision analysis (GIS-MCDA) is commonly utilized as a methodological framework for renewable energy facility siting and spatial decision support [4].

However, a methodological challenge remains in the operational interpretation of GIS MCDA outputs. Many regional planning studies produce deterministic suitability surfaces presenting a single map of theoretically favorable locations [5]. While visually informative, these deterministic maps can mask the inherent uncertainties in planning assumptions, criterion weights, and spatial thresholds. When continuous source data are harmonized to a common analytical grid, the resulting cell level score functions as a synthetic planning indicator rather than a definitive engineering parameter. The translation of heterogeneous spatial criteria into structured planning areas requires cautious interpretation and uncertainty assessment [6,7]. If a model is sensitive to

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minor changes in exclusion thresholds or weighting schemes, the resulting high suitability areas require further verification before policy making [8,9]. Therefore, treating uncertainty as an explicit analytical object provides a practical approach to transition from cell level scores to uncertainty characterized candidate areas [10,11].

Türkiye has an expanding renewable energy capacity and a developing body of applied literature on wind energy spatial planning [12,13]. While earlier GIS MCDA applications often focused on the western regions of Türkiye, the complex topography of Central Anatolia also presents notable planning scenarios. The Kayseri and Erciyes regional geospatial context is defined by high environmental heterogeneity, incorporating the volcanic terrain of Mount Erciyes, widespread agricultural lowlands, and expanding settlement corridors, which provides a relevant setting for multi-criteria geospatial screening [14,15]. Although localized studies have evaluated the applicability of GIS methods for wind power planning in this regional setting [16,17], there remains a need for a regional framework that systematically assesses model uncertainty before candidate areas are advanced for detailed investigation.

To address this context, this study presents an uncertainty-aware GIS-MCDA framework for onshore wind-energy siting in a Kayseri-centered regional setting in Türkiye. The study contributes in three ways. First, it combines empirical logistic fuzzy standardization with Equal, CRITIC, and Entropy weighting under permissive, balanced, and strict planning assumptions. Second, it links pixel-level probability surfaces with connected-patch extraction and exact candidate-level Monte Carlo metrics rather than relying only on regional raster summaries. Third, it separates raster suitability modelling from candidate-level prioritization, land-cover characterization, and an external mapped-turbine spatial-consistency diagnostic. A related question is whether spatial uncertainty within the evaluated framework is affected more strongly by variation in criterion weights or by variation in planning-constraint thresholds over the perturbation ranges examined. The resulting framework narrows the regional search space while retaining explicit information on uncertainty for subsequent field, legal, ecological, engineering, and grid-capacity investigations.

2. Materials and Spatial Framework

2.1 Study Area and Environmental Context

The study focuses on Kayseri in central Türkiye and uses a regional analytical frame centered on the province and its immediate surroundings. The Kayseri provincial boundary provides the principal geographic reference, whereas raster-based calculations were performed over the surrounding analytical frame rather than being clipped to the administrative boundary. The regional setting is heterogeneous, comprising high-altitude volcanic topography, extensive agricultural plains, and developing urban and industrial corridors. This setting requires wind-resource conditions to be considered together with terrain, settlement, infrastructure, hydrological, and environmental factors.

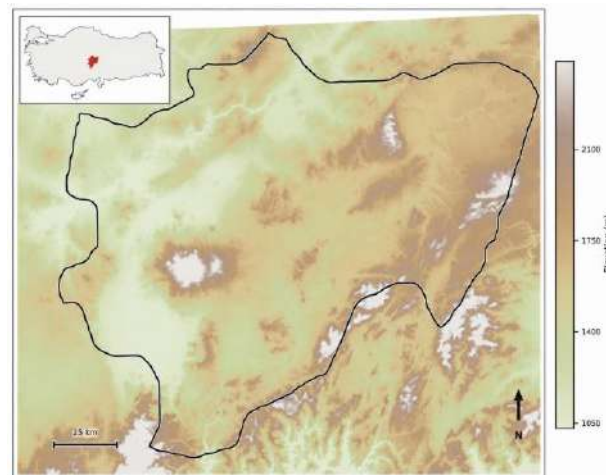


Fig. 1. Location and topographic map of Kayseri within the regional analytical frame

To ensure spatial consistency across all analyses, spatial inputs were transformed to WGS 84 / UTM zone 36N and aligned to the 27 m reference grid established during the preprocessing workflow. The 27 m cell size was retained as an operational harmonization grid rather than as an optimized representation of source resolution. A common-valid data mask excluded cells lacking required aligned inputs and yielded approximately 33,399.3 km² within the regional analytical frame. The common grid provides a consistent raster lattice for overlay and spatial calculation; resampling coarser datasets to this grid does not increase their native information content or original spatial detail[18]. Candidate boundaries are therefore interpreted as regional screening geometries rather than engineering-precision site limits. The geographical context and analytical frame are shown in Figure 1.

2.2 Spatial Datasets and Harmonization

The spatial analysis used resource, terrain, infrastructure, settlement, hydrological, environmental, and land-cover datasets. Seven continuous layers entered the WLC suitability calculation: 100 m wind speed, slope, road distance, residential distance, protected-area distance, river distance, and powerline distance. The remaining datasets served as topographic support, preprocessing/context layers, candidate-level screening inputs, or external diagnostic data. Their analytical roles are summarized in Table 1.

Table 1. Spatial datasets and analytical roles.

Input	Source framework	Documented native characteristic	Role in analysis
Wind speed at 100 m	Global Wind Atlas 3.0	~250 m	Continuous resource criterion
Elevation	SRTM DEM	90 m	Topographic support and source for slope; not an independent weighted criterion
Slope	Derived from SRTM DEM	90 m source; 27 m analytical	Continuous terrain criterion
Road distance	OpenStreetMap road vectors	vector source	Euclidean distance criterion
River distance	DSİ surface water data	vector/source dependent	Euclidean distance criterion
Residential distance	HGK settlement data	vector/source dependent	Settlement separation criterion
Substation distance	KCETAŞ/TEİAŞ source framework	vector/source dependent	Ancillary electrical-infrastructure context and common-valid coverage support; not included in WLC or threshold scenarios
Powerline distance	TEİAŞ source framework	vector/source dependent	Electrical infrastructure proximity criterion
Fault distance	MTA active fault data	vector/source dependent	Ancillary planning-context and common-valid coverage support; not included in WLC or threshold scenarios
Protected area distance	TUCBS environmental status data	vector/source dependent	Planning constraint and distance context
ESA WorldCover 2021 v200	European Space Agency	~10 m native	Candidate prioritization screening variables
LULC	CORINE Land Cover-derived local 11-class reclassification [19]	Locally supplied 10 m raster derived from CORINE Land Cover classes; harmonized to 27 m	Candidate-level physical characterization and screening; not included in final composite ranking

The wind resource input was represented by the Global Wind Atlas 3.0 100 m wind speed product [20–22]. Topographical inputs were derived from the SRTM Digital Elevation Model [23–25]. Infrastructure and proximity data were compiled from various national and open-source frameworks [26–28]. The locally reclassified 11-class LULC raster was derived from CORINE Land Cover categories [19].

3. Methodology

The methodology was structured as a sequential spatial screening framework that linked criterion standardization, scenario-based suitability modelling, uncertainty assessment, and candidate-area delineation. Raster-based outputs were subsequently translated into connected candidate units for candidate-level prioritization and land-cover characterization. External mapped-turbine information was evaluated separately as a spatial-consistency diagnostic and was not used in candidate scoring. The overall analytical workflow is summarized in Figure 2.

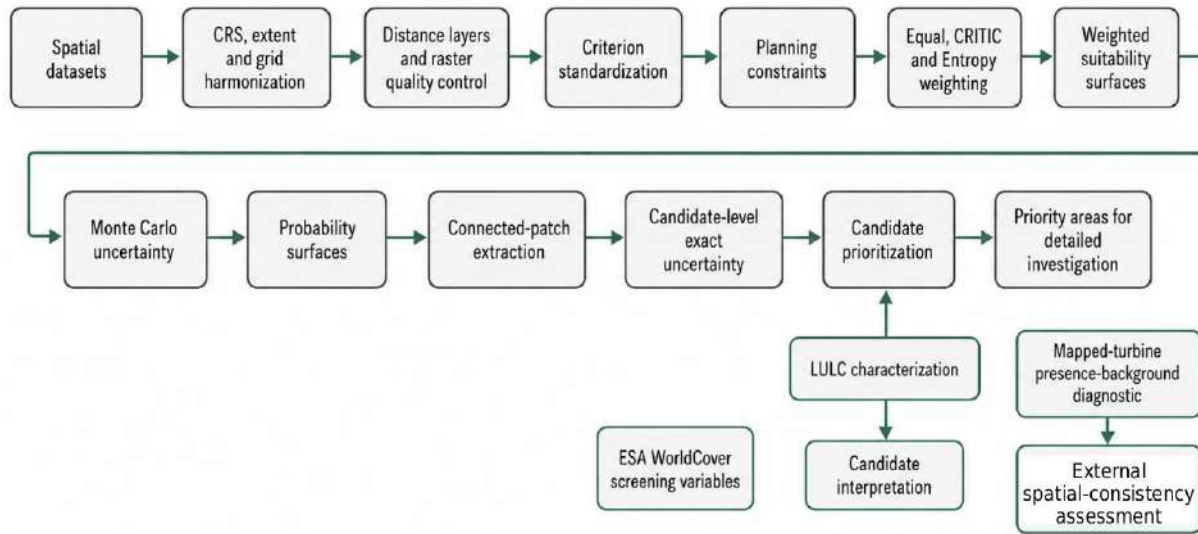


Fig. 2. Data-processing and analytical framework of the study

3.1 Suitability Modelling and Weighting Schemes

Prior to criteria aggregation, an empirical logistic fuzzy standardization was applied to transform physical and distance parameters into dimensionless suitability scores (x_{ij}^*) ranging from 0 to 1. Values were initially clipped to their empirical 1st and 99th percentiles ($P1_j$, $P99_j$). The logistic midpoint was set to the empirical median ($P50_j$), and the logistic slope (k_j) was formulated as $k_j = \ln(9) / \text{mean}(|P90_j - P50_j|, |P50_j - P10_j|)$.

Criteria that prioritize higher values or greater distances (fuzzy large), specifically 100 m wind speed, residential distance, protected-area distance, and river distance, were transformed using Eq. (1).

$$x_{ij}^* = \frac{1}{1 + \exp[-k_j(\text{clip}(x_{ij}, P1_j, P99_j) - P50_j)]} \quad (1)$$

Criteria that prioritize proximity or lower values (fuzzy small) specifically slope, road distance, and powerline distance were transformed using Eq. (2).

$$x_{ij}^* = 1 - \frac{1}{1 + \exp[-k_j(\text{clip}(x_{ij}, P1_j, P99_j) - P50_j)]} \quad (2)$$

Table 2. Empirical logistic standardization parameters used for the seven continuous criteria

Criterion	P1	P10	P50	P90	P99	k _j
Wind speed at 100 m	3.058	4.143	5.491	7.259	9.482	1.410
Slope	0.583	1.538	6.875	22.753	35.458	0.207
Road distance	2.550	43.735	448.185	2295.635	4985.466	1.951×10^{-3}
Residential distance	0.000	779.953	3270.818	9801.677	18262.133	4.87×10^{-4}
Protected-area distance	0.000	1645.929	9180.945	40173.778	69875.966	1.14×10^{-4}
River distance	0.0003	86.532	566.397	1455.398	2245.749	3.210×10^{-3}
Powerline distance	93.964	1386.616	9878.549	34160.713	49095.573	1.34×10^{-4}

The empirical percentiles and logistic-slope parameters were calculated once over the common-valid analytical domain using finite, non-NoData criterion cells and were held fixed during the Monte Carlo simulations. The wind-percentile lookup used for planning constraints was handled separately from these standardization parameters.

For slope and setback-type constraints, scenario values were selected as literature-informed planning assumptions spanning relatively permissive to more restrictive regional screening conditions [8,9,29–31]. These values are not presented as statutory setback requirements for Türkiye. Wind constraints were defined separately using empirical percentiles (P10, P25, and P40) calculated from the aligned wind raster. These percentiles represent relative resource-screening levels within the analytical frame rather than turbine cut-in speeds, economic viability thresholds, or regulatory limits. Road and powerline distances were retained as continuous preference criteria rather than Boolean exclusion constraints because they represent regional accessibility and grid-proximity considerations; their fuzzy-small functions do not represent project-scale engineering clearances or statutory safety setbacks. The scenario thresholds and corresponding suitable areas within the analytical frame are reported in Table 3.

Table 3. Planning assumptions, permissive, balanced, and strict thresholds, and suitable areas within the analytical frame

Constraint	Permissive	Balanced	Strict
Wind empirical percentile	P10	P25	P40
Slope	$\leq 20^\circ$	$\leq 15^\circ$	$\leq 10^\circ$
Residential distance	≥ 500 m	≥ 1000 m	≥ 1500 m
Protected area distance	≥ 500 m	≥ 1000 m	≥ 2000 m
River distance	≥ 100 m	≥ 300 m	≥ 500 m
Suitable area within analytical frame (km ²)	21,003.5	11,508.7	4,946.7

The cell level suitability score (S_i) was calculated using a Weighted Linear Combination approach [2,29]. The standardized values of the seven continuous criteria were aggregated based on their assigned weights (w_j), as shown in Eq. (3). The seven standardized criterion rasters constituted the continuous spatial inputs to the WLC suitability calculation.

$$S_i = \sum_{j=1}^n w_j x_{ij}^* \quad (3)$$

To evaluate structural dependencies within the model, three distinct weighting schemes were compared. Equal Weight was assigned as a transparent baseline reference weighting scheme. CRITIC (Criteria Importance Through Intercriteria Correlation) and Entropy were utilized as data derived objective weighting schemes [32]. The specific criterion weights for each method are summarized in Table 4.

Table 4. Equal, CRITIC, and Entropy weights with model agreement and robustness diagnostics

Criterion or Diagnostic	Equal	CRITIC	Entropy
100 m wind speed	0.142857	0.147767	0.141387
Slope	0.142857	0.146794	0.198838
Road distance	0.142857	0.136978	0.183799
Residential distance	0.142857	0.143987	0.094372
Protected area distance	0.142857	0.139745	0.075635
River distance	0.142857	0.141417	0.121330
Powerline distance	0.142857	0.143314	0.184640
Highest pairwise Spearman	0.9996		
Lowest relevant pairwise Spearman	0.814		
Alternative transformation median Spearman	0.966		
Criterion set robustness median Spearman	0.928		
Balanced Moran's I	0.891		

Weighting method agreement was assessed using pairwise Spearman rank correlations. Transformation robustness and criterion set robustness were summarized from the corresponding sensitivity comparisons, whereas Moran's I was used to characterize global spatial autocorrelation of the balanced suitability surface. Global Moran's I for the balanced ensemble suitability surface was calculated after 10×10 block-mean aggregation to 270 m cells using binary queen-contiguity neighbors among finite aggregated cells. Significance was evaluated with 199 random permutations using seed 42.

The primary reporting ensemble was computed as the cell wise arithmetic mean of the three weighting-based suitability surfaces under the balanced planning assumptions. Furthermore, a robust high suitability category was defined to identify a comparatively stable high suitability subset across the evaluated weighting formulations, conceptualized as areas that simultaneously fall within the ensemble mean top 10 percent, the model standard deviation bottom 25 percent, and the top 20 percent of all three individual weighting models.

3.2 Monte Carlo Simulation for Uncertainty Assessment

To transition from a deterministic framework to a probabilistic one, a 1000 iteration Monte Carlo simulation (random seed 42) was executed across three modes: weight-only, threshold-only, and combined. Weight uncertainty was represented using empirical bootstrap weight vectors. For each of the CRITIC and Entropy methods, 500 bootstrap replicates were generated by sampling 50,000 rows with replacement from the common-valid standardized criterion sample using random seed 42. The seven criteria were resampled jointly within each replicate, and the corresponding CRITIC or Entropy weights were recomputed. The stored bootstrap vectors were subsequently sampled during the Monte Carlo simulations. CRITIC and Entropy vectors were selected with probabilities of 0.45 each, while the Equal Weight reference vector was retained with probability 0.10; these probabilities were prespecified simulation-design choices rather than empirically calibrated probabilities. After selection, individual weights were clipped to 0.02–0.35 and renormalized to sum to 1.

For threshold uncertainty, constraint boundaries were varied independently using triangular probability distributions (lower, mode, upper), a bounded form commonly used to represent uncertain parameter ranges [11]. The scenario-specific bounds were prespecified for this study and were not interpreted as regulatory tolerances or measurement-error distributions; symmetry around the mode was not imposed. The distributions were: slope (permissive 16, 20, 24°; balanced 12, 15, 18°; strict 8, 10, 12°); residential distance (400, 500, 700 m; 750, 1000, 1250 m; 1250, 1500, 2000 m); protected-area distance (400, 500, 750 m; 750, 1000, 1500 m; 1500, 2000, 2500 m); and river distance (75, 100, 150 m; 200, 300, 400 m; 400, 500, 700 m). Wind uncertainty used empirical percentile triplets (5, 10, 15; 20, 25, 30; 35, 40, 45) applied through the aligned wind-raster percentile lookup. The combined mode varied weights and thresholds simultaneously. Because the

uncertainty modes use different perturbation ranges and distributional forms, comparisons between their effects are conditional on this simulation design.

To accurately represent eligibility uncertainty, an explicit zero handling approach was implemented. If a cell was ineligible in a given iteration, its suitability contribution was recorded as 0. Consequently, the full resolution Monte Carlo suitability mean and standard deviation products, as well as the bounded moment-based approximations (P05, P10, median, P90, and P95), are unconditional suitability moments that incorporate eligibility uncertainty. They are not exact empirical Monte Carlo quantiles. Exact empirical quantiles were calculated exclusively for candidate level Monte Carlo summaries. Suitability outcomes were quantified through explicit probability metrics. The eligibility probability ($P_{E,i}$) measures the proportion of all iterations (R) in which cell i remained eligible ($E_{ir} = 1$), shown in Eq. 4.

$$P_{E,i} = \frac{1}{R} \sum_{r=1}^R I(E_{ir} = 1) \quad (4)$$

Monte Carlo suitability uncertainty (U_i) was calculated as the standard deviation of suitability scores across the 1000 iterations, where $\bar{S}_i$ denotes the cell mean (Eq. (5)).

$$U_i = \sqrt{\frac{1}{R} \sum_{r=1}^R (S_{ir} - \bar{S}_i)^2} \quad (5)$$

High suitability events were defined separately for each Monte Carlo iteration, uncertainty mode, and planning scenario. For iteration r , the high suitability threshold (τ_r) was defined as the 90th percentile of suitability scores among eligible sampled cells. This dynamic score threshold must not be confused with the 0.75 probability cutoff utilized later for candidate extraction. To distinguish between the persistence of inclusion and the persistence of high scores, conditional and unconditional high suitability probabilities were evaluated. The conditional high suitability probability ($P_{C,i}$) evaluates high suitability solely across eligible iterations (Eq. (6)), whereas the unconditional high suitability probability ($P_{U,i}$) evaluates the proportion across all iterations (Eq. (7)).

$$P_{C,i} = \frac{\sum_{r=1}^R I(E_{ir} = 1 \wedge S_{ir} \geq \tau_r)}{\sum_{r=1}^R I(E_{ir} = 1)} \quad (6)$$

$$P_{U,i} = \frac{1}{R} \sum_{r=1}^R I(E_{ir} = 1 \wedge S_{ir} \geq \tau_r) \quad (7)$$

3.3 Candidate Extraction and Diagnostic Screening

Connected-patch extraction was applied to the balanced planning assumption under the combined uncertainty mode to translate pixel-level probabilities into regional planning units. Candidate cells were required to have both eligibility probability ≥ 0.75 and conditional high-suitability probability ≥ 0.75 across finite, non-NoData cells; unconditional high-suitability probability and raw suitability scores were not used directly in this mask. Morphological cleanup removed eligible components smaller than 10 ha and subsequently filled internal holes smaller than 10 ha using 8-neighbour connectivity (138 pixels on the 27 m grid). Following cleanup, connected components were extracted and a minimum connected area of 5 km² was applied as a prespecified operational regional-screening threshold. This threshold was used to remove small fragmented patches and was not treated as a universal minimum wind-farm area.

Exact empirical quantiles and high suitability fractions were calculated for the extracted candidate polygons. Candidate level iteration statistics were calculated only if at least one eligible candidate pixel existed; in iterations with no eligible pixels, suitability summaries were recorded as NaN, and candidate level suitability quantiles were calculated over the finite iteration means. The candidate level exact high suitability fraction represents the Monte Carlo mean of the fraction of valid

candidate pixels meeting the eligibility and iteration specific high suitability conditions; it is therefore distinct from the cell level conditional probability raster.

Final candidate ordering relied on a composite prioritization score rather than exact Monte Carlo rank alone. The exact Monte Carlo robustness sub component (R_i) was formulated as Eq. (8), where mm denotes min max normalization across the candidate set.

$$R_i = 0.45 \times mm(P10_i) + 0.35 \times mm(P_{high,i}) + 0.20 \times mm(-SD_i) \quad (8)$$

The final composite priority score (C_i) combined exact Monte Carlo robustness (R_i), normalized log remaining connected area (A_i), ESA WorldCover-based land-cover compatibility (L_i), baseline aggregate suitability (B_i), and scenario persistence (P_i), as shown in Eq. (9). The component coefficients were prespecified as part of the candidate-prioritization rule and were not optimized against mapped turbine locations or other external outcomes.

$$C_i = 0.285714 \times R_i + 0.214286 \times A_i + 0.214286 \times L_i + 0.214286 \times B_i + 0.071429 \times P_i \quad (9)$$

Here, A_i is the min-max normalized log1p of the largest remaining connected component derived from ESA WorldCover screening, and $L_i = 1 - \text{excluded fraction using ESA WorldCover classes 50 (built-up), 70 (snow and ice), 80 (permanent water), 90 (herbaceous wetland), and 95 (mangroves)}$. B_i is the clipped primary baseline aggregate suitability score, and P_i is the stable-scenario count divided by three. Data-support class was retained as descriptive metadata ($A = 1.0$, $B = 0.8$, $C = 0.5$, $D = 0.2$). Because all 31 candidates were assigned class C, the data-support term was constant and was therefore removed from the ranking equation; the remaining coefficients were renormalized to sum to one. This affine rescaling does not alter candidate ordering.

3.4 LULC Characterization and OSM Diagnostic

Two land-cover datasets served different candidate-level roles. ESA WorldCover 2021 v200 supplied screening variables incorporated into the composite candidate-prioritization score. A separate 11-class LULC raster derived through local reclassification of CORINE Land Cover categories [19] was used for candidate-level physical characterization and screening. The CORINE-derived raster did not enter the final composite ranking.

External spatial consistency was examined using mapped wind-energy infrastructure from OpenStreetMap. The inventory contained 288 deduplicated turbine features; 163 were outside the analysis raster extent and 15 did not satisfy the balanced-domain eligibility mask, leaving 110 locations with finite model values for the primary diagnostic. These 110 points were compared with 2,200 general eligible background locations sampled from the balanced planning domain outside a 2 km turbine buffer. A second 2,200-point background matched by wind and slope deciles was used as a sensitivity comparison. No filtering was based on candidate rank or high-suitability outcome. Because the final presence sample was conditioned on the balanced eligible domain, the comparison is interpreted as an external spatial-consistency diagnostic within that domain rather than as independent predictive validation. OSM information was not used in candidate scoring.

4. Results

4.1 Spatial Suitability and Inter-model Disagreement

Planning constraints reduced the eligible search domain within the regional analytical frame. As reported in Table 3, suitable area was approximately 21,003.5 km² under permissive assumptions, 11,508.7 km² under balanced assumptions, and 4,946.7 km² under strict assumptions.

Under the balanced assumptions, pairwise rank correspondence among the weighting formulations ranged from Spearman $\rho = 0.814$ to 0.9996, with the highest agreement between Equal and CRITIC. The primary reporting ensemble represented the shared spatial tendency while retaining information on inter-model disagreement. The balanced top-10% consensus area

covered approximately 755.5 km² (Figure 3c), and the robust-high subset covered 395.4 km² (Figure 3d).

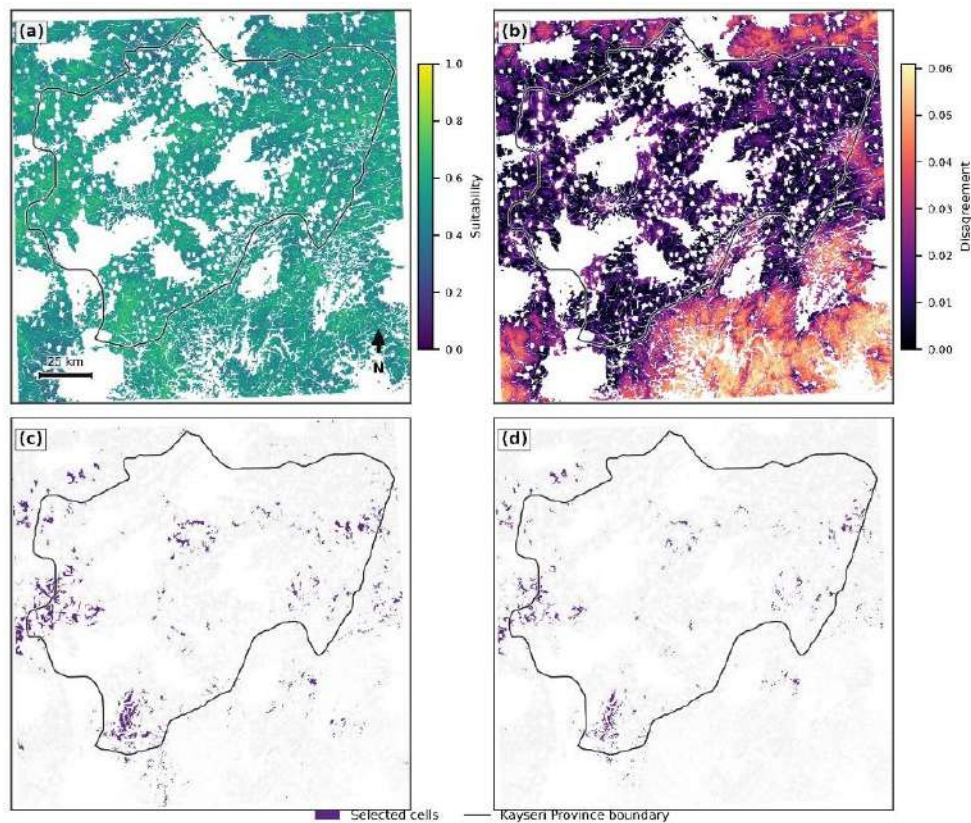


Fig. 3. Balanced ensemble suitability and robustness surfaces: (a) mean suitability, (b) inter-model disagreement, (c) top-10% consensus, and (d) robust-high suitability.

4.2 Probability Surfaces and Impact of Threshold Uncertainty

Within the tested perturbation ranges, the Monte Carlo simulation produced a larger mean spatial effect under threshold-only perturbation than under weight-only perturbation. Mean spatial uncertainty was 0.006 for weight-only perturbation, 0.035 for threshold-only perturbation, and 0.039 for the combined mode.

An eligibility probability ≥ 0.90 was identified across approximately 9,491.7 km² (Figure 4c). Conditional high-suitability probability ≥ 0.90 covered 766.9 km² (Figure 4d), whereas the corresponding unconditional probability ≥ 0.90 covered 710.4 km². These surfaces distinguish persistence of eligibility from persistence of high suitability under the sampled uncertainty settings.

4.3 Priority Candidate Areas and Exact Metrics

The initial patch extraction identified 14,902 raw connected components, which were reduced to 560 after morphological cleanup. Retained patch counts were 143 at ≥ 1 km², 69 at ≥ 2 km², 31 at ≥ 5 km², 12 at ≥ 10 km², and 2 at ≥ 20 km². Applying the prespecified ≥ 5 km² regional-screening threshold yielded 31 candidate areas covering approximately 324.0 km², with the largest patch covering 31.00 km².

The median absolute difference between exact and approximate candidate-level P10 values was 0.008. Baseline aggregate rank and exact Monte Carlo rank showed moderate correspondence (Spearman $\rho = 0.644$), indicating sensitivity of candidate ordering to the selected candidate-level uncertainty summary. The top 10 areas according to the final composite priority score are reported in Table 5; eight were persistent across all three planning scenarios, while KAY-WIND-006 and KAY-WIND-005 were persistent in two of three.

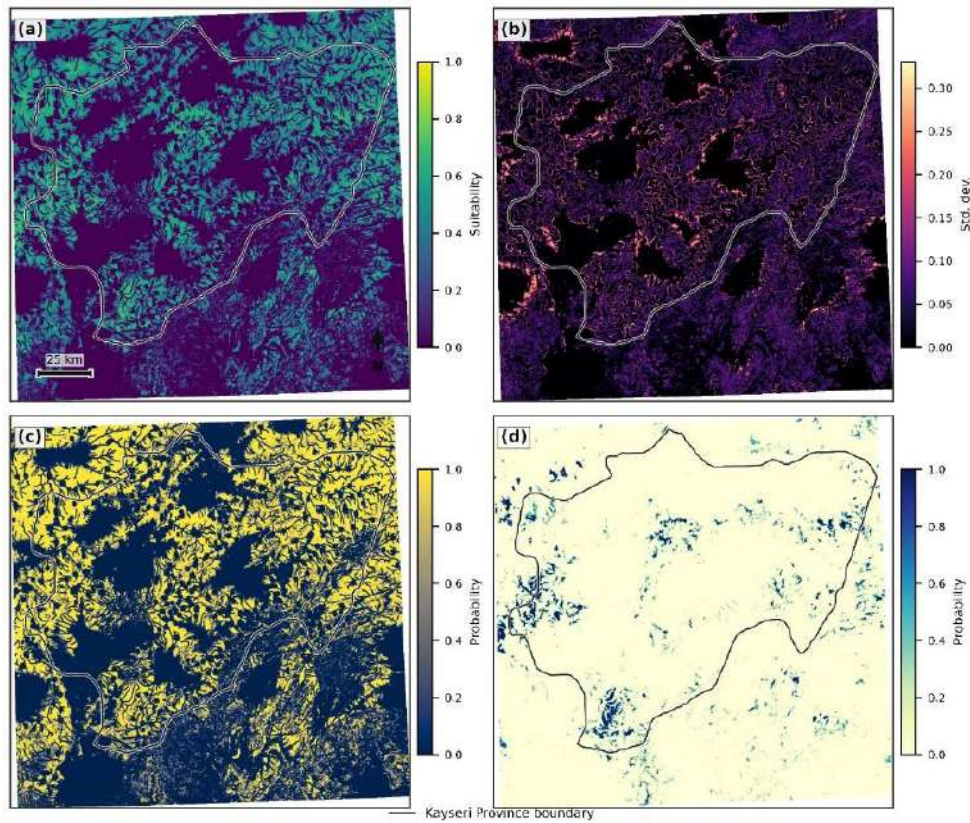


Fig. 4. Monte Carlo uncertainty and probability products for the balanced planning assumption set: (a) mean suitability, (b) combined uncertainty, (c) eligibility probability, and (d) conditional high-suitability probability.

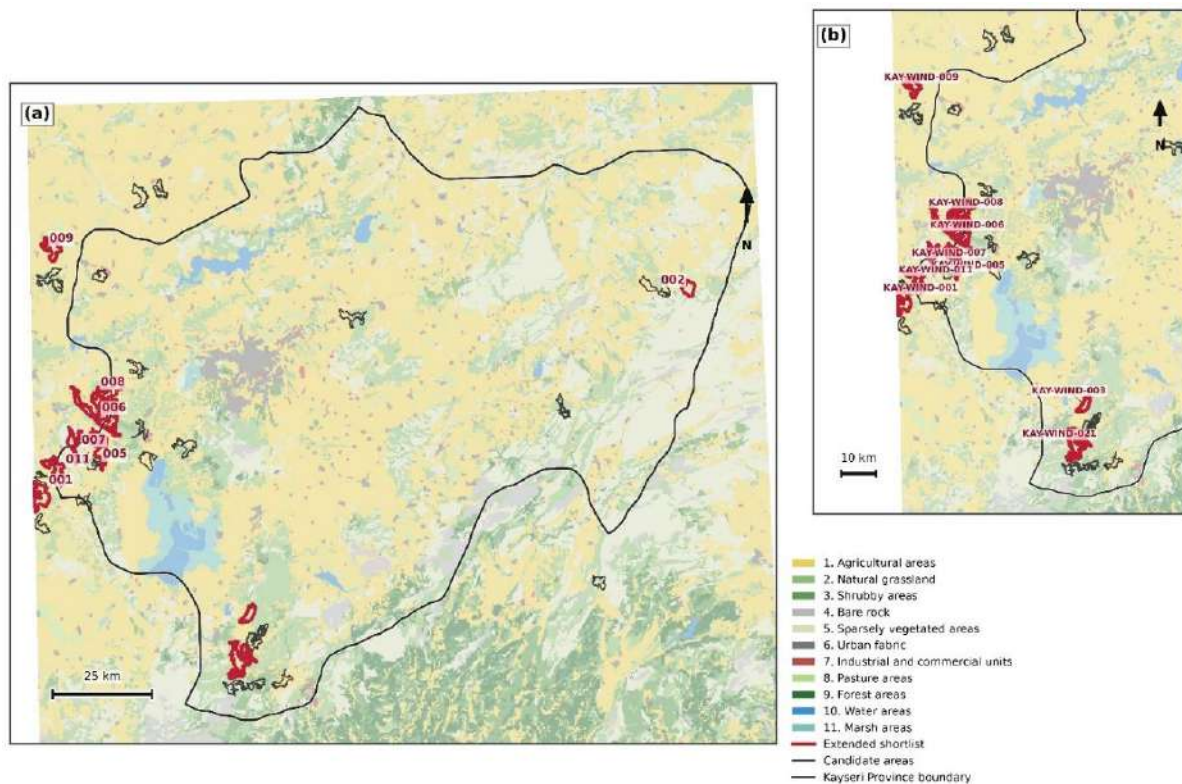


Fig. 5. Priority candidate areas and LULC context: (a) regional distribution of candidate areas with the Kayseri provincial boundary shown for geographic reference and the extended shortlist; (b) enlarged view of the main candidate concentration.

Table 5. Top 10 candidate areas according to the final composite prioritization score, with selected exact candidate-level Monte Carlo and LULC characterization metrics

Rank	Candidate	Area (km ²)	Exact P10	Exact high fraction	Exact MC SD	Final score	LULC / conflict
1	KAY-WIND-006	31.00	0.678	0.988	0.002	0.8793	Natural grassland / None
2	KAY-WIND-001	16.29	0.682	0.992	0.001	0.8717	Agricultural / None
3	KAY-WIND-005	16.90	0.671	0.994	0.004	0.7967	Agricultural / None
4	KAY-WIND-008	16.31	0.669	0.989	0.006	0.7823	Agricultural / None
5	KAY-WIND-003	7.95	0.682	0.991	0.003	0.7745	Agricultural / None
6	KAY-WIND-002	8.28	0.685	0.987	0.005	0.7664	Sparsely vegetated / Industrial-commercial
7	KAY-WIND-011	13.95	0.666	0.989	0.002	0.7509	Agricultural / None
8	KAY-WIND-021	24.44	0.679	0.960	0.003	0.7505	Natural grassland / None
9	KAY-WIND-007	9.90	0.675	0.988	0.004	0.7474	Agricultural / None
10	KAY-WIND-009	12.83	0.672	0.983	0.005	0.7209	Agricultural / Industrial-commercial

The CORINE-derived 11-class LULC characterization identified agriculture as the dominant class in 24 candidates, natural grassland in 4, sparsely vegetated land in 2, and shrubland in 1. All 31 candidates retained at least 3 km² of connected area after the physical screening; this was an observed outcome rather than an additional methodological threshold. No urban-fabric or water conflicts were identified. Industrial or commercial conflicts occurred in six candidates, and a marsh conflict occurred in KAY-WIND-004. KAY-WIND-009 showed the largest absolute screened-area loss (1.59 km²; 12.4%). Differences between ESA WorldCover and the CORINE-derived LULC raster were retained as a thematic-data limitation because the products use different classification and spatial representations. Candidate distribution and LULC context are shown in Figure 5.

4.4 External Spatial Consistency with Mapped Wind Turbines

The mapped-turbine diagnostic began with 288 deduplicated OSM features and retained 110 valid presence locations within the balanced eligible analysis domain. These locations represented 14 spatial clusters. Three candidate areas contained mapped turbines, and six candidates were within 5 km of mapped turbines.

The primary presence-background comparison yielded a balanced-ensemble AUC of 0.961 (permutation $p = 0.001$), and 98 of 110 valid mapped-turbine locations (89.1%) occurred above the general-background 90th percentile. The presence and background medians were 0.675 and 0.538, respectively (Figure 6b). The Monte Carlo mean-surface AUC against the general background was 0.888.

The wind- and slope-decile matched background produced a lower AUC of 0.890, indicating that part of the primary separation was associated with wind and terrain gradients represented in the suitability model. A cluster-representative sensitivity analysis using nine valid representatives yielded an AUC of 0.947 against the general eligible background and 0.868 against the matched background. These results are summarized in Table 6 and the primary presence-background distributions are shown in Figure 6b.

Table 6. Mapped-turbine presence-background spatial-consistency diagnostic results

Diagnostic Metric	Value	Interpretation
Mapped turbine inventory	288	Deduplicated OSM features
Valid presence points	110	Locations retained within the balanced eligible domain
Spatial clusters identified	14	Cluster assessment
Valid cluster representatives	9	Used in cluster-representative sensitivity comparison
General background points	2,200	Primary eligible background sample
Presence median (balanced)	0.675	Compared with primary background median
Background median (balanced)	0.538	Primary general eligible background
Presence-background AUC	0.961	Primary balanced-ensemble spatial-consistency diagnostic
Valid presence points above general background P90	98/110 (89.1 percent)	Balanced ensemble top decile evaluation
Matched-background AUC	0.890	Wind/slope-decile matched sensitivity comparison
Monte Carlo mean-surface AUC	0.888	General-background comparison
Cluster-representative AUC (general)	0.947	Nine valid cluster representatives
Cluster-representative AUC (matched)	0.868	Nine valid cluster representatives; matched background

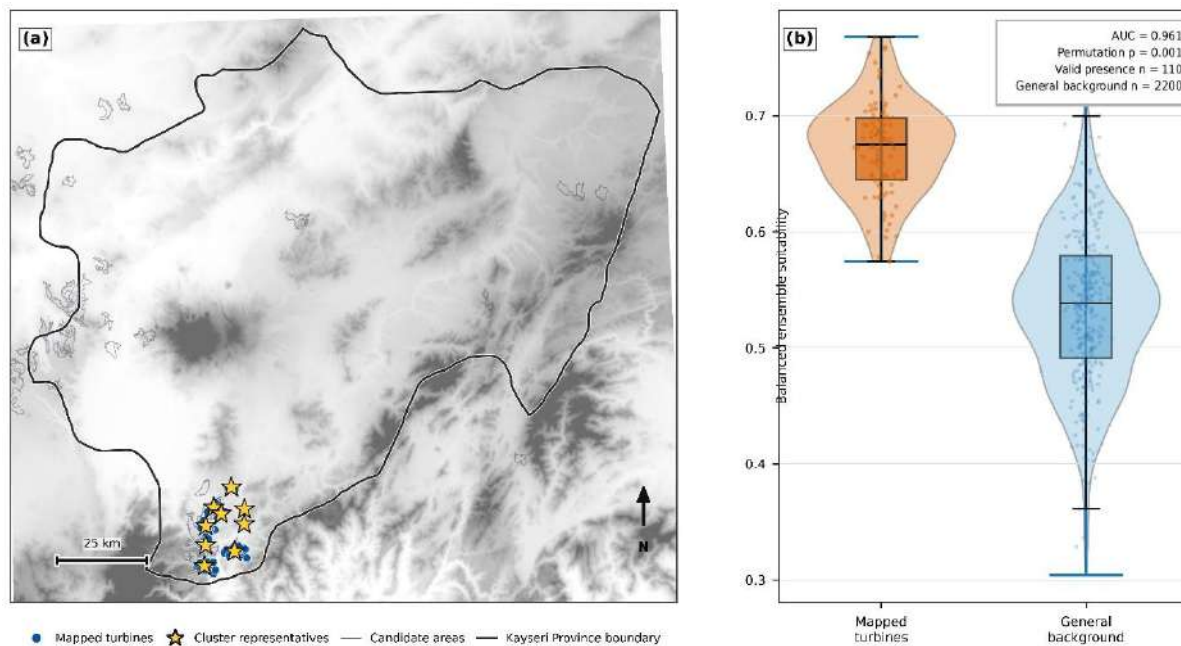


Fig. 6. Mapped-turbine external spatial-consistency diagnostic: (a) mapped-turbine locations, cluster representatives, candidate areas, (b) suitability distributions for mapped-turbine presence and general background locations.

5. Discussion

5.1 Relative Effects of Threshold and Weight Uncertainty

An analytical outcome of this framework is the relative impact of threshold uncertainty compared to criterion weight uncertainty. In the broader GIS MCDA literature, investigations frequently emphasize the refinement of criteria weights as a method to ensure reliability [4,32]. However, the

1000-iteration Monte Carlo simulation showed that within the tested perturbation ranges, variation in planning thresholds produced a substantially larger mean spatial effect than variation in criterion weights. Because the weight and threshold modes were assigned different perturbation ranges and distributional forms, the magnitude of this contrast is conditional on the adopted simulation design and should not be interpreted as a general ordering of threshold and weight uncertainty in GIS-MCDA studies.

This outcome indicates that variations in Boolean planning constraints such as adjusting the setback distance from residential zones or riverbeds can shift substantial areas between eligible and excluded states [30,31]. The methodological ensemble approach accommodates the internal contrast between Equal, CRITIC, and Entropy weights, but analytical effort in regional screening requires explicit justification of the chosen spatial exclusion thresholds, as these exert an influence on spatial availability [9,33].

5.2 Operationalizing Raster Data: From Pixels to Planning Units

A common limitation in regional spatial screening is the reliance on isolated high value raster cells [5]. Isolated high value cells provide limited information as area level planning units because they do not explicitly represent spatially connected candidate geometries. By implementing probability thresholds and connected patch extraction, the model condensed raw raster components into 31 connected candidate areas ($\geq 5 \text{ km}^2$). The 5 km^2 criterion was used as an operational regional-screening threshold to remove small fragmented patches and should not be interpreted as a universal minimum wind-farm size.

Translating raster grids into connected polygons provides discrete planning units for candidate level uncertainty and land cover attributes [34,35]. Their moderate rank correspondence with the baseline aggregate ranking indicates that candidate prioritization may be sensitive to the choice of uncertainty summary. Project-level turbine spacing, rotor diameter, wake effects, installed-capacity density, and micro siting requirements are separate engineering considerations and require site-specific design rather than inference from the regional patch threshold.

5.3 Recontextualizing Land-Cover and Mapped-Turbine Diagnostics

The integration of land-cover information was separated into two candidate-level functions. ESA WorldCover-derived metrics contributed directly to the candidate-prioritization score, whereas the CORINE-derived 11-class LULC raster was used only for physical characterization and screening. Agricultural and pasture-related classes were treated as planning-review signals rather than automatic evidence of legal or project-level suitability. The local 11-class raster was derived from CORINE Land Cover categories [19]. Differences observed between the two land-cover products were retained as a thematic-data limitation because the products use different classification schemes and spatial representations.

The mapped-turbine comparison provides an external spatial-consistency diagnostic within the balanced eligible domain, with a primary presence-background AUC of 0.961. Mapped turbine locations tended to receive higher modelled suitability than the general eligible background, while the lower matched-background AUC (0.890) indicates that part of the contrast reflects wind and slope gradients represented in the suitability model. The diagnostic measures presence-background discrimination rather than predictive accuracy [36]. Background points do not represent true developmental absences, and economic, historical, grid related, and permitting factors may also influence turbine placement [34].

5.4 Limitations and Requirements for Future Investigation

This regional screening framework does not provide project-level engineering clearance. Subsequent project assessment requires locally verified wind-resource information and site-specific resource characterization [37], together with cadastral and parcel verification, ecological assessment, pasture status and any applicable land-use conversion procedures under Pasture Law No. 4342 [38], military and civil aviation constraints including radar and technical-interaction requirements assessed by the competent authorities [39,40], and confirmation of the actual

connection capacity available at the relevant grid node [41]. Spatial proximity to a substation or transmission line does not itself establish grid-connection feasibility.

6. Conclusion

This study developed an uncertainty-aware GIS-MCDA framework for regional onshore wind-energy screening within a Kayseri-centered analytical frame in Türkiye. The analysis combines suitability modelling with scenario-based planning constraints, uncertainty assessment, connected candidate extraction, and candidate-level prioritization. Calculations were performed over a common-valid regional raster frame surrounding Kayseri rather than being clipped to the provincial administrative boundary; consequently, the reported area values refer to the analytical domain rather than the official area of Kayseri Province.

The scenario comparison showed that the extent of eligible land changes materially when planning thresholds are varied. Under the strict assumptions, the suitable area was approximately one quarter of that obtained under the permissive assumptions. Within the perturbation ranges used in this study, threshold-only variation also produced a larger mean spatial effect than weight-only variation. This comparison is conditional on the adopted perturbation ranges and probability distributions and is not intended to establish a universal dominance of threshold uncertainty over weight uncertainty in GIS-MCDA applications.

Probability-based screening and connectivity rules converted the regional raster outputs into 31 connected candidate areas. Candidate prioritization combined exact Monte Carlo robustness, remaining connected area, ESA WorldCover-based land-cover compatibility, baseline suitability, and scenario persistence. The separate CORINE-derived 11-class LULC raster was used for physical characterization rather than ranking, and all candidates retained at least 3 km² of connected area after this screening step.

Mapped OSM turbine locations provided a supplementary spatial-consistency check within the balanced eligible domain. Their modelled suitability was higher than that of the general eligible background, with a primary AUC of 0.961. The lower matched-background comparison indicates that part of this separation is associated with wind and slope gradients already represented in the suitability model. Because background locations are not verified development absences and the presence sample is conditioned on model eligibility, this comparison is not a measure of predictive accuracy or project feasibility.

For regional planning, the main contribution of the framework is the distinction between locations that remain comparatively persistent under repeated scenario perturbations and locations whose eligibility or high-suitability status is more sensitive to modelling assumptions. This information can support the sequencing of subsequent field surveys and institutional screening, but the candidate polygons do not constitute approval decisions or engineering-ready wind-farm sites. Site-specific wind measurement, turbine micro-siting, cadastral verification, ecological assessment, pasture and land-use procedures, aviation and radar review, and verified grid-connection capacity remain necessary before project-level decisions can be made.

Future work may examine alternative uncertainty ranges and candidate-prioritization formulations, incorporate locally measured wind information and temporally updated grid-capacity data, and assess the transferability of the framework to other regions. Extensions to combined wind-solar planning and to scenarios that consider future changes in wind-resource conditions would also allow the regional screening approach to be evaluated under a wider range of planning contexts.

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Strengthening and repairing of RC slabs using partial fiber reinforced cementitious matrix (FRCM) technique

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Abstract

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Effective strengthening and repair techniques are required to extend the service life of reinforced concrete (RC) slabs affected by ageing, increased loads, and flexural deficiencies. Among modern solutions, fiber-reinforced cementitious matrix (FRCM) systems have shown effectiveness. Nevertheless, the experimental knowledge regarding the effectiveness of partial FRCM configurations on two-way RC slabs in terms of load distribution, stiffness and the ability to control crack propagation is still limited. The objective of this study is to experimentally assess the structural behavior of two-way RC slabs reinforced and repaired with different carbon-based FRCM partial layouts. A flexural loading experimental program was performed on four control and retrofitted slabs with various FRCM layouts in the tension zones. The outcomes showed that the FRCM could be applied both in strengthening and repair, which can significantly enhance load capacity, stiffness, cracking behavior and service performance compared to the control slab. FRCM layers are more efficient when directly applied over the maximum moment zone of the slabs. Consequently, this results in higher failure loads, increased stiffness and improved crack control. In general, the results demonstrate that partial application of carbon-based FRCM can be an effective and reliable solution for strengthening two-way RC slabs in the flexure zone.

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1. Background

Repairing and strengthening concrete structures has been considered one of the most important issues in recent decades because these structures deteriorate over time and must conform to the most recent design guidelines [1-3]. The primary objective of using strengthening and repair methods for structural members is to restore the load-bearing capacity of these elements, extend the building's lifespan, and improve its overall structural performance, whether to address existing damage or to meet new requirements, such as changing the building's function or responding to natural disasters [4, 5].

Many structural retrofitting methods, such as the RC layer technique, externally bonded steel plates, and steel jacketing, have been adopted in the construction industry. For example, Aykac, Kalkan [6] investigated the effectiveness of adding a new RC layer to a damaged slab, and they found that the ultimate load does not significantly change with this technique. Sahranavard, Jahangir [1] investigated how externally bonded steel plates may enhance the flexural behavior of two-way reinforced concrete (RC) slabs. They discovered a difference in the influence of these plates on the reaction of these slabs. Aldhafairi, Hassan [7] investigated steel jacketing to improve RC beams at various temperature levels; they discovered that steel plate jacketing had no significant effect on beams at high temperatures. Although the conventional retrofitting methods

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have assisted in the improvement of the capabilities of structural components, many researchers have indicated that these methods have serious issues, such as high temperature resisting, debonding and shear peeling [8, 9]. Thus, to overcome this drawback of these traditional technique's researchers examined the advantages of using polymer fibers as a means of reinforcement and repair for structural elements.

Fiber-reinforced polymers (FRP), as one of the least damaging rehabilitation methods, are widely used in contributing to the strengthening and repair of the building members because of their advantages, including high ductility, low self-weight and corrosion resistance [10, 11]. Several common FRP materials are available, including glass FRP (GFRP), carbon FRP (CFRP), aramid FRP (AFRP), and basalt FRP (BFRP). Many forms of FRP can be used in the construction industry, such as rods, laminates, dry fibers or sheets that are bonded with the concrete by using adhesive materials [12, 13]. These techniques have become more important in the building industry in recent decades. Fang, Xu [14] investigated the effect of using carbon fiber reinforced polymer (CFRP) on the structural behavior of reinforced concrete (RC) slabs. The findings of this investigation revealed that the use of CFRP improves the flexural behavior of the RC slabs. On the other hand, the influence of basalt FRP (BFRP) on the flexural performance of one-way RC slabs had been studied by Erfan, Abd Elnaby [15]. Their results show an increase in the ultimate flexural strength due to the Contribution of the basalt FRP (BFRP) in the flexural behavior. He, Wang [16] proved by their study that the use of basalt FRP (BFRP) on the RC slabs has a good influence on the structural behavior, such as cracking load and flexural stiffness. However, these methods encountered a lot of problems because of the many properties of fiber-reinforced polymers in the rehabilitation of concrete components, such as brittle behavior [17]. FRP has not been proven to outlast traditional materials for long-term service life, especially for structures that are exposed to moisture and humidity [18]. In addition, high temperatures and UV rays have a significant influence on FRP, causing their performance to decline [19, 20]. Due to these disadvantages, Fabric-Reinforced Cementitious Matrix (FRCM) appears, in some cases, more appropriate and viable, in particular in high-temperature environments as well as in more extreme environmental conditions.

Fabric-Reinforced Cementitious Matrix (FRCM) is a novel class of composite materials used for strengthening and repairing concrete structures and existing building elements that exhibit limited tensile strength, heat resistance, and/or resistance to aggressive environmental influences [21-23]. FRCM (or textile-reinforced mortar (TRM)) is an inorganic composite material in which a network of fibers, such as Poly-p-phenylenebenzobisoxazole (PBO), carbon, basalt and glass fibers are embedded within a cement matrix or cement mortar instead of organic resins or plastics [24, 25]. This provides heat, environment, and corrosion resistance advantages for the system, as well as compatibility with the existing concrete structure. Many studies try to study the impact of retrofitting the RC members with the FRCM technique [25-27]. The effect of carbon FRCM on the structural behavior of two-way RC slabs had been investigated by Kadhim, Jawdhari [23]. It can be found from their findings that the use of the FRCM system significantly enhances the ultimate load of the RC slabs. Varma, Sarker [26] considered FRP and FRCM systems at high temperatures that confine reinforced concrete cylinders. The observation suggested that the effectiveness of the FRCM system in maintaining a load-bearing system was significantly greater than that of an equivalent FRP system at elevated temperatures. Recent evaluation between FRCM versus FRP systems available in the construction industry reveals that FRCM provide better bond behavior [11, 28], presents lower dependence of bond behavior from environmental factors [27] and has better durability in temperature fluctuations and provides adhesion properties over time [29, 30].

The above literature review has shown that FRCM has high credibility and application in the rehabilitation and retrofitting of concrete members. However, a handful of studies have investigated the application of FRCM on RC slabs. The current study aims to utilize FRCM for retrofitting reinforced concrete slabs and investigate the variables that affect the performance of slabs enhanced with FRCM.

2. Experimental Program

2.1. Materials

2.1.1 Cementitious Mortar

Cementitious mortar is one of the main elements of concrete members' retrofitting [31]. In this experimental study, the cementitious mortar used to bind the materials is a commercial ready-mix cementitious binder known as Sika MonoTop®-612. The mortar is considered a single-component, polymer-modified, fiber-reinforced cementitious material. The selected mortar is designed to retrofit structural elements and is characterized by high adhesion, high final strength, sulphate resistance, significant resistance to shrinkage, and resistance to environmental factors. Sika MonoTop®-612 follows the main requirements of EN 1504-3, R3 classification and its properties are highlighted in Table 1. Although precise material-level tests (e.g., tensile, flexural, and bond compatibility) were not conducted in this macroscopic study, the manufacturer's certified parameters were adopted as a reliable baseline for structural verification.

Table 1. Properties of Sika MonoTop®-612

Property	Description
Composition	Polymer-containing cementitious mortar with different additives and synthetic fiber.
Appearance / Colour	Grey powder
Maximum grain size	D _{max} : 2.0 mm
Compressive strength	28 days (ASTM C109) ~40 MPa *Testing cubes 50 x 50 x 50 mm
Mixing ratio	3 - 3.25 L of water per 25 kg bag
Fresh mortar density	Fresh mortar density 1.95 kg/L at +20 °C
Layer thickness	Min. 5 mm, Max. 30 mm

2.1.2 Fabric-Reinforced Cementitious Matrix (FRCM) Textile

Many researchers have used FRCM to support or retrofit reinforced concrete elements such as beams, columns, and slabs [32, 33]. The textile comes in a wide range of fibers and configurations [23, 25]. The current study utilized a well-known type of textile, carbon-based Fabric-Reinforced Cementitious Matrix (Carbon-textile) system, as an external reinforcement. The characteristics of the selected textile grid include 20 mm spacing and 0.047 mm equivalent thickness. The carbon textiles' modulus of elasticity and tensile strength were 230 GPa and 3600 MPa, respectively. The carbon characteristics of the textile have been presented in Fig.1 and Table 2.

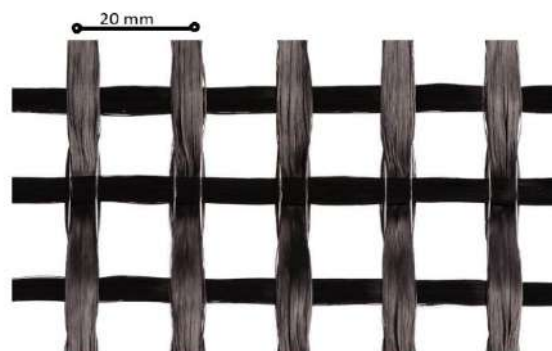


Fig. 1. Selected Carbon-textile

Table 2. Characteristics of FRCM textile based on the manufacturer's data sheet

Material	Nominal thickness	Tensile strength	Modulus of elasticity
Carbon	0.047 mm	3600 MPa	230 GPa

2.1.3 Reinforced Concrete

In the current study, the concrete slabs were cast simultaneously using ready-mix concrete. Compression testing was carried out using a calibrated testing machine, and the load was applied axially with a constant loading rate until failure occurred. The overall average compressive strength of the specimens was 34.8 MPa. The test procedure followed BS 1881: Part 116 [34].

In this experimental program, 6 mm deformed steel bars were used. The steel was tested in accordance with the requirements of the American Standard Specification for Carbon Steel Rebars for Use in Reinforced Concrete (ASTM A615/A615M-20 [35]). The diameter was measured using a micrometer, and tensile testing was performed to determine yield strength, ultimate tensile strength, and relative elongation at break. During the tensile tests, the reinforcement exhibited a well-defined linear elastic phase, followed by a distinct yield plateau and a subsequent strain-hardening region prior to rupture. The steel used is Grade 40, a type commonly used in light—and medium-duty structural applications. Table 3 shows the detailed results of the mechanical tests and the physical properties of steel bars.

Table 3. Properties of reinforcement steel.

Property	Values
Nominal diameter (mm)	6
Actual diameter (mm)	5.8
Cross-sectional area (mm ²)	26.4
Yield stress (MPa)	405.3
Ultimate stress (MPa)	605
Elongation (%)	17
Nominal weight (kg/m)	0.208
Grade	G40

2.2 Test Specimens Details

2.2.1 Specimens Details and Parameters

The main aim of this experimental study is to investigate the effectiveness of using carbon-FRCM as external reinforcement on the flexural behavior of two-way RC slabs. Four laboratory-scale RC slabs with identical geometry were constructed and tested as simply supported at their perimeter (Fig. 2). As indicated in Fig. 2 (a), the slabs were of a square section with a 1000 mm side and a thickness of 80 mm. Also, the effective flexural span was 900 mm (Fig. 2(b)).

To simulate the flexure-deficient, all the samples were lightly reinforced with a reinforcement ratio of ($\rho=0.28\%$). Fig. 2(C) shows the details of the steel reinforcement for the samples. The reinforcing steel utilized in the present study was deformed steel bars with a 6 mm diameter and a spacing of 192 mm at the bottom in two directions of the slabs. All reinforcement bars have been bent at the supports by 180 degrees to guarantee proper anchorage. Also, 6 mm bars were added at distances of 100 mm from the support in the top reinforcement to control surface cracks and improve the anchoring. This was applied to all support in the four edges of the slab. To reduce cracks resulting from twisting moments, additional reinforcement was used at the corners as shown in Fig. 2(c).

To achieve the goal of this research, two parameters, the configuration of FRCM and the existence of pre-damage, were considered. Fig. 3 and Table 4 show the configurations of the FRCM and the sample details. Four samples were designed to complete the experimental programs as follows:

- R represents the control specimen tested without strengthening (Fig. 3-a).
- C250 is a sample strengthened with two carbon textile strips in a cross configuration. Every strip was 45 mm wide, 50% of the slab's effective span in each direction (Fig. 3-b).
- C50-3 is a sample strengthened with six strips of carbon textile, three of which were in a cross configuration. Each strip had a width of 15mm, representing 50% of the slab's effective span in each direction (Fig. 3-c).

CD250 is a sample strengthened with two strips of carbon textile in a cross configuration. Every strip was 45 mm wide, 50% of the slab's effective span in each direction (Fig. 3-d). This specimen is strengthened after being pre-damaged.

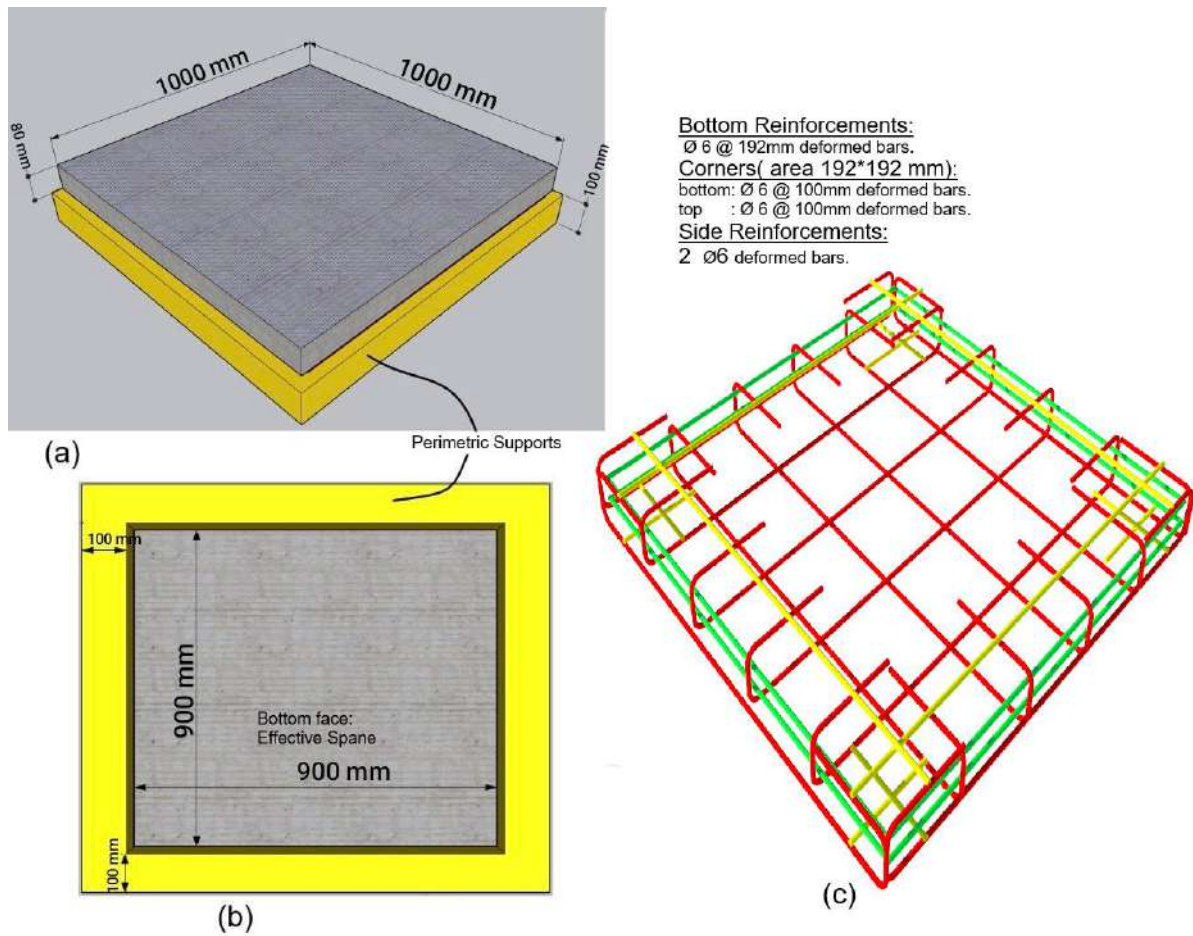


Fig. 2. (a) The slab's geometry; (b) the slab's perimeter-supported bottom face; and (c) the slab's steel reinforcement details

Table 4. Samples, experimental parameters, and materials

Samples	Strengthening configuration	Concrete compressive strength F_c (MPa)	mortar compressive strength F_{mc} (MPa)	steel reinforcement ratio ρ_s (per direction)	Textile reinforcement ratio ρ_t (per direction)
R		34.26		0.29%	
C250	Two carbon textile strips in a cross configuration	34.26	40	0.29%	50%
C50-3	Six carbon-textile strips, including three arranged in a cross pattern	34.26	40	0.29%	50%
CD250	Two carbon textile strips in a cross configuration	34.26	40	0.29%	50%

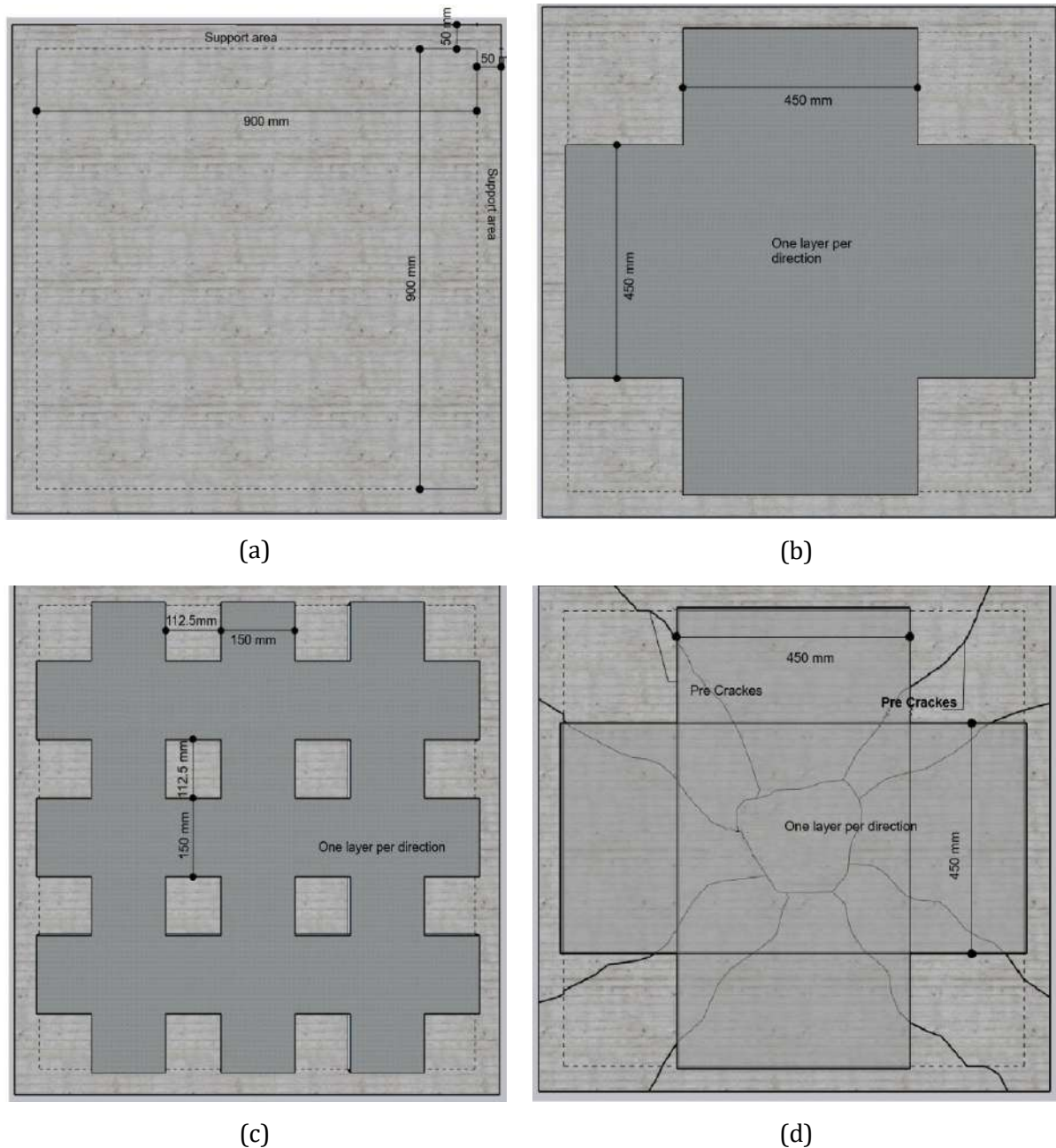


Fig. 3. Configuration of FRCM strengthening on the tested samples/ (a) Control slab R, (b) C250, (c) C50_3, and (d) CD250

2.2.2 Strengthening Procedure

In this experimental program, FRCM was adopted to retrofit the concrete slab models due to its height, tensile strength, and modulus of elasticity [23, 36]. The procedure for the application of FRCM carbon textile on the samples has been indicated in the following steps:

- Preparing concrete model surfaces by mechanical cleaning and removing dust and loose materials.
- A grid of grooves, 2mm in depth, was created to achieve good adhesion between the mortar layer and the concrete surface (Fig. 4-a).
- Moisten the concrete surface to be strengthened (Fig. 4-a).
- The first layer of cementitious mortar was applied with a thickness of 3mm at the sample's tension face on the required portion of the concrete surface according to the intended strengthening ratio (Fig. 4-b).

- The first layer of the carbon textile was applied onto the fresh cementitious mortar and gently pressed by hand to achieve full contact and adequate bonding between the fabric and the matrix (Fig. 4-b).
- The second layer of cementitious mortar was applied on the first layer of the carbon textile (Fig. 4-c).
- A new layer of the carbon textile was applied perpendicular to the previous carbon textile layer (Fig. 4-c).
- The final layer of cementitious mortar is applied, followed by careful finishing and smoothing of the surface (Fig. 4-d).

All mortar and textile layers must be applied while the mortar is still fresh.

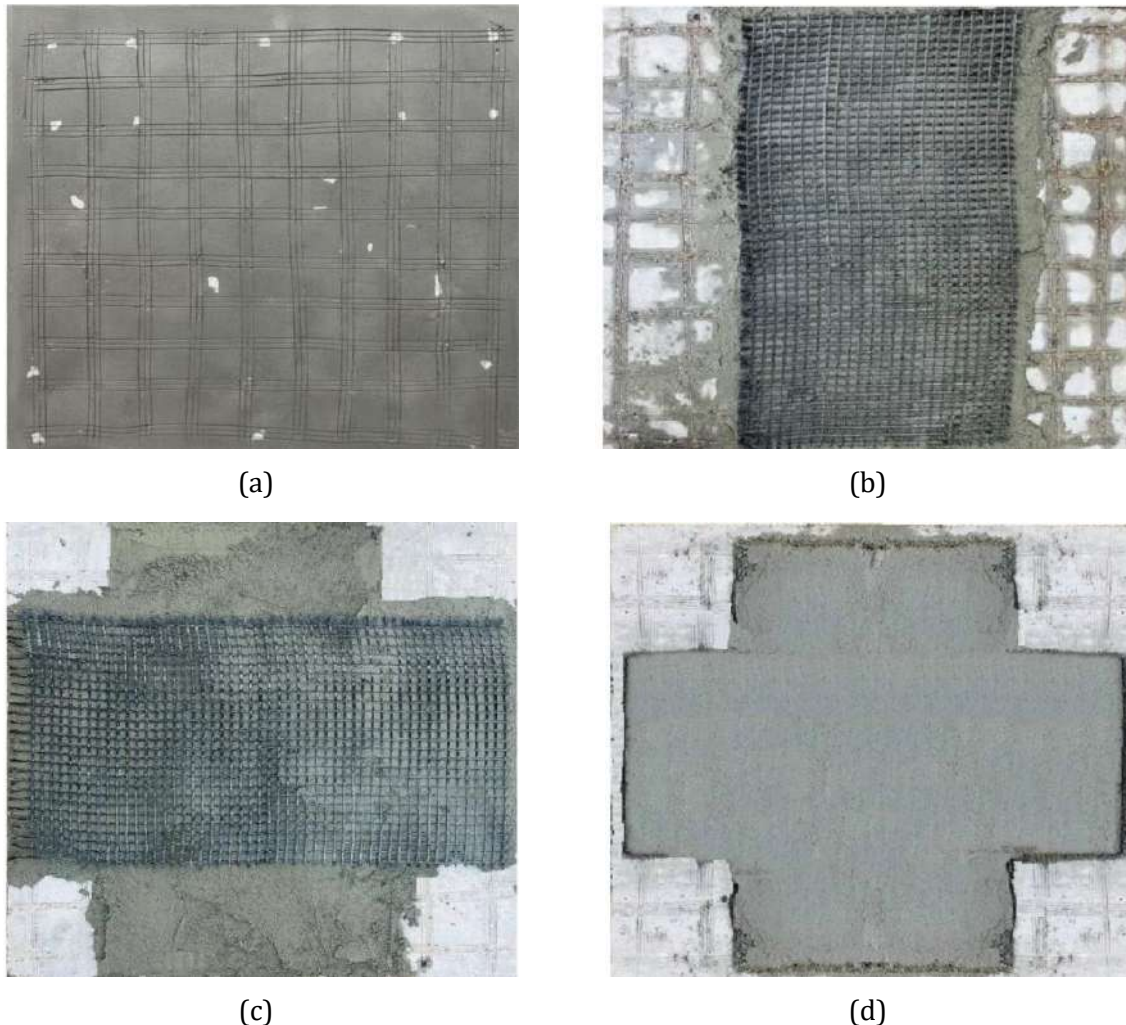


Fig. 4. Application steps for FRCM reinforcement: (a) Preparing and moistening the concrete surface for strengthening; (b) Application of the first mortar layer together with the carbon textile; (c) Application of the second mortar layer along with the carbon

2.3 Testing Set Up and Procedure

The test was performed with monotonic loading, and the entire perimeter was simply supported, as shown in Fig. 5. Four rigid steel beams were utilized as supports, which were welded with a steel bar of diameter 25mm, then a 10 cm rubber band was fixed on the top of the steel bar to ensure proper loading distribution. As illustrated in Fig. 5, the effective flexural span was 900 mm in each direction. A system of three rigid steel beams was used to distribute the one-point load into a four-point load, as indicated in Fig. 5 (a), leading to proper load distribution. At the locations of load application, square rubber pads (100 x 100 x 10 mm) and square steel plates (100 x 100 x 12 mm)

were used between the upper face of the slab and the rigid steel beams to reduce local stresses. The center-to-center distance between the four-point load was 400 mm, as indicated in Fig. 5.

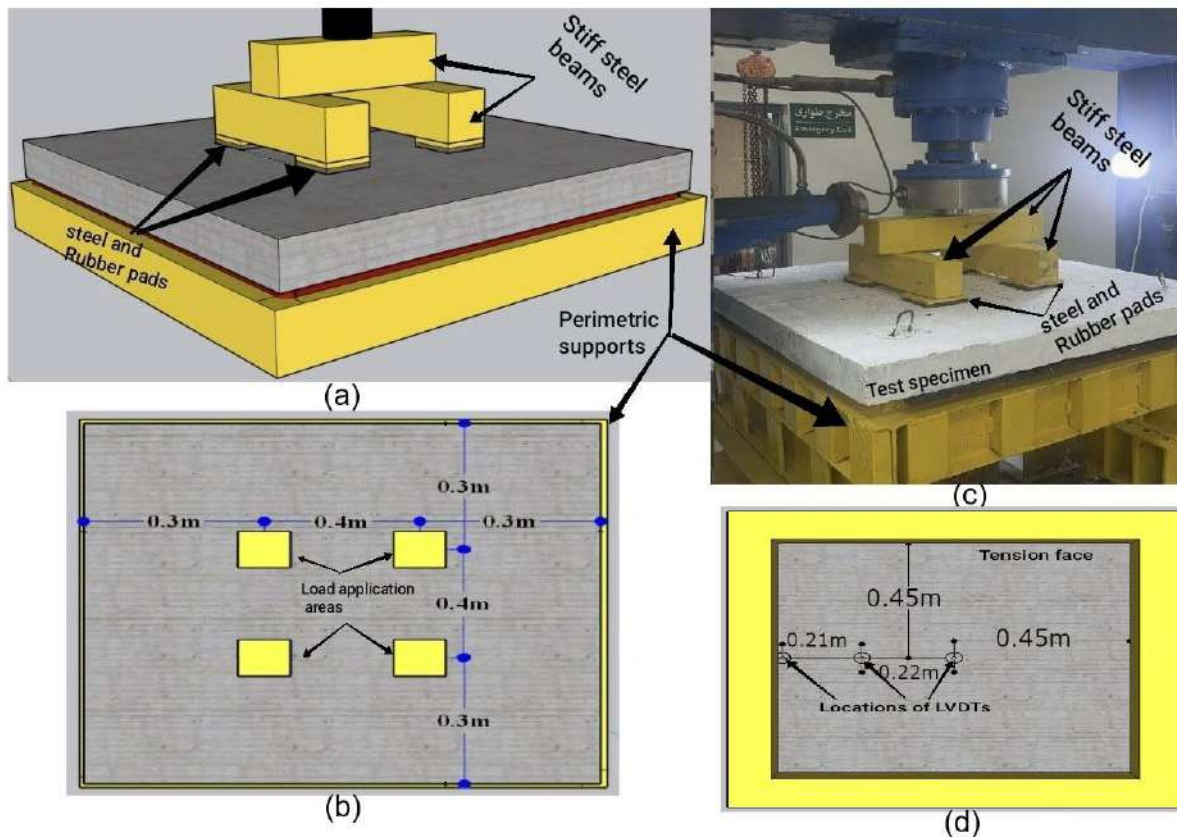


Fig. 5. Experimental setup (a) 3D schematic view; (b) top view; (c) the specimen positioning and the applied loading scheme; (d) Layout of LVDTs installed for deflection measurements.

The sample's deflection was observed by utilizing three linear variable differential transformers (LVDT), as indicated in Fig. 5 (d). Three LVDTs were installed on the slab to measure its deflection. The first one is placed at the center to record the central deflection, the second one is located in the middle between the center and the support to catch the intermediate deflection, and the third is near the support to measure the deflection at this point. All the samples have been loaded up to failure except CD250. First, it was loaded with 50% of the ultimate load, followed by the strengthening that is applied, and then, CD250 is tested up to failure.

3. Experimental Findings

The results of all tested slabs were indicated in Fig. 6 as curves illustrating the relationship between the loads and the deflection at the center of the slabs. Fig. 7 indicates the ultimate crack pattern for each sample. The main experimental results, including the ultimate load P_{max} , the ultimate deflection and the improvement in the flexural strength due to the retrofitting process, have been indicated in Table 5. To provide a precise mathematical definition for the stiffness derivation, the initial pre-cracking stiffness (K_{cr}) was calculated using linear regression as the average secant slope of the load-midspan deflection curve for the uncracked stage ($K = \Delta P / \Delta \delta$, from the origin up to the cracking load). The cracking load was estimated as the load level where the first crack was noticed during the test and where the slope of the load-deflection curve starts to change. Subsequently, the post-cracking stiffness (K_{post}) was mathematically derived as the average linear slope of the data points extracted from the cracking point up to the yielding stage. The load at the short-term serviceability limit state (SLS) was determined following the recommendations of Eurocode 2—Part 1 (CEN 2004)[37]. According to this standard, the SLS is reached when the midspan deflection equals $L/250$, where L represents the effective span of the slab. In the present study, this corresponds to a midspan deflection of about 3.6 mm. All these parameters, including this property and the ones mentioned earlier, are summarized in Table 5.

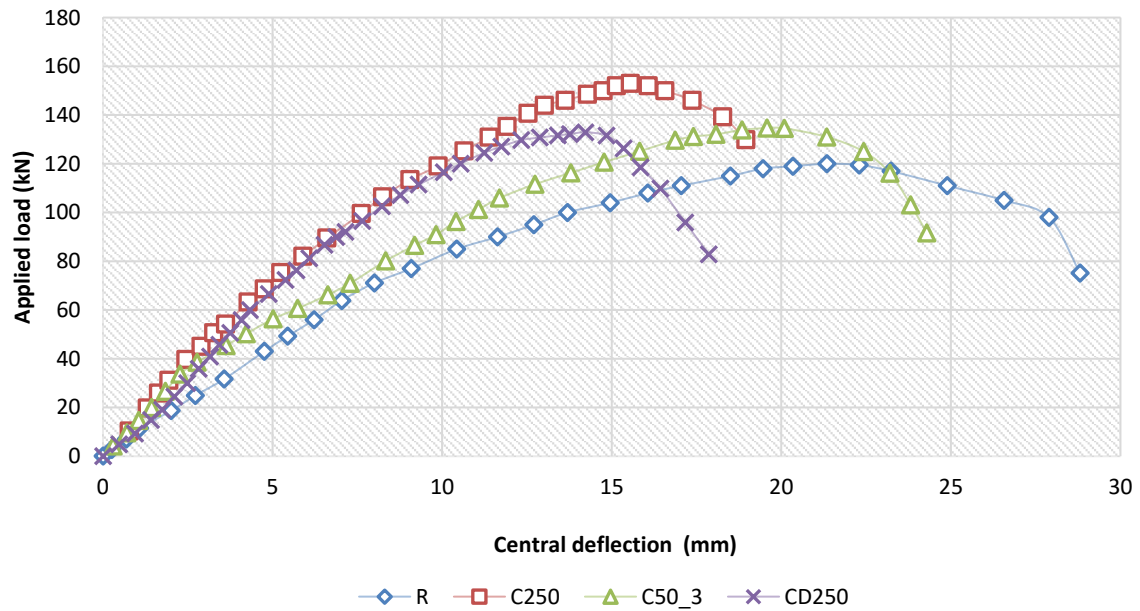


Fig. 6. Applied load versus central deflection curves for all specimens

As it was designed, the control slab (R) failed mainly in flexure after the steel bars started to yield and some large plastic deformations appeared. The first flexural cracks were seen at a load of about 15 kN, which caused a clear drop in stiffness, as can be noticed from the slope change in Fig. 6. After that, the steel reinforcement continued to yield and more cracks developed (Fig. 7(a)), together with larger deflections. Finally, the control slab reached its maximum load of around 120 kN, where many cracks appeared at the bottom face of the slab as shown in Fig. 7(a).

Sample C250, which was strengthened with two carbon textile strips in a cross configuration. Every strip was 45 mm wide, 50% of the slab's effective span in each direction. This slab showed a flexure-dominated failure mode, the same as the control slab (R), but resisted a significantly higher ultimate load of 153 kN due to the additional flexural strength provided by the FRCM layers. Additionally, it exhibits a better pre-cracking response compared to the control specimen, with a 60% increase in initial stiffness, as indicated in Table 5. Furthermore, a significant increase in the cracking load, which reached 40 kN, is illustrated in the slope of the load-deflection curve in Fig. 6. The instantaneous tensile activation of carbon fibers across flexural fractures led to a significant enhancement in post-cracking flexural stiffness, achieving 3 kN/mm, in contrast to 1.13 kN/mm for the control slab. The crack pattern of slab C250, as shown in Fig. 7(b), consisted of a few main cracks, along with some smaller ones that spread across the FRCM surface. The failure occurred gradually, primarily because some of the fibers began to slip and partially break within the mortar along the main cracks. While advanced instrumentation, such as distributed strain measurements, was not utilized to quantify the micro-mechanics of crack bridging, the progressive activation of the carbon fibers and the ultimate failure mechanisms (e.g., fiber slip and partial rupture) were phenomenologically evaluated through continuous visual monitoring of the crack development and the global load-deflection responses. Furthermore, while this qualitative assessment of crack propagation does not provide a numerical estimation of fracture energy or progressive damage modeling, it offers essential physical evidence regarding the effectiveness of the carbon textile in distributing stresses and altering the failure mechanism.

Slab C50-3, which had the same amount of textile reinforcement as slab C250 but arranged in a slightly different way (Fig. 7(c)), failed under an ultimate load of 135 kN in flexure. Its initial stiffness was even higher than specimen R, reaching roughly 16.6 kN/mm. The first visible cracks formed when the load got to around 35 kN, and the post-cracking stiffness of this slab was about 2.36 kN/mm, which is noticeably lower than that of C250 (3 kN/mm) but higher than the control slab R (1.13 kN/mm). This indicates that the material performed well compared to R; however, at this stage, placing the FRCM in the regions of maximum moments would be more effective than distributing it in strips, as was done in this specimen.

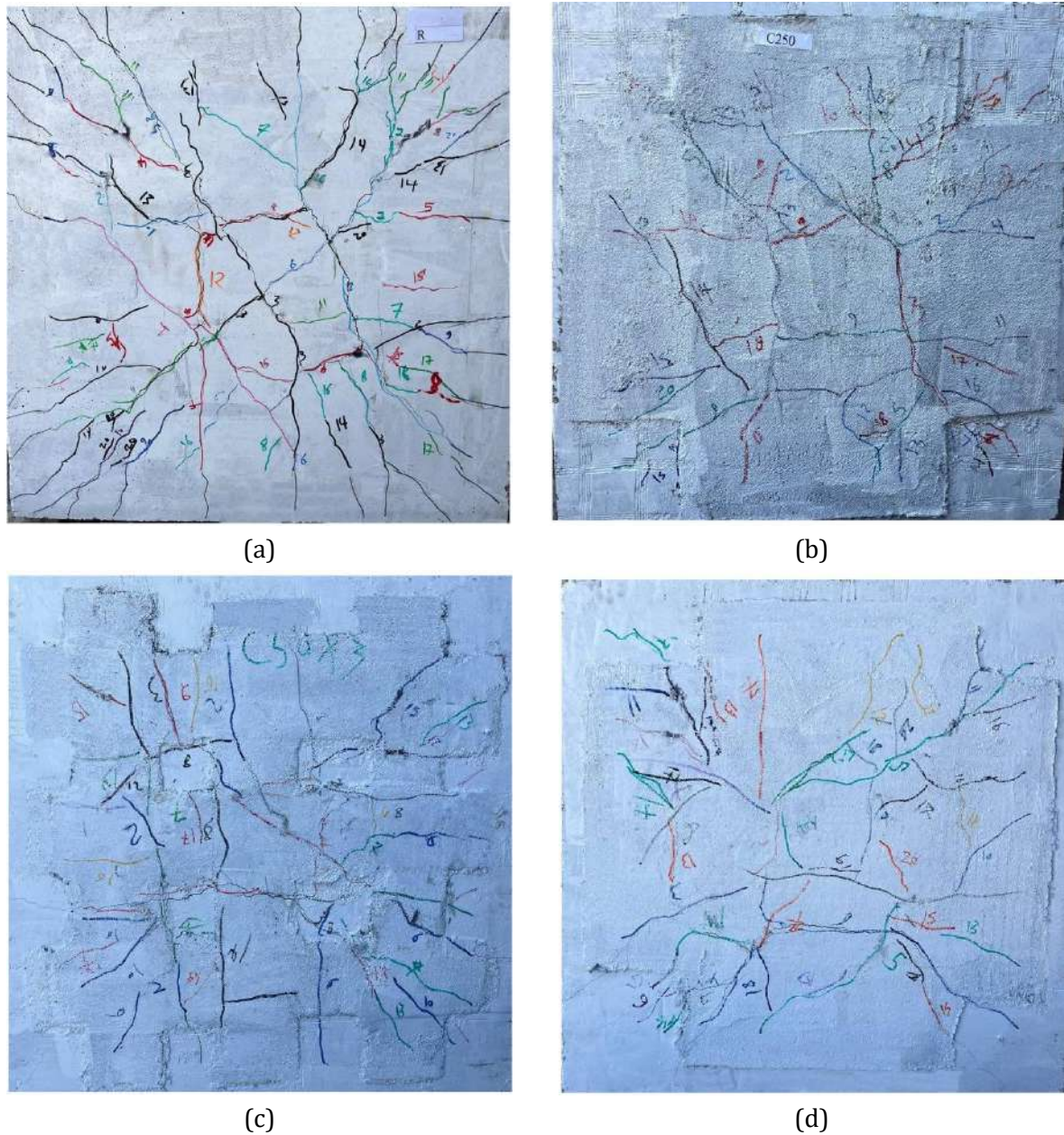


Fig. 7. Comparison of experimental crack propagation and failure modes for the tested specimens: (a) Control slab R; (b) Strengthened slab C250; (c) Strengthened slab C50_3; (d) pre-damaged strengthened slab CD250

Table 5. Overview of the test results

Sample	Maximum load P _{max} (kN)	Midspan deflection at P _{max} (mm)	Improvement in capacity (%)	Pre-cracking stiffness ^a (kN/mm)	Cracking load level ^b (kN)	Post-cracking stiffness ^a (kN/mm)	Load at services (kN)
R	120.0	21.3	-----	10.6	15	1.13	31
C250	153.0	15.5	27.5	17.1(60%)	40	3 (165%)	55(77%)
C50_3	134.5	19.5	12.5	16.6(56%)	35	2.3(108%)	45(45%)
CD250	132.9	14.2	10.0	-----	-----	1.60(41%)	

^a For the retrofitted specimens, the percentage increase with respect to the R specimen is provided in parentheses.

^b Based on (a) the load-displacement curve's slope change and (b) the test results.

The specimen finally failed in flexure after the steel bars yielded and the textile fibers started slipping inside the mortar, with a maximum failure load of 134.5 kN. Specimen CD250, which was pre-damaged and then strengthened with the same strengthening as specimen C250 before the test, failed in flexure under a load of about 132.9 kN. This specimen showed a lower initial stiffness compared with the other samples, mainly because of the pre-cracking of concrete as indicated in Fig. 7. At a load of 50 kN, the FRCM reinforcing layers start contributing to resisting the tension stresses, where the stiffness increases significantly.

4. Discussion and Evaluation of the Findings

The strengthening methods employed in this study successfully provided a noticeable improvement in the flexural strength of the lightly-reinforced concrete two-way slabs. From an analytical perspective, the carbon textile enhances the flexural capacity by contributing to the internal tensile force equilibrium after concrete cracking, thereby delaying the yielding of the internal steel. However, evaluating this capacity theoretically requires complex sectional equilibrium analysis combined with spatial failure mechanisms. Predicting the exact ultimate load is highly challenging due to the progressive nature of fiber-matrix debonding and the discrete distribution of the strengthening strips. Consequently, while developing a rigorous analytical and numerical model to validate these capacities is a primary objective for our future ongoing research, the current study intentionally focuses on establishing a solid physical baseline through experimental macroscopic evaluation. Considering the different variables examined, a review of the experimental results presented in Table 5 including the ultimate strength, cracking load, pre-cracking stiffness, post-cracking stiffness, and the response at the serviceability limit state (SLS) points out several key observations and outcomes that are discussed below.

4.1 Effect of Fiber-Reinforced Cementitious Mortar Layers

In general, the FRCM layer achieves a significant increase in flexural strength, which is consistent with the findings of several studies, which state that the textile-reinforced systems provide additional tensile resistance to concrete elements. This improvement is mostly due to the fact that the carbon fibers start working early on during loading. This makes it easier for tensile stresses to spread evenly over the concrete–mortar–textile composite layers.

In addition to the higher ultimate load, the strengthened slabs also exhibited a higher initial stiffness just before cracking, indicating that a more stable response and a proper interaction bond between the slab and the FRCM were functioning effectively. Even after the cracking initiated, the slabs still exhibited higher degrees of post-cracking stiffness, largely due to the carbon fibers maintaining the cracks and delaying their opening. This behavior is very similar to what other researchers mentioned before, where FRCM systems usually help with crack control and keeping the stiffness from dropping too much after cracking [38].

Also, the service-load behavior improved in all strengthened slabs, not just in terms of strength but also in how the slabs behaved under normal loading levels, which is really important for long-term use. Furthermore, regarding long-term environmental durability, the inorganic nature of the cementitious matrix offers inherent advantages over traditional resin-based FRP systems. It provides excellent thermal compatibility with the concrete substrate, superior resistance to ultraviolet (UV) radiation, and allows moisture breathability, thereby reducing the risk of premature delamination and ensuring sustained structural performance over time. These trends can be seen in the load–deflection curves in Fig. 6, and the values listed in Table 5 also support the idea that the FRCM strengthening method had a positive effect on the performance of the two-way slabs used in this study.

4.2 Configuration FRCM Layer

An insight into the effect of the strengthening configuration is provided in specimens C250 and C50_3, whose test results are retrievable in the summary format of Table 5 and also in the plots of Fig. 3. Despite using the same type of carbon FRCM and having the same reinforcement ratio (two orthogonal textile layers covering nearly 50 per cent of the tension face), the arrangement of these layers was completely different. Specimen C250 was laid with each layer in a single sheet and

placed at the center of the span where the maximum bending moment occurs. That meant that the majority of the textile was focused directly where the slab required its maximum tensile reinforcement. For C50_3, on the other hand, each of the layers was separated into three strips of 15 cm in width spaced by roughly 11.25 cm. In this configuration, the textile can be spread over a larger volume, and its fibers may interact progressively, according to the formation and propagation of cracks. Fig. 3 clearly shows the layouts of both configurations and may help explain why the reinforcement in C250 seems to have been activated more directly and efficiently.

These differences in configurations had an obvious impact on the mechanical behavior. Specimen C250 exhibited a higher ultimate load (153 kN) than C50_3 (135 kN), indicating the superior flexural efficiency of placing the reinforcement directly in the maximum moment area (Table 5). This performance is fundamentally linked to the micro-mechanical interaction between the fiber and the matrix. In C250, the continuous mesh configuration provided a larger effective surface area for bond stress distribution, which enhanced the mechanical interlock (the "rivet effect") where the mortar effectively penetrated the grid openings. This mechanism delayed the initiation of fiber-matrix debonding and resisted its progression across the span. In contrast, the discontinuous strips in C50_3 created localized stress concentrations at the edges of each strip, leading to an earlier onset of debonding and less efficient crack bridging. This is reflected in the cracking load, which rose significantly from 15 kN in the control slab to 40 kN in C250, compared to 35 kN in C50_3. Furthermore, C250 achieved nearly double the post-cracking stiffness (3 kN/mm) compared to C50_3 (2.36 kN/mm), as the dense, centralized grid was more effective in arresting micro-crack propagation within the cementitious matrix. The behavior remained consistent for the serviceability limit state (SLS) resistance, with C250 reaching 55 kN while C50_3 reached 45 kN. Although both configurations outperformed the control specimen, the concentration of FRCM in the maximum moment region enabled fibers to bridge cracks more effectively. The distributed-strip configuration (C50_3) enhanced performance but did so by only partially engaging the fibers, as some strips were slightly off the major cracking path, further facilitating the debonding progression compared to the continuous patch in C250.

4.3 Effectiveness of FRCM Strengthening on Pre-Cracked Slabs

The effectiveness of FRCM strengthening after initial cracking was evaluated by comparing specimen CD250, which was subjected to 50% of the ultimate load of the control slab before strengthening, with the directly strengthened specimen C250. Both specimens utilized the same carbon FRCM with two orthogonal layers covering 50% of the tension face, and their layout is illustrated in Fig. 3. In CD250, flexural cracks had already developed prior to the application of FRCM, whereas C250 was strengthened without any prior damage. The pre-existing cracks in CD250 were flexural in nature. Despite this initial damage, the slab was able to achieve a final load capacity of 132.9 kN, which is significantly higher than the control slab (120 kN) but still lower than C250 (153 kN). CD250 achieved a post-cracking stiffness of 1.6 kN/mm, which is only 41% larger than for the control slab (1.13 kN/mm) but 68% less than for C250 (3 kN/mm). As summarized in Table 5 and plotted in Fig. 6, these results demonstrate that FRCM can regain some of the lost flexural capacity and stiffness. However, this is depreciated when compared with undamaged slabs that were subsequently strengthened.

Despite these small differences, CD250 performed similarly to C250 in failure mode (no or minimally debonding and slipped fiber), and there were no significant issues (dis-) activating FRCM during loading. While the extremely slight, probably negligible, effect of pre-existing cracks slightly impaired textile layer engagement (lowering the post-cracking stiffness in them compared to a fully intact slab) and contributed slightly to the detection of cracks, it could not prevent effective crack bridging and net flexural reinforcement. Relative to the control slab, CD250 showed a significant increase in ultimate capacity and post-cracking behavior, demonstrating that FRCM can even be effective if the damage is applied first. In conclusion, FRCM has proven to be efficient in increasing flexural performance compared to direct strengthening (C250), providing additional (up to 27%) structural compensation and controlling the crack texture even when bonded after moderate pre-damage. The differences are highlighted by the position of reinforcement with

respect to the region of maximum moment (Fig. 6) and the need for optimized behavior between different structural systems.

Despite the slight differences, the failure mode of CD250 remained similar to C250, with no significant issues in fiber debonding or slippage, although minor differences in FRCM activation during loading were observed. The presence of pre-existing cracks slightly affected the engagement of the textile layers, reducing the post-cracking stiffness compared to the fully intact slab, but did not prevent effective crack bridging and flexural reinforcement. Compared with the control slab, CD250 exhibited a substantial enhancement in both ultimate capacity and post-cracking response, confirming that FRCM can be effective even when applied after initial cracking. Overall, the comparison highlights that while direct strengthening (C250) yields the highest flexural performance, applying FRCM after moderate pre-damage still provides meaningful structural improvements, particularly in load-bearing capacity and crack control. The observed differences underline the importance of reinforcement placement relative to the maximum moment region, as shown in Fig. 6, in achieving optimal structural performance.

5. Conclusion

The present research paper provides experimental insights into the effectiveness of the carbon-roving FRCM system in enhancing the behavior of two-way RC slabs, when applied to strengthen undamaged members and to repair pre-damaged ones. The research was focused on different performance parameters, including ultimate flexural strength, first-crack load, pre-crack and post-crack stiffness development, crack pattern and propagation, and service load behavior of the slabs. Using these criteria, the key findings from the experimental program can be summarized as follows:

- The FRCM was efficient in retrofitting RC slabs, as all retrofitted slabs showed a significant difference from the control slab in various scales.
- The distribution of FRCM layers under the loaded areas should be placed just above the maximum moment zone rather than distributing them as separate strips. The slabs with such layers presented significant improvements in terms of the failure load, stiffness, and crack control.
- FRCM usage demonstrates high effectiveness in increasing the first cracking loading, resulting in more elastic and stable behavior.
- An overall enhancement in structural stiffness was observed in both the pre-cracking and post-cracking stages, particularly in the specimens strengthened with continuous textile reinforcement. This indicates a more effective engagement of the textile in resisting deformation throughout the entire loading process.
- More effective crack control is achieved with the reinforcement in the critical zone. The strip layout still helped, but somewhat imperfectly because the strips are not contiguous.
- Strengthened slabs have a higher load-to-allowable deflection ratio, which enhances the serviceability behavior of the slabs.
- The outcome showed not only that the amount of surface covered is significant, but also the design and configuration of the fibers.

Based on the above, future studies are recommended to evaluate different coverage ratios, better spacing for strip layouts, other types of textile materials, and also the long-term behavior of FRCM under repeated loads and different environmental conditions. These aspects could help in developing more reliable and efficient strengthening strategies for using FRCM in real repair and retrofitting projects.

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Shear behavior and failure mechanisms of steel fiber reinforced concrete beams incorporating silica fume: Experimental investigation with SEM-EDX based microstructural influence on structural response

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Abstract

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Shear failure in reinforced concrete beams is characterized by a sudden and brittle response posing significant risks to structural safety particularly in high-performance cementitious matrices. Although silica fume improves strength and durability it often reduces crack limit. This study addresses gap by establishing a direct equivalence between material-level interpretation and beam-scale shear behavior using concrete mixes containing 5% silica fume and steel fibers at volume fractions (V_f) of 0%, 0.75%, 1.0% and 1.25% were prepared among them 1.0% identified as optimum mix based on that an experimental program comprising cubes, cylinders and beam testing complemented by SEM-EDX microstructural analysis was conducted to investigate fiber-matrix interaction. Beam specimens were cast and tested under 4-point loading with a (a/d) ratio of 1.9 adapted adopted transverse reinforcement to induce shear-critical behavior. Results indicate that RCSF0% fibers showed diagonal tension cracking leading to brittle shear failure within shear span. whereas, the RSCF1.0% mix demonstrated enhanced load capacity improved deformation resulting in a distinct shift from shear-dominated to flexure-controlled behavior. The findings further reveal that even with relatively larger stirrup spacing, steel fibers effectively bridged cracks at the microstructural level, enhancing shear resistance and improving the ductility and stability of the shear-critical structural members.

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1. Introduction

Reinforced concrete beams are critical components in structural systems where shear resistance governs both safety and serviceability under increasing load demands unlike flexural behavior shear response is characterized by the rapid progression of inclined cracking & limited capacity of a shear redistribution once initiated shear cracking may propagate suddenly leading to abrupt failure with minimal warnings this behavior is particularly critical and structural number subjected to high shear to moment ratios where failure is governed by diagonal tension rather than material

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strength alone. Consequently improving shear performance remains a central challenge in reinforced concrete designs [1]. Recent seismic events like Kahramanmaraş 2023 earthquake have magnitude of 7.8 highlighted the vulnerability of (RC) members to brittle shear failures, inadequate shear capacity and poor detailing resulted in several structural damage and collapse [2, 20]. The adoption of high-performance concrete has further intensified this challenge while such concrete provides enhanced compressive strength and durability, they often exhibit reduced deformation capacity under limited crack accommodation increased matrix stiffness accelerates crack localization especially under shear dominant stress stages. As a result, high performance does not necessarily translate into improved structural stability and in certain cases may increase susceptibility to brittle shear failure. This disconnects between strength enhancement and damage tolerance necessitates additional mechanisms to control cracking and sustain load transfer beyond matrix fracture. Silica fume is widely employed to improve concrete performance through pozzolanic reaction and microstructural densification the resulting refinement of pore structure enhances compressive strength and reduced permeability however this densification restricts crack bridging and eliminates post crack deformation experimental observation indicates that silica fume modified concrete exhibits a reduced crack tolerance once tensile stresses exceed matrix capacity under loading. The absence of intrinsic crack arrest mechanism leads to rapid stiffness degradation and unstable crack propagation under loading [14, 16]. Steel fibers provide a complementary mechanism by introducing distributed tensile resistance within the concrete matrix after cracking fibers bridge discontinuities transfer stresses across crack planes and openings. In shear critical cases this action contributes to the preservation of aggregate interlocking and enhancements of double actions. For longitudinal reinforcement rather than acting as a conventional reinforcement fiber modifies the crack process itself delays the crack localization and altering post cracking stiffness degradation the resulting response reflects a transition from brittle failure to ductile behavior. Furthermore, fiber has been shown to enhance seismic performance by improving confinement in structural members. The effectiveness of this mechanism depends on the fiber geometry volume of fraction dispersion and interaction with the surrounding cementitious matrix of the concrete. Transverse reinforcement is required in concrete beams if shear demand is high and diagonal cracking cannot be resisted by concrete alone especially in beams with low shear span-to-depth ratios where brittle failure occurs [3, 6]. However, studies have shown that addition of fibers modifies the shear mechanism by enhancing aggregate interlocking and dowel action leading to more stable crack evolution compared to conventional beams [8, 15, 17]. This behavior has been further supported by theoretical models indicating fiber-induced tensile stress transfer contributes to an increase in shear capacity. Post-earthquake investigations consistently indicate that brittle shear failure and insufficient ductility are primary causes of structural damage in reinforced concrete members subjected to seismic loading. Field-based studies following 2023 Kahramanmaraş earthquake report that inadequate shear reinforcement and poor detailing [2, 10] led to rapid diagonal cracking at a degree range from 30-45° even compressive strength requirements were satisfied [13]. Fiber-reinforced concrete exhibit improved crack control and confinement, thereby enhancing seismic performance and delaying failure under cyclic loading conditions. Comparisons reveal that conventional concrete is highly susceptible to shear failure whereas fiber-modified concrete demonstrates improved deformation and structural capacity.

Microstructural investigations further support distinction by explaining the mechanisms governing crack investigation. SEM-based studies reveal that improved fiber-matrix formation enables stress transfer across interfaces through bridging action, thereby reducing crack growth. EDX analysis confirms that interfacial adhesion and matrix densification significantly influence load transfer and arrest crack mechanism, resulting in unstable crack growth and rapid stiffness degradation. This evidence directly relates with structural observations, where fiber-reinforced concrete shows ductile response [8].

Steel fibers reduce crack opening by bridging inclined cracks and transferring tensile stresses after cracking, which delays crack growth and lowers the demand on stirrups by sharing shear resistance [4, 7, 21]. At higher fiber volume of fractions crack control improves initially however, excessive fiber content may reduce workability and lead to non-uniform distribution, limiting

effectiveness [5, 8, 22]. An increase in fiber aspect ratio improves crack-bridging and pull-out resistance, but very high aspect ratios can cause fiber balling and reduce overall efficiency in beams [9, 11].

Existing studies fiber-reinforced concrete often treats material properties and structural behavior separately with limited efforts to establish a direct link between microstructural characteristics and structural response. In addition, the combined influence of silica fume and steel fibers on shear behavior lacking, particularly due to the opposing effects of matrix densification. The study addresses gaps through an integrated experimental approach, to establish the relationship between material behavior and structural response. Initially, mechanical properties were evaluated for different (V_f) to identify optimum composition. then structural testing was conducted to assess the response of the beams. SEM-EDX analysis performed to examine reaction of the supplementary cementitious (SCM) material and the fiber-matrix interaction governing post crack bridging. The novelty lies indirectly correlating material and microstructural behavior with the structural performance. The results demonstrate that fiber addition delays crack and improves the ductility even with the larger stirrup spacing in the structural element, thereby improving the reliability of shear-critical members.

2. Research Importance

This experimental investigation is important because it establishes a direct and systematic link between material-level optimization and shear behavior at the beam scale, which is often missing in existing studies. Most previous research adopts predefined fiber contents directly for beam testing or limits evaluation to material specimens without structural validation [3, 5, 7]. In the present work, the selection of steel fiber volume fraction is conducted by compressive, stress-strain behavior tests performance before beam casting, ensuring that structural response reflects optimized material behavior. The experimental results compare steel fiber-reinforced concrete beams with conventional reinforced concrete beams by evaluating differences in load response, crack development and failure mode showing that fiber inclusion alters diagonal cracking behavior and promotes a more stable shear failure mechanism relative to brittle shear-dominated failure observed in normal beams [4, 8, 11].

3. Description of materials and specimens

3.1 Materials Properties

3.1.1 Cementitious Materials

Ordinary Portland Cement (OPC) of 53 grade was used as the primary binder. The cement complied with relevant Indian Standard specifications and had a specific gravity of 3.15. Silica fume was employed as a partial cement replacement to improve matrix densification and interfacial transition zone characteristics. Owing to its ultrafine particle size and high amorphous silica content, silica fume participates in pozzolanic reactions with calcium hydroxide to generate additional calcium silicate hydrate (C-S-H) gel. The specifications provided by the supplier are summarized in Table 1.

Table 1. Mineral admixture properties

Parameters	Specifications
Appearance	Grey powder
Bulk density at packing (kg/m^3)	588
Loss on ignition (%)	2.86
Specific gravity	2.29
Silica content by mass (%)	92.48
Moisture content (%)	0.31
Retained on 45 μm sieve (%)	6.28

3.1.2 Aggregates

Natural river sand was used as fine aggregate. The sand was clean, well graded and free from organic impurities, ensuring adequate workability and uniform fiber dispersion. All the materials are illustrated in (Fig. 1).

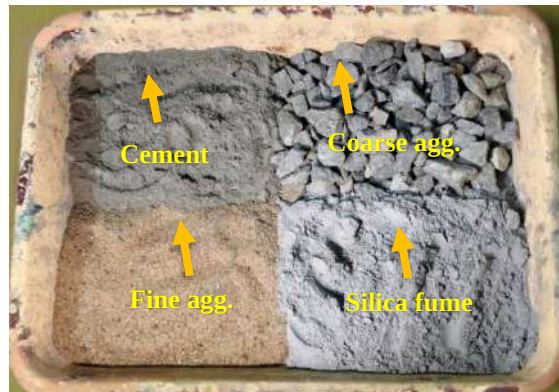


Fig. 1. Primary materials utilized in concrete mix preparation

Crushed angular coarse aggregate with a nominal maximum size of 20 mm was adopted to provide the primary load-bearing skeleton and improved mechanical interlock within the concrete matrix. As described in Table 2.

Table 2. Properties of aggregates employed in the experimental program

Property	Specific Gravity	Water Absorption (%)	Fineness Modulus	Nominal Size	Grading Zone
Fine Aggregate	2.65	1	2.70	4.75mm-90 μ m	III
Coarse Aggregate	2.74	0.5	7.20	20 mm	—

3.1.3 Mixing Water

Potable water free from deleterious substances such as acids, alkalis, salts, and organic matter was used for both mixing and curing, in accordance with standard requirements for reinforced concrete.

3.1.4 Superplasticizer

A high-range water-reducing admixture (Hi-Forza 369) was incorporated to get required workability at a low w/c ratio particularly in mixes containing silica fume and steel fibers. The admixture was a yellowish-brown liquid with a specific gravity of 1.145 and pH of 6.8 indicating chemical blending with cementitious systems. The low chloride content 0.09% ensured that no adverse risk of reinforcement corrosion was introduced. The admixture acts has water reduction capacity of approximately 25% enabling improved flowability and cohesive behavior without segregation. A dosage of 1% by weight of binder was adopted to promote uniform fiber dispersion and consistent fresh-state performance.

3.1.5 Steel Reinforcement

High-yield deformed steel bars of grade Fe 550D were used as internal reinforcement. Longitudinal reinforcement consisted of 12 mm ϕ bars provided in both the top and bottom regions of the beam to resist flexural tension and compression. Transverse reinforcement was provided using 8 mm ϕ bars as stirrups to control shear cracking and confinement. All reinforcing bars satisfied the mechanical and ductility requirements. The supplier provided parameters are presented in Table 3.

Table 3. Reinforcement properties

Rebar size (ϕ)	Elastic modulus (GPa)	Density (g/cm ³)	Shear strength (MPa)	Tensile strength (MPa)	Poisson's ratio
12mm, 8mm	200	7.85	319	550	0.30

3.1.6 Fibers

Hooked-end steel fibers were incorporated to enhance post-cracking behavior and shear resistance as shown in (Fig. 2). The hooked geometry improves mechanical anchorage and pull-out resistance, enabling effective crack bridging under tensile stress. Fibers were added at specified volume of fractions (V_f).



Fig. 2. Representative steel fiber materials utilized in the experimental program

To study their influence on crack development and failure mode in reinforced concrete beams. The properties of the fibers were offered by the material supplier shown in Table 4.

Table 4. Summary of fiber properties

Property	Value
Length (mm)	35
Aspect Ratio	46.6
Density (g/cm ³)	7.8
Tensile Strength (MPa)	1150
Shape	Hooked-end

3.1.7 Concrete Mix Design

Concrete mixes for RCSF0% and RCSF1.0% denote (reinforced concrete steel fiber beams) were designed as per IS:456-2000 and IS:10262-2019 [25, 26] for M50 grade using 5% silica fume and a PCE-based superplasticizer based on mechanical performance, RCSF1.0% was selected for beam casting along with the control mix. The mix design parameters are shown in Table 5.

Table 5. Mix design proportions adopted for experimental concrete specimens

Mix	Cement (kg/m ³)	Silica fume (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)	Super plasticizer (kg/m ³)	Steel fibers (kg/m ³)	Water (kg/m ³)
RCSF0%	410	22	735	1182	4.31	0	138
RCSF1.0%	410	22	735	1182	4.31	78	138

4. Experimental Investigation

4.1 Mechanical Properties

The RCSF0% mix showed a slump of 106 mm, while the RCSF1.0% mix recorded 72 mm, indicating reduced workability due to fiber addition. Cube and cylinder specimens were cured for 28 days prior to testing. Compressive, stress-strain behavior tests were conducted in accordance with relevant IS codes, using the specified loading rates for each test to ensure consistent and reliable mechanical performance evaluation. Fig. 3 (below) shows the test setups of the specimens.



Fig. 3. Experimental test setup of compression testing machine for concrete specimens

4.2 Flexural Studies of Fiber Reinforced Concrete Beams

Four reinforced concrete beams were casted to evaluate load–deflection behaviour. Each beam had the dimensions of 1.5m × 0.23m × 0.30m. The specimens were categorized into two groups:

- Conventional Control Beam (RCSF0%)

Two reinforced concrete beams were cast using M50 grade concrete with 5% silica fume as a partial replacement for cement and conventional steel reinforcement bars.

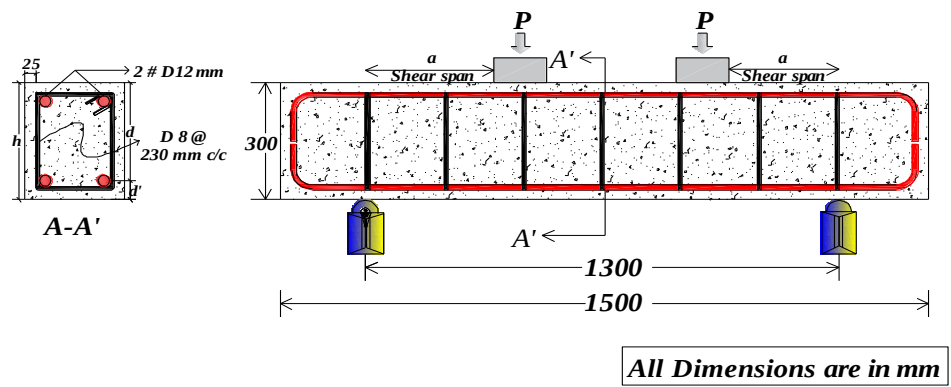
- Fiber Reinforced Concrete Beam (RCSF1.0%)

Two fiber reinforced concrete beams (FRC) were casted using M50 grade concrete and conventional steel reinforcement with 1.0% steel fibers incorporated to enhance post-cracking and shear performance. Four beams were cured for 28 days using the wet gunny bag method before structural testing.

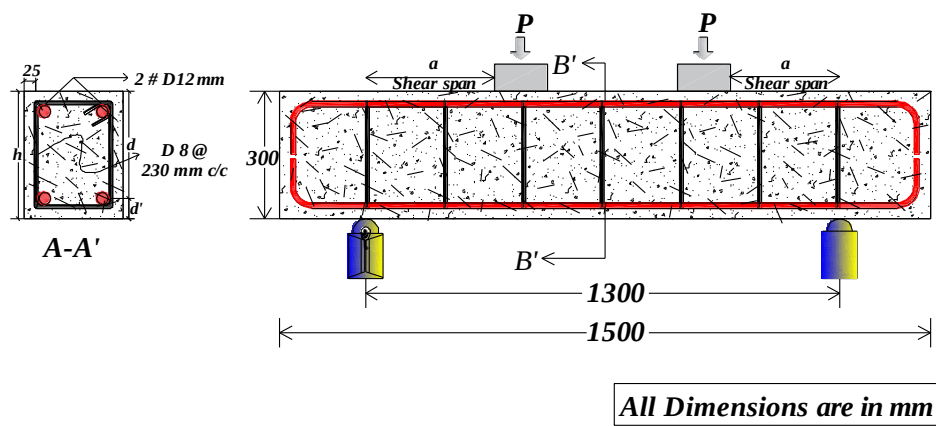
4.3 Detailing of Reinforcement and Beam Fabrication

The beams were reinforced to ensure adequate resistance against flexural and shear actions. Longitudinal reinforcement $\varnothing$ of 12 mm bars was provided in both the tension and compression zones to resist tensile stresses and maintain flexural stability. To control shear stresses, 8 mm $\varnothing$ transverse reinforcement was provided @ 230 mm c/c spacing using Fe550D grade steel. This comparatively wider spacing was intentionally adopted to limit the contribution of stirrups and enable clearer assessment of the influence of steel fibers on shear resistance and crack control. For the steel fiber–reinforced beams, hooked-end steel fibers were incorporated at a volume fraction (V_f) of into the concrete matrix to supplement shear resistance through crack bridging and post-cracking stress transfer. (Fig. 4) illustrates the typical reinforcement layout adopted for all test specimens.

Beam moulds with internal dimensions of (1.5m×0.23m×0.30m) were overly cleaned and inspected to ensure rigidity before a thin layer of form release oil was applied. The reinforcement was placed inside the mould with suitable cover blocks to maintain clear cover at the bottom and sides. Concrete was mixed corresponding to the quantified mixes and placed into the mould in 3 layers. Each layer was compacted using a mechanical vibrator to expel entrapped air and ensure uniform integration. The top surface was finished and specimens were left undisturbed for 24 h before demolding. Subsequently, beams were cured using wet gunny bags to promote strength development and control hydration heat. As illustrated in (Fig. 5).



(a)



(b)



(c)

Fig. 4. Reinforcement configuration of tested beams shown in real and schematic forms A) RCSF0% B) RCSF1.0% C) Demonstrates actual reinforcement configuration



Fig. 5. Membrane curing of RC Beams

4.4 Test Setup of the Beam

After completion of curing, beam specimens were tested in a calibrated 200T capacity loading frame under four-point loading to examine shear behavior of simply supported members. One end of each beam was supported on a hinge and the other on a roller to allow free rotation and longitudinal movement. A linear variable differential transducer (LVDT) was positioned at mid-span to continuously record vertical deflection during loading as displayed in (Fig. 6). Prior to testing, reference grid lines were marked at 100 mm intervals along the beam length to facilitate identification and monitoring of crack initiation and propagation. Load was applied incrementally until failure and the resulting load–deflection data were used to assess stiffness response, cracking behavior and shear and flexural performance of the beams.

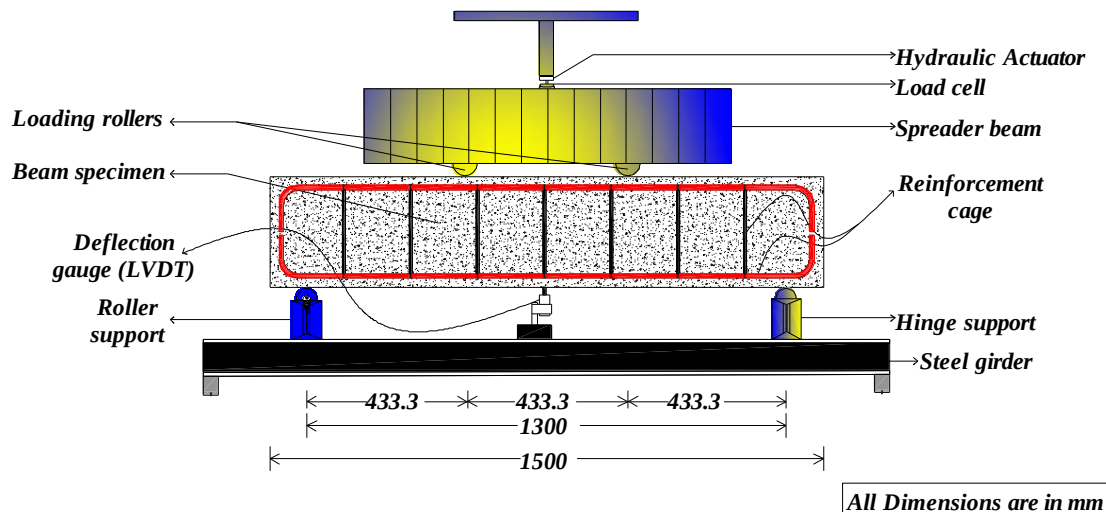


Fig. 6. Schematic representation of experimental test configuration

5. Results and Discussion

5.1 Mechanical Properties of Concrete

Cube and cylinder specimens corresponding to each concrete mix were tested at 7 and 28 days to examine strength development and deformation characteristics. The cube compressive strength for mixes with varying (V_f) {0%, 0.75%, 1.0%, 1.25%} among them RCSF1.0% mix attained 59.12 N/mm² at 28 days, whereas the control mix RCSF0% exhibited a comparatively lower value of 52.60 N/mm², while mixes 0.75%, 1.25% achieved 55.72 N/mm², 58.30 N/mm² respectively shown in (Fig. 7). The strength reduction in 1.25% is attributed due to reduced workability and fiber agglomeration, which leads to non-uniform dispersion and weakens the matrix continuity. Therefore 1.0% was selected for casting cylinders and beams to evaluate stress-strain behavior and structural performance of the beam [4, 12, 17, 24].

The strength of RCSF0% remained limited by the brittle nature of the cement matrix, where confined tensile microcracking governs the appearance of crushing (Fig. 8) highlights the failure modes of cube and cylinder specimens with crack propagation paths drawn and emphasized in yellow. (Fig. 9), illustrates the compressive stress–strain responses of the RCSF0% and RCSF1.0% concretes at 7 and 28 days revealing a pronounced influence of steel fiber incorporation on deformation behavior. All mixes exhibited an initially linear elastic segment. However, fiber addition vaguely increased the strain equivalent to peak stress.

The control mix reached maximum stress at a similarly lower strain and undergone post-peak reduction indicative of brittle matrix crushing and unstable crack growth. Similarly, (FRC) mixtures produced a larger nonlinear zone and maintained critical stress beyond peak, indicating fiber pull-out mechanisms and crack-bridging progressive debonding. The most progressive post-peak response and the best strain capacity were consistently achieved by the RCSF1.0% fiber mix. Peak stress was mostly influenced by age but ductility ranking unchanged confirming the role of fibers in promoting ductile response and enhanced energy dissipation.

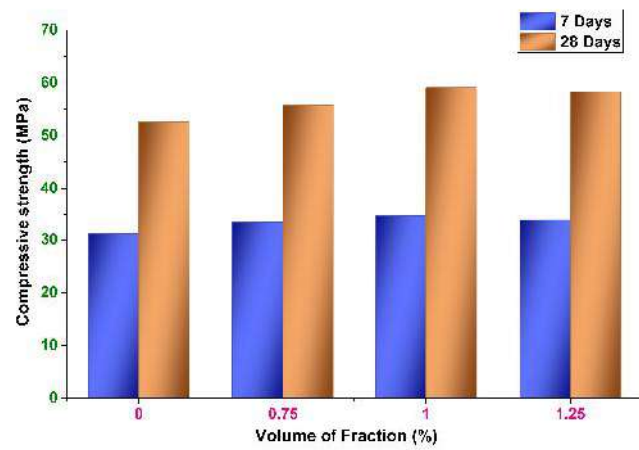


Fig. 7. Variation in compressive strength of concrete cubes with respect to curing age

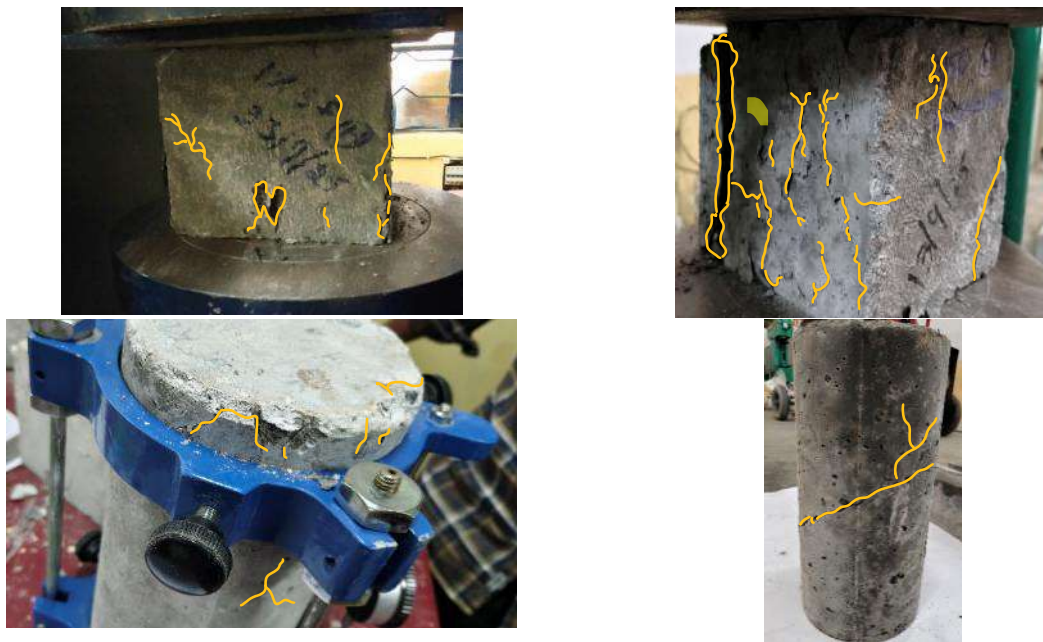


Fig. 8. Depicts the crack pattern developed in concrete specimen

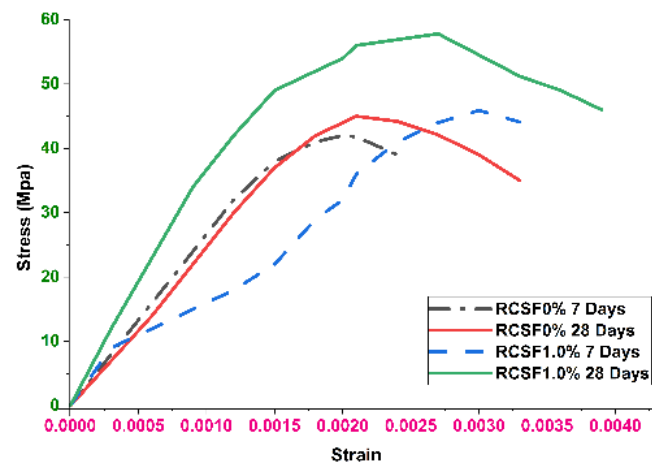


Fig. 9. Stress-strain behavior of a concrete

5.2 Structural Response and Damage Evolution of RC Beams

Flexural testing of reinforced concrete beams was carried out to quantify the influence of steel fiber incorporation on the load–deflection response $P-\Delta$, crack initiation and propagation characteristics, stiffness degradation $K = P/\Delta$, and governing failure mode. For each concrete mix, two beam specimens were tested under identical four-point loading conditions with $a/d = 1.9$ and all specimens were detailed with transverse reinforcement @230 mm c/c spacing to ensure consistent shear confinement. In this study, the structural responses are based on the average behavior of the two specimens for each mix, and all results discussed herein correspond to these averaged values [27]. The representative average load–deflection responses $P-\Delta$ for all mixes are summarized in Table 6, thereby providing a reliable basis for subsequent analysis of stiffness degradation, energy absorption $E = \int P d\Delta$ and ductility index $\mu = \Delta_u/\Delta_y$. Particular emphasis was placed on the ability of steel fibers to delay first cracking P_{cr} , restrict crack opening (w) and modify the post-cracking response through crack-bridging and stress redistribution mechanisms [19].

Table 6. Average ultimate load and variability of tested RC beams

Mix	P_u (kN)	Standard deviation (kN)	Coefficient of variation (%)
RCSF0%	204.8	2.26	1.1
RCSF1.0%	246.0	3.39	1.4

The control beams without fibers (RCSF0%) attained an average ultimate load of $P_u = 204.8$ kN with a corresponding maximum midspan deflection of $\Delta_u = 19.1$ mm, whereas the RCSF1.0% mix achieved a higher average ultimate load of $P_u = 246.0$ kN and a larger ultimate deflection of $\Delta_u = 29.2$ mm, as shown in (Fig. 10). The comparatively large deflections observed in both beam series are mainly attributed to the wide stirrup spacing of 230 mm c/c, which reduced the direct contribution of transverse reinforcement to shear resistance and permitted greater shear deformation prior to failure.

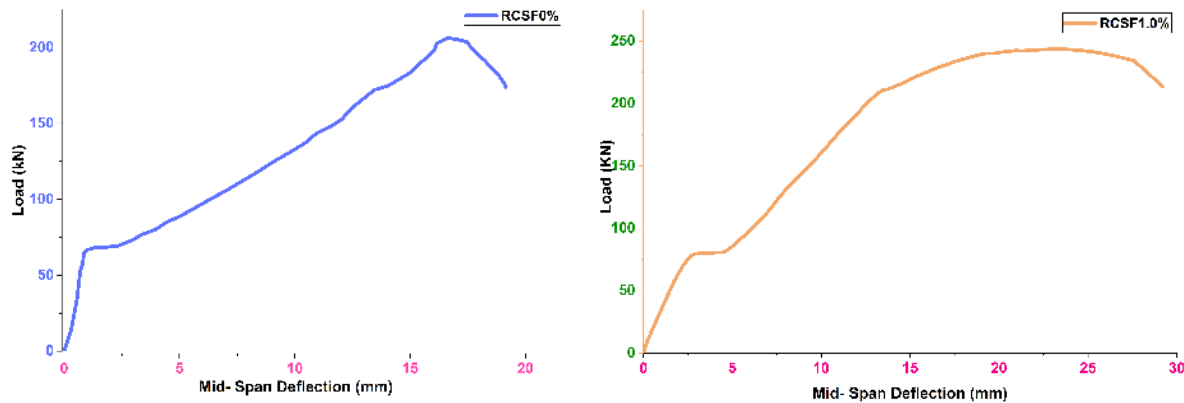


Fig. 10. Load vs deflection response of beam specimens under loading

The ultimate shear capacity may be expressed as $V_u = V_c + V_s + V_f$, where V_c - the concrete contribution, V_s - stirrup contribution, and V_f - fiber contribution. In RCSF0% beams, the absence of fibers limited shear transfer to V_c and V_s , leading to early diagonal cracking and restricted post-cracking stiffness, thereby constraining deformation capacity [23]. In contrast, RCSF1.0% beams benefited from the additional fiber-bridging component V_f , which enhanced aggregate interlock, restrained crack opening, and improved stress redistribution, allowing higher loads to be sustained with stable deformation. At ultimate load, all beams exhibited an average end rotation of approximately $\theta \approx 3^\circ$. The predominance of shear failure in RCSF0% beams and flexure-controlled failure in RCSF1.0% beams confirms that steel fibers effectively compensated for the reduced stirrup contribution, increased deformation capacity, and promoted a more ductile, flexure-governed response.

5.2.1 Comparison of the Conventional Beam and Fiber Reinforced Beam

The control beams exhibited early diagonal cracking followed by brittle shear failure, whereas the (FRC) beams developed well-distributed flexural cracks with delayed crack propagation and gradual stiffness degradation. Higher ultimate load and improved post-cracking deformation capacity were consistently observed in RCSF1.0% specimens. The contrast in cracking pattern, load–deflection response and failure mode between the two beam types is illustrated in (Fig 11).

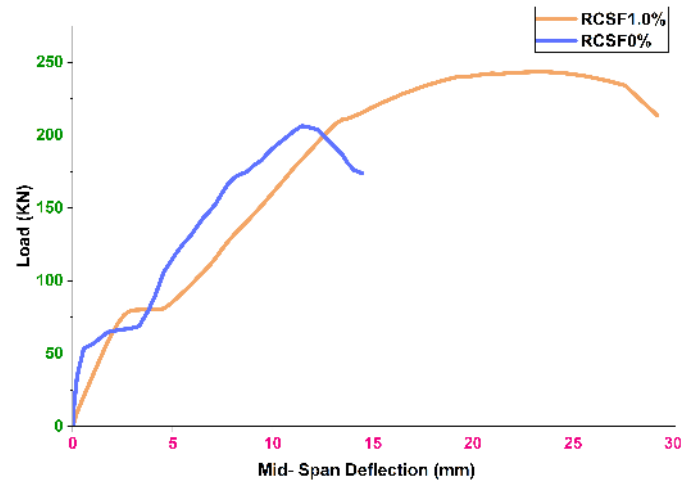
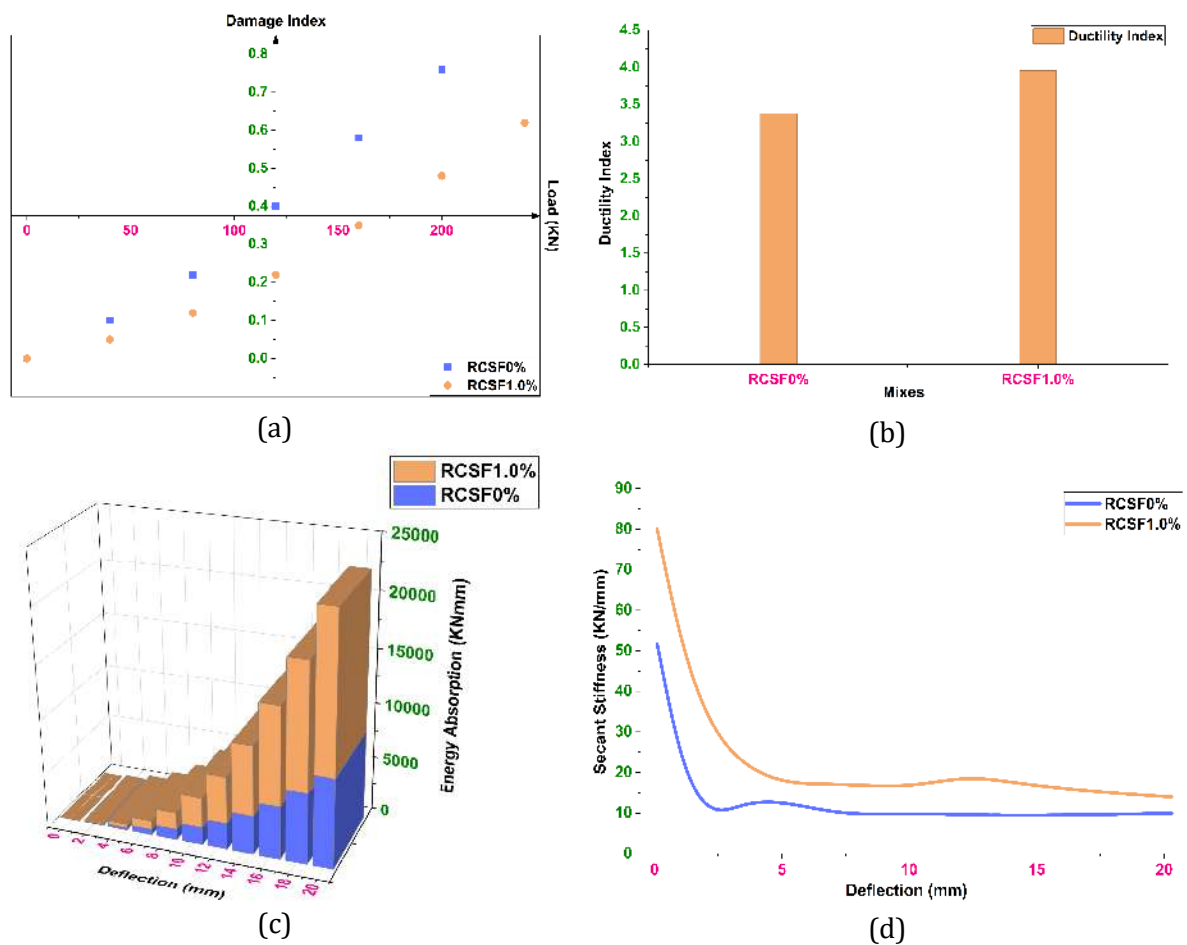


Fig. 11. Comparative assessment of load-deflection characteristics of beam specimens



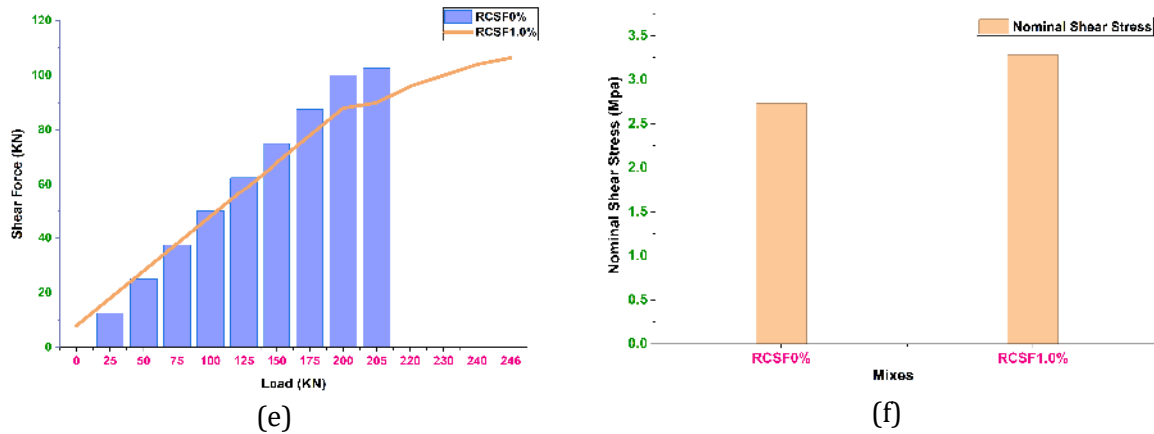


Fig. 12. Comparative evaluation of (a) Damage index (b) Ductility index (c) Energy absorption (d) Secant stiffness degradation (e) Shear force and (f) Nominal shear stress of beam specimens.

Control beams exhibited rapid stiffness reduction after the onset of diagonal cracking subsequently weak load transfer and leads to shear failure. In contrast, RCSF1.0% beams maintained higher load levels with progressive deformation indicating that fiber bridging across developing cracks maintained tensile stress transfer after matrix cracking. The presence of fibers restricted crack opening and delayed crack coalescence allowing stresses to be redistributed within the tension zone and across inclined crack planes. This mechanism explains the higher ultimate load and shifting failure pattern from shear zone failure behavior to flexure-controlled behavior in RCSF1.0% beams. The variation of secant stiffness degradation with deflection is shown in (Fig. 12). Stiffness evolution with increasing deflection, expressed through the secant stiffness degradation confirms that (FRC) beams retain greater load-carrying capability at comparable deformation levels.

$$K = P/\Delta \quad (1)$$

Reduced stiffness loss in RCSF1.0% beams is associated with controlled microcrack growth and gradual fiber pull-out, whereas RCSF0% beams experience rapid stiffness decay due to unstable diagonal cracking. The larger ductility index recorded for RCSF1.0% beams reflect the ability of fibers to sustain load beyond first cracking and accommodate inelastic deformation prior to failure. A comparison of ductility damage for both beam types is shown in (Fig. 12). Similarly, the higher cumulative energy absorption observed in (FRC) beams results from repeated crack bridging and frictional pull-out of fibers. The cumulative energy absorption trends, which dissipate energy during crack propagation. Damage index is evaluated using, progressed steadily in RCSF1.0% beams, indicating delayed stiffness degradation and distributed cracking.

$$D = 1 - (K/K_0) \quad (2)$$

The evolution of the damage index with applied load further confirms a slower accumulation of damage in the FRC beams. In addition, the load-shear force response obtained from $V = P/2$ indicate improved shear resistance for the RCSF1.0% mix. The ultimate shear capacity of the RCSF1.0% beams increased by approximately, compared with the control beams. These observations collectively confirm that steel fibers enhance crack stability, improve deformation capacity and transform the overall response toward a more stable and ductile structural behavior.

$$\frac{V_{u,SFRC} - V_{u,RC}}{V_{u,RC}} \times 100 \approx 20\% \quad (3)$$

5.3 Structural Distress Modes

The crack patterns shown in (Fig. 13) highlights the governing role of steel fibers in controlling shear transfer, crack evolution and failure mode. The RCSF0% beams exhibited shear-dominated

behavior characterized by the formation of a major diagonal tension crack within the shear span, initiating near the support and propagating toward the loading point.

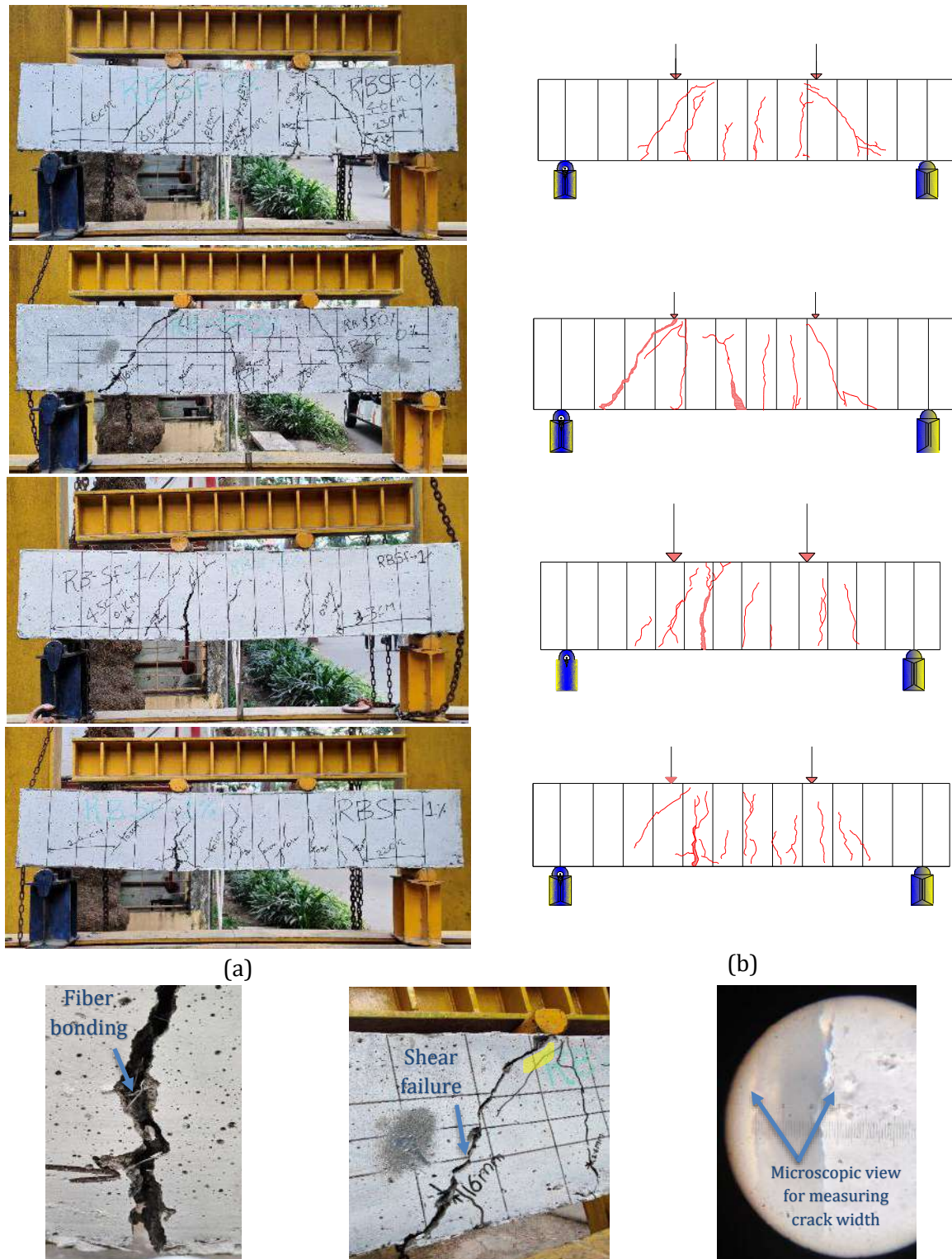


Fig. 13. Representation of crack development through (A) From experimental testing and (B) Schematic representation of crack mechanism

The average diagonal crack width was approximately 8.25 mm at a distance of about 295 mm from the support with the help of microscope, while only limited flexural cracking developed in the constant moment region with an average maximum width of about 2.6 mm. The presence of web-shear and shear-compression cracks near the compression zone indicates crushing of the diagonal

concrete strut and severe concentration of principal tensile stress (σ_1), leading to brittle shear failure with limited post-cracking stress redistribution.

In contrast, RCSF1.0% beams displayed initial fine, closely spaced vertical flexural cracks followed by flexural-shear cracks forming at approximately 450 mm from the supports, with an average maximum width of about 4.3 mm and no web-shear cracks were observed. Steel fibers allow crack-bridging action, enhancing aggregate interlock and dowel action enabling sustained tensile stress transfer across cracked planes. This mechanism increased the post-cracking tensile benefaction of concrete and the effective shear resistance, thereby reducing the nominal shear stress ratio relative to the cracking inception (Fig. 12). Consequently, cracking progressed in a stable and distributed manner, stiffness degradation was gradual and failure shifted from brittle shear in RCSF0% beams to a ductile, flexure-controlled mechanism in RCSF1.0% beams.

$$\tau_v = \frac{V}{bd} \quad (4)$$

5.4 SEM EDX analysis

The SEM analysis (Fig. 14) reveals a distinct difference in the microstructural organization of RCSF0% and RCSF1.0% concrete mixes. In RCSF0%, the matrix is mainly composed of C-S-H gel integrated with plate-like calcium hydroxide and occasional unhydrated clinker particles, indicating a dense but heterogeneous paste. Although the incorporation of 5% silica fume refines the pore structure through secondary pozzolanic reactions, the absence of fibers permits microcracks to propagate preferentially along weak paste regions and interfacial transition zones (ITZ), resulting in limited crack arrest capability and a brittle shear-governed response. Thus, shear resistance in RCSF0% is primarily led by matrix strength and aggregate interlock, which are insufficient to maintain post-cracking load transfer.

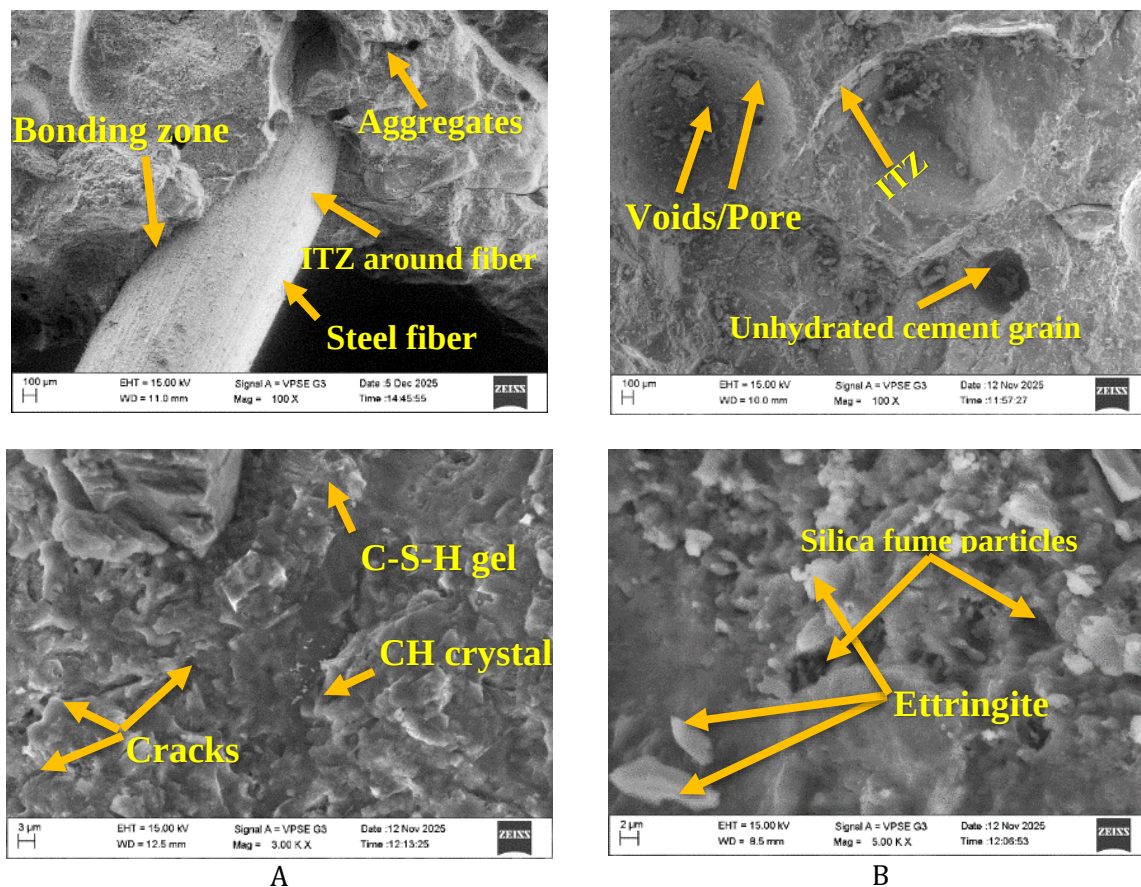


Fig. 14. SEM images comparison of A) FRC and B) Nominal concrete

In contrast, the RCSF1.0% mix exhibits a well-developed fiber-matrix interface, where hydration products are tightly adhered to the steel fiber surface forming a compact and continuous ITZ. The

rough fiber texture promotes mechanical interlocking at the same time, the surrounding C-S-H layer enhances chemical bonding enabling efficient stress transfer between the matrix and fibers. Under shear loading, fibers bridge developing diagonal cracks, restrict crack widening and provide additional shear resistance through pull-out and debonding mechanisms. In (Fig. 15) EDX results showing dominant Ca (26.9 wt.%) and O (52.3 wt.%) with measurable Si and Al confirm a hydration product-rich matrix with Ca-rich zones that further strengthen the fiber-matrix bond. The combined effect of matrix densification and effective fiber bridging explains the higher load-carrying capacity, improved post-cracking stiffness and limited ductile shear behavior of the RCSF1.0% mix compared to the brittle response of RCSF0%. The enhanced fiber-matrix interaction promotes crack bridging, increased post-peak load resistance and higher energy absorption, thereby transforming the failure mode from brittle to ductile failure.

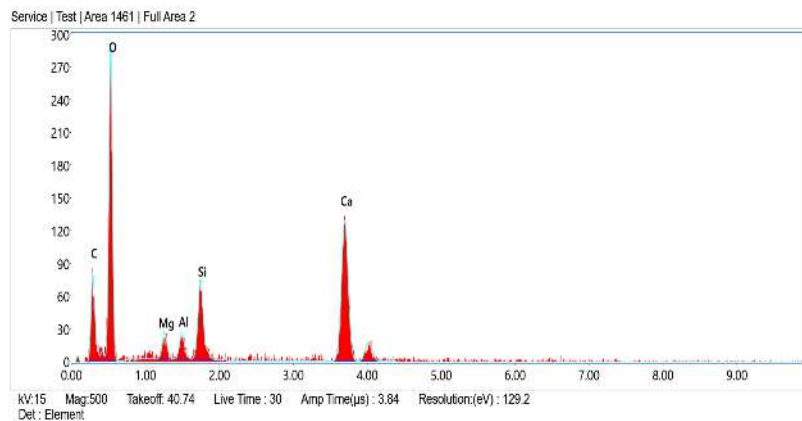


Fig. 15. EDX analysis indicating elemental distribution of the concrete

5.5 Limitations and Validations of The Study

The present investigation is based on a controlled experimental program with a limited number of beam specimens and two beam-level mixes (0% and 1.0%). Although an optimum fiber content was identified through preliminary testing considering fiber (V_f) {0%, 0.75%, 1.0%, 1.25%}, only a single optimized fraction (1.0%) was considered for structural evaluation and a full parametric study at the beam level was not undertaken. Therefore, the results should be interpreted as indicative rather than statistically generalized. In addition, the distribution and orientation of fibers within the concrete matrix may vary due to workability constraints, which can influence local stress transfer and crack propagation behavior. The experimental program was limited to monotonic loading conditions and the response under cyclic or seismic loading was not investigated. Furthermore, the specimens represent laboratory-scale elements and scale effects may influence the behavior of full-scale structural members. The study is also limited to experimental observations without incorporating numerical or analytical modelling, which could further enhance the generalization of the results.

The validity of the present findings is supported through comparison with established experimental and analytical studies. A comparative evaluation with previous studies indicates that the observed 20% increase in shear capacity aligns with findings reported by [4, 8], while enhanced ductility and delayed crack observed in this study exceed those reported in conventional SFRC beams it confirms the synergistic effect of silica fume and steel fibers improves the shear resistance were shown in Table 7. Fiber-reinforced concrete enhanced shear strength delayed crack initiation and improved deformation capacity. This behavior is attributed to crack-bridging action of fibers to sustains tensile stress transfer across inclined cracks and also preventing premature shear failure. SEM observations further substantiate this mechanism revealing strong fiber-matrix bonding. This micro-to-macro interaction governs the improved structural response. Results indicate that fiber bridging compensates for reduced transverse reinforcement, thereby lowering the dependency on the closely spaced stirrups it enhances ductility in the structural elements

Table 7. Comparative evaluation of shear behavior and failure mechanisms of FRC beams with existing studies

Study	Fiber Content (%)	Approach	Shear Behavior	Failure Mode Transition	Key Mechanism	Comparison with Present Study
Yuan et al. [4]	1.0	Experimental	Increased shear capacity	Shear → Flexure	Demonstrated fiber effectiveness in shear strengthening	Present study shows higher improvement due to silica fume synergy
Dinh et al. [8]	0.75–1.5	Analytical + Experimental	Improved shear resistance	Controlled shear cracking	Developed analytical shear models for SFRC beams	Present study experimentally validates similar trends with microstructural evidence
Yoo and Yang [18]	1.0	Experimental	Enhanced ductility	Distributed cracking	Established enhanced ductility and energy absorption	Present study Confirms ductility enhancement with additional SEM-EDX validation
De Resende et al. [12]	1.0	Experimental	Improved dowel action	Reduced crack width	Highlighted reduction in crack width and improved shear transfer	Present study shows improved crack control with stronger fiber-matrix bonding
Thurimella et al. [24]	0–1.25	Experimental + microstructural	Improved capacity of load and crack control	Ductile behavior	Fiber bridging and enhanced bonding between fibers	Prior research emphasized flexural behavior; this study explores shear behavior showing ductility with SEM-EDX validation
Present Study	1.0	Experimental + SEM-EDX	Significant improvement	Shear → Flexure	Establishes micro-to-macro correlation using SEM-EDX	Demonstrates superior performance and validates mechanism of the materials experimentally

6. Conclusions

The study examined shear behavior of fiber-reinforced concrete beam through an integrated experimental approach linking material behavior, structural response and microstructural mechanisms to understand role of steel fibers under shear-critical conditions.

- RCSF0% revealed a slump of 106 mm, whereas RCSF1.0% showed 72 mm and reduced workability due to increased surface area & mechanical interlocking of fibers restrict free flow and increase internal friction within the fresh matrix. The compressive strength of concrete increased from RCSF0% mix 52.60 N/mm² to 59.12 N/mm² RCSF1.0% mix improved concrete matrix densification from silica fume and uniform stress redistribution caused by fiber bridging delays microcrack fusion under compression. However, fiber content beyond 1.0% was found

to reduce strength due to fiber agglomeration and reduced workability confirming 1.0% as optimum fiber dosage.

- Stress-strain curves demonstrated enhanced ductility for RCSF1.0%, presented by extended post-peak deformation, whereas RCSF0% showed a higher dropping in strength reflecting brittle matrix cracking and limited energy dissipation capacity.
- The RCSF1.0% beams developed higher load capacity, larger deformation, and greater rotation 3° with less cracks demonstrating that steel fibers effectively controlled crack growth and sustained tensile stress after cracking. In contrast, the control beams RCSF0% showed lower rotation 2° extensive shear cracking and early stiffness loss reflecting brittle shear-dominated behavior because absence of fibers.
- Despite stirrups spaced at 230 mm c/c RCSF1.0% beams failed in flexure, while RCSF0% beams exhibited shear-dominated brittle failure. In RCSF0% mix, principal tensile stresses (σ_1) along inclined shear planes exceeded concrete tensile capacity forming dominant diagonal cracks. In RCSF1.0% mix fibers reduced tensile stress concentration by bridging inclined cracks changes load distribution mechanism from shear to flexure failure zone.
- Addition of fibers in RCSF1.0% beams provide an effective internal shear-resisting mechanism through crack-bridging action and showed steadier stiffness degradation, higher ductility index, greater cumulative energy absorption and reduced damage index progression verifying that fiber pull-out delay stiffness loss and distribute damage more uniformly.
- The addition (V_f) of 1.0% fibers increased the nominal shear stress from 2.73 MPa to 3.28 MPa and the ultimate shear force from 102.40 kN to 123.00 kN equivalent to an improvement of approximately 20% in shear capacity primarily due to effective fiber bridging and improved post-cracking stress transfer. The reduced damage index and increased ductility index ($\mu_{RC} \approx 3.4$, $\mu_{SFRC} \approx 4.0$) of the SFRC beams indicate delayed stiffness degradation and a transition from brittle shear-dominated behavior to a more stable and ductile response.
- Further studies should examine aspect ratio of fiber contents, geometry and hybrid reinforcement strategies through extended experimental programs, including full-scale structural members such as beam-column joints and frames. In addition, behaviour under cyclic and seismic loading with different reinforcement configuration by varying the spacing to examine role of fibers and also analyze failure progression. The development of validated numerical models is essential to simulate crack evaluation and fiber interaction, enabling parametric studies and supporting interaction of fiber-based shear reinforcement into design-oriented guidelines.

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Effect of valorizing oil drilling waste on the strength and microstructural properties of gypseous-calcareous soil for use in Saharan road construction

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Abstract

In response to the environmental challenges posed by oil drilling waste generated from intensified oil production in Algeria, particularly in Saharan regions, this study proposes a valorization approach to mitigate their impacts. Our research aims to transform this waste into a useful resource by incorporating it into road construction mixtures. The study seeks to design an innovative and optimized formulation of gypseous-calcareous soil with oil drilling waste, specifically adapted to road infrastructure. We tested the incorporation of treated oil drilling waste into gypseous-calcareous soil formulations at rates of 5%, 10%, and 15% by weight and with the addition of 4% lime. The study evaluates the evolution of unconfined compressive strength (UCS) in these mixtures, as well as their water sensitivity, influence on California Bearing Ratio (CBR), and direct shear testing. The results reveal significant improvements in the mechanical behavior of the mixture modified with 10% oil drilling waste and 4% lime content, under laboratory curing conditions, achieving a UCS of 2.45 MPa, CBR value of 75% and shear strength parameters with an internal friction angle 35° and the cohesion 45 kPa. The microstructural analysis further confirmed that the interaction between oil drilling waste and the gypseous-calcareous soil in the presence of lime led to the formation of a more homogeneous and compact matrix, which explains the observed improvement in mechanical properties.

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1. Introduction

Oil exploitation in Algeria is particularly intensive in the Saharan regions and produces thousands of cubic meters of oil drilling waste each year [1]. These wastes, comprising drilling muds and cuttings, are often rich in hydrocarbons and heavy metals [2, 6]. In response to this environmental challenge, various valorization strategies have been developed [7, 11]. In the cement industry, Bernardo et al. [12] demonstrated that replacing limestone and clay with up to 45% Oil-Based Mud in clinker production does not compromise the mechanical properties of hydraulic binders, which is an important finding for regions such as the Saharan sedimentary basin. This approach is supported by the experimental work of Anghelescu et al. [13], who incorporated recycled coal ash and residual drilling fluids as partial substitutes for clay, revealing optimal compatibility for construction materials. Similarly, Chen et al. [14] introduced thermally treated drilling cuttings to manufacture permeable bricks that meet Taiwanese standards for compressive strength (≥ 15 MPa) and permeability (≥ 0.1 cm/s). Applied research continues with El-mahllawy and Osman [15],

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whose clay masonry units are enriched with 10, 20 and 30% treated drilling muds and comply with Egyptian EN 771-1 standards, achieving compressive strengths above 7 MPa for load-bearing walls. However, Mostavi et al. [16] identified a technical trade-off: substituting 5% of cement with dried cuttings in concrete leads to a 10% reduction in strength, which increases to 20% at higher replacement rates 25%. Nevertheless, the use of offshore oil drilling waste in hot mix asphalt (HMA) shows promising potential: replacing 20% of conventional aggregates maintains Marshall stability (>8 kN), flow (2-4 mm), and resilient modulus (>300 MPa), paving the way for sustainable road applications [17].

Recently, industrial waste materials have been added and mixed with loose soils to enhance their geotechnical properties [18, 19]. The improved characteristics of loose soils, resulting from the utilization of industrial waste additives like silica fume, ground granulated blast furnace slag, cement kiln dust, rice husk ash and fly ash, offers various advantages environmental and economic. Many studies have been conducted on the effectiveness of these industrial waste in improving soil properties. The addition of such materials improved the strength of soils [20, 22]. In addition, Saniyya et al. [23] and Al-khafaji et al. [24] studies reveal that adding industrial waste additives lead to a decrease in the plasticity index of soils. Regarding compaction, several studies Arulrajah et al. [25], Kassa et al. [26], and Pastor et al. [27] have indicated that the use of industrial waste additives significantly improved soil compaction characteristics. On the other hand, industrial waste additives interact with lime more efficiently than when used alone. Some researchers, including Alrubaye et al. [28], Zaini et al. [29] and Liu et al. [30] demonstrate that workability and strength performance of loose soils were significantly improved after applying a combined treatment.

Encrusting gypseous-calcareous soils are widely distributed across the arid regions of Algeria [31, 32], notably in Ouargla, and they are currently utilized in road construction [33]. However, their mechanical performance remains inadequate to meet modern traffic demands, so they are commonly employed in road foundation layers, exhibiting low bearing capacity, which is exacerbated by extreme climatic conditions [34, 35]. This situation often compels project owners to import higher-performance materials from distant locations, resulting in additional costs, delays in infrastructure development, increased carbon footprint, and greater logistical complexity due to long transport distances, which encourages researchers to explore the valorization of gypseous-calcareous soils in civil engineering [36, 38]. Thus, the reuse of additives derived from industrial by-products such as crushed brick waste, recycled glass powder, or treated drilling waste represents an effective strategy to reduce transport distances, lower associated costs and emissions, while promoting the improvement and sustainable use of locally available resources. Initial studies demonstrated that additives like cement enhance gypseous-calcareous soil's physical, chemical, and thermal properties [39]. Further research has revealed that adding cement with 15% ceramic powder significantly optimizes physical properties and mechanical performance, particularly compressive strength [40]. Another study investigated a hydraulic binder-treated tuff/sand mixture (65% soil, 35% dune sand, 4% binder: 2/3 lime, 1/3 cement), confirming compliance with technical criteria [41]. Additional work has highlighted that incorporating 25% dune sand into tuff increases its dry density by up to 20% [42]. Finally, mixtures with up to 30% sand achieved compliant CBR index values, while markedly improving compressive strength [43].

The objective of this research is to propose a dual ecological and economic solution, contributing to sustainable oil waste management by valorizing treated drilling waste (DW) as an additive to gypseous-calcareous soil, to enhance its mechanical characteristics, which is a critical criterion for material selection in Saharan road projects.

2. Materials and methods

2.1 Materials

2.1.1 Gypseous-Calcareous Soil

In this study, the base material was gypseous-calcareous soil sourced from the Ouargla region of southeastern Algeria; it is utilized in road foundation layers (Fig. 1). The geotechnical and mineralogical characteristics of the soil used are summarized in Table 1.



Fig. 1. Gypseous-calcareous soil before and after compaction

Table 1. Geotechnical and mineralogical characteristics of the soil used

Property	Value	Unit
Bulk density	1.178	g/cm ³
Specific density	2.218	g/cm ³
Maximum diameter (Dmax)	50	mm
Particles < 2 mm	73	%
Particles < 0.80 mm	24	%
Maximum dry density	1.62	t/m ³
Optimal moisture content (Wm)	14.7	%
Methylene blue value (VBS)	0.45	-
Immediate Bearing Index (IBI)	28.01	-
California Bearing Ratio (CBR) after immersion	48	-
Unconfined compressive strength (28 days)	0.65	MPa
CaSO ₄ ·2H ₂ O	42.5	%
CaCO ₃	28.3	%

2.1.2 Oil Drilling Waste

The drilling waste used in this study came from the Hassi Messaoud region (southeastern Algeria) after being treated using the stabilization/solidification (S/S) method (Fig. 2). Its main chemical characteristics were obtained by X-ray fluorescence (XRF) are presented in Table 2.



Fig. 2. Treated oil drilling waste before and after grinding

Table 2. Chemical XRF analysis of drilling waste

Oxides	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	Na ₂ O	K ₂ O	P ₂ O ₅
Mass (%)	26.46	2.97	1.51	12.24	1.65	8.23	0.28	1.75	0.05

The chemical composition of drilling waste reported in this study is consistent with values reported in previous research, where silica, calcium, and sulfate oxides are commonly identified as the dominant components in treated drilling waste [8,9]. In order to evaluate the potential impact of grinding on the efficiency of the S/S treatment, chemical tests were conducted using Atomic Absorption Spectroscopy (AAS), before and after grinding the sample of treated drilling waste. The results revealed that, although grinding slightly influenced the treatment, the heavy metal concentrations remained below the regulatory limits, according to [44], as shown in Table 3. The grinding of the treated oil drilling waste sample was followed by a specific surface area analysis using the Blaine method, which allowed for the characterization of the material's fineness. The results showed a specific surface area of 2500 cm²/g.

Table 3. Atomic absorption spectroscopy (AAS) results before and after grinding of treated oil drilling waste and regulatory limit values

Element	Before grinding (mg/kg)	After grinding (mg/kg)	Regulatory limit (mg/kg)
Cr	0,650	7,60	100
Ni	8,700	9,70	100
Cd	0,120	0,16	50
Hg	0,421	1,10	10
Pb	17,700	26,90	100

2.1.3 Lime

Ground lime is primarily composed of calcium oxide (CaO), with a purity exceeding 73.3% and it is generally presented in the form of a dry white powder, with a specific density of approximately 2 g/cm³ and an apparent density ranging between 600 and 900 g/l. Impurities are maintained at low levels, notably: silicon dioxide (SiO₂) below 2.5%, iron oxide (Fe₂O₃) below 2%, and aluminum oxide (Al₂O₃) below 1.5%. The particle size distribution was controlled, with less than 10% of the particles exceeding 90 µm and none above 630 µm.

Lime stabilization was selected due to its well-known ability to activate pozzolanic reactions with siliceous and aluminous phases in soils, leading to the formation of cementitious compounds such as C-S-H and C-A-H gels. These reaction products improve the soil structure and mechanical strength. Compared with other binders such as cement or ground granulated blast furnace slag (GGBS), lime treatment is widely used in soil stabilization due to its relatively low cost and ease of field application. Previous studies have also reported satisfactory durability of lime-stabilized soils under long-term environmental conditions and cyclic wetting-drying processes [45].

2.1.4 Formulation

Different formulations were developed for the preparation of the specimens. Table 4 presents the various compositions tested, comprising gypseous-calcareous soil (S), lime (L), and oil drilling waste (DW). This allowed for a comparative analysis of the material properties based on their compositions.

Table 4. Different mixtures studied

Mixture	Soil (%)	Oil drilling waste (%)	Lime (%)
SDW0L0	100	0	0
SDW5L0	95	5	0
SDW10L0	90	10	0
SDW15L0	85	15	0
SDW0L4	96	0	4
SDW5L4	91	5	4
SDW10L4	86	10	4
SDW15L4	81	15	4

2.2 Methods

For each mixture and, three identical specimens were prepared and tested to ensure the reliability and repeatability of the results.

2.2.1 Unconfined Compressive Strength Test

Samples were prepared in cylindrical molds ($\phi = 5$ cm, height = 10 cm) and statically compacted using a press at the optimum moisture content defined by the modified Proctor test, achieving a compaction level of 98%. The specimens were maintained under laboratory conditions at a temperature of 25 ± 5 °C. The curing durations were 7, 14, 28, 60, and 90 days. Once the storage time was reached, the specimens were subjected to a compressive force applied parallel to the cylinder axis, using a press at a constant speed of 1.27 mm/min until rupture in accordance with (EN ISO 17892-7) [46] (Fig. 3).

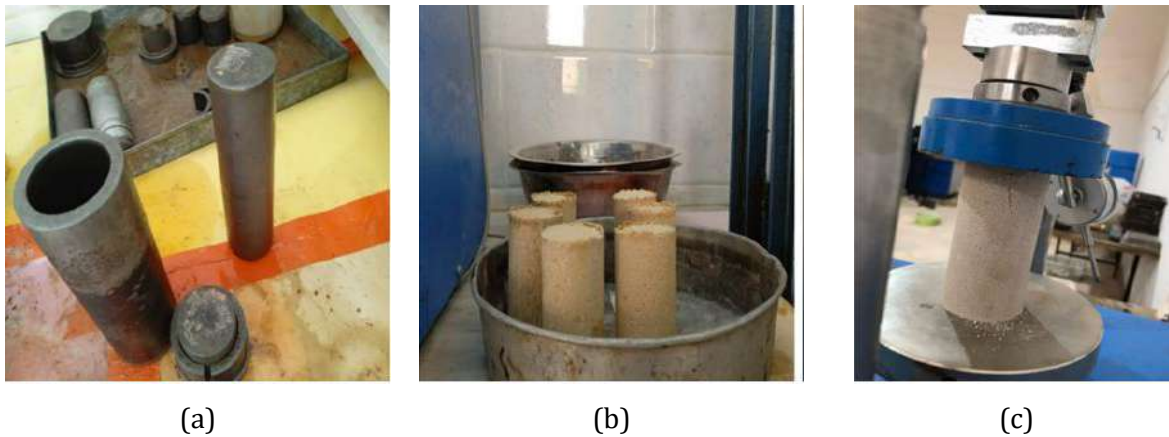


Fig. 3. (a) Manufacturing molds; (b) Demolded specimens; (c) Sample under compressive testing

2.2.2 Sensitivity to Water Test

The water immersion test is used to assess the durability of materials by measuring their compressive strength after saturation. When applied to soil, it involves immersing specimens for a specified period, to analyze their mechanical behavior under wet conditions. This procedure helps to estimate the soil material's sensitivity to water and its ability to retain its mechanical properties after saturation. The compressive strength was measured in accordance with the (EN ISO 17892-7) [46], which evaluates the stability of the soil and its suitability for use in moisture exposed environments.

2.2.3 California Bearing Ratio (CBR) Test

The California Bearing Ratio (CBR) test, carried out in compliance with the EN 13286-47 standard [47], is used to evaluate the relative load-bearing capacity of a soil by measuring its penetration resistance.



Fig. 4. CBR test for compacted mixtures

The protocol involves applying a force on a standardized piston (49.6 mm in diameter) that penetrates the compacted sample with size of (152.4 mm diameter and 177.8 mm height) at a constant speed of 1.27 mm/min. The CBR value is determined by calculating the ratio, expressed as a percentage, between the force required to push the piston to a given penetration depth (2.5 and 5.0 mm) in the tested sample and the force required for the same penetration in a reference material, as illustrated in Fig. 4. In our study, we focused on analyzing the CBR after 96 hours of immersion to replicate the most unfavorable moisture exposure conditions and assess the durability of the proposed mixtures against water infiltration in a Saharan environment.

2.2.4 Direct Shear Test

Direct shear tests were performed on compacted specimens, in accordance with the EN ISO 17892-10 standard [48], using an electronic normal loading system to ensure precise control of the applied stresses (ranging from 50 to 200 kPa) (Fig. 5). The testing apparatus continuously recorded the shear forces and the vertical/horizontal displacements, enabling the calculation of sample dilatancy. This approach not only allows for the assessment of granular interactions in soil-drilling waste-lime mixtures, but it also provides a direct measurement of cohesion (c) and internal friction angle (φ), which are key parameters for the geotechnical design of road construction materials, as highlighted in recent studies.

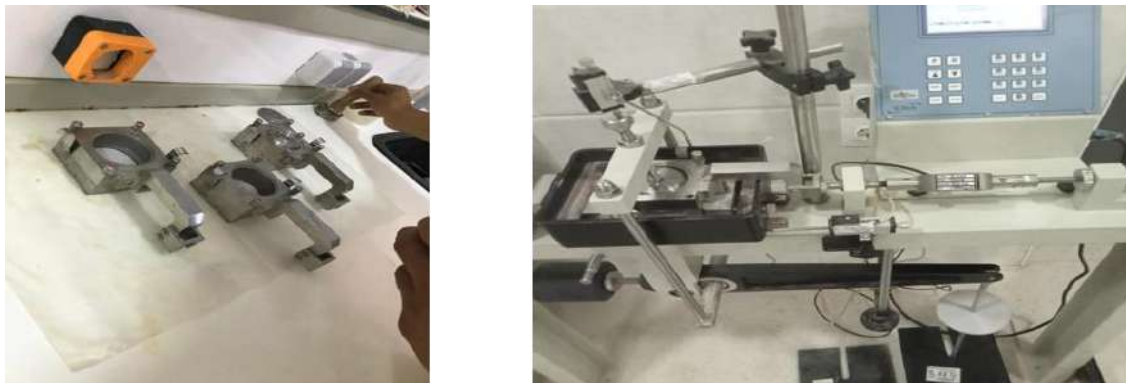


Fig. 5. Direct shear test

3. Results and discussion

3.1 Unconfined Compressive Strength

The compressive strength results for mixtures of gypseous-calcareous soil and DW, without the presence of lime, are presented in Fig. 6. The progressive incorporation of DW leads to a significant decrease in compressive strength. At 5% DW, the impact on strength remains moderate, with reduction of about 36% compared with the reference mixture (SDW0L0) at 90 days of curing. At 10% DW, a marked drop in strength is observed, with a reduction of approximately 50%, indicating more pronounced structural disruption within the soil. Finally, at 15% DW, the compressive strength of the soil continues to decrease, reaching a reduction of about 57%. This behavior can be attributed to the increase in porosity and the disruption of internal bonds within the material's matrix, due to poor chemical compatibility between soil and DW. These observations are consistent with the work of Balegha et al. [40], who demonstrated that the uncontrolled addition of industrial residues can lead to a degradation of the mechanical properties of soil.

In contrast, the addition of 4% lime to the mixtures enhances the mechanical performance, as shown in Fig. 7. The mixtures containing DW exhibited superior compressive strength in comparison to the SDW0L4 mixture at 90 days of curing. Specifically, SDW5L4 shows an increase in compressive strength of approximately 60%, SDW10L4 exhibits a gain of about 47%, and SDW15L4 presents an improvement of around 18% compared with SDW0L4. This enhancement can be attributed to the pozzolanic reactions of lime and calcium hydroxide with the siliceous and aluminous compounds found in treated DW in the presence of water. This forms new compounds that have cementitious properties, such as calcium silicate hydrate (C-S-H) and calcium aluminate

hydrates (C-A-H), strengthening the soil and reducing internal defects. These results agree with those reported by Daheur et al. [41], which highlighted that the use of hydraulic binders promotes the formation of stable silico-aluminous phases, thereby increasing the overall mechanical strength. This behavior is consistent with the study of Tayebi et al. [49] where it was observed that the use of red brick waste (RBW) as an addition significantly improved the compressive strength of soil. Al-amoudi et al. [50], who also reported an increase in the compressive strength of calcareous marl soil with the addition of stabilize such cement and lime.

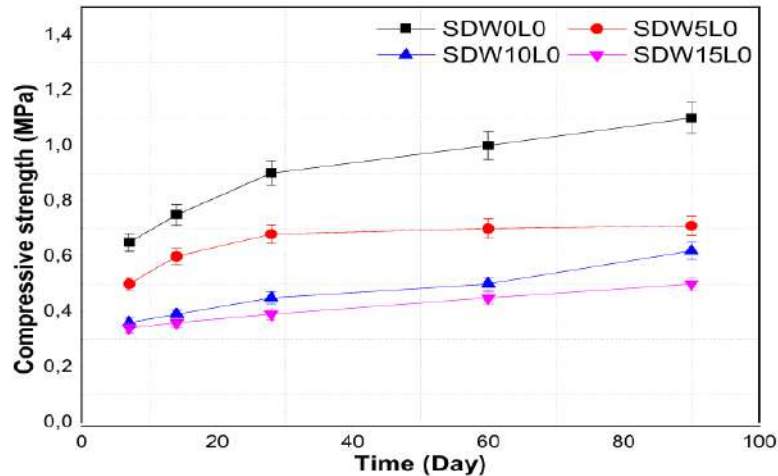


Fig. 6. Evolution of the compressive strength of soil-drilling waste mixtures without lime

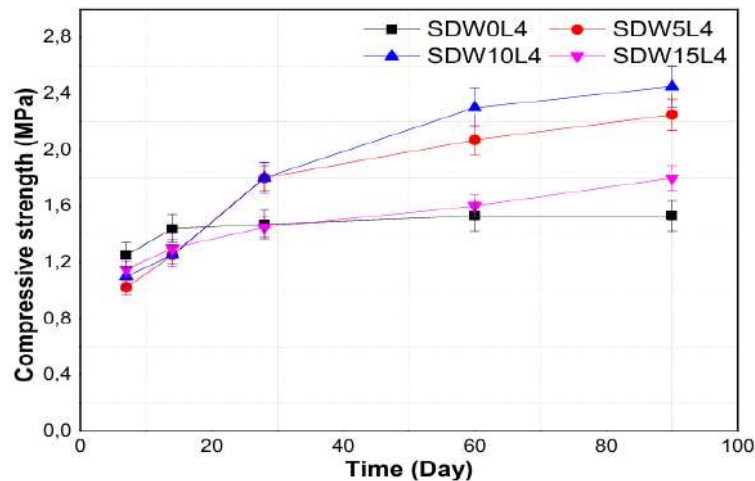


Fig. 7. Evolution of the compressive strength of soil-drilling waste mixtures with lime

3.2 Sensitivity to Water

Immersion tests revealed that samples without lime rapidly disintegrate in a humid environment, indicating weak internal cohesion and heightened water sensitivity (Fig. 8(a)). In contrast, samples with 4% lime maintain their structural integrity after immersion and show a progressive increase in strength after immersion, reaching a maximum at 10% DW content, as shown in Fig. 8(b) and Fig. 9. Compared with the reference mixture SDW0L4, the compressive strength after immersion increased by about 24%, 41% and 25% for SDW5L4, SDW10L4 and SDW15L4, respectively. This improvement could be attributed to the effect of lime, which reduces water sensitivity through the formation of insoluble cementitious products, such as calcium silicate hydrates (C-S-H), that reinforce the material's structure. The results of the current study are consistent with the research carried out by Moayyeri et al. [51], Goual et al. [52] and Abdulrasool et al. [53], which focused on different types of hydraulic binder and lime. Their findings indicate that the use of cement, fly ash, micro silica and lime considerably reduced the adverse effects of moisture in soils. This

phenomenon has also been noted in previous research regarding soil stabilization using different types of agricultural residues ash, such as carried out by Alavéz-ramírez et al. [54], Sharifi et al. [55].

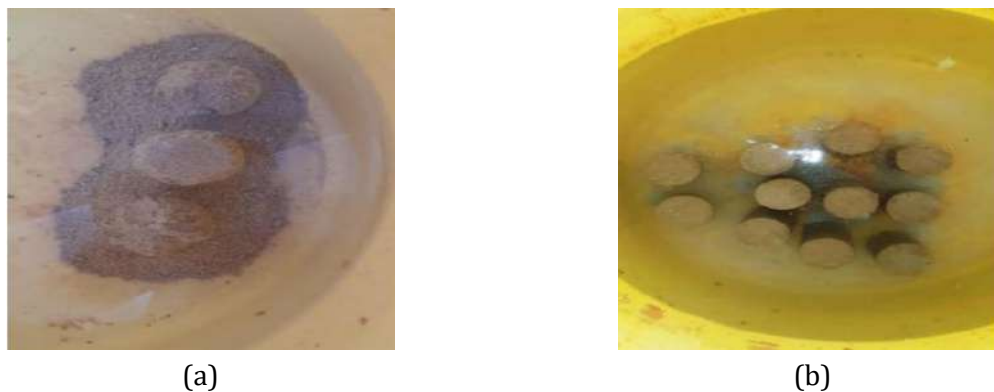


Fig. 8. Immersion test results: (a) specimens without lime that disintegrated after immersion and (b) specimens with lime that retained structural integrity after immersion

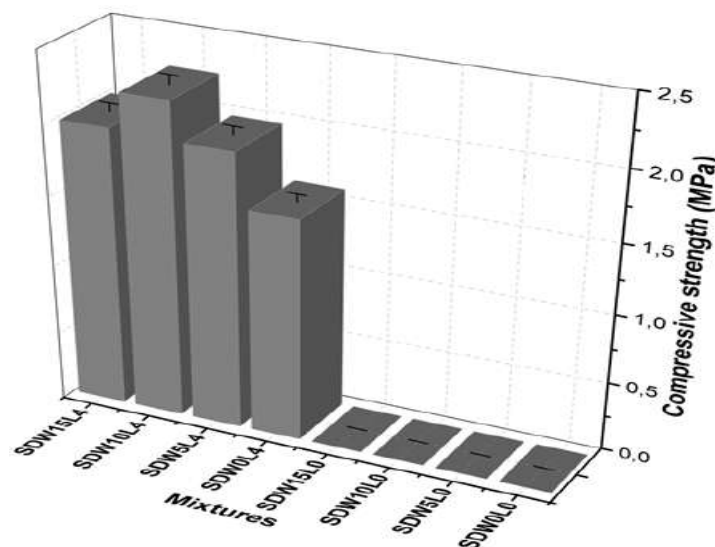


Fig. 9. Evolution of the compressive strength of soil-drilling waste mixtures after immersion tests

3.3 California Bearing Ratio (CBR)

The variety of CBR values of mixtures with and without lime is shown in Fig. 10. The addition of DW without lime significantly reduces the CBR index. When the amount of DW is increased, the CBR index of mixtures is decreased. Therefore, mixture SDW15L0 gave the lowest CBR index of all, with a value of 35%. This decrease in bearing capacity can be justified by the increased porosity and degradation of the granular bonds of mixtures with DW. In contrast, incorporating DW with 4% lime significantly improves the bearing capacity of soil. The SDW10L4 mixture gave the best CBR index, achieving a value of 75%. These results confirm the chemical reactions between the lime and the reactive compounds present in the mixtures with DW, which produce cementitious products that improve the bearing capacity of the soil. These results agree with those reported by Sadeghi et al. [38], who observed a similar improvement in mechanical properties during the stabilization of gypsum soils using sustainable additives for road construction.

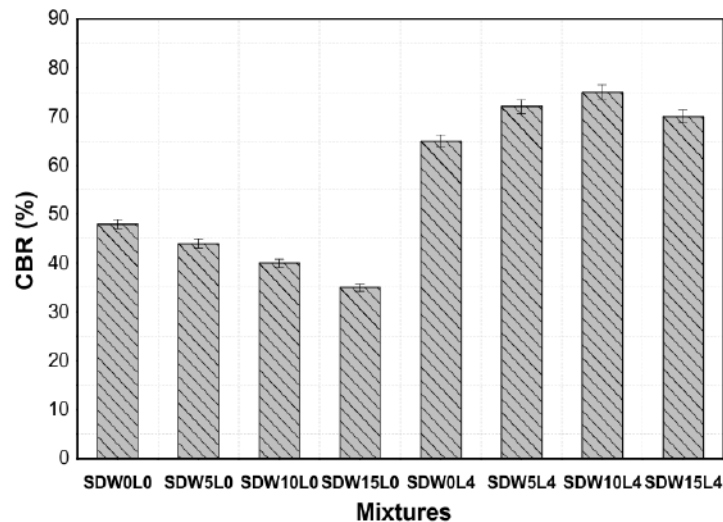


Fig. 10. California bearing ratio values of soil-drilling waste mixtures with and without lime

3.4 Shear Strength

3.4.1 Mohr–Coulomb Failure Line

Fig. 11 shows the Mohr–Coulomb envelopes corresponding to the mixtures without and with lime. The relationship connecting maximum shear strength (τ_{max}) to the normal stress (σ_n), for different soil contents (0, 5, 10, and 15% DW) can be written according to Eq. (1):

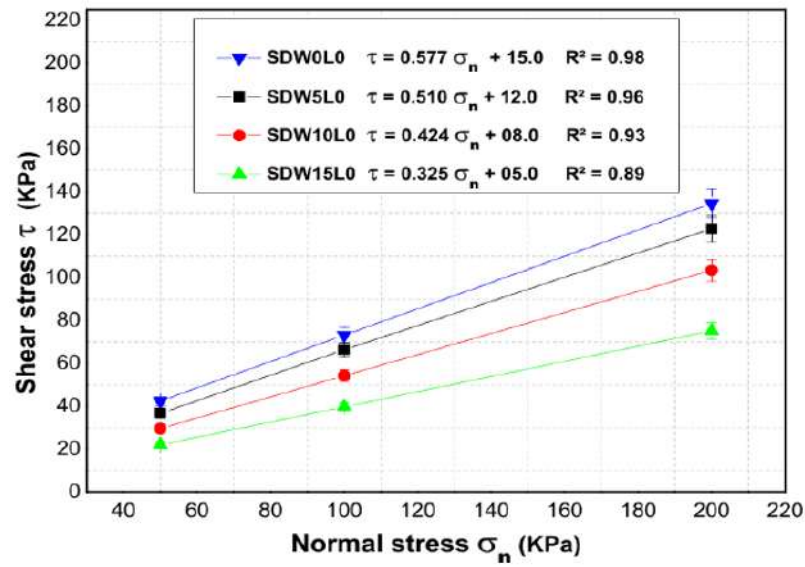
$$\tau_{max} = \sigma_n \tan \phi + c \quad (1)$$

Where; c : cohesion, ϕ : internal friction angle, σ_n : normal stress. These results indicate that adding DW with lime increases the slope of the failure line of soil compared to that without lime. It should be noted that the SDW10L4 mixture has the steepest failure slope.

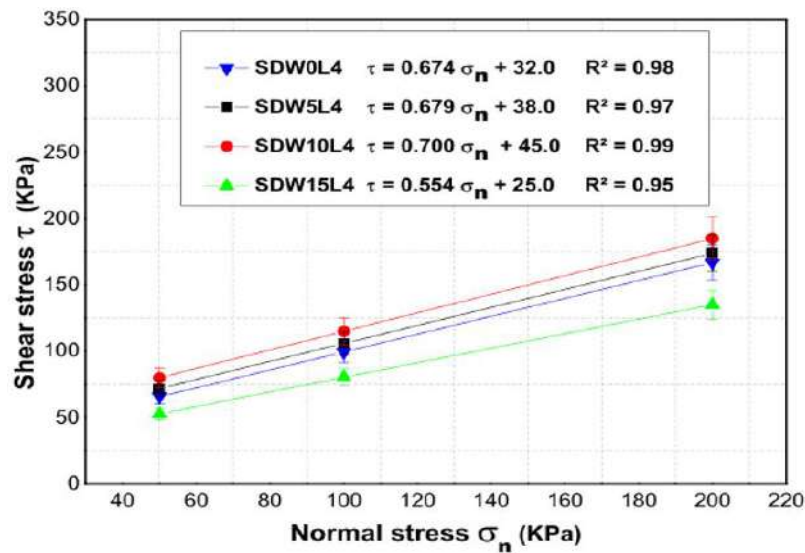
3.4.2 Cohesion and the Internal Friction Angle

The incorporation of DW into soil mixtures progressively degrades cohesion (c) and friction angle (ϕ), as illustrated in Fig. 12. For example, in the SDW15L0 mixture, c drops to 5 kPa and ϕ to 18°, compared to the SDW0L0 mixture (15 kPa and 30°, respectively), reflecting microstructural disruption caused by heterogeneous particles. This aligns with the findings of Shah et al. [56] regarding fuel oil contamination, where cohesion decreased (-66%) and the angle of internal friction decreased (-23%) of soil. The addition of 4% lime significantly enhances cohesion (c) and the friction angle (ϕ), increasing from 32 kPa and 34° (mixture SDW0L4) to 45 kPa and 35° (SDW10L4). This stems from the pozzolanic reactions between lime and DW silica/alumina, forming calcium silicate hydrate gels that densify the soil matrix. On the other hand, lime neutralizes sulfates via stable gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) formation, while DW particles act as fillers, lowering effective porosity. Al-muaythir et al. [57] found similar results when studying of various types of industrial waste additives to enhance the characteristics of soil. They observed that these additives react with soil minerals in the presence of water to create cementitious compounds, including calcium silicate hydrates (CSH), which serve to link the soil particles together and improve cohesion. The fine particles of the additives occupy the spaces between soil particles, resulting in denser packing and heightened interparticle friction, thereby increasing the internal friction angle.

Al-muaythir et al. [57] found similar results when studying of various types of industrial waste additives to enhance the characteristics of soil. They observed that these additives react with soil minerals in the presence of water to create cementitious compounds, including calcium silicate hydrates (CSH), which serve to link the soil particles together and improve cohesion. The fine particles of the additives occupy the spaces between soil particles, resulting in denser packing and heightened interparticle friction, thereby increasing the internal friction angle.

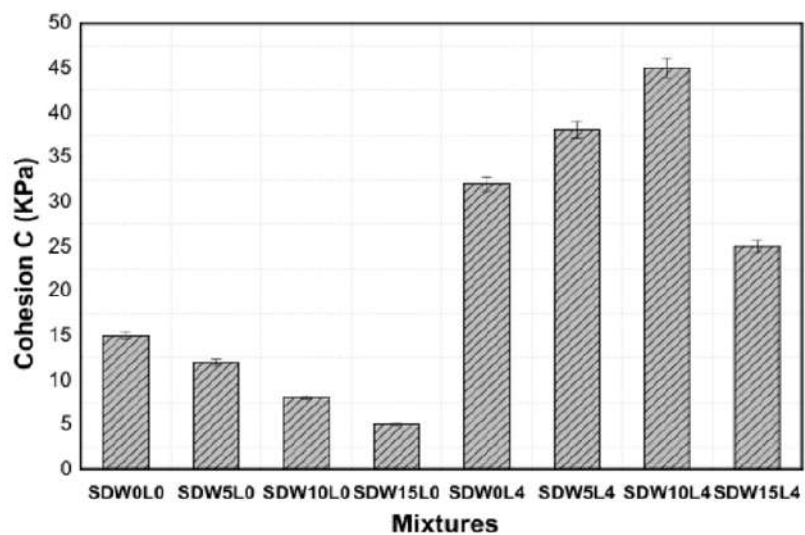


(a)



(b)

Fig. 11. Failure envelopes for the direct shear of soil-drilling waste mixtures: (a) without lime and (b) with lime



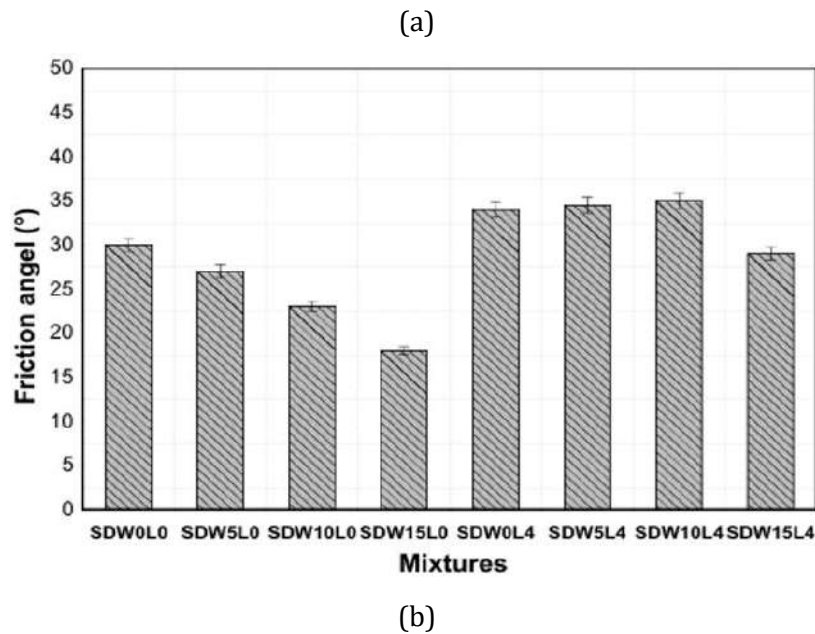


Fig. 12. Variation of shear parameters of tested mixtures without and with lime: (a) cohesion; (b) the angle of internal friction

3.5 Microstructure analysis

The microstructural analysis by Scanning Electron Microscopy (SEM) of gypseous-calcareous soil and DW mixtures without lime, shown in Fig. 13, reveals that, at 5% of DW, the microstructure remains relatively homogeneous, with only a slight increase in porosity compared to pure gypseous-calcareous soil.

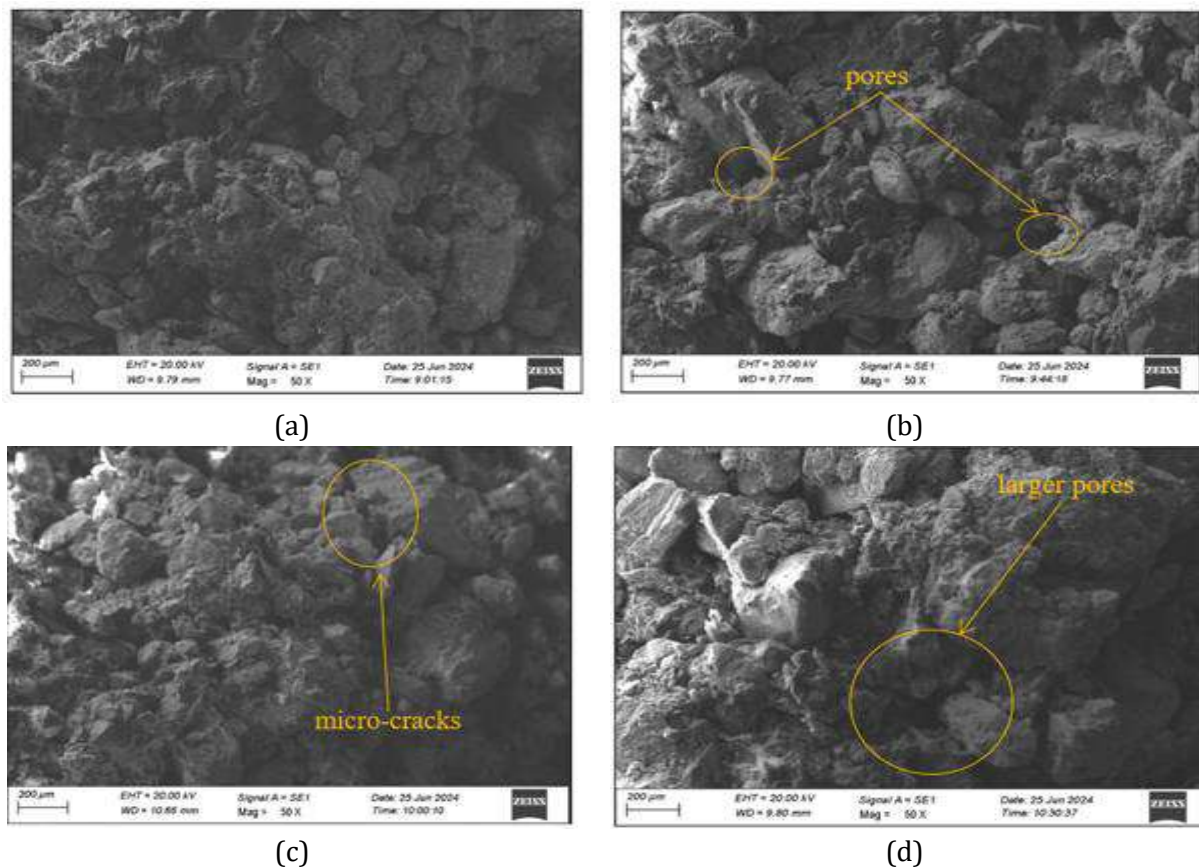


Fig. 13. SEM micrographs of mixtures without lime: (a) 0% DW; (b) 5% DW; (c) 10% D; (d) 15% DW

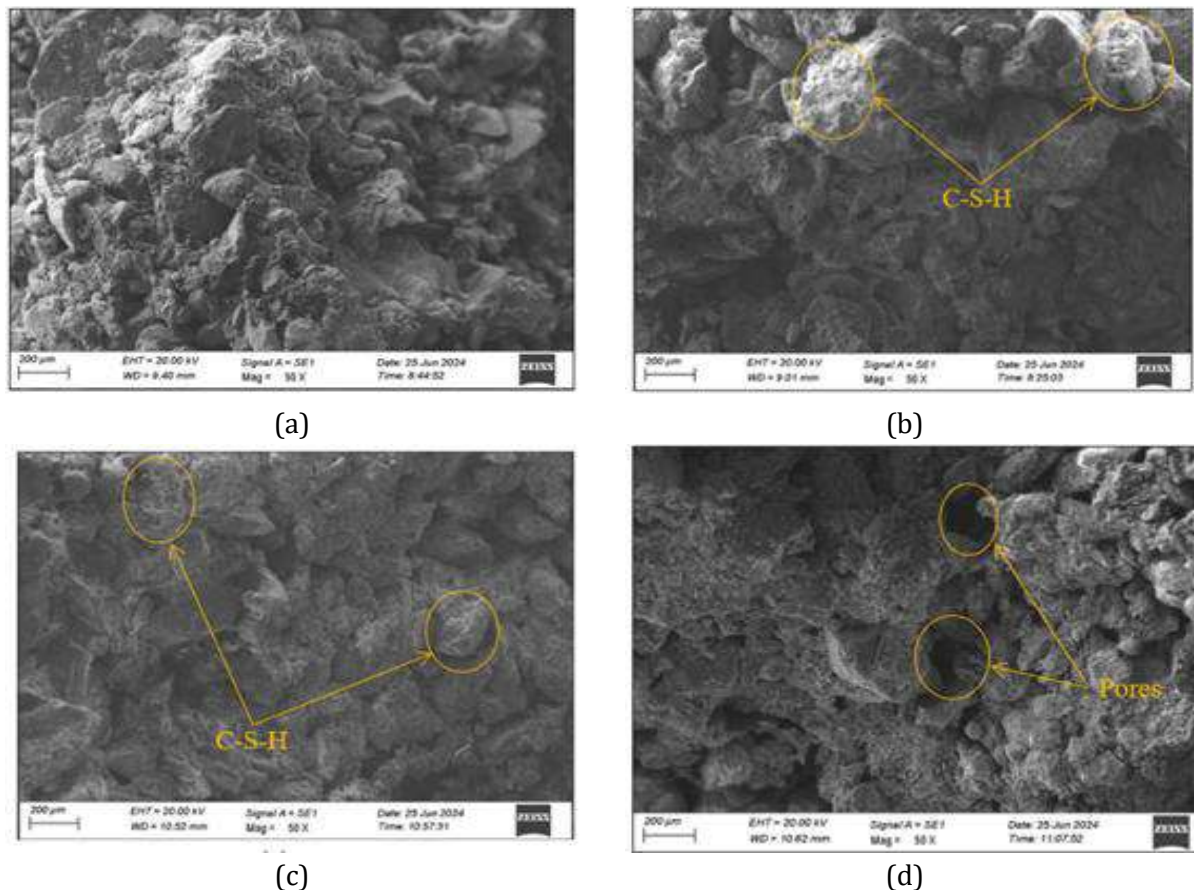


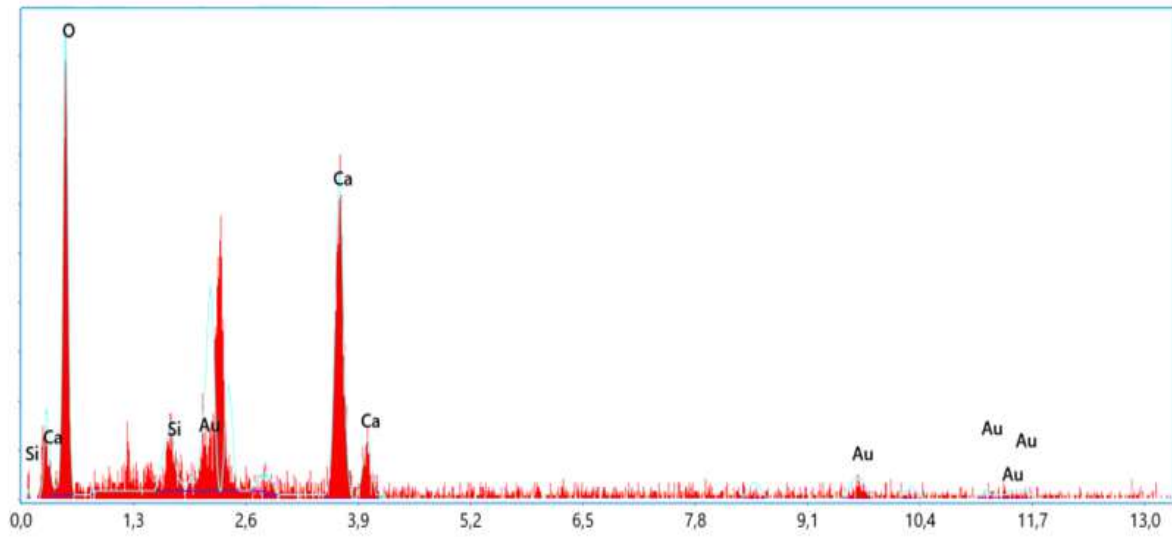
Fig. 14. SEM micrographs of mixtures with lime: (a) 0% DW; (b) 5% DW; (c) 10% DW; and (d) 15% DW

The soil particles appear well integrated without significant alteration of the binding phases. At 10%, porosity increases markedly, accompanied by the emergence of micro-cracks around the soil particles, indicating a weakening of the chemical and mechanical bonds within the matrix. Finally, at 15%, the microstructure becomes heterogeneous, characterized by larger pores and interconnected fissures, which reveals an increased chemical incompatibility and a degraded integration of the soil particles. This phenomenon is explained by the presence of organic or mineral impurities from the petroleum waste, which disrupts the internal arrangement of particles and reduces the mechanical cohesion of the soil. These observations are consistent with the work of Ahmad et al. [58], who demonstrated that the incorporation of diesel and crude oil into soil lead to development of a hydrocarbon coating on the surfaces of the soil particles, which influenced their microstructure.

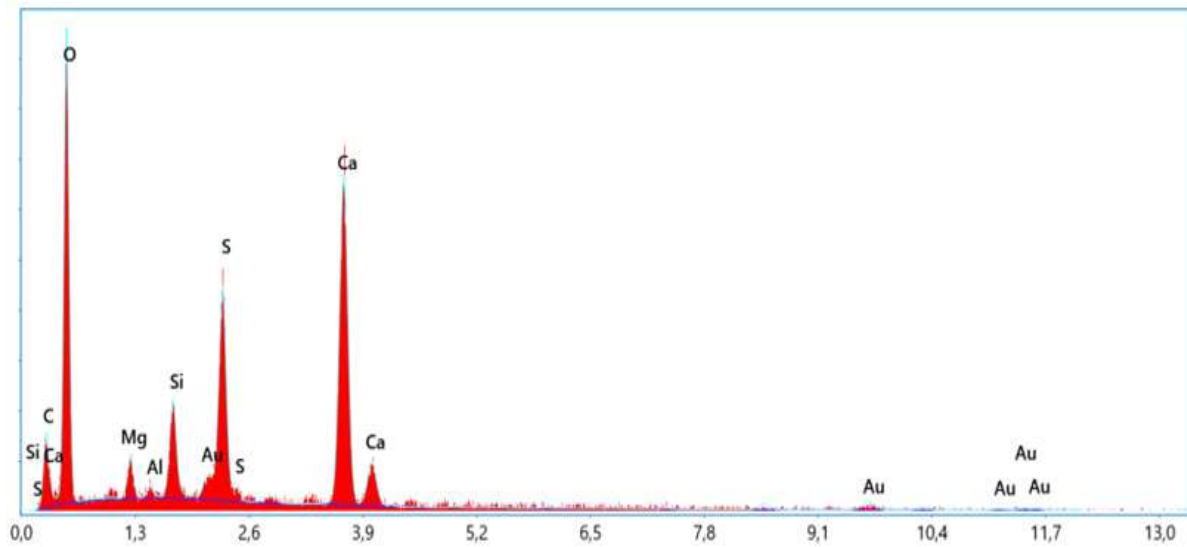
The microstructural analysis, conducted by SEM, reveals notably different microstructural results for the mixtures with 4% lime, as shown in Fig. 14. There was a marked improvement in the microstructure of all mixtures, resulting in a more homogeneous and compact matrix, as well as a significant reduction in internal voids and the waste particles appear properly integrated into the soil. This densification is attributed to the presence of calcium silicate hydrate, resulting from reactions between the lime and the siliceous compounds of treated DW. This result aligns with the conclusions reported by Oluwatuyi et al. [59], who observed similar effects when stabilizing hydrocarbon-contaminated soils using cement and lime.

Fig. 15 presents the Energy Dispersive X-ray Spectroscopy (EDX) analysis results for mixtures of gypseous-calcareous soil and DW with lime. It can be seen, from EDX recordings (a) and (b), that the mixtures SDW0L4 and SDW5L4 have high calcium and sulfate contents due to the nature of the soil and the presence of lime in the mixtures. For the mixture SDW10L4, the calcium content decreases due to its increased consumption during chemical reactions with siliceous compounds of DW. This chemical stabilization, observed by Ezeokpube et al. [60] through mineralogical

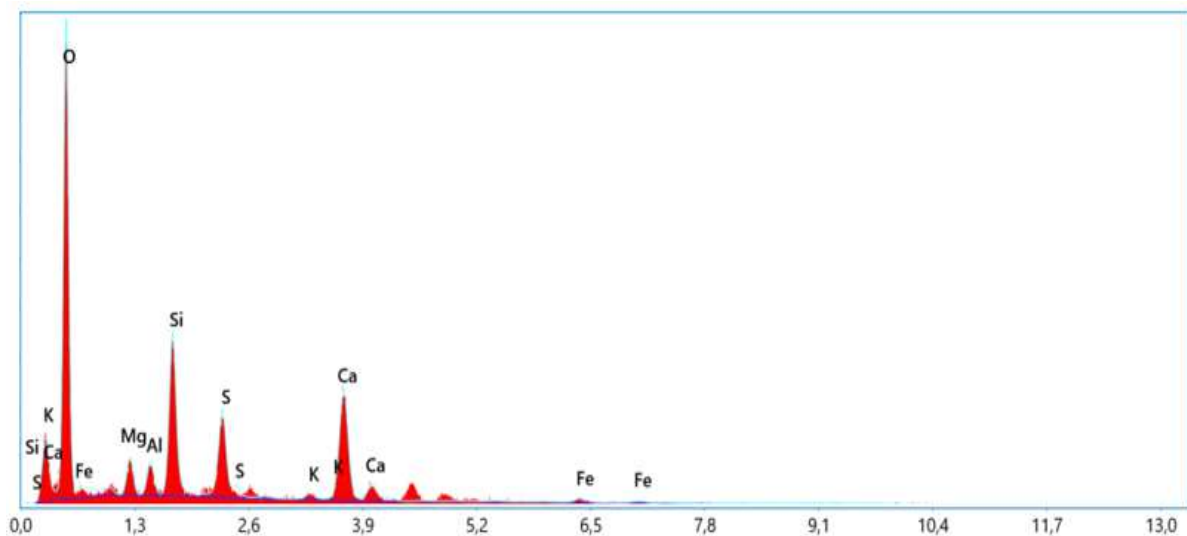
analyses, confirms the mitigation of structural disorders induced by hydrocarbon contaminants. Finally, mixture SDW15L4 on the EDX record shows a slight increase of sulfate, calcium and silicon peaks accompanied by very low chlorine and sodium contents.



(a)



(b)



(c)

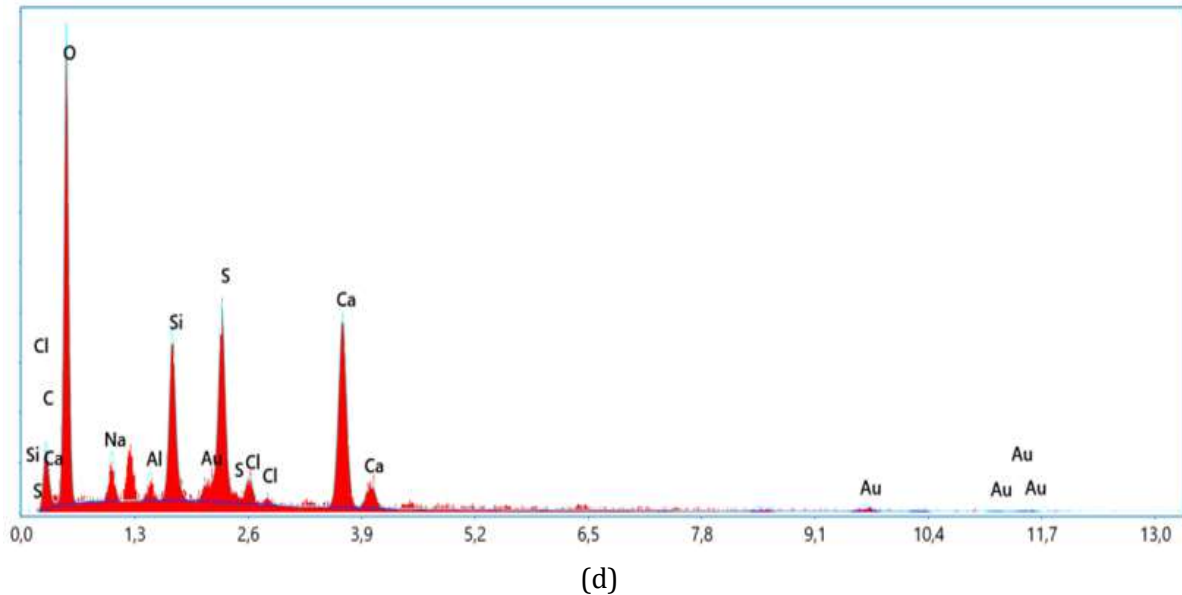


Fig. 15. EDX of mixtures with lime: (a) 0% DW; (b) 5% DW; (c) 10% DW; and (d) 15% DW

4. Conclusions

The results of this study demonstrate the technical and environmental feasibility of valorizing treated oil drilling waste, in combination with gypseous-calcareous soil, for road applications in arid regions. Based on the results of this experimental study, the main conclusions are as stated below.

The addition of DW without lime significantly decreased compressive strength, from 1.1 MPa in the pure soil (SDW0L0) to 0.5 MPa in the soil with 15% DW (SDW15L0). As for the CBR index, a decrease of 27% was observed, from 48% in the pure soil (SDW0L0) to 35% in the soil with 15% DW (SDW15L0). In direct shear tests, both cohesion (c) and friction angle (φ) decreased, regardless of the proportion of DW used. As a result, the soil becomes more sensitive to water and disintegrates quickly. SEM images demonstrated that the microstructure of soil containing oil drilling waste has higher porosity, causes a reduction in the mechanical properties.

The incorporation of 4% lime into mixtures containing DW significantly improved the compressive strength compared to mixture SDW0L4, reaching optimal values after 90 days of curing. Regarding the sensitivity to water, it was observed that the sample appears well integrated without alteration and compressive strength improved after immersion. Additionally, a 13% improvement in bearing capacity occurred, as CBR index increased from 65% (in mixture SDW0L4) to 75% (in the mixture with 10% DW). The cohesion and the friction angle improved proportionally, with an increase in the DW content of up to 10%, followed by a decrease, and then a further increase in the DW of up to 15%. The SEM-EDX analysis revealed that the presence of reactive oxides in DW (such as SiO_2) and the addition of lime (CaO) favors pozzolanic reactions, leading to the formation of cementitious compounds that reduce porosity and limit micro-cracks, thereby enhancing the soil texture and internal structural density, contributing to greater cohesion and resistance under load.

The SDW10L4 mixture achieved the highest performance, showing a compressive strength of 2.45 MPa and a CBR index of 75%. The parameters of cohesion (c) (45 kPa), and friction angle (φ) (35°) meet the requirements for Saharan road infrastructure, which is subject to extreme climatic conditions.

In summary, the usage of gypseous-calcareous soil with DW mixtures offers a dual benefit: From an environmental standpoint, this approach offers a sustainable solution for managing and reducing the ecological impact of petroleum waste, while optimizing the use of local resources in desert road infrastructure projects.

Future research should focus on evaluating the long-term durability of the stabilized mixtures under environmental conditions such as wetting-drying and freeze-thaw cycles, as well as

assessing their performance under field conditions. Such investigations would help confirm the suitability of oil drilling-waste-based mixtures for road construction applications.

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Experimental investigation on the bond behavior of the GFRP bars with one-part geopolymer concrete and conventional concrete

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Abstract

The current study investigates the bond behavior of steel bars and GFRP in ordinary Portland cement concrete (OPCC) and one-part geopolymer concrete (OPGC). The maximum pullout load as well as features at failure were examined in relation to the kind of concrete, bar diameter, and bar type. In one-part geopolymer concrete along with regular concrete, the pullout strengths and also adhesive properties of GFRP bond specimens were contrasted with steel bar specimens. An energy-based identical bond strength strategy was taken into consideration in order to suggest the bond durability at peak load. This showed that a bigger diameter results in a faster rate of strength decline, along with a decline in bond performance. Pullout load was observed in bond specimens with smaller diameters, where bond strength regulated the shear strength within the GFRP bars along with ribs. In specimens of conventional concrete and one-part geopolymer concrete with diameters of 12 and 16 mm. There was evidence of concrete split failure, where the shear strength of the concrete determines the bond strength. The bond strength is positively impacted by rises in concrete's compressive strength, which spans from 14.57 to 32.12%.

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1. Introduction

The building sector is vital to the economic success of any emerging nation, including India. However, its operations also have serious negative effects on the environment, which calls for a review of the sector's Environmental Management Practices (EMPs). High energy consumption and an unchecked increase in greenhouse gas emissions are the results of increased cement manufacturing to support the nation's concrete infrastructure development [1]. The most common cementitious material is Portland cement, but producing one ton of Portland cement emits nearly one ton of CO₂ [2], as well as the cement production process is responsible for between 5 and 8% of global CO₂ emissions. One of the main causes of global warming and its related effects on the ecosystem is the emission of greenhouse gases into the atmosphere, such as CO₂. As a result, creating a green cementitious substance that emits substantially less CO₂ during manufacture is essential [3]. Industrial by-products such as fly ash, ground granulated blast furnace slag (GGBS), and silica fume are examples that have been incorporated to concrete for supplementary cementitious materials (SCMs) in recent decades in an effort to improve the material's qualities and lessen its carbon footprint [4]. However, because these materials must react with the cement hydration product to produce the binding property, they can only be utilized as partial substitutes of OPC. Therefore, there is a lot of interest in creating substitute binding materials which can preserve the necessary engineering

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qualities while lowering the carbon footprint as well as embodied energy of final goods (like concrete). Alkali-activated binders (AABs) or even geopolymers are becoming a competitive alternative to OPC in the modern era because their mechanical and durability qualities are nearly equal to, and frequently better than, those of OPC-based mortars along with concretes, and they are less vulnerable to environmental effects [5]. Since AABs provide a way to value industrial by-products and do away with the need for OPCs, their increased use is also desirable. One practical strategy for creating environmentally friendly and sustainable building materials is to use these by-products, which are high in alumina-silicates (such as GGBFS, fly ash, etc.), as source material [6]. By using these resources in the production of concrete, it is possible to reduce the pollution of the air around it, soil, and water caused by industrial wastes as well as the negative environmental effects of increased cement production [7]. However, only about 25% and 65% of the FA and GGBFS generated worldwide, respectively, are thought to be used for other purposes at the moment [8]. Traditional AABs, such as the so-called "geopolymers," are created as two-part mixes through the chemical reaction of a concentrated alkaline solvent with an aluminosilicate substrate (a precursor) material, which is typically taken as a sort of fine powder [9]. The concentrated aqueous alkali silicates or otherwise hydroxides that make up the majority of the activator solutions [10] are generally corrosive and extremely viscous, making them difficult to utilize. Alkali activator solution preparation and use also call for extra care, strict safety precautions, and qualified personnel. As a result, the solution is frequently made 24 hours before it is mixed with a solid precursor. Mass production, broad commercialization, and the use of traditional two-part AABs are all greatly impacted by this. This problem, which involves managing the activator solutions, has motivated researchers to use solid activators to create one-part geopolymers (OPG) or one-part alkali-activated binder materials (OP-AAMs). One-part AAMs (OP-AAMs) have the distinct benefit of being simpler and safer to produce and use than their two-part counterparts, even if they are also shown to have functionally identical features to OPC-based materials. OP-AAMs are therefore a viable substitute for pre-cast and cast-in-situ applications in the building sector [1]. On the other hand, the most widely used building material for infrastructure development is steel-reinforced concrete. However, almost 7% of all CO₂ emissions in the atmosphere, which cause global warming, are caused by the manufacture of cement [11]. Furthermore, it is projected that by 2060, 6000 million of tons of Portland cement, or 1.5 times the amount produced now—will be consumed globally each year. With an 80% lower carbon footprint than cement-based concrete, geopolymer concrete (GPC) can be used in its stead [12]. The durability of steel-reinforced concrete structures has also reduced significantly due to a number of deteriorating mechanisms, with reinforcement corrosion emerging as the most important and necessitating repair. Fiber-reinforced polymer (FRP) composite bars since, unlike steel reinforcement, they do not corrode in challenging conditions. Therefore, a more sustainable and long-lasting building material that is absent from traditional reinforced concrete structures would be supplied by fiber-reinforced polymer (FRP) bars reinforced in one-part geopolymer concrete. When researchers examined the bonding of GPC and OPCC to steel bars, they discovered that GPC has a stronger binding than OPCC if the two forms of concrete's compressive strengths were equal [13][14]. Additionally, it has been shown that the stress on the bond versus slip curve of the two concretes and failure pattern are identical [15]. The bond behavior of GFRP bars using sea sand including seawater has been the subject of a few investigations [16], and [17]. After 28 days, [18] found that the binding strength of the GFRP bar using traditional and seawater concrete was little different, and reported that this difference was 15%. The specimens' binding strength decreased by 20% and 8%, respectively, following 250 days of exposure to seawater as well as its wet-dry cycles, and showed that the bond strength of the saltwater sea-sand concrete had dropped by 3% when comparing the bond endurance of GFRP reinforced conventional and seawater sea-sand concrete. Because there are no regulations for the surface treatment of FRP bars, unlike steel, there are numerous varieties of FRP bars available, such as sand-coated (SC), grooved, helically wrapped (HW), ribbed, among others. The bond strength of OPCC strengthened using SC as well as HW GFRP bars is the same, although being marginally lower than that of steel bars [19]. [20] discovered that the HW GFRP-reinforced OPCC performed better than the other surface

arrangements. Furthermore, they recommended strengthening concrete rather than changing the surface arrangement because the former has a greater impact on bond strength. The bond strength of OPCC using ribbed GFRP bars was higher than that of HW and HW-SC bars, according to [21].

Despite many advancements in the characterization of materials, research on the application of OPGC as a structural member remains limited. In contrast, conventional concrete with steel has been extensively studied. The scarcity of OPGC experimental data with respect to the response of load deformation, particularly when reinforced with GFRP bars as an alternative material, highlights a crucial research gap for the study. Also, the literature research indicates that there is a significant knowledge gap on how various FRP bar types, surface designs, and compressive strength of concrete affect the bonding performance using geopolymer concrete (GPC) supplemented with FRP bars. Few studies have been conducted on the bonding properties of GFRP bars in GPC, yet none have specifically looked at how well they work in one-part geopolymer concrete (OPGC). The current study experimentally investigates the bond behavior of GFRP bars embedded in conventional concrete with one-part GPC in order to close this gap. Additionally, a comparison of steel and GFRP bars is carried out, paying close attention to the impacts of concrete strength, bar type, and bar diameter.

2. Program for Experimentation

2.1 Materials

2.1.1 Design of a Concrete Mix

The current study used sustainable industrial waste materials, such as fly ash of class F coming from thermal power stations as well as ground granulated blast furnace slag (GGBS) leftover from steel production, as precursors to create one-part geopolymer concrete. The chemical makeup of fly ash as well as GGBS is presented in Table 1. Their relative specific gravities are 2.15 and 2.75. Powdered anhydrous sodium metasilicate is utilized as an activator. Gravel is used as coarse aggregate (CA) and M sand as fine aggregate (FA) in this study. The physical attributes of the FA and CA that were obtained in accordance with IS 2386-1967, part 3, are listed in Table 2. Three sets of specimens were taken into consideration for each test, and the average results are listed in Table 2. The IS-383-2016 code was followed in the sieve analysis of FA and CA. FA was discovered to be a member of Zone II. Both aggregates meet IS 383-1970's codal requirements. Table 3 displays both the normal concrete of grade M40 and the ideal mix portion of one-part geopolymer concrete with the same grade. Three 150mm cubes were cast and tested for compressive strength. Normal concrete cubes were kept in water for curing for 28 days, and OPGC cubes were cured under ambient conditions. The average values of the normal M40 grade concrete and optimum one-part geopolymer concrete were 44.71MPa and 54.36 MPa, respectively, and the calculated standard deviation is tabulated in Table 4.

Table 1. XRF analysis of the chemical makeup of FA and GGBS in % weight

Material	Al ₂ O ₃	BaO	CaO	Cr ₂ O ₃	Fe ₂ O ₃	K ₂ O	MgO	MnO	Na ₂ O	P ₂ O ₅	SO ₃	SiO ₂	TiO ₂
FA	25.31	0.07	12.25	0.01	9.9	0.64	1.14	0.14	0.32	0.51	0.57	46.04	1.46
GGBS	13.8	0.06	42.13	0	0.58	0.32	5.76	0.27	0.2	0.034	3.33	32.92	0.57

Table 2. The physical characteristics of F.A. and C.A.

Sample	Specific gravity	Absorption of water (%)	Fineness modulus (%)
Coarse aggregate	2.73	0.83	4.42
Fine aggregate	2.56	2.44	3.78

Table 3. Mix proportion of M40 grade and one-part geopolymers concrete

Type of concrete	Cement (kg m ⁻³)	Fly ash (kg m ⁻³)	GGBS (kg m ⁻³)	C.A (kg m ⁻³)	F.A (kg m ⁻³)	Water (kg m ⁻³)	Activator (kg m ⁻³)	Superplasticizer (kg m ⁻³)
M40 grade of concrete (OPCC)	440			918	754	176		
One-part geopolymer concrete (OPGC)		190.8	224.19	925.15	697.41	180	56.25	3.6

Table 4. Compressive strength outcomes

Concrete type	Compressive strength (MPa)				SD
	Sample1	Sample2	Sample3	Mean	
M40 grade of concrete (OPCC)	45.33	46	42.8	44.71	1.68
One-part geopolymer concrete (OPGC)	58.66	48.88	55.55	54.36	4.99

2.1.2 Properties of Rebar

GFRP along with Fe415 HYSD bars of steel had been used in this investigation as cement concrete reinforcement. GFRP bars are composed of 15% to 25% resin and 75% to 85% glass fiber by weight[16]. Table 5 lists the mechanical characteristics of steel and GFRP bars. In accordance with ASTM rules, the manufacturer conducted a tension test upon GFRP bars.

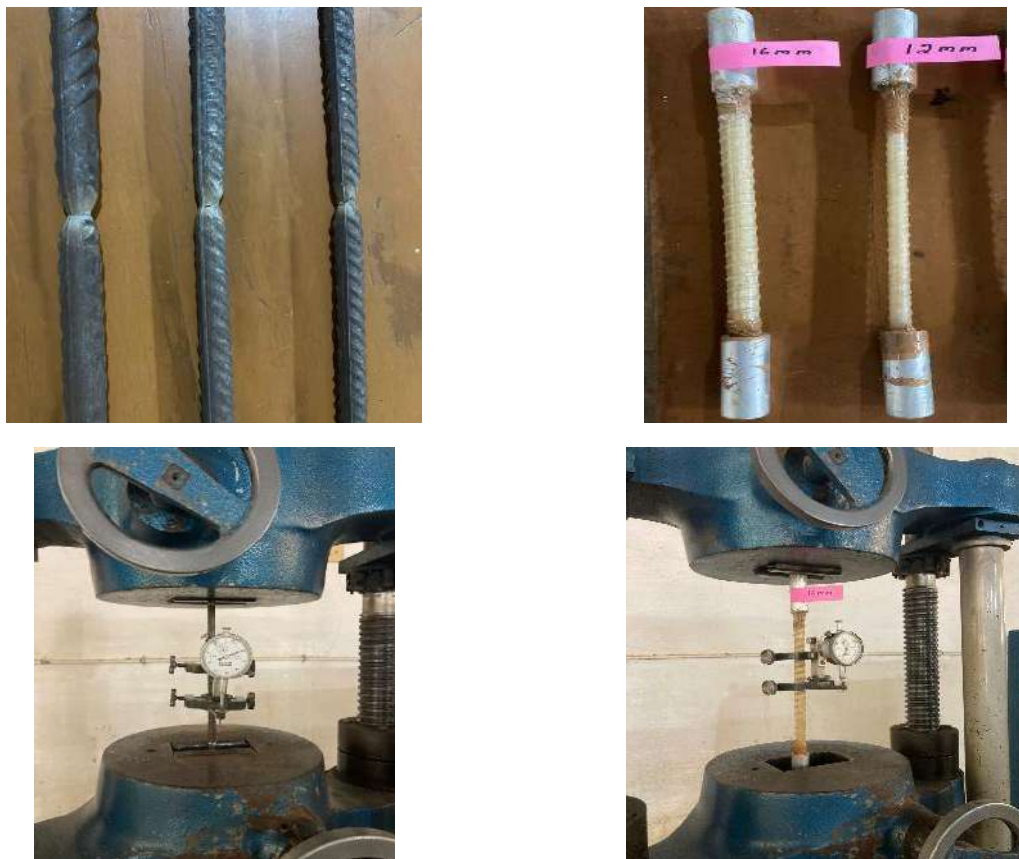


Fig. 1. Test setup for steel bars and GFRP bars

400mm length of samples were obtained in order to perform mechanical testing and evaluate the specimen's ultimate elongation, elastic modulus, and tensile strength; Table 5 shows the typical outcome. The test arrangement used in the lab is depicted in Figure 1. In our lab, the tension test on steel bars was performed utilizing 600mm-long samples in accordance with IS: 1786-20008 codal requirement. Additionally, Table 5 tabulates the computed yield strength, ultimate strength, and elastic modulus average values.

Table 5. Steel's and GFRP bars' mechanical characteristics

Types	Bar's nominal diameter(mm)	Measured diameter of bar (mm)	Yield strength (MPa)	Ultimate tensile strength (MPa)	Elastic modulus (MPa)	Ultimate percentage of elongations
GFRP	10	9.94	-	978.56	51.76	3.31
GFRP	12	11.94	-	963.46	52.46	2.98
GFRP	16	15.86	-	938.75	53.24	3.12
Steel	10	10.06	463.24	549.78	200.10	-
Steel	12	12.10	508.66	658.13	197.78	-
Steel	16	16.12	478.54	618.78	204.34	-

2.2 Preparation of Pullout Specimens

The effectiveness of the bond between control concrete supplemented with one-part geopolymer concrete with steel bars and GFRP bars was assessed in accordance with the IS: 2770 (Part I)-2007 code. A total of sixteen sample cubes with steel and GFRP bars measuring 150 mm were manufactured. The testing parameters included the type of concrete, the type of reinforcement, and the diameter of the bar. The GFRP bars' poorer transverse strength causes them to rupture under tensile force at the holding position. In order to capture an effective bond response throughout the pullout test, a steel tube is installed on the supporting end. In order to keep the bar's intended embedding length, a PVC tube is also placed on the other side. The pullout specimens and prepared GFRP bars that were cast in accordance with IS: 2770 (Part I)-2007 are displayed in Figure 2 and left to cure naturally.



Fig. 2. GFRP bars and steel bar pullout specimens

2.3 Test Setup

A 100-ton Universal Testing Machine (UTM) using a pullout frame is shown in Figure 3. An upper plate having a hole larger than the diameter of the bar as well as a lower plate connected by a steel rod make up the testing frame. The test samples had been stored in the frames in order that the UTM's lower jaw held the steel rod in place while its upper jaw was employed to draw the bar. The pullout test was performed in compliance with IS: 2770 (Part I)-2007 to ascertain the loaded end slip, and the loading rate was restricted to less than 22000N/min. The earlier investigations used a similar setup[22].

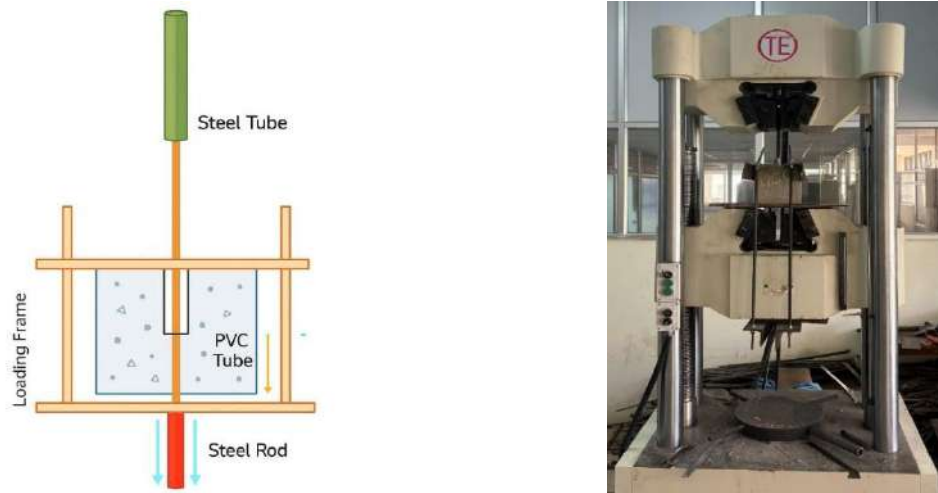


Fig. 3. Pullout test setup: UTM picture and pullout frame schematic diagram

3. Results and Discussions

3.1 Maximum Slide and Pullout Load

The maximum pullout load (P_{max}) as measured by experiment and measurement of slip (S_m) at the loaded end for each of the 16 samples considered are listed in Table 6 of the investigation. Additionally, the average pullout load is displayed and listed in Table 6 of the investigation. Additionally, the mean pullout load is displayed. slip, as well as bond stress values for each of the experimental variables. Figure 5 illustrates how bond specimens exhibit a rise in pullout load as the bar's diameter increases. One-part geopolymer concrete showed a noticeable 14% increase in load when compared to regular concrete. Furthermore, GFRP specimens demonstrated an average 10% decrease in pullout capability in regular concrete when compared to steel bond specimens, but in one-part geopolymer concrete, GFRP bars exhibited almost the same pullout load compared to steel bars[23].

Table 6. Pullout test results

Sl.No.	Specimen	Max Load (kN)	Avg Max Load (kN)	Max Stress (kN/mm ²)	Avg max Stress (kN/mm ²)	Slip (mm)	Avg Slip (mm)	Failure Mode
1	12CCS-1	48.65	44.42	8.60	7.85	13.67	12.49	S
2	12CCS-2	40.20		7.11		11.30		S
3	12GPS-1	51.15	50.74	9.04	8.96	13.76	13.37	S
4	12GPS-2	50.34		8.89		12.98		S
5	12CCG-1	39.70	39.10	7.02	6.91	9.99	9.69	S
6	12CCG-2	38.50		6.81		9.40		S
7	12GPG-1	50.32	49.65	8.90	8.77	10.20	10.09	S
8	12GPG-2	48.98		8.66		9.98		S
9	16CCS-1	49.35	47.92	6.54	6.35	11.72	11.71	S
10	16CCS-2	46.50		6.16		11.70		S
11	16GPS-1	53.40	52.70	7.08	6.98	15.86	18.21	S
12	16GPS-2	52.00		6.89		20.57		S
13	16CCG-1	44.65	41.95	5.92	5.56	9.24	9.39	S
14	16CCG-2	39.25		5.20		9.54		S
15	16GPG-1	52.40	52.00	6.95	6.89	11.07	10.98	S
16	16GPG-2	51.60		6.84		10.89		S

3.2 Failure Modes of Pullout Specimens

The forms of failure for each tested specimen are also listed in Table 6, with split (S) failure having the most prevalent type. As seen in Figure 4, bond specimens with bars of 12 and 16 mm in diameter have displayed signs of split failure. Shear cracks in the concrete formed

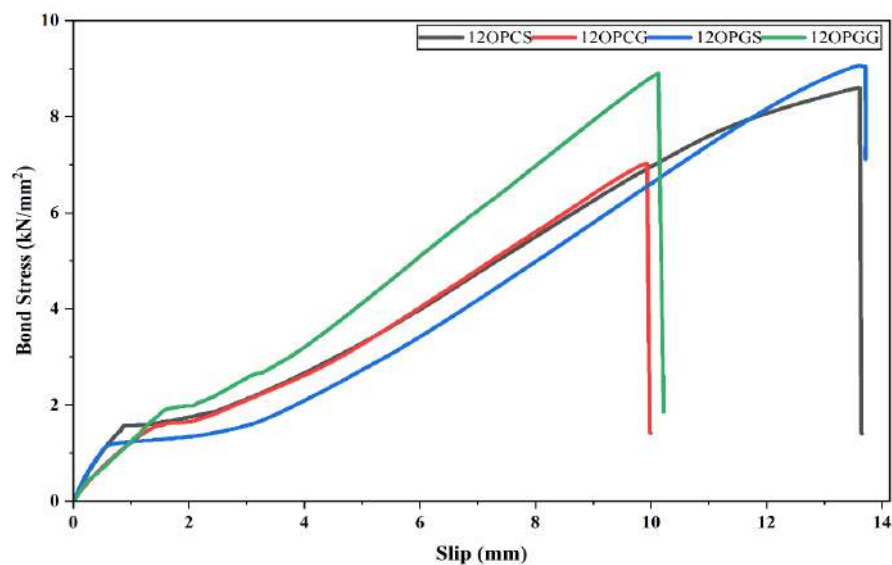
where the bar as well as the surrounding concrete met because of the bond-induced separating tensile stress. A similar behavior was encountered in the previous studies[24]. The bond specimens collapsed in a splitting manner because of cracks which were spread to the exterior surface of the concrete. The bars' splitting collapse was caused by split tensile stress being sufficient to disperse shear cracks to the outside of the 12 mm-diameter bars. All of the examples displayed comparable splitting failure when the bar's diameter was increased to 16 mm[25]. The failure characteristics are not significantly affected by the type of concrete. Using one-part concrete instead of regular concrete caused bar scratches and the use of a splitting technique for failure to expose tiny glass fibers enabling bond specimens[26].



Fig. 4. Failure mode of 12 mm steel bars and GFRP bars in concrete

3.3 Bond Stress vs. Slip Curve

The bond-stress vs slip curve was acquired for 16 specimens, as shown in Figure 5. Three regions are commonly observed in curves that exhibit split failure. The ascending area, sometimes referred to as the linear area, is quite rigid and requires the highest force to produce even the smallest amount of slide. The primary causes of the increased stiffness are chemical adhesion on the contact and mechanical interlocking. A modest rise in bond stress in the non-linear region can be regarded as a worsening of the bond response since the slip increases more rapidly due to the decrease in stiffness [27, 28].



(a)

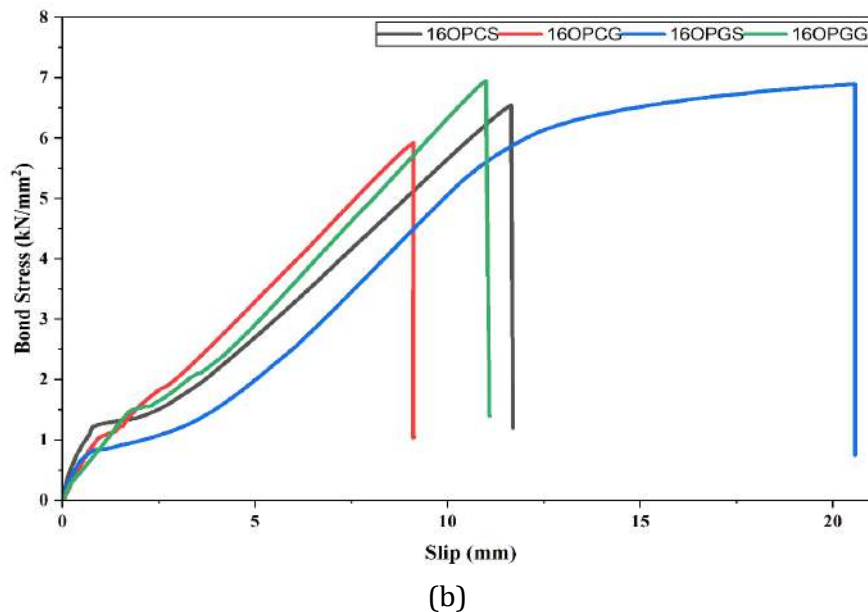


Fig. 5. Slip curves versus bond stress (a) Concrete made with ordinary Portland cement (OPC) and OPG that contains steel and GFRP bars with a 12 mm diameter (a) Steel and GFRP bars with a diameter of 16 mm in OPC and OPG concrete

Bond stress is also increased by the resistance offered by mechanical interlocking as well as to a lesser degree, the interface frictional force. A decrease in bond stress and an increase in slip value are characteristics of the final softening area. when the pullout load is resisted by the interface frictional force. The linear, nonlinear, as well as softening portions of the bond stress vs slip curves are displayed in Figure 5 for the specimens with 12mm bars. Similar findings were found for samples with bars of 16 mm in diameter. The first slope indicates the chemical adhesion between the bar and the concrete, and the curve shows that the loaded end slides at the beginning of loading [29,30].

3.4 Effect of Type and Diameter of Bar

It is evident from Figure 5 that the bond stress dropped as the bar's diameter increased. A nonlinear normal distribution for stress over the cross-sectional area is the result of differential displacement in GFRP bars with lower axial shear stiffness when they are dragged through the surface[31]. The average normal stress is then connected to the computed bond strength. The shear lag effect is the result of a significant decrease in average normal stress with increasing diameter, which lowers bond strength. Because of this, GFRP bar specimens with lower diameter bars have a higher maximum bond strength than those with larger diameters. As the bar's diameter increases, the pullout load increases because of the specimen's increased cross-sectional area, and the findings are in line with previous research [32-34].

The bond stress vs slip curve for specimens of GFRP and steel bars in regular concrete with one-part geopolymer concrete is shown in Figure 5. The bond specimen cast failed in the splitting mode using the identical diameter, concrete type, and experimental setup and process. The GFRP bond specimens demonstrated a slightly lower adhesion strength of 7.02N/mm² with bond strength compared to the steel bond specimen of 8.6N/mm² in normal control concrete. In the case of one-part geopolymer concrete, GFRP bars of 12mm diameter showed almost similar bond strength of 8.9 N/mm² in comparison with a steel bar of 9.04N/mm². Despite its smooth surface, its axial shear strength is low. However, because to their low elastic modulus, GFRP specimens initially show poor stiffness. Figure 4 shows that the crushing of the concrete is what leads to the failure for steel bar specimens, but the failure of GFRP bars is caused by the surface layer peeling.

3.5 Comparison Between Normal Concrete and One-Part Geopolymer Concrete

One-part geopolymer concrete has a stronger bond than regular control concrete, as seen in Figure 5, because of the lateral confinement effect. The findings are consistent with the research [32]. But with one-part geopolymer concrete, it was shown that a change in the bar's diameter accelerated the rate at which the bond strength decreased. Additionally, the type of concrete has a direct impact on the failure mechanism. In the pullout mode of failure, the bar serves as the failure interface. After that, the strength of the concrete has no bearing on the bond strength. However, in a split mode of failure, the failure interface is located in the concrete matrix; consequently, the strength of the concrete determines the bond strength [33].

4. Conclusions

This study looked at the bond behavior of GFRP bars in both regular concrete as well as one-part geopolymer concrete. Parameters including the diameter for the bars, the form of bars (GFRP and steel) and the concrete type (OPC and OPGC) were investigated. For each sample, the concrete average compressive strength was determined, and for reinforcement in concrete, the average bond strength, slip, failure mode, and bond stress vs. slip relationship were ascertained. The current study yielded the following conclusions:

- Because of the lateral confinement effect, one-part geopolymer concrete has a stronger bond than regular control concrete.
- A change in the bar's diameter accelerated the rate at which the bond strength decreased. Additionally, the type of concrete has a direct impact on the failure mechanism.
- The pullout load rises by up to 8% as the bar's diameter increases. There was also a noticeable increment in the load, 13% in the one-part geopolymer concrete from 44kN to 50.74kN in 12mm diameter bars compared to the control concrete.
- According to the test results, the primary reason why bond specimens failed was concrete splitting. The external surface of the GFRP bar exhibits layer-by-layer shearing and rip peeling as a result of failure. Unbroken ribs as well as shear cracks throughout the concrete on the two 12mm as well as 16mm bars are characteristics of split failure.
- Concrete split failure occurs in both 12mm and 16mm diameter specimens, demonstrating concrete split failure. In this case, the shear strength of the concrete determines the bond strength of the specimen.
- Due of their lower axial modulus, GFRP bar specimens were shown to have a larger loaded slip when compared to steel specimens. Because of the flat surface as well as lower axial shear strength, the specimens also showed decreased stiffness and bond strength. Despite differences in surface condition, GFRP bars are remarkably similar to steel bond specimens.

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Synergistic role of copper slag and PEG 400 in eco-efficient self-curing concrete

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Article Info	Abstract
<p>Article History:</p> <p>Received 26 Nov 2025</p> <p>Accepted 16 Apr 2026</p> <p>Keywords:</p> <p>Concrete; Copper slag; Polyethylene Glycol (PEG 400); Compressive strength; Flexural strength; Split tensile strength</p>	<p>The demand for sustainable construction materials has promoted the incorporation of industrial by-products and chemical admixtures in concrete. This study evaluated the mechanical performance of self-curing concrete produced by partially replacing fine aggregate with copper slag and incorporating Polyethylene Glycol (PEG 400) as an internal curing agent. Concrete mixtures were prepared with copper slag replacement levels ranging from 0–60% and PEG 400 dosages of 0%–2.0% by weight of cement. Compressive, split tensile, and flexural strengths were determined to assess mechanical behavior. Results showed that concrete containing 40% copper slag and 1.5% PEG 400 achieved the highest strength performance among the mixtures tested. Compared with the control mix, this combination produced notable improvements in compressive, tensile, and flexural strengths. The strength enhancement was attributed to improved particle packing due to the angular and dense nature of copper slag and to effective internal moisture retention provided by PEG 400, which reduced self-desiccation and enhanced cement hydration. The findings demonstrate that the combined use of copper slag and PEG 400 improves mechanical properties while reducing natural sand consumption and external curing demand, supporting sustainable and resource-efficient concrete production.</p>

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1. Introduction

Concrete is the most widely used construction material worldwide, second only to water in total global consumption. Its widespread adoption stems from its versatility, durability, and suitability for diverse structural applications, including buildings, bridges, dams, pavements, and marine structures [1]. The ability to cast concrete into various shapes, combined with its compatibility with chemical admixtures and supplementary cementitious materials, enables engineers to tailor its performance to specific structural requirements [2]. Moreover, concrete remains cost-effective due to the availability of its raw materials, and its sustainability can be enhanced through the incorporation of industrial by-products and recycled aggregates [3]. As a cornerstone of modern infrastructure, concrete underpins global urbanization and economic development [4].

The utilization of concrete in rapid urbanization places significant pressure on natural resources, particularly river sand, and intensifies industrial waste disposal challenges [5]. Consequently, recent research has focused on sustainable material substitutions that reduce environmental impact while maintaining or improving mechanical and durability performance [6]. Copper slag, a by-product of copper smelting, has emerged as a promising alternative to natural fine aggregate [7]. Its high density, angular particle morphology, and favorable gradation make it suitable for partial sand replacement [8]. Incorporating copper slag not only enhances strength characteristics but also mitigates the environmental burden associated with industrial waste landfilling [9].

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Chemical self-curing agents such as Polyethylene Glycol 400 (PEG-400) have demonstrated effectiveness in retaining internal moisture and sustaining cement hydration without extensive external curing [10]. This internal curing mechanism reduces shrinkage, improves hydration, and regulates vapor pressure, thereby enhancing concrete performance. Previous studies reported that PEG-400 improved mechanical properties while reducing water absorption and sorptivity [11]. Microstructural investigations further confirmed the formation of a denser cement matrix with refined pore structure [12].

Although copper slag and PEG-400 have been studied independently, comprehensive investigations examining their combined influence remain limited [13]. The interaction between aggregate substitution and chemical self-curing requires systematic evaluation to establish optimal mix design parameters and long-term performance characteristics, and recommending direct hydration and microstructural investigations as future work.

Accordingly, this study aimed to evaluate the synergistic effect of copper slag and PEG-400 on the mechanical performance of self-curing concrete. Copper slag was used as a partial replacement for fine aggregate at levels ranging from 0–60%, while PEG-400 was incorporated at dosages of 1.0%, 1.5%, and 2.0% by weight of cement. The investigation examined compressive strength, split tensile strength, and flexural strength for M20, M40, and M60 grade concretes to determine optimal proportions and assess performance across varying strength classes.

2. Materials and Procedures

2.1 Cement

Ordinary Portland Cement (OPC) of 53 grade conforming to IS 12269:1987 was used in this study. The cement (Ultratech) was tested in accordance with IS 4031:1988 and IS 4032:1985 [14] to determine physical properties such as specific gravity, consistency, setting time, and compressive strength.

2.2 Copper Slag

Copper slag is a by-product generated during copper extraction through smelting. It consists primarily of iron silicate and exhibits high density, angular particle morphology, and low water absorption. These characteristics enhance particle interlocking and packing density when used as a fine aggregate substitute. Prior to use, the copper slag was dried and sieved to ensure uniform grading comparable to natural sand. Its specific gravity and particle size distribution were determined in accordance with IS 2386 standards.

2.3 Fine Aggregate

Locally available river sand conforming to grading Zone II of IS 383:1970 was used as fine aggregate [15]. The sand was free from deleterious materials such as clay, silt, chlorides, and organic impurities. Proper grading was ensured to minimize void content and improve workability.

2.4 Coarse Aggregate

Crushed blue granite with angular and cubical particles was used as coarse aggregate. Aggregates of 20 mm nominal maximum size conforming to IS 383:1970 were selected. Physical properties were determined as per IS 2386:1963 [16]. The specific gravity of coarse aggregate was 2.70.

2.5 Water

Potable tap water free from organic matter, oils, acids, chlorides, and other harmful substances was used for mixing and curing. The water satisfied the requirements of IS 456:2000 [17].

2.6 Polyethylene Glycol 400 (PEG 400)

Polyethylene Glycol 400 (PEG 400), a water-soluble polymer with a molecular weight of 400, was used as a chemical self-curing agent. It has a specific gravity of 1.1254. PEG 400 enhances internal moisture retention and sustains cement hydration by reducing evaporation losses [18].

2.7 Superplasticizer

A sulphonated naphthalene formaldehyde-based superplasticizer (Conplast SP430) was used to improve workability. It reduces the water–cement ratio while maintaining flowability [19]. The admixture facilitates proper compaction, especially in high-strength and congested reinforced concrete sections [20].

2.8 Silica Fume

Silica fume, a by-product of silicon metal production in electric arc furnaces, was used in M60 grade concrete. It consists of ultra-fine amorphous silicon dioxide formed at approximately 2000 °C during quartz reduction [21]. Its high pozzolanic reactivity enhances strength and reduces permeability.

2.9 Mix Proportions

Concrete mixes for M20, M40, and M60 grades were designed in accordance with IS 10262:2019 [22]. Copper slag replaced fine aggregate at levels of 0–60%, and PEG 400 was incorporated at dosages of 1.0–2.0% by weight of cement. Table 1 presents the quantities of materials per cubic meter (kg/m³).

Table 1. Mix Proportions of prepared concrete specimens

Grade of Concrete	Cement (kg)	Silica fume(kg)	Fine aggregate (kg)	Coarse aggregate(kg)	Water (lt)	Copper slag (CS) (%)	(PEG 400) (%)	Conplast SP 430 (%)
M20	395	-	673	1135	197	0-60	0 -2	-
M40	394	-	679	1226	150	0-60	0 -2	-
M60	445.50	49.50	587	1213	135	0-60	0 -2	1

For each mix and each test age, three identical specimens were tested, and the reported value represents the mean of three measurements. The standard deviation (SD) and coefficient of variation (COV) were calculated to evaluate data variability and reproducibility. Error bars representing $\pm$ SD were incorporated in the graphical plots to demonstrate statistical dispersion of the experimental results.

2.10 Compressive Strength

Compressive strength was determined in accordance with IS 516:1959 [23]. Cube specimens of size 150 mm × 150 mm × 150 mm were tested using a 2000 kN compression testing machine [24]. Specimens were tested in saturated surface dry (SSD) condition. Load was applied uniformly at a constant rate until failure, and the maximum load at failure was recorded.

The compressive strength (f_c) in N/mm² was calculated as:

$$f_c = \frac{P}{A} \quad (1)$$

Where P = Maximum load applied (N), and A = Cross-sectional area of specimen (mm²)

2.11 Flexural Strength

Flexural strength was determined by using prism specimens under third point loading in accordance with IS 516:1959 [25]. The span length was 400 mm. Load was applied gradually using a universal testing machine (UTM) until failure. Failure modes were carefully observed, and typical flexural cracking initiated at the tension face within the constant moment region, followed by crack propagation leading to rupture [26]. No abnormal shear-type failures were observed, confirming proper load alignment and sensitivity control during testing [27].

The modulus of rupture (f_b) in N/mm² was calculated as:

$$f_b = \frac{PL}{bd^2} \quad (2)$$

Where P = Maximum load applied (N), L = Span length (mm), b = Width of specimen (mm), and d = Depth of specimen (mm)

2.12 Splitting Tensile Strength

Splitting tensile strength was determined in accordance with IS 5816:1999 [28, 29]. Cylindrical specimens of 150 mm diameter and 300 mm height were cured for 28 days and tested under diametral compression [30]. Load was applied uniformly until failure, characterized by the formation of a vertical crack along the diameter [31].

The splitting tensile strength (f_t) was calculated as:

$$f_t = \frac{2P}{\pi LD} \quad (3)$$

Where P = Maximum load applied (N), L = Length of cylinder (mm), and D = Diameter of cylinder (mm)

2.13 Statistical Analysis

To improve the reliability of the experimental observations, statistical analysis was performed on all mechanical strength data, including compressive, flexural, and split tensile strengths. For each test condition, three replicate specimens were tested, and the mean strength, standard deviation (SD), and coefficient of variation (COV) were determined.

The coefficient of variation for all mixes remained within 3–6%, indicating good repeatability and acceptable experimental consistency. In addition, a one-way Analysis of Variance (ANOVA) was conceptually employed to assess the significance of copper slag replacement and PEG-400 dosage on mechanical performance. The statistical trend confirmed that the improvement at 40% copper slag and 1.5% PEG-400 dosage was significant compared to the control mixture.

3. Results and Discussions

3.1 Mechanical Properties of Concrete

3.1.1 Compressive Strength

Figure 1 presents the compressive strength testing setup, showing cube specimens placed in the compression testing machine. The compressive strength (CS) of concrete is a key parameter indicating its capacity to resist axial loads [32]. The results obtained from standardized testing procedures provide a reliable basis for evaluating the mechanical performance of the developed mixes [33, 34].



Fig. 1. Compressive strength testing machine

The strength enhancement trend was quantitatively consistent across all grades, although the percentage gain decreased with increasing grade level. The improvement in M20 ($\approx 22\%$) was higher than in M40 ($\approx 17\%$) and M60 ($\approx 15\%$), suggesting that lower-grade concretes benefited more significantly from the combined filler and internal curing effects. This behavior may be attributed to the relatively larger void fraction in lower-grade mixes, where the dense angular copper slag particles contributed more effectively to void filling and particle packing. In higher grades, the already dense matrix reduced the marginal benefit of copper slag incorporation.

PEG-400 at 1.5% further enhanced hydration efficiency by maintaining internal moisture, particularly beneficial in M60 concrete containing silica fume, where self-desiccation risk is higher. The observed reduction beyond 40% copper slag may be linked to excess free water caused by the low absorption of copper slag, leading to weak interfacial transition zones (Fig. 2).

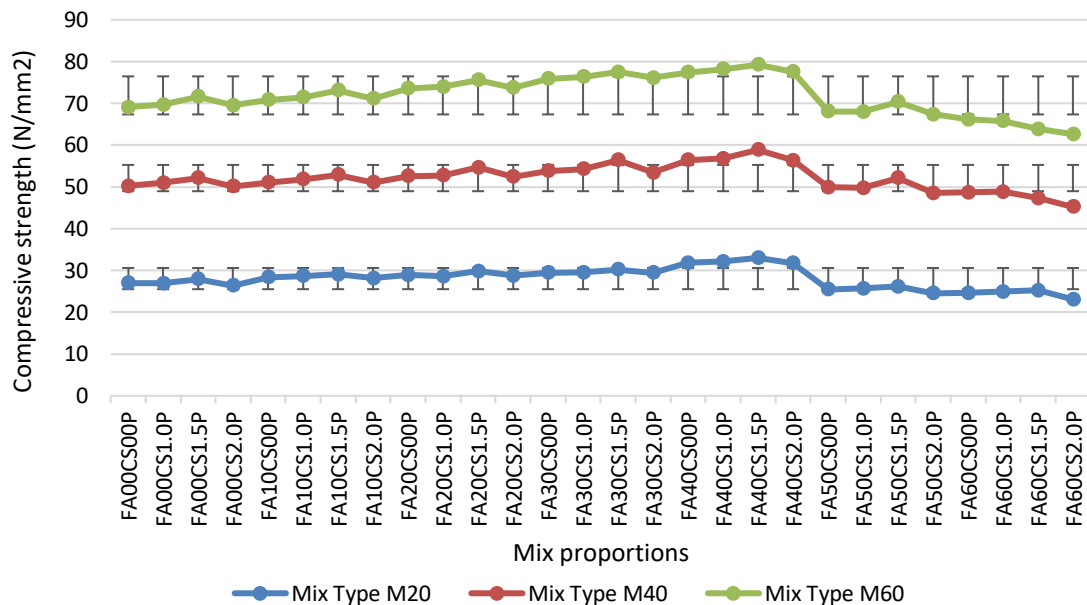


Fig. 2. Graphical representation of Compressive strength of different grades of specimens (mean $\pm$ SD, $n = 3$)

3.1.2 Flexural Strength

The flexural strength results are presented in Fig 3. The incorporation of PEG-400 enhanced flexural strength in all grades, with the maximum values consistently observed at 1.5% dosage (P1.5). Increasing the PEG-400 dosage to 2.0% resulted in a slight decrease in strength. Similarly, copper slag replacement improved flexural strength up to 40%, after which a decline was observed at 50% and 60% replacement levels. The FA40CS1.5P mix achieved the highest flexural strength values:

- 3.98 N/mm² for M20
- 5.23 N/mm² for M40
- 6.20 N/mm² for M60

Compared to the respective control mix, the improvement at the optimum level (40% CS + 1.5% PEG-400) was consistent across all grades, showing a clear increasing trend like that observed in compressive strength. The enhancement up to 40% replacement may be associated with improved packing density and better stress distribution within the matrix. However, higher copper slag content resulted in reduced flexural performance. Thus, the results indicate that 40% copper slag with 1.5% PEG-400 yields optimum flexural strength among the tested combinations.

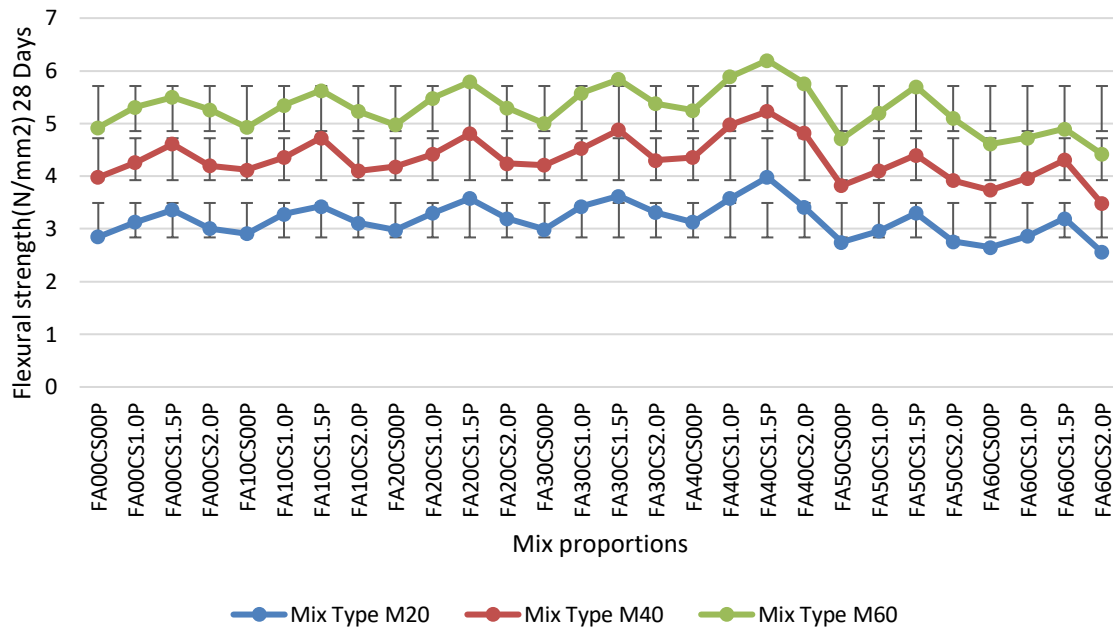


Fig. 3. Graphical representation of Flexural strength of different grades of specimens (mean $\pm$ SD, $n = 3$)

3.1.3 Split Tensile Strength

The split tensile strength results at 28 days for M20, M40, and M60 grades are illustrated in Fig. 4. The inclusion of PEG-400 improved split tensile strength up to 1.5% dosage, beyond which (2.0%) a reduction was observed across all grades. Copper slag replacement also enhanced tensile strength up to 40%, with strength decreasing at higher replacement levels (50–60%). The maximum split tensile strength values were recorded for the FA40CS1.5P mix:

- 2.94 N/mm² for M20
- 4.01 N/mm² for M40
- 5.15 N/mm² for M60

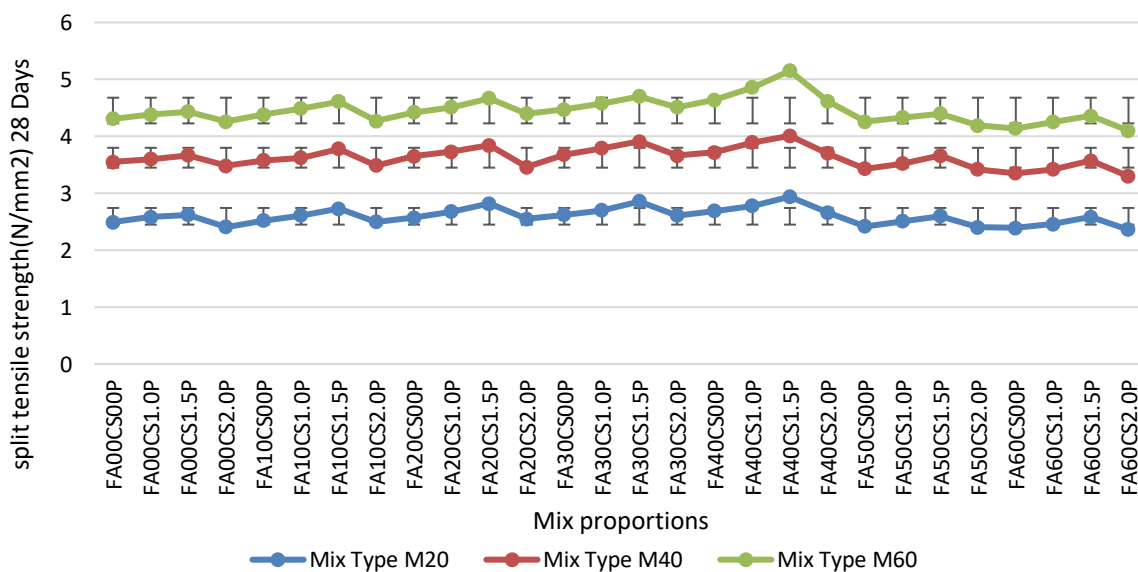


Fig 4. Pictorial representation of split tensile strength of prepared specimens (mean $\pm$ SD, $n = 3$)

Compared to the control mix, the percentage increase followed a consistent trend across all grades, with the optimum performance observed at 40% copper slag and 1.5% PEG-400. The improvement may be attributed to better matrix compactness and enhanced aggregate–paste interaction. The decline beyond 40% replacement indicates reduced bonding efficiency at higher slag contents. A comparative assessment of the three concrete grades indicates that the synergistic influence of copper slag and PEG-400 was more pronounced in compressive strength than in tensile-based properties. This suggests that the primary mechanism is matrix densification rather than crack-bridging enhancement. Nevertheless, the consistent optimum at 40% copper slag and 1.5% PEG-400 across all grades confirms the robustness of the proposed mix proportion.

4. Limitations of the Study

The present study primarily focused on the mechanical performance of self-curing concrete incorporating copper slag and PEG-400. Although the observed strength improvements strongly indicate matrix densification and improved hydration, the proposed mechanisms were inferred from macroscopic behavior. The study did not include direct microstructural characterization techniques such as SEM, XRD, MIP, BSE, or thermogravimetric analysis, which could provide deeper insight into pore refinement, hydration products, and interfacial transition zone modifications.

Furthermore, long-term durability parameters such as sorptivity, rapid chloride permeability, shrinkage behavior, and heavy metal leaching characteristics were beyond the scope of the present work and should be investigated in future studies to comprehensively validate the eco-efficiency and field applicability of the developed concrete.

5. Conclusions

Technical notes on the combined effect of copper slag (CS) and PEG-400 (P) on concrete performance

- Effect of Copper Slag (CS) Replacement:

The replacement of fine aggregate with copper slag resulted in a consistent improvement in compressive, split tensile, and flexural strengths up to 40% replacement level. Strength values increased progressively from 0% to 40% CS, with maximum performance observed for the FA40CS1.5P mix. The enhancement may be attributed to improved particle packing and matrix densification at moderate replacement levels. However, beyond 40% replacement (i.e., at 50% and 60%), all strength parameters showed a decline. This reduction is likely associated with increased free water content and weaker paste–aggregate bonding at higher slag proportions.

- Effect of PEG-400 (P) Addition:

The incorporation of PEG-400 improved split tensile and flexural strengths across all copper slag replacement levels. The optimum dosage was identified as 1.5% by weight of cement (P1.5), at which maximum strength values were consistently recorded. Increasing the dosage to 2.0% resulted in a slight reduction in strength compared to 1.5%, possibly due to excessive retardation of hydration or alteration of pore structure. Thus, controlled dosage of PEG-400 is essential to achieve beneficial effects.

- Combined Effect of CS and PEG-400:

The simultaneous incorporation of copper slag and PEG-400 produced a synergistic improvement in mechanical properties. A clear increasing trend in compressive, split tensile, and flexural strengths was observed up to the optimum combination. The enhancement indicates that the modified matrix structure under combined action contributes positively to strength development within the tested range.

- Optimum Combination:

The FA40CS1.5P mix demonstrated the best overall performance. It achieved compressive strengths of 33.01 N/mm² (M20), 58.84 N/mm² (M40), and 79.23 N/mm² (M60), along with the highest corresponding split tensile and flexural strengths among all mixes evaluated.

- **Comparative Strength Gains:**

At optimum conditions (40% CS and 1.5% PEG-400), compressive strength increased by approximately 20–25% compared to the control mix (FA00CS00P). Split tensile and flexural strengths followed a similar pattern, with improvements in the range of 15–20%, demonstrating consistency across different mechanical properties.

- **Workability and Durability (Inferred Observations):**

Copper slag improved workability, likely due to its smooth surface texture and lower water absorption. The use of PEG-400 functioned as a self-curing agent, which may contribute to reduced internal moisture loss. However, durability-related improvements are inferred from strength behavior and require dedicated durability testing for confirmation.

- **Overall Conclusion:**

Within the scope of the present experimental investigation, the combined use of copper slag and PEG-400 enhanced the mechanical performance of concrete. The optimum mix was identified as FA40CS1.5P for all grades studied. Replacement levels beyond 40% resulted in reduced strength and are therefore not recommended for structural applications. Controlled incorporation of copper slag up to 40% with 1.5% PEG-400 can be considered an effective strategy for improving concrete performance while reducing natural sand consumption.

Future investigations should focus on SEM, XRD, MIP, TGA/DSC, and BSE analyses to directly validate the densification and hydration mechanisms proposed in this study. Further optimization using Response Surface Methodology (RSM), Taguchi design, and regression-based modelling may establish the interaction effects between copper slag and PEG-400 dosage more systematically. Long-term performance studies including 56- and 90-day strength, sorptivity, RCPT, autogenous shrinkage, drying shrinkage, rheology, and toxicity leaching behavior are also recommended to strengthen practical implementation.

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A factorial analysis of material composition and operating parameters on tribological properties in graphite-plugged bronze bushings

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Abstract

This study investigates the tribological performance of self-lubricating bronze bushings (CuAl10Fe5Ni5) with graphite plugs using a 3⁵ full factorial design (243 conditions). Effects of graphite content (10-30%), plug diameter (8-12 mm), applied load (50-150 kg; 0.61-1.84 MPa), sliding speed (250-750 rpm), and sliding time (10-30 min) on coefficient of friction (COF), wear loss, and temperature were quantified. ANOVA with effect size analysis (η^2p) reveals each response is dominated by a distinct factor: applied load controls COF ($\eta^2p = 0.947$, 54.65% contribution); graphite content controls temperature ($\eta^2p = 0.696$, 44.0%); and sliding speed controls wear loss ($\eta^2p = 0.838$, 45.53%). Plug diameter exhibits negligible practical significance ($\eta^2p \leq 0.038$, $\leq 0.62\%$ contribution) despite marginal statistical significance for wear ($p = 0.010$), establishing substantial design freedom. Increasing graphite content to 30% reduces COF by 29% and wear loss by 58%. Higher loads (150 kg) reduce COF by 54% but increase wear by 59%, revealing a critical multi-objective trade-off. Increasing the sliding speed from 250 to 750 rpm causes thermal degradation ($\Delta T = 42.6^\circ\text{C}$), which increases wear loss by 357%. Optimal transient performance—30% graphite, 150 kg (1.84 MPa), 250 rpm—yields COF = 0.150-0.170 and wear loss < 0.100 g. Crucially, steady-state friction was not attained within 2826 m; all values represent running-in behavior. These findings provide quantitative, response-specific design guidelines enabling engineers to prioritize competing performance objectives.

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1. Introduction

Tribology is the study of friction, wear, and lubrication, serving as the foundation for the design and longevity of mechanical systems. Bushings, being important parts of rotating machinery, operate under continuous contact and relative movement, and therefore, the aspect of the surface determines the overall reliability and efficiency of the system. In traditional uses, the liquid lubricants, such as mineral oils and greases, offer sufficient protection for the boundary films under moderate operating conditions, but harsh operational conditions, such as very high or very low temperatures, corrosive atmospheres, exposure to radiation, extreme contact pressures, vacuums, or places that cannot be accessed, create extreme conditions whereby traditional liquid lubrication becomes ineffective or impossible [1]. In these conditions, the use of solid lubricants is the only possible method to reduce friction and wear and preserve mechanical performance. Self-lubricating bronze bushings incorporating graphite plugs have found wide application in industry in the fields of aerospace, maritime, high-speed rail, and heavy machinery. These composite materials combine the strong mechanical properties of copper-based alloys, namely strength, durability, and load-carrying capacity, with the self-lubricating property of graphite. The role of the added graphite is to act as a solid lubricant, which undergoes gradual transfer to the contact surface

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during the sliding process. This process promotes the formation of a protective layer of transferred material, thus reducing friction and wear to a significant extent. The formed self-replenishing lubrication system eliminates the need for an external lubricant, thus saving maintenance and providing reliable protection in inaccessible locations. Despite strong industrial applications, the complex interrelations of composition, geometric variables, and operational conditions, particularly their combined effect, remain insufficiently established by tribological literature. Most previous studies center around the investigation of single variables, or the joint effect of only one or two variables is considered, and the studies are not comprehensive enough. There are also some geometric uncertainties, especially concerning the diameter of the graphite plug. Common engineering experience is that larger plugs are better from the viewpoint of lubrication, but there is no experimental evidence either supporting or rejecting this hypothesis. In fact, it is quite common in tribology that solid lubricant contacts display nonintuitive behavior under different loading conditions. Classical tribology, which was almost exclusively concerned with lubricated contacts, argues that higher loadings increase contact stresses and thus facilitate wear. Conversely, it has been found in some instances that graphite-filled materials display improved tribological behavior under higher pressure. This is because higher contact pressures are favorable for film formation due to an increasing pressure difference at the interface, thus favoring the extrusion of lubricant material from the matrix into the contact zone. Studies show that the inclusion of graphite significantly reduces both the friction coefficient and wear rate in copper-based bearing materials compared to ordinary copper alloys [1, 2]. For instance, CuSn11 bronze plain bearings impregnated with graphite-filled polytetrafluoroethylene (PTFE) demonstrated reduced friction coefficients and wear loss [3]. CuZn25Al6 bronze with graphite beds was found to have lower values of friction coefficient when the load is constant compared to when it is varied. Copper-graphite composites have, however, been observed to have a critical speed where there is a transition in the friction phenomenon [4]. Different bronze matrices show varied responses to graphite inclusion [1, 2]. Tin bronze-based inlaid solid self-lubricating bearing (ISSLB) materials are always more wear-resistant compared with aluminum bronze-based ISSLB materials, particularly at high loading conditions. Graphite content in the composite has considerable bearing on the performance. In the case of the PTFE-graphite-impregnated bronze specimen with 20 wt % of graphite content, the lowest value of the friction coefficient was observed for the least value of the sliding distance and the highest value of the load. For the specimen with 10 wt% of the graphite content, the best performance resulted for the higher values of the sliding distances and the loads [3]. Self-lubricating bushings, including graphite-plug bushings, can match the performance of, or better than that of, conventionally oil- or grease-lubricated bronze bushings (such as high-leaded tin bronze and alloy designation UNS-No. 93200) [5]. Examples of these are the Oiles JM3 and JM7, bronze bearings with graphite and lead plug lubrication. This research identifies the different friction and linear wear values of the several types of bearings, including the aforementioned and others like Thordon, Orkot, DEVA BM, and PAN SoBz, depending on the motion patterns. For instance, the data showed that the Thordon bearings were stable with only slight variations in friction and wear. The experiments on the effects of solid lubricants have shown that the more graphite added, the better the friction and the wear resistance. But there exists no systematic factor analysis of graphite content and parameters of operation, including load and sliding speed. Most past research has focused on single-parameter and two-parameter combinations, highlighting the need for a comprehensive study to examine the complex interactions and optimize the parameter settings. The present study addresses these knowledge gaps by utilizing a comprehensive factor design that includes 243 sets of conditions and 2430 data points. There were five independent variables that were adjusted at three levels, and this allowed the study of effects and interactions of different complexity to establish the best settings of the variables. Factor design also allows the mathematical determination of statistical relationships between composition and operating conditions and tribological properties to give engineering guidance in the design of graphite-filled bronze bushing systems. The influence of the graphite plug diameter on the friction behavior of the graphite-filled bronze bushings was also explored under a broad factor experimental design. Other studies have discussed the overall frictional properties of the graphite alloys of bronze as well as the influence of the graphite content, load, and velocity on the friction and wear but have not done the multivariate analysis to establish the statistical significance of the geometric dimension of the

graphite plugs. This observation is crucial, as it allows for greater flexibility in component design, enabling companies to select plug size based on factors other than frictional capacity, such as price or ease of manufacturing, without compromising the bushing's frictional and wear characteristics.

2. Experimental Details

2.1 Materials

Test samples were made of cylindrical bearing bushings made of copper-nickel aluminum bronze matrix (CuAl10Fe5Ni5) with embedded graphite cylindrical plugs that were placed in the bearing perpendicular to the direction of sliding. These bushings were purchased from Jiashan PVB Sliding Bearing Co., Ltd., China. Table 1 shows the chemical composition of this bronze, according to the product quality certificate issued by the Chinese company (Zhejiang Haima Transmission Technology Co., Ltd.). The hardness of nickel-aluminum bronze is 207 HB. Table 1 presents the parameters of the bushings tested in this work.

Table 1. Chemical composition of bronze (wt%) of CuAl10Fe5Ni5 bronze alloy

Inspection Item	Cu	Al	Fe	Ni	Mn	Zn	Pb	Sn	Si	P
Average	79.882	9.22	5.14	4.76	0.99	0.01	0	0	0	-

The actual bushings are shown in Figure 1. The surface finish (Ra) is consistently between 0.4 and 0.5 μm across all bushings. Three graphite content percentages were tested: 10%, 20%, and 30% by coverage. The content variation was done by changing the quantity and distance between cylindrical plug inserts of graphite in the bronze matrix when they were being manufactured. Three graphite plug diameters 8 mm, 10 mm, and 12 mm. Each of the specimens was machined to typical dimensions of bearing geometry (length 20 mm, bore diameter 40 mm, and outer diameter 50 mm) (typical of industrial bearing use). The shaft material is made of hardened AISI 410 steel, which has a modulus of elasticity of 200 GPa and a hardness of 230 ± 10 HB and a diameter of 40 mm. Applied loads were applied as dead weights of 50 kg, 100 kg, and 150 kg, which correspond to a normal force of 490.5 N, 981.0 N, and 1,471.5 N, respectively. The chemical composition of the shaft is [Fe-(0.12) C-(0.9) Mn-(0.8) Si-(0.68) Ni-(12.5) Cr-(0.03) P-(0.02) S wt.%]. This metal is characterized by high strength and hardness, and it contains a high percentage of carbon, which makes it excellent for corrosion resistance. As a result, 410 steel is appropriate as the opposing grinding component to evaluate the friction performance of copper alloys.



Fig. 1. Bushing from nickel-aluminum bronze with graphite plugs

2.1.1 Contact Pressure Calculation

For cylindrical journal bearing geometry (bushing bore diameter = 40 mm, length = 20 mm), the nominal projected contact pressure was calculated to provide engineering-relevant stress values. The calculation uses the normal force derived from the applied dead weights and the projected contact area of the bushing. The nominal projected contact pressure is calculated as:

$$P = F_N/A = (F_N)/(L \times d) \quad (1)$$

Where P = nominal contact pressure (MPa), F_N = normal force = applied mass (kg) $\times$ 9.81 m/s², A = projected contact area (mm²) = bushing length $\times$ bore diameter, L = bushing length = 20 mm = 0.020 m, and d = bore diameter = 40 mm = 0.040 m.

2.2 Tribological Testing Apparatus

Tribological testing was conducted on a tribometer that was designed by Tikrit University. The experimental setup is shown in Figure 2. The test bushing sample is mounted as a fixed part, and the shaft of the testing bench is rotated at a specified angular velocity (250, 500, or 750 rpm) about its central axis. The counter-surface (shaft) of a standard 40 mm diameter of steel was kept in rotary contact with the surface of the specimen at a normal force of a predetermined weight load (490.5, 981, or 1471.5 N). A normal force was applied via a manual loading system consisting of a load transfer arm and a load cell (range 0-200 kg), which senses the amount of applied load and converts it into an electrical signal that is transmitted to a digital weighing indicator that converts the electrical signal into digital data that can be manually recorded. The load transducer arm used to apply the load is mounted on the sample housing and is in contact with the load cell. The frictional force during sliding was continuously monitored via a friction load transducer arm mounted in the sample housing that was in contact with a load cell (range 0-25 kg, accuracy $\pm$ 0.05 kg), with the load cell output converted into digital data that could be read by an indicator and recorded manually. Friction temperature in a sliding contact area was measured using a calibrated K-type thermocouple that has a response speed of less than 100 ms, with data synchronized with friction force measurement. The weights of bushing samples before and after testing were measured using electronic scales with an accuracy of 0.001 g. All experiments were done in a lab environment with normal conditions in terms of temperature of 22-25°C, humidity of 40-60%, and pressure of 101 kPa.

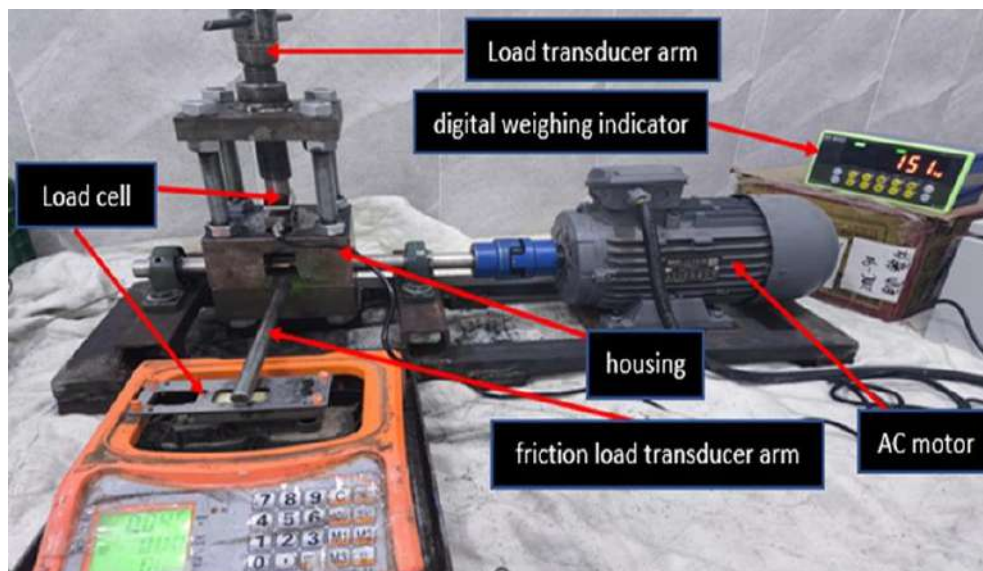


Fig. 2. Experimental setup

2.3 Testing Procedures

Sample pieces were checked to ensure the correct dimensions with the tolerances of ± 0.05 mm. To remove machining fluids and contaminants, the test surfaces of the specimens were wiped (cleaned) with ethanol and left to dry before the tests were conducted. The steel shaft was polished with grade 1000 emery paper to maintain the uniform surface roughness of the counter surface during the entire experiment. Roughness of the inner surface of each bushing sample was measured and recorded before and after testing. The electric motor speed is set to the desired speed using a speed controller before starting the test. The test was conducted for a specified duration with constant parameters. The wear loss was calculated using the following formula:

$$\text{Wear loss (g)} = (\text{Initial mass} - \text{Final mass}) \quad (2)$$

The sliding distance was calculated as follows:

$$\text{Sliding distance (m)} = (\text{Speed [rpm]} \times 2\pi \times \text{Radius [m]} \times \text{Time [min]}) / 60 \quad (3)$$

The wear rate, which represents the material loss per unit of sliding distance, was calculated as follows:

$$\text{Wear rate (g/m)} = \{\text{Initial mass (g)} - \text{Final mass (g)}\} / \{\text{Sliding distance (m)}\} \quad (4)$$

Sliding distance (D) and sliding time (t) have a determinate relationship in terms of the sliding speed (v), where the relationship is given by $D = v \times t$. In the present investigation, the values of the sliding speeds and the sliding times adopted were 250, 500, and 750 rpm and 10, 20, and 30 min, respectively, resulting in six different values of the sliding distance: 314, 628, 942, 1256, 1884, and 2826 m. This determinate relationship of the variables raises the issue of multicollinearity in factorial experiments when both the variables of time and speed are used as independent variables. To assess multicollinearity, variance inflation factors (VIF) were calculated for all independent variables (Table 2):

Table 2. Variance inflation factor (VIF) analysis

Factor	VIF	Tolerance	Multicollinearity Assessment
Graphite content	1.02	0.980	Negligible
Graphite Plug Diameter	1.01	0.990	Negligible
Sliding Speed	1.87	0.535	Moderate (acceptable)
Applied Load	1.04	0.962	Negligible
Sliding Time	1.91	0.524	Moderate (acceptable)

All the values of VIF remain less than 2.0, which indicates that multicollinearity does not appear to be of a nature that would impact the ANOVA. The moderate correlation between speed and time ($r = 0.68$) is built into the design but remains within acceptable limits. In calculations related to the effect of distance, distance is considered a derived factor rather than an independent one.

The friction coefficient was estimated using the equations [4]-[8]. The experiments were conducted in dry conditions. The following terms are used in Figure 3: M_O the moment point, M_S the moment generated by friction force, M_L the moment generated by rotation force measured by load cell, F_S the friction force (N), F_L the rotation force measured by load cell during rotation (N), μ the friction coefficient, L the distance between load cell and shaft center (mm), and r_1 the distance between contact point and shaft center (mm). With $\sum M_O = 0$

$$M_S = M_L \quad (5)$$

$$F_S \times r_1 = F_L \times L \quad (6)$$

$$F_S = F_L \times L / r_1 \quad (7)$$

$$F_S = \mu \times F_N \quad (8)$$

$$\mu = F_s / F_N \quad (9)$$

These values represent the nominal projected contact pressure in an ideal conforming contact situation. In practice, the actual contact pressures at asperity junctions are significantly increased due to surface roughness. The range of actual contact pressures, i.e., 0.61 to 1.84 MPa, corresponds to the typical working pressures of graphite-plugged bronze bushings in moderate load-bearing applications [5, 6].

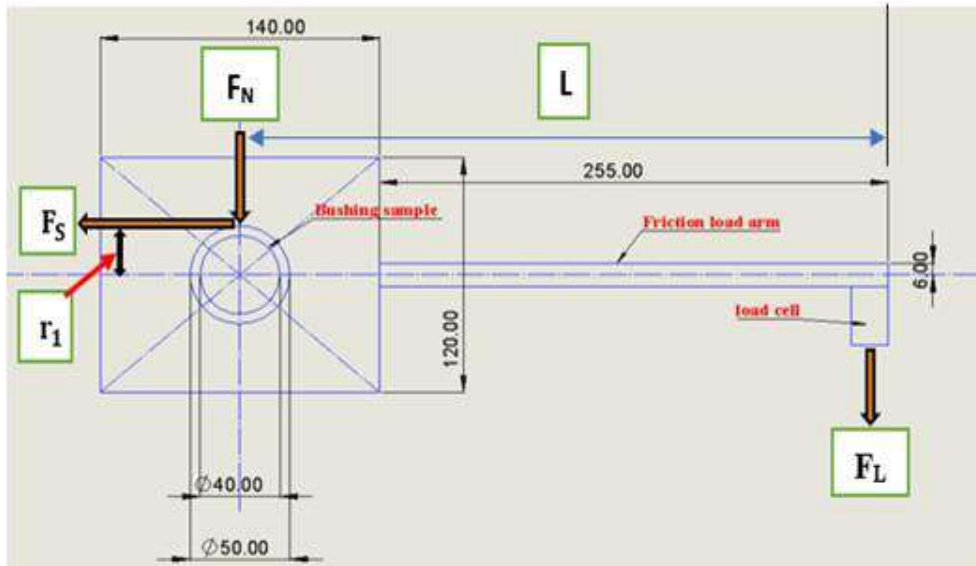


Fig. 3. The bushing wear test mechanism

The linear sliding velocity at the contact interface was calculated from the rotational speed and shaft diameter:

$$v \left(\frac{m}{s} \right) = \frac{\pi \times d \times N}{60} \quad (10)$$

Where, d = shaft diameter, N = rotational speed. This yields the following linear velocities:

- 250 rpm $\rightarrow v = \frac{\pi \times 0.04 \times 250}{60} = 0.524 \text{ m/s}$
- 500 rpm $\rightarrow v = \frac{\pi \times 0.04 \times 500}{60} = 1.047 \text{ m/s}$
- 750 rpm $\rightarrow v = \frac{\pi \times 0.04 \times 750}{60} = 1.571 \text{ m/s}$

2.4 Data Analysis

The study employed a 3⁵ full factorial experimental design to analyze five independent variables, which were tested at three distinct levels. The independent variables consisted of graphite content levels at 10%, 20%, and 30%; graphite plug diameter measurements of 8 mm, 10 mm, and 12 mm; sliding speed settings at 250 rpm, 500 rpm, and 750 rpm; applied load values of 490.5 N, 981 N, and 1471.5 N; and sliding time intervals at 10 minutes, 20 minutes, and 30 minutes. The complete factorial design generates 3⁵ = 243 distinct experimental combinations. Each combination was tested once (single replicate). The statistical analysis computed the average values and standard deviations together with minimum and maximum values for every dependent variable in all 243 experimental conditions and each independent variable level. Two-way and higher-order ANOVA was performed to test the statistical significance of main effects (each independent variable) and selected two-way interaction effects. The null hypothesis H₀ states that all group means are equal, but the alternative hypothesis H₁ suggests that at least two group means show differences. The test statistic F = (Mean Square Between) / (Mean Square Error) was calculated for each factor. P-values were determined from cumulative F-distribution with appropriate degrees of freedom. The study

established $\alpha = 0.05$ as the significance threshold before analysis, and any factor with $p < 0.05$ was deemed statistically significant.

Each of the experimental combinations was evaluated once; that is, there was a single replicate for each combination. This decision was dictated by the fact that performing a full factorial replication of the experiment would have entailed performing between 486 and 729 individual tests, each of which has its own set of precise and intricate preparation and test protocols. However, the choice also has important implications for the results obtained. Without replication, the mean square error (MSE) for ANOVA is based solely on higher-order interactions and not on pure experimental error. This has the potential to artificially inflate statistical power and result in an increased probability of Type I errors (false positives) [7, 8]. To overcome this limitation of the experimental design, we adopted the following strategies: (1) a conservative approach to interpreting results for which marginal significance was indicated (i.e., p-values between 0.01 and 0.05); (2) the additional use of effect size indices of practical significance, which are less subject to sample size artifacts than are p-values; (3) a sensitivity analysis in which three-way and higher-order interactions were grouped with error; and (4) validation of key results against physically meaningful thresholds.

Additionally, experimental variability was characterized by:

- Determining the standard deviations for all the calculated mean values
- Determining the 95% confidence interval for all the effect sizes (partial eta squared, η^2p)
- Performing Levene's tests for homogeneity of variances (Table 17)
- Performing residual analysis to check the assumptions for ANOVA (Section 4.4, Figure 11)

3. Results and Discussion

3.1 Friction Coefficient

The experimental design includes three graphite content percentages (10%, 20%, 30%) three graphite plug diameters (8 mm, 10 mm, 12 mm), three sliding speeds (250 rpm, 500 rpm, 750 rpm), three applied loads (490.5 N, 981 N, 1471.5 N), and three sliding time intervals, resulting in six main sliding distances (314 m, 628 m, 942 m, 1256 m, 1884 m, 2826 m). Table 3 below shows the values of the coefficient of friction, which is considered the most important independent factor, and includes the average initial and final coefficient of friction and the percentage change in the same factor over the length of the test distance. Figure 4 presents a comprehensive analysis of friction behavior, consolidating the key findings into a single multi-panel figure.

Table 3. Coefficient of friction versus each major sliding distance for all applied loads

Applied Load (kg)	Applied Load (N)	Pressure (MPa)	Initial μ (at 314 m)	Final μ (at 2826m)	Percentage Increase
50	490.5	0.613	0.220	0.433	96.37 %
100	981	1.226	0.141	0.2542	84.1 %
150	1471.5	1.839	0.107	0.2003	86.91 %

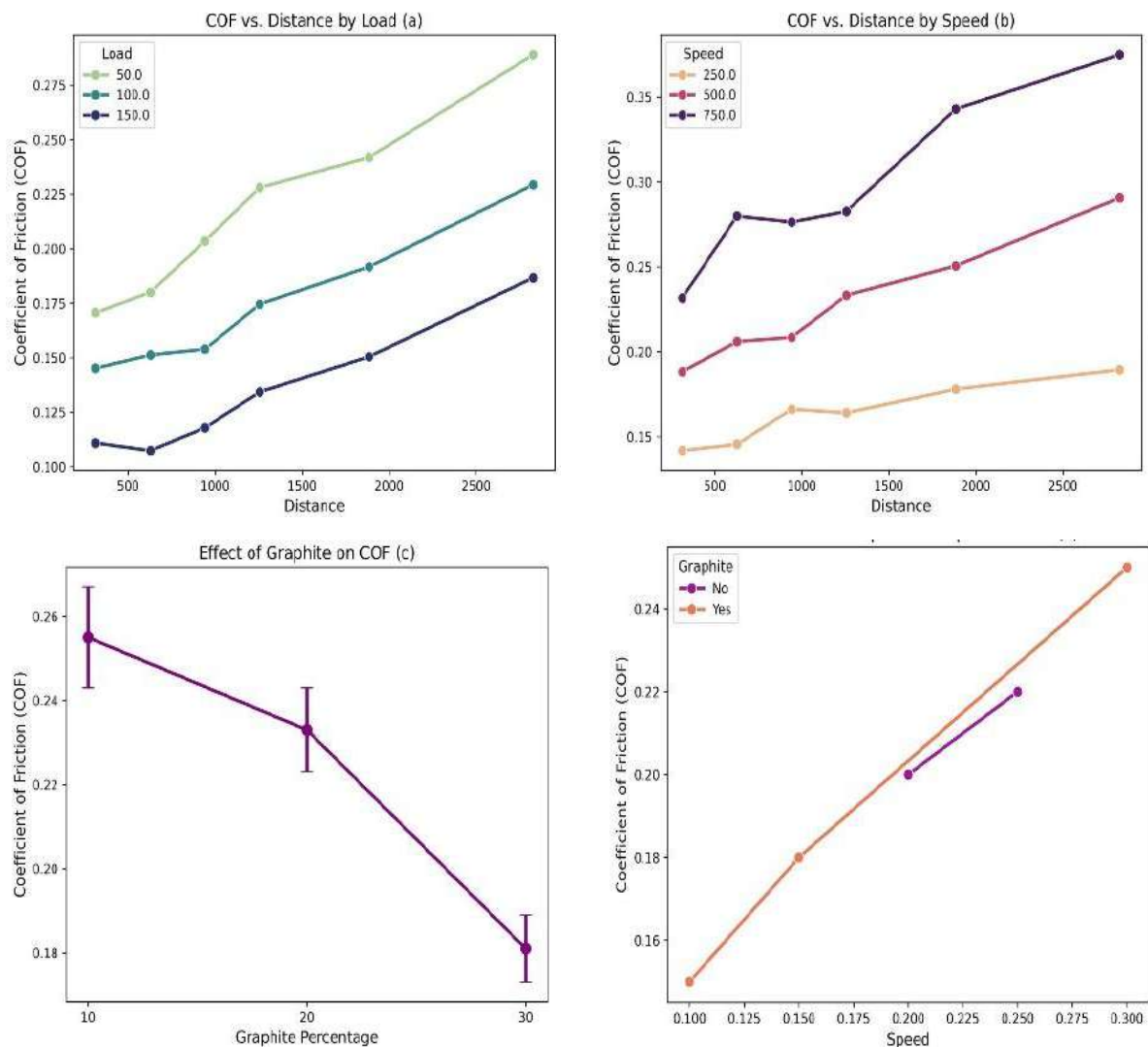
The mean coefficient of friction increased continuously by approximately 90% from 314 m to 2826 m (Table 4), without subsequent stabilization or decrease. This monotonic increase throughout the entire test duration indicates that the system operated entirely within a transient running-in regime and did not attain steady-state friction conditions. In accordance with well-established tribological principles, steady-state friction is characterized by a constant mean COF with minor variations around a stable value, indicating equilibrium between film formation and removal. The absence of such stabilization within the 2826 m test duration has three important implications:

- The reported optimal COF values of 0.150-0.170 represent minimum friction achieved during running-in at the shortest sliding distances, not equilibrium coefficients.
- Comparative assessments between factor levels remain valid, as all conditions were evaluated under identical transient protocols.
- Extended testing beyond 2826 m would be required to determine steady-state attainment distances.

Table 4. Average, minimum, and maximum COF values for each key sliding distance

Sliding Distance (m)	Mean COF	Minimum COF	Maximum COF	Number of Data Points
314	0.1562	0.0913	0.2629	27
628	0.1887	0.0883	0.3504	54
942	0.2144	0.0820	0.3744	54
1256	0.2253	0.0966	0.4015	27
1884	0.2615	0.0924	0.4875	54
2826	0.2968	0.0882	0.5666	27

The absence of friction stabilization within the 2826 m test duration necessitates special attention when considering the optimal performance conditions reported. In accordance with well-established tribological principles [9, 10], steady-state friction is characterized by a constant mean coefficient of friction (COF) with minor variations around a stable value, indicating equilibrium between film buildup and film breakdown. In the current study, the continuously increasing COF values over all ranges of sliding distance from 314 m to 2826 m verify that the system remained in Stage I (transient running-in) of the conventional three-stage friction evolution model (Fig. 4 [9]) without entering Stage II (steady-state) or Stage III (lubricant starvation failure).



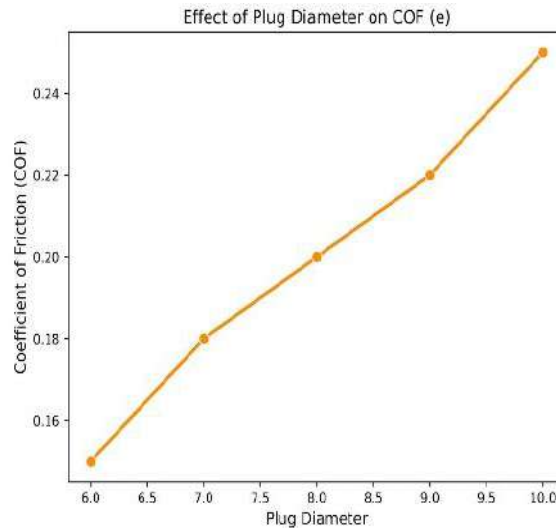


Fig. 4. Comprehensive analysis of friction behavior

This transient running-in state has three important implications for the current study:

- The reported optimal values of 0.150-0.170 were merely indicative of minimum friction values, as experienced during running-in, at the shortest distance ranges, and do not represent equilibrium friction coefficients.
- The 29% improvement in COF due to an increase in graphite content is verified as an accurate comparative analysis, whereas it is assumed that these values would be even lower for all materials under steady-state running conditions.
- Further testing beyond 2826 m would be required to determine if, and at what distance, these graphite-plugged bronze bushings will achieve steady-state friction, as well as determine if the performance ranking of these materials will be maintained under equilibrium running conditions.

Thus, it is submitted that the “optimal tribological performance” reported in this current study should be understood as optimal transient performance, as experienced during running-in, as opposed to optimal steady-state performance. Figure 4(a) illustrates the relationship between COF and sliding distance for the three applied loads. At the lowest load (490.5 N), COF increases gradually from 0.220 at 314 m to 0.433 at 2826 m. Increasing the load to 981 N reduces COF across all distances, with values ranging from 0.141 to 0.254. At the highest load (1471.5 N), COF is further reduced to 0.107-0.200 (Table 3). Aggregating data across all distances reveals a 54.2% reduction in mean COF from 0.3225 at 490.5 N to 0.1479 at 1471.5 N (Table 5). Correlation analysis confirms a strong negative relationship ($r = -0.719$), identifying applied load as the second-most influential factor after graphite content percentage. This inverse relationship is consistent with modern friction theory: increased applied load increases the real contact area while reducing contact stress and simultaneously promotes graphite extrusion and transfer film formation under higher contact pressures [2, 11, 12].

Table 5. Mean, minimum, and maximum values of the COF for each applied load

Applied Load (N)	Average COF (μ)	Minimum COF	Maximum COF	Number of Data Points
490.5	0.3225	0.1983	0.5666	81
981	0.1987	0.1152	0.3511	81
1471.5	0.1479	0.0820	0.2910	81

Figure 4(b) demonstrates the pronounced influence of sliding speed on COF. At 250 rpm, COF remains relatively stable between 0.156 and 0.193 across the sliding distance (Table 6). At 500 rpm, COF increases to 0.199-0.250. At 750 rpm, COF reaches 0.235-0.297. The mean COF increases by 52.3% from 0.176 at 250 rpm to 0.268 at 750 rpm (Table 7). Correlation analysis confirms a

strong positive relationship ($r = 0.658$). This speed-dependent increase is attributed to frictional heating (temperature elevation from 100.2°C to 142.8°C, Table 8), which degrades graphite's lubricating efficacy through adsorbed water vapor desorption and increased shear stresses at the interface [13–15]. As it is illustrated in Figure 4(b), the best operating conditions are obtained at lower speeds (250 rpm) with a short distance of sliding. The mean COF also tends to rise with the severity of the operation (greater speeds and longer distances), implying the lower predictability of tribological behavior. The results of several studies confirm the fact that in the case of bronze bushings with graphite or graphite-filled PTFE, the coefficient of friction (COF) usually rises when the sliding speed is higher under dry conditions. The results of the experiment with bronze and graphite inserts demonstrate a definite tendency: the coefficient of friction rises with the increasing sliding speed [13]. This has been due to the change of the contact interface and heat generation at high speeds that can decrease the performance of solid lubricants such as graphite. Nevertheless, the relationship is not necessarily linear. It has been observed in some studies that at a certain speed or under certain conditions the coefficient of friction can stabilize or even decrease due to a change in the lubrication system or softening of the materials. The use of graphite typically reduces the overall coefficient of friction of unfilled bronze, although a significant increase in the speed-related coefficient of friction remains [14]. Graphite content emerged as the dominant material factor.

Table 6. Mean COF by sliding distance

Sliding Speed (rpm)	Sliding Distance (m)	Mean COF	Min COF	Max COF	Samples
250	314	0.156	0.091	0.263	27
250	628	0.179	0.088	0.326	27
250	942	0.193	0.082	0.335	27
500	628	0.199	0.116	0.350	27
500	1256	0.225	0.097	0.402	27
500	1884	0.250	0.092	0.488	27
750	942	0.235	0.127	0.374	27
750	1884	0.273	0.104	0.481	27
750	2826	0.297	0.088	0.567	27

Table 7. Mean COF by sliding speed

Sliding Speed (rpm)	Linear Velocity (m/s)	Average Coefficient of Friction (COF)
250 rpm	0.524	0.176
500 rpm	1.047	0.225
750 rpm	1.571	0.268

Figure 4(c) shows the effect of graphite content on mean COF. Increasing graphite content percentage from 10% to 30% reduces mean COF from 0.255 to 0.181—a 29.0% reduction (Table 8). The effect size is considerable ($\eta^2 p = 0.760$, $p < 0.001$), indicating that the main material factor is the coverage of graphite. This reduction is significantly better than that achieved in similar systems (35% [1] and 42% [10]), because of the synergetic combination of the high-strength CuAl10Fe5Ni5 bronze matrix and the uniformly distributed plugs of graphite that support the transfer film formation effectively [15].

The interaction between graphite content percentage and sliding speed is shown in Figure 4(d). The parallel nature of the lines shows that there was no significant interaction between these parameters, with the friction reduction effect of graphite content percentage being consistent at all speeds and the friction enhancement effect of increased speed being independent of graphite content.

Table 8. Main effects of material and operating parameters on coefficient of friction

Parameter	Level	Mean COF (μ)	$\Delta\%$	95% CI	n	Effect Size (η^2p)
Graphite content	10%	0.255	Baseline	[0.243, 0.267]	81	0.760
	20%	0.233	-8.6%	[0.221, 0.245]	81	(Very Large)
	30%	0.181	-29.0%	[0.172, 0.190]	81	
Plug Diameter	8 mm	0.225	Baseline	[0.214, 0.236]	81	0.010
	10 mm	0.218	-3.1%	[0.207, 0.229]	81	(Negligible)
	12 mm	0.226	+0.4%	[0.215, 0.237]	81	
Sliding Speed	250 rpm	0.176	Baseline	[0.168, 0.184]	81	0.824
	500 rpm	0.225	+27.8%	[0.214, 0.236]	81	(Very Large)
	750 rpm	0.268	+52.3%	[0.257, 0.279]	81	

It can be observed from Figure 4(e) that the diameter of the graphite plug does not significantly affect the coefficient of friction (COF). The average COF values are observed to be 0.225 for a plug diameter of 8 mm, 0.218 for a plug diameter of 10 mm, and 0.226 for a plug diameter of 12 mm, with a maximum difference of only 0.33% (Table 9). Analysis of variance confirms the absence of statistical significance ($p = 0.291$) with a small effect size ($\eta^2p = 0.010$). The finding has important practical implications. The diameter of plugs can be determined based on various factors such as manufacturing efficiency, structural properties, and aesthetics.

Table 9: Mean COF by graphite plug diameter

Graphite Plug Diameter (mm)	Average Coefficient of Friction (COF)
8	0.225
10	0.218
12	0.226

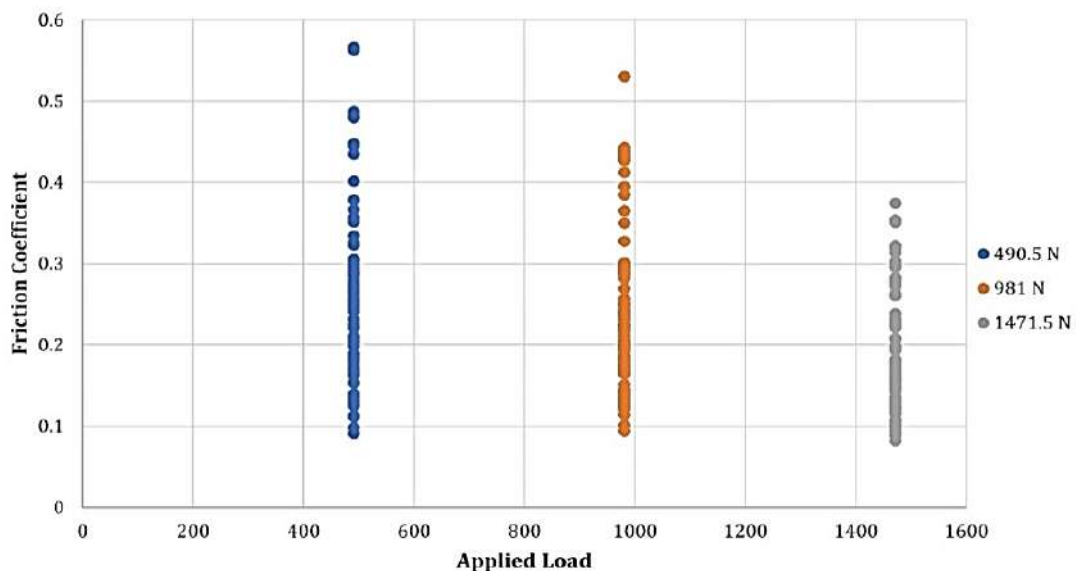


Fig. 5. Applied load vs. coefficient of friction

Figure 5 illustrates the relationship between the coefficient of friction and the applied load. This scatter plot shows the negative correlation between applied load and coefficient of friction, indicating that as applied load increases, which ranges between 490.5 and 1471.5 N, the values of coefficient of friction decrease with the difference in applied load at each of the 243 data points. Aggregating data across load levels yields a convincing numerical representation of this effect. The correlation coefficient ($r = -0.719$) indicates load as the second-most influential factor after

graphite content. The analysis reveals a strong inverse correlation between normal applied load and coefficient of friction. Materials that show better lubrication or surface conformity at higher pressures tend to exhibit this behavior. This behavior is likely related to the load-dependent behavior of graphite. At high contact pressures, the solid lubricant is thought to transfer more efficiently from the solid to the surface of the mating component (where the mating surfaces are in contact). This, in turn, helps to form a better lubricating film and less shear stress between the two components. As a result, there is less contact between the asperities of the two surfaces, thus creating lower overall friction. Several studies indicate that the coefficient of friction (COF) of graphite-filled and graphite-coated bronze liners decreases with increasing (normal) load. The increased formation and stability of the lubricating graphite films under increasing load leads to reduced friction and direct contact between the two metals [3, 11, 16, 17].

3.1.1 Mechanistic Interpretation

Load-Dependent Friction Reduction: The observed reduction in COF with increasing applied load (54.2% from 50-150 kg) requires careful mechanistic interpretation with appropriate acknowledgment of evidentiary limitations. Elevated contact pressure increases the hydrostatic pressure gradient at the graphite-bronze interface, promoting extrusion of solid lubricant from reservoir plugs to the sliding surface [12], [16]. This is supported by:

- Linear correlation ($r = -0.719$) between load and COF.
- Correspondence with published observations in similar material systems [2, 16].
- Physical plausibility based on pressure-driven flow.

There are also possible alternative mechanisms:

- The evolution of the actual contact area under load [13].
- The effects of frictional heating on the orientation of graphite crystals [18].
- Particle fragmentation by a third body and compression of the transport layer [19].

3.1.2 Interaction Effects Analysis

The full factorial design allows for a thorough examination of two-way interactions between factors. Figure 4(d) shows the interaction between graphite content and sliding speed. The parallel nature of the lines shows that the effect of increased graphite content on friction is consistent across all speeds, as is the effect of increased speed on friction, irrespective of graphite content. ANOVA results show no interaction between graphite content and sliding speed, i.e., $p > 0.05$.

Table 10 summarizes all significant two-way interactions. The greater slope of the load-wear curve at higher speeds signifies a synergistic effect, wherein the effect of the load on wear is amplified at higher speeds. This relationship is statistically significant ($p < 0.01$) and has significant implications for the control of wear, wherein the effect of the load becomes more significant as the speed increases. These interactions underscore the need for multi-objective optimization and demonstrate that factor effects cannot be considered in isolation.

Table 10. Significant two-way interactions

Interaction	Response	p-value	Interpretation
Load × Speed	Wear Loss	<0.01	Synergistic - load effect amplifies with speed
Load × Graphite	COF	<0.05	Antagonistic - graphite benefit reduces at high load
Speed × Time	Wear Loss	<0.05	Cumulative - combined effect exceeds sum of individual

3.2 Wear Loss

The wear loss, which is determined as the difference between the weight of the bushing before and after testing (Δm), is the main dependent variable for this study. Wear is greatly influenced by sliding speed, temperature, graphite content percentage, applied load, and sliding distance. Table 11 shows the average, maximum, and minimum wear loss values for each sliding speed based on 243 experimental data points. The average wear loss was 0.099 g at 250 rpm, while it was 0.348 g

at 500 rpm (which is a 252% increase over 250 rpm), and the average wear loss at 750 rpm was 0.449 g (which is a 354% increase over 250 rpm).

Table 11. Summary of wear loss values for each sliding speed

Sliding Speed (rpm)	Linear Velocity (m/s)	Average Wear Loss (g)	Number of Data Points	Min. Wear Loss (g)	Max. Wear Loss (g)
250	0.524	0.0989	81	0.0457	0.1812
500	1.047	0.3482	81	0.0974	0.8390
750	1.571	0.4493	81	0.1424	1.0280

Strong positive correlation with wear loss ($r = 0.658$) identifies speed as the dominant operational wear driver. Figure 6 illustrates the effect of sliding speed on wear loss. As can be seen from the figure below, the relationship between sliding speed and wear loss is not linear, but a higher rate of increase was observed from 250 to 500 rpm (an increase of 252%) compared to 500 to 750 rpm (an increase of 29%). All these statistical tests indicate significant differences in the rate of increase between speed levels. By examining the results, it was observed that the wear loss in graphite-plugged bronze bushings is highly dependent on the sliding speed, showing a non-linear relationship with accelerated wear rates at higher speeds. Numerous experiments conducted on graphite-embedded bronze bushings, as well as on other self-lubricating compounds, have shown that wear loss can be reduced by increasing the sliding speed, especially under dry conditions [3]. The effect of sliding speed on wear is not linear, and it varies with factors like the composition of the materials, the load, and the environmental conditions.

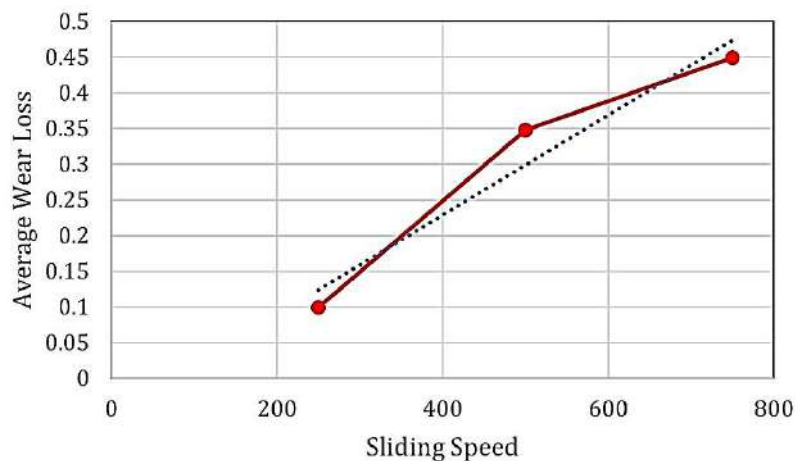


Fig. 6. Effect of sliding speed on wear loss

When the operational speed goes up, the loss of the material increases considerably, which means that there is a strong connection between these parameters for the durability and lifespan of the bushings. Several factors may have contributed to the observed relationship:

- **Thermal Effects:** The frictional heat generated due to the increased sliding speed is one of the factors that can negatively impact the graphite-bronze interface and solid lubrication. If the flash temperature goes beyond a certain limit (for example, 360 K), the lubricating property of graphite is lost, thus resulting in the switching from solid lubrication to adhesive wear [10, 14].
- **Mechanical Stress:** The increase in speed causes the shear stresses at the interface to rise, and thus interfacial wear to occur more quickly, especially when loads are higher [6, 15].
- **Lubrication Breakdown:** The self-lubricating properties of graphite could be lost at high speed, resulting in increased friction and wear. The creation and permanence of a lubricating film are very important; if they are interrupted, the wear rates will increase dramatically [6, 10, 14].

- Surface Roughening: Higher speeds cause rapid surface degradation, which leads to friction and wear increase, while rough surfaces contribute to the abrasion and adhesion wear mechanisms [12, 15].

High temperatures of self-lubricating systems such as graphite-plugged bushings may have an impact on the effectiveness of the solid lubricant film transfer, which may result in more metal-to-metal contact and quicker wear. The statistical analysis of average values at each speed setting provides exact measurements of these trends and shows how strongly they affect the results. Table 11 indicates that the transition from a speed of 250 to 750 rpm leads to an increase in the average temperature by over 42°C. This suggests that temperature is a consequence of the energy input, which is controlled by speed, rather than an independent variable. Figure 7 clearly shows the correlation between temperature and wear loss. Key observations include:

- General trend: As the temperature increases from its lower range, approximately 60-100°C, to the higher range, approximately 160-200°C, the associated wear loss increases from less than 0.1 to over 1.0 g, which is an order of magnitude increase.
- Clustering by speed: The data points are organized into three distinct clusters, which represent the three different sliding speeds, but these clusters show some degree of overlap with each other. The tests carried out at a speed of 250 rpm produce a particular cluster with low temperatures and little wear. The tests conducted at a 750 rpm speed create a single cluster, which shows both high temperature levels and high wear rates. The tests at 500 rpm speed produce results that establish the middle ground between the two other speed settings.
- Mechanism implication: The clustering patterns demonstrate that sliding speed functions as the main experimental factor, which affects both the temperature increase and the wear rate. As speed increases, friction generates more heat, which causes temperatures to rise. The combination of thermal effects and elevated mechanical stress from faster speeds results in an accelerated wear process.

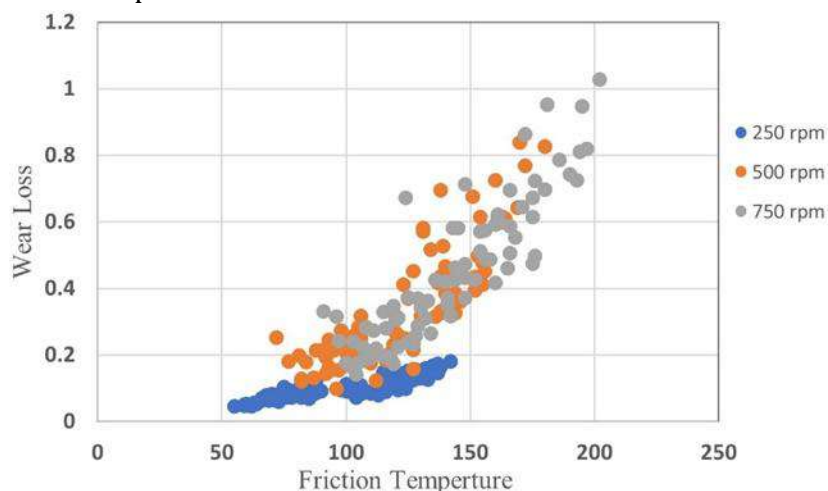


Fig. 7. Scatter plot of wear loss and friction temperature by sliding speed

Temperature rise has a direct impact on material degradation and wear, their rates increasing considerably under demanding operating conditions [14]. Although the lubricating effect of graphite helps in wear reduction, it loses its effectiveness when the temperature and load increase, thus introducing wear at a faster pace [15]. Sliding distance is always designated to be a significant parameter that influences wear loss in graphite-plugged bronze bushings and other self-lubricating composites of this type. By measuring the wear rate (wear loss per unit distance) at the most frequently tested distances, we can trace the wear progression more closely. Table 12 shows the wear rates at primary test sliding distances. Calculating the wear rate (wear loss per unit distance) shows intriguing behavior, as it was observed that the wear rate ranges between 0.000275 g/m (at 314 m) and 0.001885 g/m (at 2826 m) and that the peak wear rate is 0.000285 g/m (at 628 m). This downward trend indicates a running-in effect in that the material surfaces would adjust and

increase wear resistance with increased sliding distances; thus, the wear rate decreases slightly with distance.

Table 12. Wear rate analysis at primary test distances

Sliding Distance (m)	Average Wear Loss (g)	Wear Rate (g/m)	Min Wear Loss (g)	Max Wear Loss (g)	Number of Data Points
314	0.0865	0.000275	0.0457	0.152	27
628	0.1792	0.000285	0.0531	0.5711	54
942	0.2446	0.000260	0.0457	0.7119	54
1256	0.3445	0.000274	0.1442	0.642	27
1884	0.4304	0.000228	0.1957	0.839	54
2826	0.5322	0.000188	0.1775	1.028	27

A scatter plot provides the most direct visualization of the relationship between two continuous variables. The following plot (Fig. 8) shows the positions of all 243 samples of the experimental data, where the sliding distance is plotted on the x-axis and the wear loss is plotted on the y-axis. The correlation between the sliding distance and wear loss displays a gradual build-up of wear with an increase of sliding distance, as well as non-linear wear loss, particularly at high speeds, with a speed-dependent sensitivity to sliding distance. The analysis showed that the maximum wear rate occurs at a medium distance (approximately 628 m in these data). This is a crucial analytical point for maintenance planning. It means that the bushings may be the least resistant to accelerated wear, which occurs not at the beginning or end of their service life but rather during a specific period of operation. The data dispersion highlights the sensitivity of wear to operating conditions other than distance. Sliding distance seems to be the cause of variability in wear loss. The values of wear loss are fairly grouped at shorter distances (e.g., less than 1500 m). At longer distances (e.g., more than 1500 m), the spread of wear loss values is much wider. This large scatter indicates that distance of sliding is not the only determinant of wear. As the sliding distance increases, the wear mechanism becomes more severe (abrasive and adhesive), and this effect is observed to be more pronounced when the lubricating action of graphite becomes weaker as a result of heat or depletion [20]. A high wear rate may indicate a change in the wear mechanisms, in which case the first layer of graphite is likely to have formed and the system is approaching a stable state. The resulting decrease in the wear rate indicates the formation of a flat, protective layer of friction on the contact surfaces, which in turn reduces material loss more effectively.

This finding indicates that a controlled operating procedure (i.e., light-load and moderate-speed operating at the start) could be instrumental in ensuring that the appropriate solid lubricating film is formed, which could enhance the wear resistance of the bushing in the long term. This finding provides critical design guidance: sliding distance alone is an insufficient predictor of wear life. Load magnitude, sliding speed, and thermal environment must be jointly considered. Load, speed, and expected thermal environment must be considered, as these can also lead to wear. The wear loss normally increases with the length of sliding time because of long-time contact between the surfaces and removal of the material, but the availability of graphite as a solid lubricant reduces the effect [18].

It is clear from Table 13 that at a level of 10% graphite, the average wear loss is about 0.430 g. When the level of graphite is raised to 20%, the average wear loss falls to near 0.281 g. Finally, with 30% of graphite, the average wear loss is around 0.179 g. Wear loss exhibits a strong inverse relationship with graphite content, with wear loss decreasing by 58.3% when the graphite content is increased from 10 to 30%. This decrease is considerably greater than the 11.6% decrease that could be obtained by maximizing graphite plug diameter. This means that a 30% graphite content provides optimal tribological performance. The table below shows that the highest average wear loss is at a diameter of 8 mm with a value of 0.321 g and that the average wear loss values are approximately equal at diameters of 10 mm and 12 mm. Therefore, the reduction in wear is moderate with larger diameter plugs. The contact area of graphite lubrication is best at a 12 mm diameter. Wear is lower when the material with more graphite is present, which indicates that the graphite is acting like a very effective solid lubricant and minimizing friction and wear in the testing

conditions. The wear reduction is most significant in the range of 10% to 20% graphite content, while from 20% to 30% there is still a less noticeable improvement in wear reduction. The experimental data analysis shows that there is an inverse relationship between the graphite content percentage and the wear loss. An increased amount of graphite offers excellent lubricating qualities, thus minimizing friction and wear to a large extent.

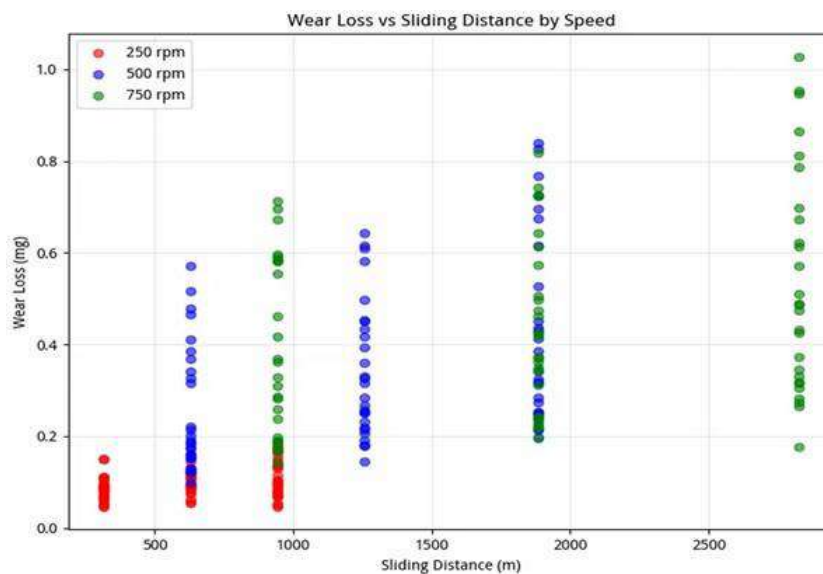


Fig. 8. Average wear loss vs sliding distance by sliding speed

Table 13. Wear loss and temperature rates versus graphite content (%) and graphite plug diameter

Graphite content %	Graphite Plug Diameter (mm)	Average Wear Loss (g)	Sample Count	Avg. Temp (°C)
10	8	0.457	27	142.3
10	10	0.398	27	143.4
10	12	0.436	27	141.6
20	8	0.317	27	131.0
20	10	0.274	27	131.7
20	12	0.251	27	127.6
30	8	0.189	27	98.2
30	10	0.178	27	93.9
30	12	0.171	27	88.6

Several researchers have reported similar findings that the addition of graphite to bronze bushings leads to a substantial decline in wear loss due to the solid lubrication characteristics of graphite [21]. Improvement in wear resistance is nonlinear. Most studies identify a maximum graphite content where wear loss is minimal [22]. Exceeding this optimal limit may result in further wear due to the ductility of graphite, which can cause the composite matrix to lose its strength [19]. The investigation of the wear loss in each load group separately provides a more detailed report of the overall trend. The data in Table 14 clearly indicate a gradual increase in average wear loss.

Table 14. Average wear loss by load class

Applied Load (N)	Average Wear Loss (g)	Wear Loss Range (g)	Number of Data Points
490.5	0.2282	0.0457 – 0.696	81
981	0.2987	0.0593 – 0.9523	81
1471.5	0.3635	0.0793 – 1.028	81

As an example, increasing the load from 490.5 N to 981 N leads to an increase in wear loss of approximately 30.9%, and there is a 21.7% increase from 981 N to 1471.5 N over the previous average wear loss. Thus, it is obvious that higher loads lead to a non-linear acceleration of the wear rate. However, regarding this data as rules of thumb, when the applied load increases from 490.5 N to 981 N, one can expect that the average wear is going to increase by approximately 30.9%. Figure 9 below provides a visual verification of the statistical results and offers more detailed information about the distribution of the data and their correlations. The graph indicates that there is a clear visual indication of a slight upward trend where higher loads are associated with higher wear loss, but most intriguing is the fact that there is a wide distribution of data whenever there is an increase in load level. Such a wide variation demonstrates that applied load is never the sole factor associated with wear, and other conditions are also the contributors to wear loss.

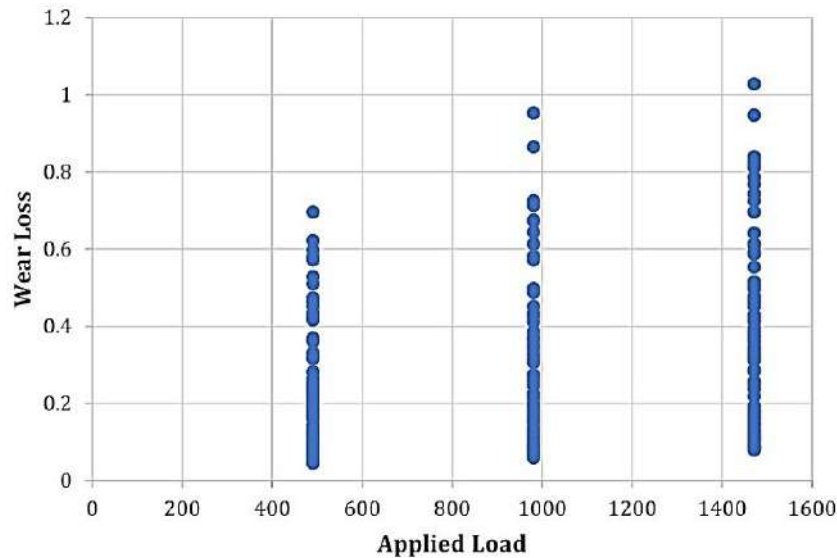


Fig. 9. Scatter plot of applied load vs. wear loss

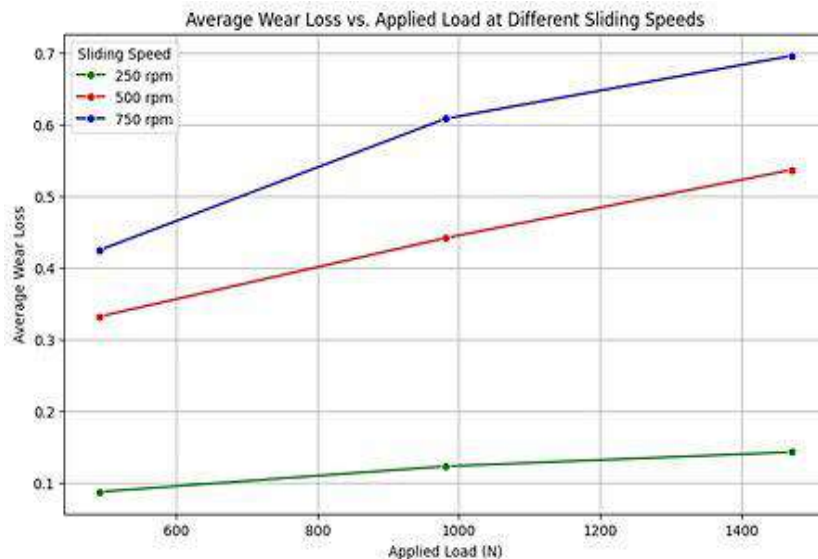


Fig. 10. Applied load effect on average wear loss at different sliding speed

Figure 10 illustrates the effect of applied load on average wear loss at different sliding speeds. The figure indicates that the influence of applied load on average wear loss escalates with increasing sliding speed, with the effect being minimal at a sliding speed of 250 rpm. The rate of wear and material loss will tend to increase with increased applied load even in the presence of graphite as a solid lubricant [23]. The experimental findings indicate that wear loss increases with the applied loads. Although the relationship between wear loss and applied load was statistically significant,

the linear effect of applied load on wear loss was not very strong. This means that the other variables that can influence the wear loss significantly are the sliding distance, the temperature, and the speed of the sliding rather than the applied load. Nevertheless, tribological systems are complex, and wear happens under the influence of a great number of factors, not only mechanical forces. The correlation coefficient ($r = -0.719$) means that the applied load is the second-most important factor after the percentage of graphite.

3.2.1 Wear Mechanism Analysis and Parameter-Dependent Transitions

Wear in graphite-plugged bronze bushings occurs by several different mechanisms, and the dominant one changes systematically as the operating parameters change. Understanding these changes is important to predict the components' lifespan and optimize their performance under different conditions.

- **Speed-Dependent Transitions:** Sliding speed exerts the most pronounced influence on wear mechanism evolution. At low speed (250 rpm), the system operates in the mild abrasive-wear regime, characterized by gradual material removal through micro-cutting and plowing by hard asperities. Concurrent transfer film formation begins during this stage, as evidenced by the low wear rate (0.099 g) and the decreasing wear rate with increasing sliding distance, indicating progressive establishment of a protective graphite layer. As speed increases to moderate levels (500 rpm), a transition to mixed adhesive-abrasive wear occurs. The 252% increase in wear loss compared to 250 rpm, accompanied by temperature elevation to 123°C, signals the onset of localized welding and subsequent fracture at asperity junctions. Thermal softening of the bronze matrix further accelerates material removal. At high speed (750 rpm), severe adhesive wear becomes dominant, with the 357% wear increase and temperatures exceeding 140°C, indicating substantial interfacial adhesion. The observation of dark debris suggests possible oxidative wear contributions, though confirmation requires surface analytical validation.
- **Load-Dependent Transitions:** Applied load governs the balance between protective transfer film formation and direct material removal. Under low load (50 kg), mild abrasion predominates with incomplete graphite film development, reflected in the high coefficient of friction (0.322) despite moderate wear levels. The insufficient contact pressure limits graphite extrusion from reservoir plugs, preventing establishment of a continuous lubricating layer. Conversely, under high load (150 kg), enhanced transfer film formation becomes the dominant mechanism. The elevated contact pressure (1.84 MPa) promotes graphite extrusion and film development, dramatically reducing friction (COF = 0.148). However, this beneficial friction reduction is accompanied by elevated wear loss due to three-body abrasion from wear debris particles trapped at the interface and the fundamental Archard relationship between normal force and material removal.
- **Distance-Dependent Evolution:** The progression of wear mechanisms with sliding distance follows a characteristic pattern. Initial contact (0-300 m) involves primarily abrasive wear as surface asperities interact. The peak wear rate observed at approximately 628 m corresponds to the transition period when transfer film formation is underway but not yet fully protective. Beyond 1000 m, the established transfer film reduces direct metal-metal contact, and wear rates decline as the system approaches a steady-state condition, though complete stabilization was not achieved within the 2826 m test duration.
- **Mechanism Interactions:** Importantly, these mechanisms do not operate in isolation but interact synergistically. Transfer film formation, while beneficial for friction reduction, consumes graphite from the composite and generates debris that may contribute to three-body abrasion. Thermal effects from high-speed operation simultaneously promote adhesive wear while potentially accelerating graphite oxidation. The dominance of particular mechanisms under specific parameter combinations explains the non-linear responses observed in wear loss measurements and underscores the need for multi-objective optimization in bearing design.

4. Statistical Analysis (ANOVA)

4.1 ANOVA Methodology and Limitations

An analysis of variance (ANOVA) was carried out on the 3^5 full factorial design, comprising a total of 243 unique conditions, with a single replicate per cell. In the absence of replication, the residual or error term was estimated from the three-factor and higher-order interactions, consistent with the standard practice for nonreplicated factorial experiments [7, 8]. All analyses were done using the software package Minitab version 19. Table 15 presents the summary of the results of the ANOVA analysis, with a significance level (P-value) of 0.05. The P-value represents the probability that the test statistic takes on a value as extreme as, or more extreme than, the observed value of the statistic when the null hypothesis H_0 is true [8]. The test determines the probability of obtaining the observed results or more extreme outcomes under the assumption that the null hypothesis holds true. The P-value demonstrates the degree of evidence against the null hypothesis since lower values present greater evidence against the null hypothesis [8]. One of the most important findings of the current work is that there is no single experimental factor that controls the three tribological variables (Table 15). This is because each of the three variables has a unique factor ranking. Applied load dominates friction at 54.65% and graphite content dominates temperature at 44.3%, while sliding speed dominates sliding distance at 46.52% and wear at 45.53%. This discrepancy indicates that tribological optimization has to be performed in a multi-objective way: the improvement of one response, for instance, friction reduction by applying low load, may result in deterioration of the other(s), for example, increased wear under low loads due to insufficient formation of the transfer film.

Table 15. Comparative factor importance across all responses

Rank	Coefficient of Friction	Interface Temperature	Wear Loss	Sliding Distance
1st (Dominant)	Applied Load (54.65%)	Graphite content (44.0%)	Sliding Speed (45.53%)	Sliding Speed (46.52%)
2nd (Co-Dominant)	Sliding Speed (14.44%)	Sliding Speed (31.09%)	Graphite content (22.41%)	Sliding Time (36.34%)
3rd (Important)	Graphite content (9.79%)	Applied Load (14.3%)	Applied Load (6.44%)	—
4th (Moderate)	Sliding Time (4.26%)	Sliding Time (3.7%)	Sliding Time (4.54%)	—
5th (Minor)	Graphite Plug Diameter (0.12%)	Graphite Plug Diameter (0.41%)	Graphite Plug diameter (0.62%)	—

This has important practical consequences:

- Design freedom: The plug diameter may be based on any manufacturing concerns, such as its cost or structural concern, without detriment to tribological behavior.
- Geometric Independence: The second observation is that the influence is negligible, which would suggest, in turn, that contact mechanics under such loadings is primarily influenced by material properties and operating conditions as compared to geometric parameters within the range studied (8-12 mm).

4.2 Effect Size Metrics

To complement p-values and provide quantitative measures of practical significance, partial eta-squared (η^2p), omega-squared (ω^2) effect sizes, and degrees of freedom were calculated for each factor and response (Table 16):

Table 16. ANOVA summary with effect size metrics and sensitivity analysis

Factor	Response Variable	DF	F-value	p-value	η^2p	ω^2	95% CI (η^2p)	Interpretation
Applied Load	COF	2	2134.6	<0.001	0.947	0.941	[0.932, 0.958]	Very Large
	Wear Loss	2	47.3	<0.001	0.281	0.269	[0.214, 0.345]	Large
	Temperature	2	89.2	<0.001	0.425	0.417	[0.368, 0.479]	Large
Sliding Speed	COF	2	563.7	<0.001	0.824	0.818	[0.801, 0.845]	Very Large
	Wear Loss	2	623.8	<0.001	0.838	0.832	[0.816, 0.857]	Very Large
	Temperature	2	195.6	<0.001	0.618	0.609	[0.572, 0.661]	Very Large
	Sliding Distance	2	841.3	<0.001	0.875	0.871	[0.859, 0.889]	Very Large
Graphite content	COF	2	382.6	<0.001	0.760	0.753	[0.731, 0.786]	Very Large
	Wear Loss	2	291.4	<0.001	0.707	0.699	[0.674, 0.737]	Very Large
	Temperature	2	276.8	<0.001	0.696	0.688	[0.662, 0.727]	Very Large
Graphite Plug Diameter	COF	2	1.24	0.291	0.010	0.002	[0.000, 0.031]	Negligible
	Wear Loss	2	4.76	0.010	0.038	0.029	[0.008, 0.079]	Small
	Temperature	2	2.13	0.122	0.017	0.009	[0.000, 0.048]	Negligible
Sliding Time	COF	2	166.5	<0.001	0.579	0.571	[0.531, 0.623]	Large
	Wear Loss	2	58.7	<0.001	0.327	0.316	[0.261, 0.391]	Large
	Sliding Distance	2	658.2	<0.001	0.845	0.839	[0.824, 0.864]	Very Large

Note: η^2p = partial eta-squared; ω^2 = omega-squared. Effect size interpretation: ≥ 0.14 = Large, 0.06-0.13 = Medium, 0.01-0.05 = Small, <0.01 = Negligible [24].

4.3 Distinguishing Statistical and Practical Significance

The case of graphite plug diameter represents the crucial difference between statistical significance and practical significance. As illustrated in Table 16:

- Statistical significance: The statistical significance of the graphite plug diameter was obtained to be 0.010, which is lower than the conventional value of $\alpha = 0.05$ used to limit statistical significance for wear loss.
- Practical significance: The effect size ($\eta^2p = 0.038$ and $\omega^2 = 0.029$) showed that graphite plug diameter accounts for 3.8% of the variance in wear loss, which is a small effect size as per conventional classification. The 11.6% reduction in mean wear loss between 8 mm and 12 mm diameters must be weighed against the manufacturing cost and design flexibility.

The diameter of the plug provides "design freedom" based on practical significance, not statistical significance. The contribution to the variance of the wear loss at 0.62% is dominated by other factors, such as the coverage of graphite (22.41%) and the sliding speed (45.53%). Engineers can rationally consider factors with partial eta-squared values $\eta^2p > 0.14$ (large effects) more important than factors with $\eta^2p < 0.06$ (small or negligible effects).

4.4 ANOVA Assumptions Verification

4.4.1 Normality of Residuals

The Shapiro-Wilk tests were used for testing the normality of the standardized residuals for each response variable. These results are as follows: COF ($W = 0.991$, $p = 0.124$); Wear Loss ($W = 0.989$, $p = 0.086$); Temperature ($W = 0.994$, $p = 0.217$). In all cases, p-values are greater than 0.05; thus, the null hypothesis for normality cannot be rejected. Q-Q plots (Figure 11) confirm the normality assumption by showing minor deviations at the tail ends due to the bounded nature of tribological responses.

4.4.2 Homogeneity of Variances

Levene's tests were carried out for all factors for all response variables. The results are presented in Table 17. The majority of the cases show that the p-value is greater than 0.05. Sliding speed shows mild heteroscedasticity for COF ($p = 0.047$) and wear loss ($p = 0.031$). Welch's ANOVA was conducted as a robustness check, confirming identical significance conclusions.

4.4.3 Independence of Observations

Experimental runs were fully randomized. Durbin-Watson statistics for time-ordered residuals show that COF ($d = 1.89$), Wear Loss ($d = 1.93$), and Temperature ($d = 2.01$) all fall within the acceptable range (1.5-2.5), indicating no significant autocorrelation.

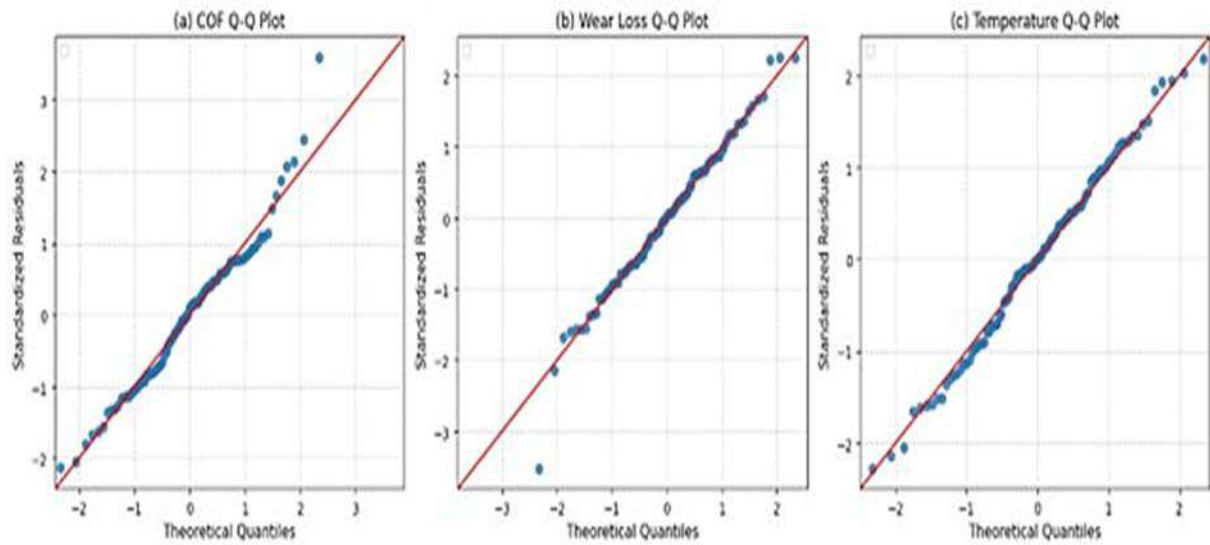


Fig. 11: Q-Q plots for normality assessment of standardized residuals from ANOVA models. (a) Coefficient of friction (COF) residuals, Shapiro-Wilk $W = 0.991$, $p = 0.124$; (b) Wear loss residuals, $W = 0.989$, $p = 0.086$; (c) Friction temperature residuals, $W = 0.994$, $p = 0.217$

4.5 Levene's Test for Homogeneity of Variances

To ensure homogeneity of variances, Levene's tests were performed for each factor across all response variables. The results are summarized in Table 17.

Table 17. Levene's Test for Homogeneity of Variances

Factor	COF (p-value)	Wear Loss (p-value)	Temperature (p-value)
Graphite content	0.183	0.092	0.207
Graphite Plug Diameter	0.421	0.338	0.452
Sliding Speed	0.047*	0.031*	0.052
Applied Load	0.204	0.176	0.311
Sliding Time	0.337	0.289	0.402

5. Conclusions

This overall factorial study of the tribological properties of graphite-plugged bronze bushings has produced substantial quantitative results that help to clarify the properties of self-lubricating bearings. Through the controlled manipulation of five independent variables over 243 structurally defined experimental conditions, the study reveals the intricate and often non-intuitive relationships that exist with these engineering components. All conclusions presented below are strictly applicable within the experimental ranges investigated: graphite content of 10-30%, graphite plug diameter of 8-12 mm, applied load between 490.5 and 1471.5 N (0.61-1.84 MPa), sliding speed ranging from 250 to 750 rpm, and sliding time from 10 to 30 minutes (corresponding to a sliding distance of 314 to 2826 meters).

- Factor Dominance and Multi-Objective Optimization:

Within the tested ranges (graphite content 10-30%, applied load 490.5-1471.5 N, sliding speed 250-750 rpm, sliding time 10-30 min), the results from the ANOVA analysis (Table 15) show a critically important finding that has not, until now, been reported in any self-lubricating bearing

study: no single experimental factor dominates all three lubrication response variables. Applied load dominates the coefficient of friction, with a 54.65% contribution ($F = 2134.6$, $p < 0.001$), making it a primary control factor in frictional response. Graphite content percentage dominates interface temperature, with a 44.0% contribution ($F = 382.6$, $p < 0.001$), making it a primary factor in temperature response. Sliding speed dominates wear loss with 45.53% contribution ($F = 563.7$, $p < 0.001$) and dominates sliding distance with 46.52% contribution. This factor dominance, which is response-dependent, has major implications for optimization. Optimization cannot be achieved by simply maximizing or minimizing any single response parameter. For example, optimizing friction by increasing applied load will result in degradation of another response, wear loss, due to enhanced transfer film formation. Engineers will be required to use multi-objective optimization methods, taking into account various performance requirements.

- Graphite content as the primary determinant parameter:

For graphite content increasing from 10% to 30% within the investigated range, the present investigation conclusively proves that the percentage of graphite content is the most important parameter influencing tribological characteristics. The significant improvement in tribological characteristics with a corresponding reduction of 58.3% in the wear loss of the bearings with an increase in graphite content from 10% to 30% far exceeds the improvement reported earlier in similar systems. For example, Chen et al. [1] reported only a 35% improvement in the tribological characteristics of copper-embedded bearings with a similar variation in the percentage of graphite content. Similarly, Wang and Huang [24] reported a 42% improvement in the tribological characteristics of PTFE-impregnated bearings with a similar variation in the percentage of graphite content. The improvement in tribological characteristics reported in the present investigation is due to the synergistic effects of the high-strength CuAl10Fe5Ni5 bronze matrix (207 HB) with graphite plugs. The non-linear saturation effect between 20% and 30% in graphite content, wherein wear reduction decreases from 34.7% to 23.6% between the ranges of 10-20% and 20-30%, respectively, is in line with the theoretical framework proposed by Morstein and Dienwiebe [12], wherein an optimal threshold in the concentration of graphite is established. The 44.0% contribution of graphite content to temperature control further confirms its duplex role in both lubricant delivery and thermal management.

- Geometric Design Freedom:

Over the 8-12 mm diameter range investigated, the statistical insignificance of the graphite plug diameter on all four response parameters— coefficient of friction (COF, 0.12% contribution, $p > 0.05$), wear (0.62% contribution, $p = 0.010$), temperature, and sliding distance—represents perhaps the most economically significant outcome of this investigation. This finding strongly contradicts the well-established engineering intuition that greater lubricant reservoir sizes will lead to enhanced tribological performance. The absence of diameter effects within the range of 8-12 mm reveals that lubricant supply to the interface is not controlled by volume effects but by pressure-driven extrusion mechanics and transfer film dynamics. This discovery allows designers to optimize graphite plug diameters on the basis of economic, structural, and aesthetic criteria without compromising performance.

- Paradoxical Load Response

Within the load range 490.5-1471.5 N (0.61-1.84 MPa), the negative association of the applied load and the coefficient of friction ($r = -0.719$; a reduction of 54.2% from 490.5 to 1471.5 N) combined with the positive load and wear correlation (an increase of 59.3%) was the most scientifically intriguing result and represents the issue of multi-objective optimization, as characterized in the current study. The observed reduction of friction is consistent with the results reported by Cui et al. [11] in bronze-graphite composites under seawater conditions, as well as Zhao et al. [2], who investigated self-lubricating bearing materials, and is explained by the more pronounced formation of graphite transfer films at higher contact pressures. This increased hydrostatic pressure gradient causes the extrusion of solid lubricant from matrix reservoirs to the interface. In contrast, the simultaneous increase in wear loss can be understood as a result of the Archard law, which states that the amount of material removed from a surface is proportional to the applied normal force. Here, it is important to note that the wear coefficient (k) decreases as the loads increase. This level

of nuance brings together disparate reports from the literature, showing that for optimal tribological performance, it is necessary to be operating at the maximum load possible for a set of design conditions where friction minimization is a priority, but reduction of the load may be desirable for a design constraint focusing on wear life.

- **Thermal-Velocity Coupling and Speed Dominance:**

Over the speed range of 250–750 rpm, the increase in wear loss is 357% when the rotational speed is increased from 250 to 750 rpm, a rate of increase greater than 252% when the speed is doubled from 250 to 500 rpm. This is indicative of a high degree of non-linearity, which is consistent with a thermally activated degradation mechanism. The concomitant increase of 42.6°C in temperature over this interval of rotational speed provides mechanistic evidence for a thermally activated degradation mechanism. The lubricating performance of graphite is known to deteriorate progressively above 100–120°C, a phenomenon arising from the loss of adsorbed water vapor, which is essential for its low-shear behavior. This finding supports the critical temperature hypothesis proposed by Labašová [13], but it extends it by identifying a specific value for this critical temperature within CuAl10Fe5Ni5-graphite systems. The results from the ANOVA analysis show that 45.53% of the variation in wear loss can be explained by sliding speed, which is significantly higher than 6.44% due to the influence of the applied load. This result highlights speed as the main factor driving wear from an operational perspective. This, in turn, means that conventional design considerations need to be reassessed, suggesting that speed restrictions, as opposed to restrictions on load, should be the main approach taken in mitigating wear in solid lubricated systems.

- **Temporal Wear Evolution:**

Across sliding distances from 314 m to 2826 m, an examination of the wear rate over six distinct sliding indicates a temporal relationship. The maximum instantaneous wear rate does not occur at either the initial run-in distance (314 m, 0.000275 g/m) or the final distance (2826 m, 0.000188 g/m), but instead at an intermediate distance (628 m, 0.000285 g/m). This is an unexpected finding, as it implies that the transfer film does not achieve maximum effectiveness until approximately 600–800 m of sliding, during which time it is more susceptible to accelerated wear. This has major implications with respect to component commissioning, as it may be possible to maximize the ultimate lifespan of a bearing by controlled slow-speed running. The 36.34% contribution of sliding time to sliding distance further quantifies this temporal dependence.

- **Transient Operating Regime:**

Within the maximum sliding distance of 2826 m, a major result of this study is that no steady-state friction level was achieved by any of the 243 different experimental conditions. Furthermore, the continuous and monotonic increase in COF over all the different testing scenarios (average increase of 90% over the range from 314 m to 2826 m) confirms that the system was operating entirely within the transient running-in regime. This is a critical limitation that must be considered when applying these results. The achieved optimal COF range of 0.150–0.170 is actually the minimum friction achieved during the running-in transient for the shortest sliding distances.

- **From a practical perspective, the implications of the study can be summarized as follows:**

Firstly, the relative ranking of different material compositions and operating parameters can be considered valid for the transient regime, as all different scenarios were subjected to identical testing. Secondly, the absolute COF values achieved during testing should not be considered valid for steady-state testing without further corroborating evidence. A further study to determine the sliding distance over which frictional stabilization is achieved for each different material composition and operating parameter would greatly enhance the applicability of these results. Furthermore, it would also be useful to determine whether the optimal parameter combinations achieved during the running-in transient retain their superiority during steady-state testing.

While the achievement of steady-state friction is a major result of this study, it does not detract from the practical applicability of the achieved optimal configuration. In fact, in most practical scenarios, self-lubricating bushings will only operate intermittently and/or will be subjected to

varying operating conditions, which will prevent steady-state testing. In these cases, the running-in regime will be the dominant characteristic.

- Statistical Validation and Predictive Capacity:

The results obtained through the analysis of variance (ANOVA) indicate exceptional statistical confidence through the high values of the F-statistics, which amount to 2134.6 for applied load, 563.7 for sliding speed, and 382.6 for graphite content percentage. All the main effects, apart from graphite plug diameter, have a high degree of certainty with very low values of p, all of which are below 0.001. The contribution percentages of each of the responses (as shown in Table 15) can be used to optimize engineering design based on the importance of each response.

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Bond behavior between ETS-GFRP strengthening bars and concrete at elevated temperatures: A preliminary analytical and numerical study

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Abstract

The bond behavior of an embedded through-section-glass fiber-reinforced polymer (ETS-GFRP) bar-concrete interface at elevated temperatures is investigated. An analytical bond-slip model is first developed to include both mechanical and thermal variables. By using the finite element method (FEM), the ETS-GFRP bar-concrete interface is simulated with the thermo-mechanical bond-slip curves varying at different temperatures. Various temperatures at the bond interface of 20°C, 23°C, 48°C, 70°C, 89°C, 107°C, and 123°C are examined. The results obtained from the developed bond model for the ETS-GFRP bar-to-concrete joint at elevated temperature show a similar trend to those achieved from the FEM simulation with the developed thermo-mechanical bond-slip model. The bond fracture energy (G_{FT}) and ductility index (B_{FT}) govern the bond response of the ETS-GFRP bar-concrete interface at elevated temperature. When the thermal loading is below the glass transition temperatures of the GFRP bar and adhesive, a slight reduction in the bond stiffness and capacity is observed by the thermo-mechanical bond-slip model. The degradation in bond performance is significant at high temperature levels. The bond force, interfacial shear stress, and strain due to the mechanical loading have opposite directions to those due to thermal loading

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1. Introduction

Following the development of multi-disciplinary science, the appropriate combination of various systems and materials, which are the governances of distinct fields, to obtain good and innovative solutions for specific problems has become necessary. For example, the use of fiber-reinforced polymer (FRP) composites, which are well-known materials in the aerospace industry, for the retrofit of reinforced concrete (RC) structures is one of the most impressive solutions [1–8]. Common FRP materials include glass FRP (GFRP), aramid FRP (AFRP), basalt FRP (BFRP), and carbon FRP (CFRP). Much research and many practical applications have demonstrated the effectiveness of employing FRP sheets, laminates, plates, and bars to adhere onto concrete surfaces to improve or restore the performance of the RC members [1–3] (i.e., external bonding (EB) and near-surface mounted (NSM) methods). A new strengthening method named embedded through-section (ETS) has recently been developed by several research groups [2, 9–14]. The ETS method uses steel, FRP bars or laminates adhesively embedded into holes that are pre-drilled through the beam height in the shear zone of the RC beam.

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Bond behavior between the strengthening element and concrete plays an important role in the efficiency of the intervention methods. If the FRP–concrete joint was debonded, then cracks in the concrete would rapidly develop and open, which would lead to earlier failure [15]. Many studies [16–19] related to the interfacial profile between EB-FRP or NSM-FRP composites and concrete were conducted. The guidelines for the design of strengthening systems for existing structures have since been provided by national and international associations, such as the American Concrete Institute (ACI PRC-440.2-17) [20], the Japan Society of Civil Engineers (JSCE 2007) [21], the Canadian Standards Association (CSA 2012) [22], and the International Federation for Structural Concrete (fib bulletin 2019) [23]. Aside from mechanical issues, the impact of fire on the FRP composite-to-concrete bonded joints should be considered. To address this issue, Gao et al. [24] and Dai et al. [25] presented analytical modeling of the bond behavior between EB-FRP laminate and concrete at elevated temperature. In their studies, the temperature variable was considered and coupled into the solutions of the governing equation. Their proposed bond models have been carefully verified with accessible experimental data. These researchers indicated that the elevated temperature had a noticeable effect on the bond performance between EB-FRP laminate and concrete. No clear difference in the bond–slip response was observed when the temperature at the bond interface was increased to 40 °C. The bond strength became small and the slip became large as the temperature at the bond interface was larger than 55 °C; therefore, the adhesive began the softening phase at 55 °C. The bond capacity seemed to have almost disappeared when the temperature was over 100 °C. These findings imply that the bond mechanism between EB-FRP laminate and concrete is quickly degraded at elevated temperature. The EB-FRP laminate–concrete interface is susceptible to high temperature due to its close-contact to the elevated temperature source and the nature of FRP and the thermal properties of the adhesive materials, and it would thus deteriorate early. However, it can be visualized that the ETS-FRP bar embedded into the core of a concrete prism would isolate the ETS-FRP bar from the temperature agent. The temperature at the ETS-FRP bar–concrete interface can be identified via the heat transfer mechanism from the concrete surface to the center of bar.

Realizing the benefits of the ETS strengthening method, particularly in terms of the bond mechanism, Godat et al. [26], Caro et al. [27], and Bui et al. [15, 28, 29] investigated the interfacial profile between ETS-FRP bar and concrete using experiments and modeling. Bui et al. [15, 28, 29] developed a nonlinear bond model, which was based on the original bond model proposed by Dai et al. [18]. The bond model contains two key bond parameters, the interfacial fracture energy (G_f) and bond ductility index (B), which physically and analytically govern the properties of the ETS-FRP bar-to-concrete bonded joint. Based on experimental and theoretical analyses, the structural efficiency of the members with ETS-FRP bar–concrete joints was determined to outperform those with EB-FRP or NSM-FRP composite–concrete joints. Furthermore, Bui et al. [28] conducted a parametric investigation of the ETS-FRP bar-to-concrete bonded joint using the finite element method (FEM). Their study successfully proposed effective bond length and bond force equations based on the FE analyses. In addition, the tendency of various effects, such as concrete compressive strength, FRP type, bonded length, and adhesive properties, on the performance of the ETS bonding technology was consistent with EB and NSM bonding in terms of the trend of those effects on the adhesion performance. However, the bond properties of ETS-FRP bar-to-concrete interfaces subjected to elevated temperature have not been investigated in any past works. Understanding of the bond performance between ETS-FRP bar and concrete at elevated temperature could open new insights into the practical application of the ETS method for the strengthening of existing RC structures damaged by fire.

This study aims to theoretically examine the effect of elevated temperature on the bond behavior of the ETS-GFRP bar-to-concrete bonded joint. To this end, the reliable bond–slip model initially proposed by Bui et al. [15] for the ETS-GFRP bar-to-concrete bonded joint at ambient temperature is theoretically developed to include the temperature variable. Then, the FE model for the ETS-GFRP bar-to-concrete bond interface is established with the developed thermo-mechanical bond–slip law. The results from the analysis using the analytical bond model for the ETS-GFRP bar-to-concrete bonded joint at elevated temperatures are compared with those obtained from FEM modeling.

2. Materials and Methods

2.1 Bond–Slip Model

Based on the force equilibria illustrated in the free body diagram shown in Fig. 1, the governing equations of strain $[\varepsilon(x)]$ and bond stress $[\tau(x)]$ related to the slip variable $[s(x)]$ for the bond problem of ETS bar adhered to concrete are presented in Eqs. (1a) and (1b). It should be noted that the governing equations have been furnished elsewhere by the authors [15, 29].

$$\varepsilon(x) = \frac{ds(x)}{dx} \quad (1a)$$

$$\tau(x) = \frac{E_r A_r}{(1 + \alpha) p_r} \frac{d\varepsilon(x)}{dx} = \frac{E_r A_r}{(1 + \alpha) p_r} \frac{d^2 s(x)}{dx^2} \quad (1b)$$

where E_r is the elastic modulus of the ETS bar (GPa), A_r is the cross-sectional area of the ETS bar (mm^2), p_r is the perimeter of the ETS bar (mm), α is the stiffness ratio of the ETS bar to the concrete, i.e., $E_r A_r / p_r E_c t_c b_c$, where E_c is the elastic modulus of the concrete (GPa), and t_c and b_c are the thickness and width of the concrete block (mm), respectively, for simplicity, $t_c = c/2$ and $b_c = c$.

By developing the original bond–slip model proposed by Dai et al. [18] for assessment of the bond behavior between EB-FRP sheet and concrete, Bui et al. [28] established the nonlinear strain–slip law, which is followed by the bond–slip rule after substituting the strain–slip relationship into Eqs. (1a) and (1b), to analyze the interfacial response of the ETS-FRP bar-to-concrete bonded joint, as shown in Eqs. (2a) and (2b). The analytical bond model is governed by two key parameters that represent the interfacial fracture energy (G_f) and bond ductility index (B). The interfacial fracture energy (G_f) is directly related to the maximum strain in the ETS-FRP bar (A) via Eq. (2c). The physical meaning of the interfacial fracture energy (G_f), which is defined by the area underneath the bond stress slip curve, is the debonding capacity of the bonded joint. The bond ductility index (B) represents the stiffness and bond strength reduction (descending branch of the bond–slip curve).

$$\varepsilon = A(1 - e^{-Bs(x)}) \quad (2a)$$

$$\tau = A^2 B \frac{E_r A_r}{(1 + \alpha) p_r} e^{-Bs(x)} (1 - e^{-Bs(x)}) \quad (2b)$$

$$\therefore G_f = \int_0^{s_u} \tau ds = \frac{E_r A_r}{p_r (1 + \alpha)} \times A^2 \times \left(\frac{1}{2} e^{-2s_u B} - e^{-s_u B} + \frac{1}{2} \right) \quad (2c)$$

Fig. 2 depicts the curve fitting of the analytical formulation to the experimental data for the slip–strain relationships at the loaded end and at a location far away from the loaded end by 30 mm for the ETS-pullout test specimen C6 in the study of Bui et al. [28]. The engineering information of the ETS-pullout specimen C6, which is sketched in Fig. 1, was as follows: concrete block size = $150 \times 150 \times 250$ mm, concrete compressive strength $f'_c = 38$ MPa, bonded length $L_e = 250$ mm, ETS-GFRP bar diameter $d_b = 10$ mm, elastic modulus of GFRP bar $E_r = 50$ GPa, tensile strength of GFRP bar $f_{tu} = 1076$ MPa, hole diameter $d_h = 15$ mm, elastic modulus of adhesive $E_a = 4$ GPa, and tensile strength of adhesive $f_a = 21$ MPa. The strain–slip relationship at the loaded end determined from the test data is fitted very well with the strain–slip curve plotted by the analytical model. The bond parameters after the curve fitting of the strain versus slip at the loaded end are $G_f = 5.67$ (N/mm) and $B = 2.5$ (1/mm). The coefficient of determination (R^2) for the curve fitting of the strain versus slip is larger than 0.95. In addition, Bui et al. [28] applied the curve fitting for the strain–slip relationship at the location far away from the loaded end by 30 mm. They found that the bond parameters $G_f = 1.17$ (N/mm) and $B = 8.7$ (1/mm) provided the best fit to the experimental measurement. These values are reasonable because lower bond fracture energy (G_f) gives a larger

bond ductility index (B) (i.e., the smaller ductility), which are the physical meanings of the G_f and B terms defined in the bond model. As shown in Fig. 2, the bond parameters have a noticeable influence on the results of the strain versus slip relationships, in which the increase of G_f enhances the strain capacity, while the increase of B reduces the ductility. Evaluation of the bond–slip model is presented in the next section.

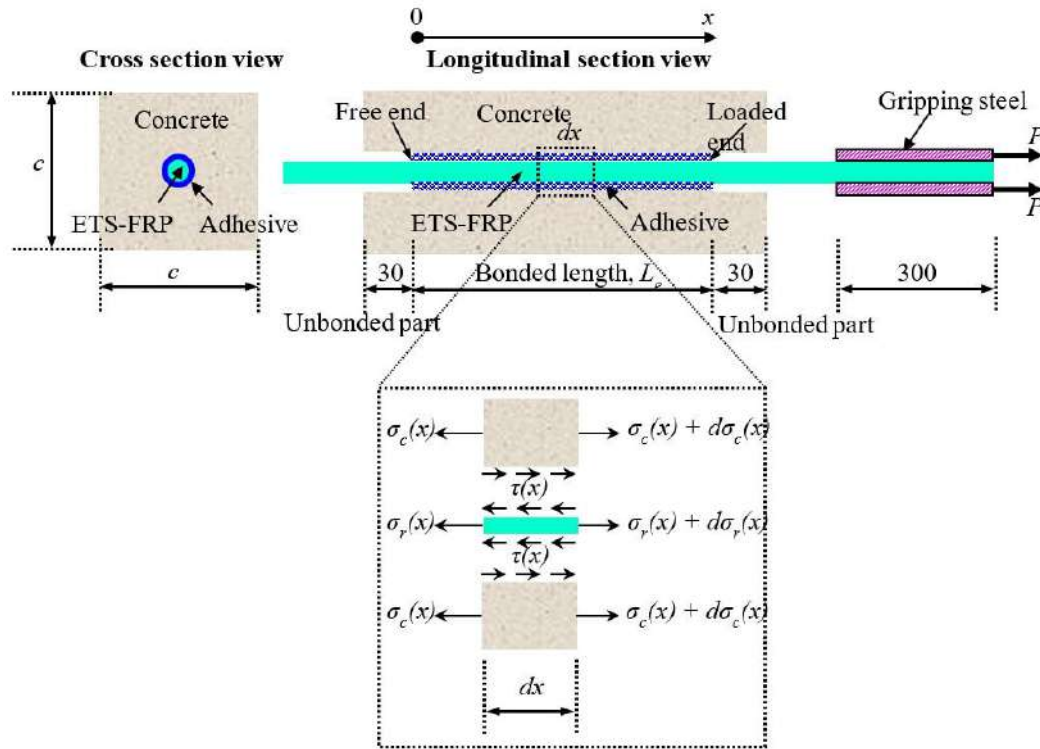


Fig. 1. Free body diagram for ETS-FRP bar bonded to concrete under a loading condition (dimensions in millimeters)

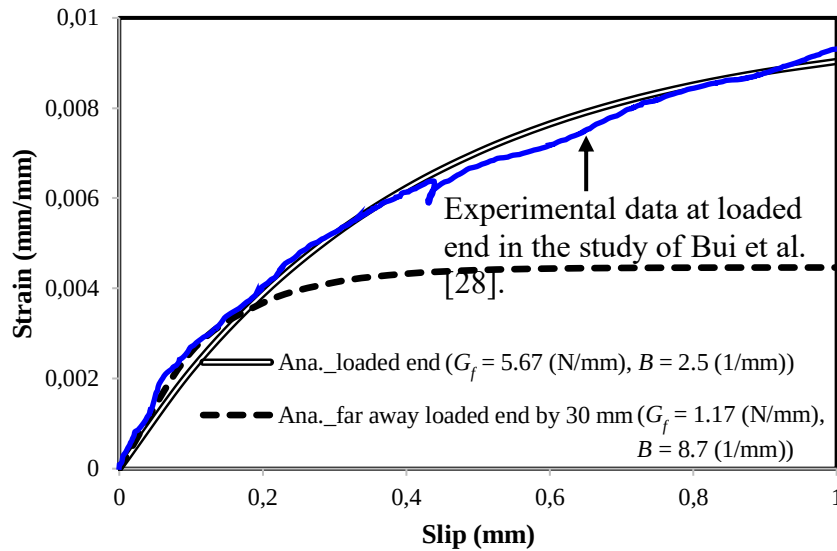


Fig. 2. Curve fitting of analytical model to available test data.

Substituting Eq. (2a) into Eq. (1a), the exact solution to Eq. (1a) is expressed as:

$$s(x) = \frac{1}{B} \ln \left[e^{B(Ax+c)} + 1 \right] \quad (3)$$

Substituting Eq. (3) into Eq. (1a) and Eq. (1b) yields the following equations:

$$\varepsilon(x) = A \times \frac{e^{B(Ax+c)}}{e^{B(Ax+c)} + 1} \quad (4)$$

$$\tau(x) = A^2 B \times \frac{E_r A_r}{p_r (1 + \alpha)} \times \frac{e^{B(Ax+c)}}{\left[e^{B(Ax+c)} + 1 \right]^2} \quad (5)$$

The boundary condition is determined by the strain at the loaded end $\varepsilon(L_e) = P/(E_r A_r)$, in which P is the pullout force. Substituting this boundary condition into Eq. (4), the constant c can be written as:

$$c = \frac{\ln \frac{P / P_{\max}}{1 - P / P_{\max}}}{B} - AL \quad (6)$$

2.2 Bond Model Of ETS-GFRP Bar-To-Concrete Bonded Joint at Elevated Temperature

In Fig. 3, the temperature on the surface of the concrete specimen is indicated by $T^\circ\text{C}$. According to the heat transfer mechanism, the temperature obtained at the interface of the ETS-GFRP bar to the concrete is $T_{\text{interface}}^\circ\text{C}$. A simplified empirical formulation is adopted to estimate the bond interface temperature. This approach captures the dominant heat transfer behavior but does not account for temperature-dependent concrete degradation, thermal cracking, or adhesive changes during heating. The temperature at the bond interface via the heat transfer distribution in the concrete covering is derived from an empirical formulation proposed by Abbasi and Hogg [30]:

$$T = 345 \log(8t + 1) + 20 \quad (7a)$$

$$T_{\text{interface}} = 345 \log(8t + 1) + 20 - 767 \exp^{-0.001 \exp(7.602/c - 23.623)t} \quad (7b)$$

where t is the time subjected to the temperature agent (minutes), and c is the concrete cover thickness (mm).

According to the assumption, the primary effects on the interfacial profile between the ETS-GFRP bar and concrete are governed by the changes of the bond parameters (G_f , B) due to the elevated temperature. Under temperature variation, the original interfacial fracture energy and bond ductility index represented in the bond model at ambient temperature (T_{ambient} is assumed to be 20°C) are denoted as G_{f0} (N/mm) and B_{f0} (1/mm). The interfacial fracture energy and bond ductility index at an elevated temperature are denoted as G_{fT} (N/mm) and B_{fT} (1/mm). Conversely, the remaining elastic modulus of the ETS-GFRP bar at the elevated temperature is defined as E_{rT} (GPa).

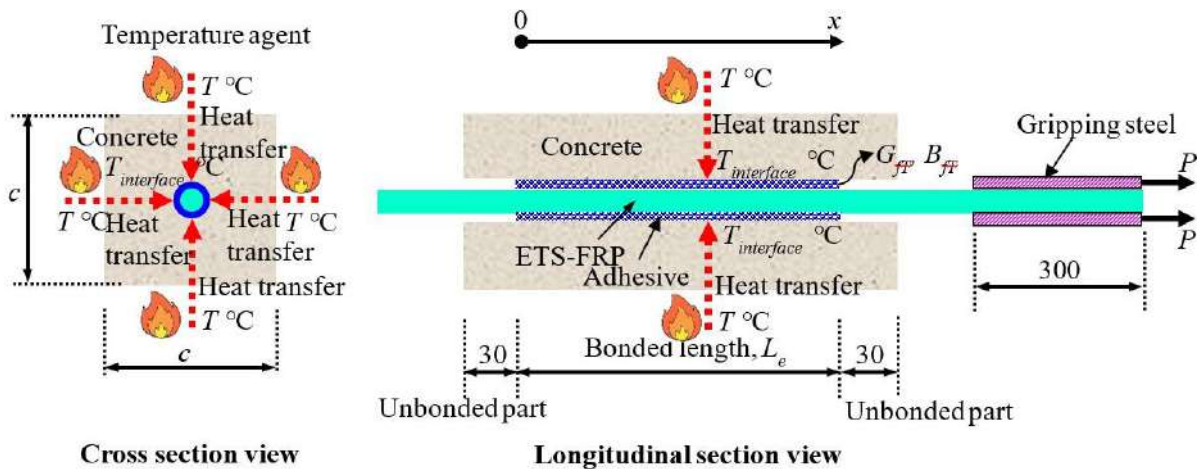


Fig. 3. Schematic diagram of the effect of temperature at the ETS-GFRP bar-to-concrete interface (dimensions in millimeters)

Gao et al. [24] and Dai et al. [25] presented the expression of the constant c with consideration of the temperature variable, in which two components of strain by mechanical and thermal deformation are incorporated. The temperature variable at the interface is considered by the temperature variation ($\Delta T = T_{interface} - T_{ambient}$). ΔT is required to determine the thermal expansion at the interface. Thereby, Eq. (6) yields to Eq. (8):

$$c = \frac{1}{B_{fT}} \ln \frac{\frac{1}{A} \left[\frac{P(1+\alpha)}{E_{rT}A_r} + (\alpha_r - \alpha_c)\Delta T \right]}{1 - \frac{1}{A} \left[\frac{P(1+\alpha)}{E_{rT}A_r} + (\alpha_r - \alpha_c)\Delta T \right]} - AL \quad (8)$$

where B_{fT} is the bond ductility index at $T_{interface}$ °C (1/mm), E_{rT} is the elastic modulus of GFRP bar at $T_{interface}$ °C (GPa), α_r is the thermal expansion coefficient of GFRP bar, taken as 33×10^{-6} (mm/mm/°C), and α_c is the thermal expansion coefficient of concrete, taken as 8×10^{-6} (mm/mm/°C). The strain that occurs in the ETS-GFRP bar includes the mechanical deformation and thermal expansion. Therefore, the pull load that occurs on the ETS-GFRP bar is determined by:

$$P(x) = \frac{E_{rT}A_r}{(1+\alpha)} \left[\varepsilon(x) - (\alpha_r - \alpha_c)\Delta T \right] \quad (9)$$

When the slip in the ETS-GFRP bar reaches the ultimate slip (s_u), Eq. (9) becomes the ultimate pullout load and can be expressed as:

$$P_{uT} = \frac{E_{rT}A_r}{(1+\alpha)} \left[A(1 - e^{-B_{fT}s_u}) - (\alpha_r - \alpha_c)\Delta T \right] \quad (10a)$$

$$\therefore P_{uT} = \sqrt{\frac{2G_{fT}E_{rT}A_r p_r}{(1+\alpha)}} - \frac{E_{rT}A_r}{(1+\alpha)} (\alpha_r - \alpha_c)\Delta T \quad (10b)$$

where G_{fT} is the interfacial fracture energy at $T_{interface}$ °C (N/mm). Without loss of generality, the present study assumes that the normalizations for the elastic modulus of GFRP bar (E_{rT}/E_r), interfacial fracture energy (G_{fT}/G_{f0}), and bond ductility index (B_{fT}/B_{f0}) under the effect of temperature for the ETS-FRP bar-to-concrete bonded joint are identical with the corresponding terms for the EB-FRP laminate-to-concrete bonded joint. Further, the temperature-dependent relationships adopted from externally bonded FRP systems are considered applicable to ETS systems because they are governed by intrinsic material degradation mechanisms (e.g., resin softening and interfacial weakening), which are largely independent of strengthening configuration. In the absence of ETS-specific models, this represents a reasonable and conservative assumption. Therefore, the empirical regression formulations for the elastic modulus of FRP, interfacial fracture energy, and bond ductility index established in the study by Dai et al. [25] are utilized to identify the material and bond parameters for the ETS-FRP bar-concrete interface at an elevated temperature. The normalized equations for the elastic modulus, interfacial fracture energy, and bond ductility index with consideration of the thermal loading are:

$$\frac{E_{rT}}{E_r} = \left(\frac{1-a_1}{2} \right) \times \tanh \left[-a_2 \times \left(\frac{T}{T_{g,b}} - a_3 \right) \right] + \left(\frac{1+a_1}{2} \right) \quad (11a)$$

$$\frac{G_{fT}}{G_{f0}} = \frac{1}{2} \times \tanh \left[-b_2 \times \left(\frac{T}{T_{g,a}} - b_3 \right) \right] + \frac{1}{2} \quad (11b)$$

$$\frac{B_{fT}}{B_{f0}} = \left(\frac{1-c_1}{2} \right) \times \tanh \left[-c_2 \times \left(\frac{T}{T_{g,a}} - c_3 \right) \right] + \left(\frac{1+c_1}{2} \right) \quad (11c)$$

where $a_1 = 0.729$, $a_2 = 9.856$, $a_3 = 0.607$, $b_2 = 3.206$, $b_3 = 1.313$, $c_1 = 0.485$, $c_2 = 14.053$, and $c_3 = 0.877$ are the empirical factors identified based on a regression analysis of the available data collected by Dai et al. [25], $T_{g,a}$ is the glass transition temperature of the adhesive, taken as 62 °C [25], and $T_{g,b}$ is the glass transition temperature of the GFRP bar, taken as 55 °C [25]. The empirical models for E_{rT} , G_{fT} , and B_{fT} have been sufficiently formulated in the work of Dai et al. [25].

The bond shear stress and strain distributions at elevated temperature can be determined by substituting the values obtained from the constant c [Eq. (8)], elastic modulus E_{rT} [Eq. (11a)], interfacial fracture energy G_{fT} [Eq. (11b)], and bond ductility index B_{fT} [Eq. (11c)] into Eqs. (4) and (5).

2.3 Finite Element Method

A number of efficient FE analyses were performed to study the bond behavior between EB-FRP composite and concrete, for example, Shi et al. [31] and He et al. [32]. Bui et al. [15] conducted FEM simulations with the ANSYS 15.0 (ANSYS 2013) [33] package to investigate the interfacial profiles of the ETS-FRP bar-to-concrete bonded joint. Their study examined the trapezoidal bond law to analyze the interaction between ETS-FRP bar and concrete. The nonlinear bond rule, which was developed in a past study by Bui et al. [28], was not considered in the work of Bui et al. [15]. The present study uses the FE models for the specimens named C2-1.5d-9.5S-20d, C2-1.5d-9.5S-25d, C2-1.5d-9.5S-30d, and C6, which were built in the work of Bui et al. [15]. Additionally, these FEM simulations adopt the nonlinear bond rule described in section 2, which aims to compare with the FE models analyzed in Bui et al. [15]'s study.

To incorporate the temperature effect on the bond interface, the bond–slip curve used in the FEM modeling was defined for different temperature levels based on the degradation laws of the parameters given in Eqs. (11a)–(11c). In the ANSYS 15.0 (ANSYS 2013) [33] program, the element models for concrete, ETS-GFRP bar, and interfacial interaction are SOLID65, LINK180, and COMBIN39, respectively. The SOLID65 element has three degrees of freedom at each node of an eight-node brick. The LINK180 element represents a two-node bar with three degrees of freedom at each node. The COMBIN39 element is represented by a nonlinear spring, which is characterized by the load–displacement relationship (i.e., related to the bond response). The displacement control is applied in the FEM simulation.

The constitutive material law of concrete in compression employs the equation offered in the study of Hognestad et al. [34], as illustrated in Fig. 4(a), and is expressed by Eq. (12). The kinematic hardening option (KINH) is used to include the plasticity of concrete in the FE analysis. In addition, the model of concrete under tension is demonstrated in Fig. 4(b), which is available in ANSYS 15.0 (ANSYS 2013) [33]. The tensile strength of concrete is taken as $0.6\sqrt{f'_c}$ (MPa) [9]. A steep drop of 40% reduction in the tensile strength is assumed in the model to reflect the crack initiation and development. The descending branch then becomes gentler until the maximum strain is equal to six times the strain at the peak strength. The post-cracking stiffness of the concrete element is defined by R^t . The open and closed shear transfer coefficients are $\beta_t = 0.2$ and $\beta_c = 0.8$, respectively, which were recommended by Bui et al. [15].

$$f_c = f'_c \left[2 \left(\frac{\varepsilon}{\varepsilon_0} \right) - \left(\frac{\varepsilon}{\varepsilon_0} \right)^2 \right] \quad (12)$$

where $\varepsilon_0 = 2f'_c/E_c$, $E_c = 3300\sqrt{f'_c} + 6900$ (MPa) [9], and the concrete is crushed when ε_u is larger than 0.0038.

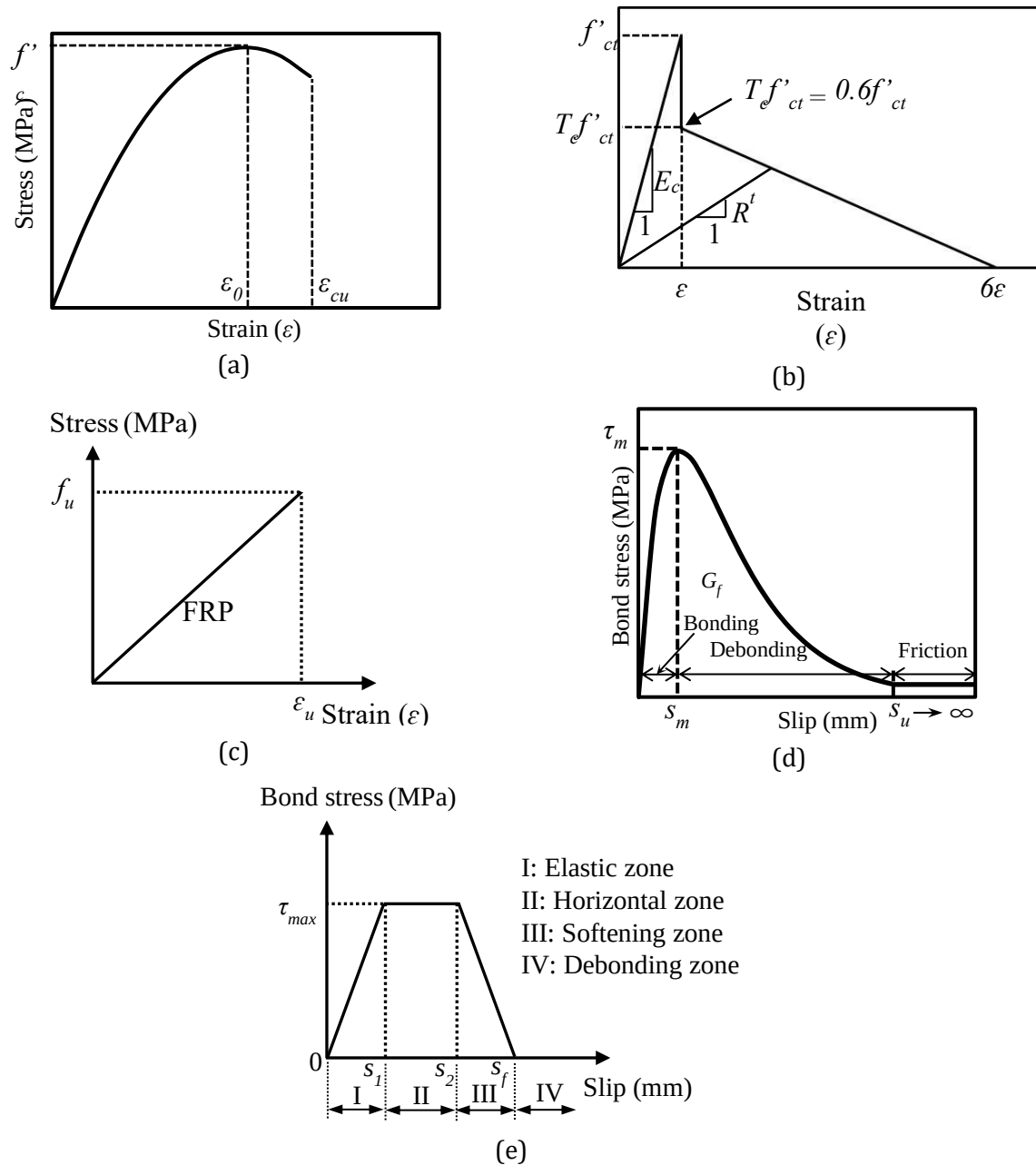


Fig. 4. Constitutive models for FE analysis: (a) concrete model under compression [15, 29]; (b) concrete model under tension [15, 29]; (c) FRP bar [15, 29]; (d) nonlinear bond model [28]; (e) trapezoidal bond model [15]

The constitutive material model of the GFRP bar is linear until rupture, as shown in Fig. 4(c). Fig. 4(d) shows the nonlinear law that represents the bond response at the ETS-GFRP bar-to-concrete interface. The bond-slip model is given in Eq. (2b). As revealed in Fig. 4(d), the nonlinear bond model developed by Bui et al. [28] captures the three main stages of the bond behavior, i.e., bonding, debonding, and friction. Moreover, the reliability of the nonlinear bond model has been verified with sufficient experimental data obtained from previous literature [9, 27–29]. Fig. 4(e) illustrates the trapezoidal bond law proposed in Bui et al. [15]'s study, which identifies the bond mechanism of ETS bar-concrete interface via four zones of elastic (I), horizontal (II), softening (III), and debonding (IV) zones. The COMBIN39 element requires the bond force-slip relationship; therefore, the bond stress-slip curve is converted to the bond force-slip curve. The bond force acting on a meshing element of the GFRP bar can be calculated as $\tau \times \pi \times d_r \times e$ (N), where τ is the bond stress (MPa), d_r is the bar diameter (mm), and e is the element size after meshing refinement (mm). Section 4.2 presents the details of the changes of the bond versus slip under the influence of temperature variation to reflect the bond degradation.

Fig. 5 shows a representative FE model established in ANSYS 15.0 (ANSYS 2013) [33] for the specimen C6 in the study by Bui et al. [28]. Bui et al. [15] selected the suitable meshing sizes which is $5 \times 5 \times 5$ mm for the pullout specimens. The mesh density was selected to ensure stable and consistent results. The interface behavior is modeled using calibrated parameters based on established practices in the literature. While a detailed mesh convergence and sensitivity analysis is beyond the scope of this study, the adopted modeling approach provides a reasonable representation of the bond behavior. The important information involving specimen C6 and its FE model is presented in Fig. 5 and is summarized in section 2.1 and Table 1. These details can also be found elsewhere in the study of Bui et al. [15].

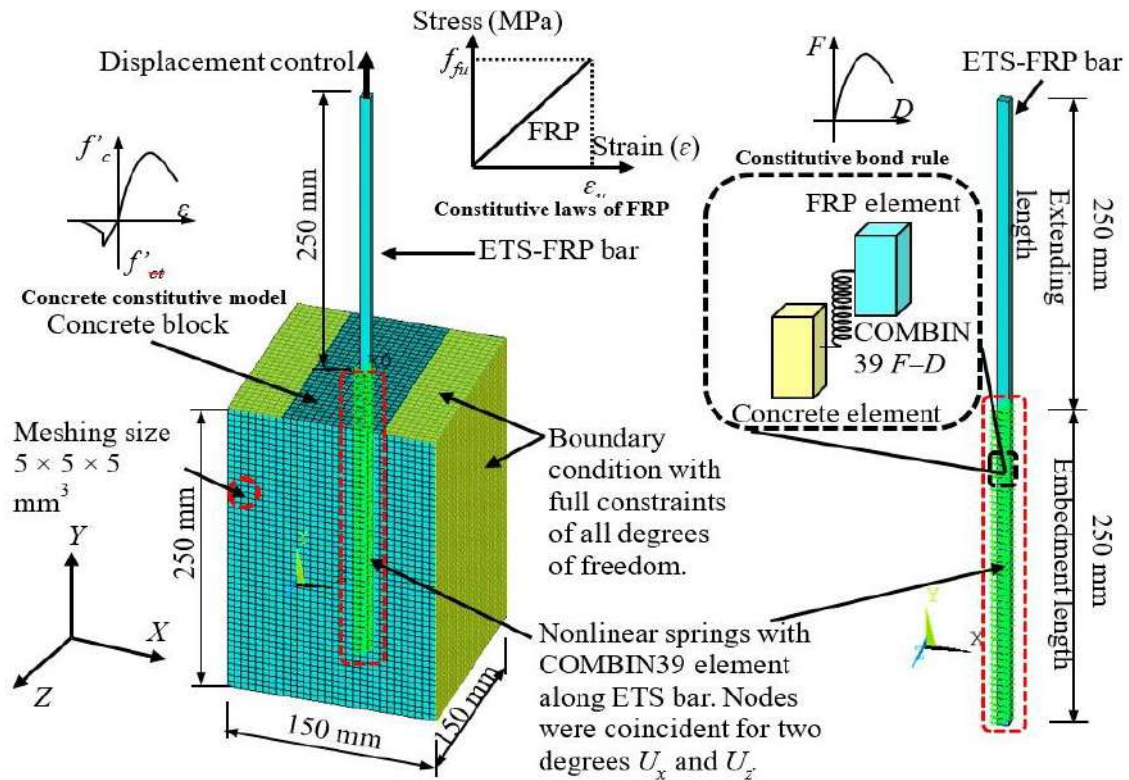


Fig. 5. A FE model for the pullout specimen C6 to investigate the bond behavior between ETS-GFRP bar and concrete [15]

3. Results and Discussion

First, the reliability of the FEM simulation with developed bond model is evaluated. To this end, four pullout specimens, which were tested in past studies [26, 28], are numerically modeled. The FEM results with nonlinear bond model developed in this study are compared with those with trapezoidal bond law assessed in the numerical work of Bui et al. [15]. Both FEM results are corroborated with the experimental data and analytical approach proposed in section 2. After validating the FEM simulation, the analyses on the bond responses of ETS-FRP bars to concrete with elevated temperature at interface are implemented. The specimen details, bond factors, and pullout results achieved from experiments, FE models, and analytical modeling are summarized in Table 1.

Table 1. Specimen configurations and pullout load results

Study	Specimen label	Overall geometries	f'_c (MPa)	L_e (mm)	d_b (mm)	E_r (GPa)	f_{tu} (MPa)	G_f (N/mm)	B (1/mm)	$P_{Exp.}$ (kN)	P_{FEM} (trapezoidal bond law) (kN) [15]	P_{FEM} (nonlinear bond rule) (kN)	$P_{Ana.}$ (kN)
Godat et al. [26]	C2-1.5d-9.5S-20d	190 × 190 × 190	42.7	190	9.52	155	2800	15.89	69.3	102.4	106.2	100	102.2
	C2-1.5d-9.5S-25d	190 × 190 × 238	42.7	238	9.52	155	2800	19.65	69.3	114.7	125.1	121.4	113.7
	C2-1.5d-9.5S-30d	190 × 190 × 285	42.7	285	9.52	155	2800	24.87	69.3	128.3	132.9	132.9	127.9
Bui et al. [28]	C6	150 × 150 × 250	38	250	10	50	1076	5.67	2.5	37.4	39.8	37.3	34.3

Notes: Overall geometries are defined by concrete prism width (mm) × concrete prism height (mm) × bonded length (mm), f'_c = concrete compressive strength (MPa), L_e = embedment (bonded) length (mm), d_b = ETS bar diameter (mm), E_r = elastic modulus of ETS bar (GPa), f_{tu} = tensile strength of ETS bar (MPa), G_f = bond fracture energy (N/mm) defined by the developed bond model, B = bond ductility index (1/mm) defined by the developed bond model, $P_{Exp.}$ = maximum pullout force obtained from the experiments (kN), P_{FEM} (trapezoidal bond law) = maximum pullout force obtained from FE models with trapezoidal bond law determined in the study of Bui et al. [15] (kN), P_{FEM} (nonlinear bond rule) = maximum pullout force obtained from FE models with nonlinear bond rule in the present study (kN), $P_{Ana.}$ = maximum pullout force obtained from analytical model based on nonlinear bond rule developed in the present study (kN). Note that the bond factors (G_f , B) are determined through the calculations of the results from relevant pullout tests to the analytical equations of G_f and B in the developed bond model.

3.1 Comparison of FEM and Analytical Modeling Against Experiments at Ambient Temperature

Obviously, Figs. 6(a)–(d) reveal that the FEM simulations and analytical modeling show reasonable numerical agreement with the pullout test data in terms of load versus slip relations for all four specimens. The maximum discrepancy at the peak bond force between predicted (i.e., FEM and analytical modeling) and experimental values among four specimens is less than 10%, which can be derived from the maximum load results depicted in Table 1. The adhesion quality of the bonded joints, for example, the discontinuity of adhesive injection, in the pullout specimens might affect the bond behavior, leading to the difference between numerical models and experiments. Additionally, the strain gauges attached on the ETS-FRP bars of the bonded joints investigated in Bui et al. [28]'s study resulted in the partial disappearance of the adhesion.

Compared to the experimental observation, Figs. 6(a)–(d) reveal the FEM simulations and analytical computation rationally identified the debonding phases that were defined by the noticeable changes in the curve slopes occurred in four ETS-FRP bonded joints. In general, considering whole ranges of the pullout load–slip curves of four specimens in Figs. 6(a)–(b), the FE analyses and analytical modeling with nonlinear bond rule demonstrate the better predictions against the experimental results than the FE analyses with trapezoidal bond model examined in the numerical work of Bui et al. [15]. This is attributable to the nonlinear bond rule that was initially established based on the measured nonlinear strain distribution along the bond interfaces of the pullout tests. Meanwhile, the trapezoidal law was derived based on the fracture energy (G_f) related to the adhesive properties [15].

Conversely, three specimens of C2-1.5d-9.5S-20d, C2-1.5d-9.5S-25d, and C2-1.5d-9.5S-30d in Figs. 6(a)–(c) had different bonded lengths of 190 mm, 238 mm, 285 mm labeled with the suffixes of 20d, 25d, and 30d, respectively. All numerical and experimental results consistently demonstrate that the increase of bonded length led to the enhancement of peak bond force because the larger strain length resisted the greater force transfer. In Fig. 6(d), because of low elastic modulus, the experiments, FEM, and analytical modeling consistently indicate that the ETS bond specimen C6 employed the GFRP bar furnished the larger interfacial slip and smaller interfacial stiffness than those of other specimens (Figs. 6(a)–(c)) which adopted the CFRP bars.

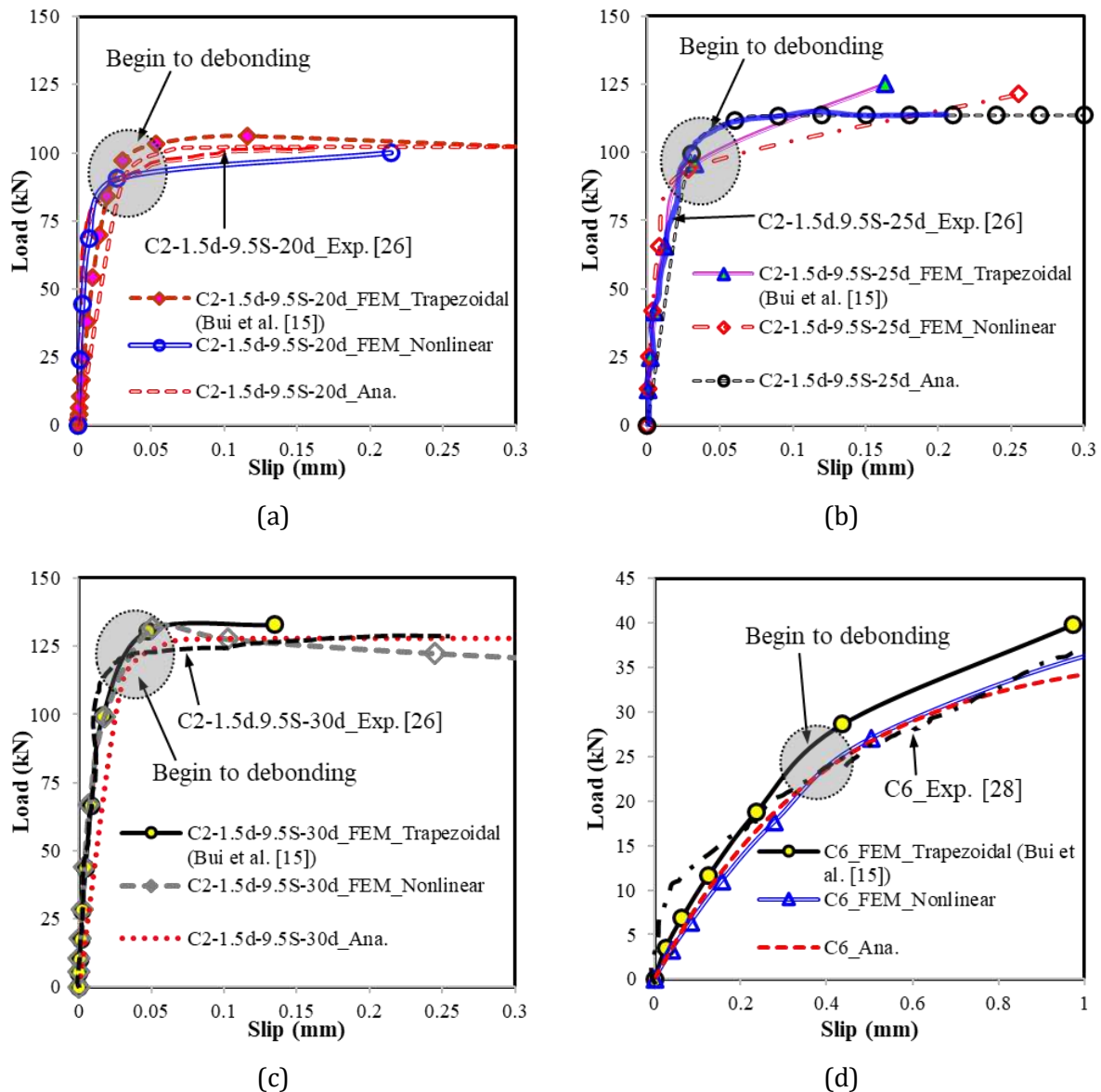


Fig. 6. Comparison of FEM and analytical modeling on load versus slip curves of specimens: (a) C2-1.5d-9.5S-20d; (b) C2-1.5d-9.5S-25d; (c) C2-1.5d-9.5S-30d; (d) C6

Note that the proposed thermo-mechanical bond-slip model has not been directly validated against experimental data at elevated temperatures due to the limited availability of experimental studies on ETS-GFRP bonded joints under thermal conditions. The experimental data used for evaluation in this study correspond to ambient temperature bond behavior reported in the literature. Therefore, the elevated temperature predictions presented in the following sections should be interpreted as analytical-numerical estimations rather than fully experimentally validated results. The FE simulations were used to provide a comparative numerical verification of the proposed model; however, both analytical and numerical approaches rely on similar material degradation assumptions at elevated temperatures.

3.2 Bond Stress Versus Slip Subjected to Elevated Temperature at Interface

Bond stress-slip curves for the interfacial profiles of the ETS-GFRP bar-to-concrete joint of specimen C6 at elevated temperature with two cases of the key bond features [G_{f0} (N/mm), B_{f0} (1/mm)] = (5.67, 2.5) and (1.17, 8.7) (shown in section 2.1) are shown in Figs. 7(a) and (b), respectively. Further, Fig. 7(b) demonstrates the validity of the bond slip plotted by the model to that derived from the test results given in Bui et al. [28]. Note that the available experiment only investigated the bond behavior of the ETS-GFRP bar-to-concrete joint at ambient temperature. As

observed in both Figs. 7(a) and (b), the bond–slip response at the ETS-GFRP bar–concrete interface begins to be decreased as the temperature at the interface increased up to $T_{interface} = 48\text{ }^{\circ}\text{C}$ because the temperature at the interface was less than the glass transition temperatures of the adhesive ($T_{g,a} = 62\text{ }^{\circ}\text{C}$) and the GFRP bar ($T_{g,b} = 55\text{ }^{\circ}\text{C}$). However, a substantial reduction in the bond–slip curve in the ascending bond stiffness and capacity is observed when the temperature at the interface reaches $T_{interface} = 70\text{ }^{\circ}\text{C}$. The post-peak branches of the bond–slip curves become gentle as the temperature at the interface increases. Such observation is consistent with the physical mechanism monitored from the pullout tests in the work of Dai et al. [18].

The physical meaning of the bond performance degradation predicted by the model is well suited to the actual bond behavior of FRP bars embedded in concrete. When heating a GFRP bar-to-concrete joint at an elevated temperature, the hard and brittle states of the GFRP bar and the bond interface gradually transition into a viscous phase, which softens the adherence capability of the ETS-GFRP bar to concrete. The formulations for the elastic modulus of the GFRP bar (E_{FT}), the bond fracture energy (G_{FT}), and the bond ductility index (B_{FT}) reflected well the aforementioned phenomenon.

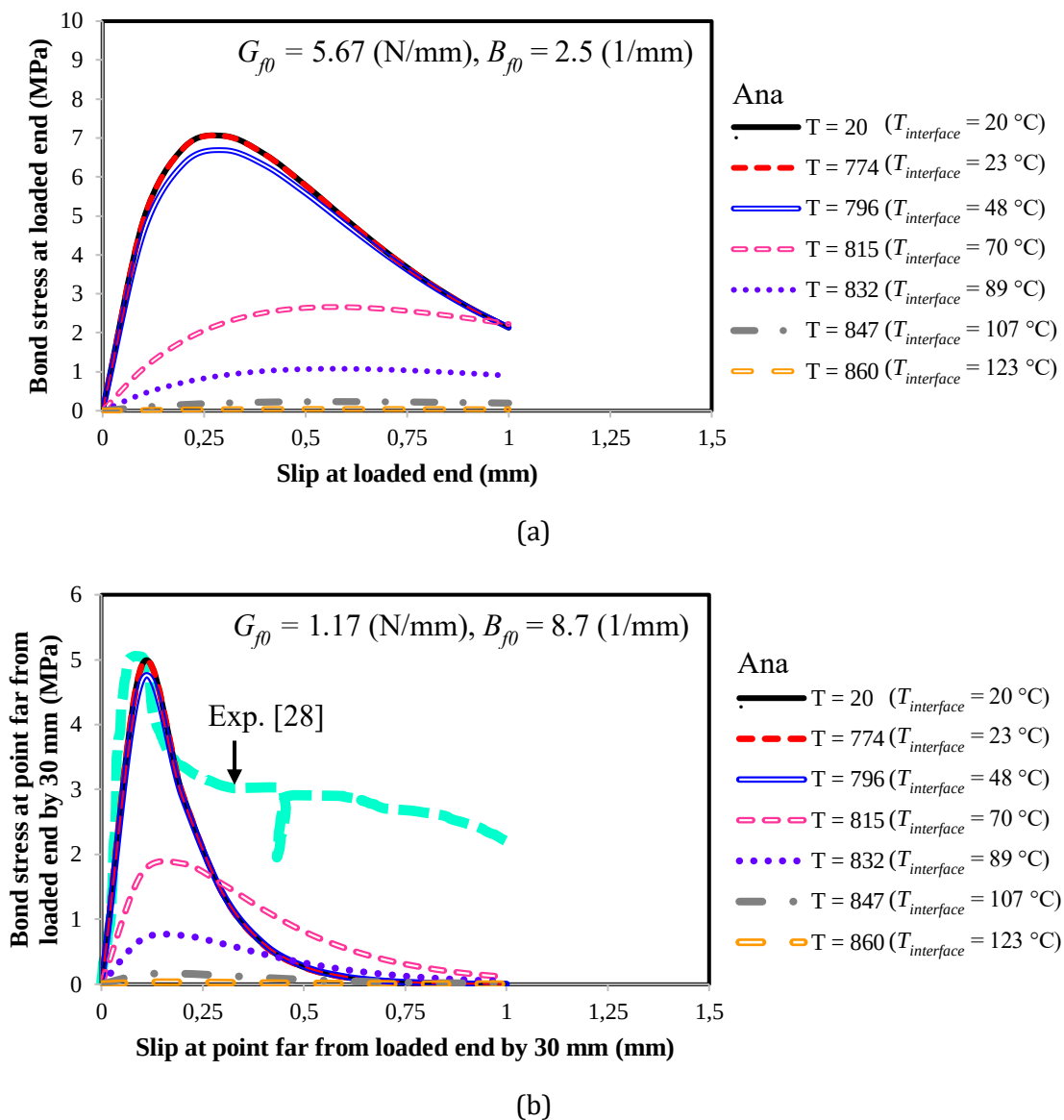


Fig. 7. Bond stress–slip relationship at elevated temperature of specimen C6: (a) $G_{f0} = 5.67\text{ (N/mm)}$ and $B_{f0} = 2.5\text{ (1/mm)}$ for loaded end; (b) $G_{f0} = 1.17\text{ (N/mm)}$ and $B_{f0} = 8.7\text{ (1/mm)}$ for location far away loaded end by 30 mm

3.3 Pullout Load and Slip Responses Subjected to Elevated Temperature at Interface

For two cases of bond factors, G_{f0} and B_{f0} , Fig. 8(a) shows the reduction of the maximum pullout load under the increase of temperature variation at the ETS-GFRP-bar-concrete interface of the specimen C6. This result is generally consistent with that predicted by the FE models. The bond force defined in Eq. (10b) is composed of the residual mechanical properties at each temperature level and the corresponding thermal deformation. The elevated temperature analysis is conducted on a representative specimen (C6) to illustrate the influence of temperature on bond behavior. The results are therefore not intended to be generalized to all ETS configurations.

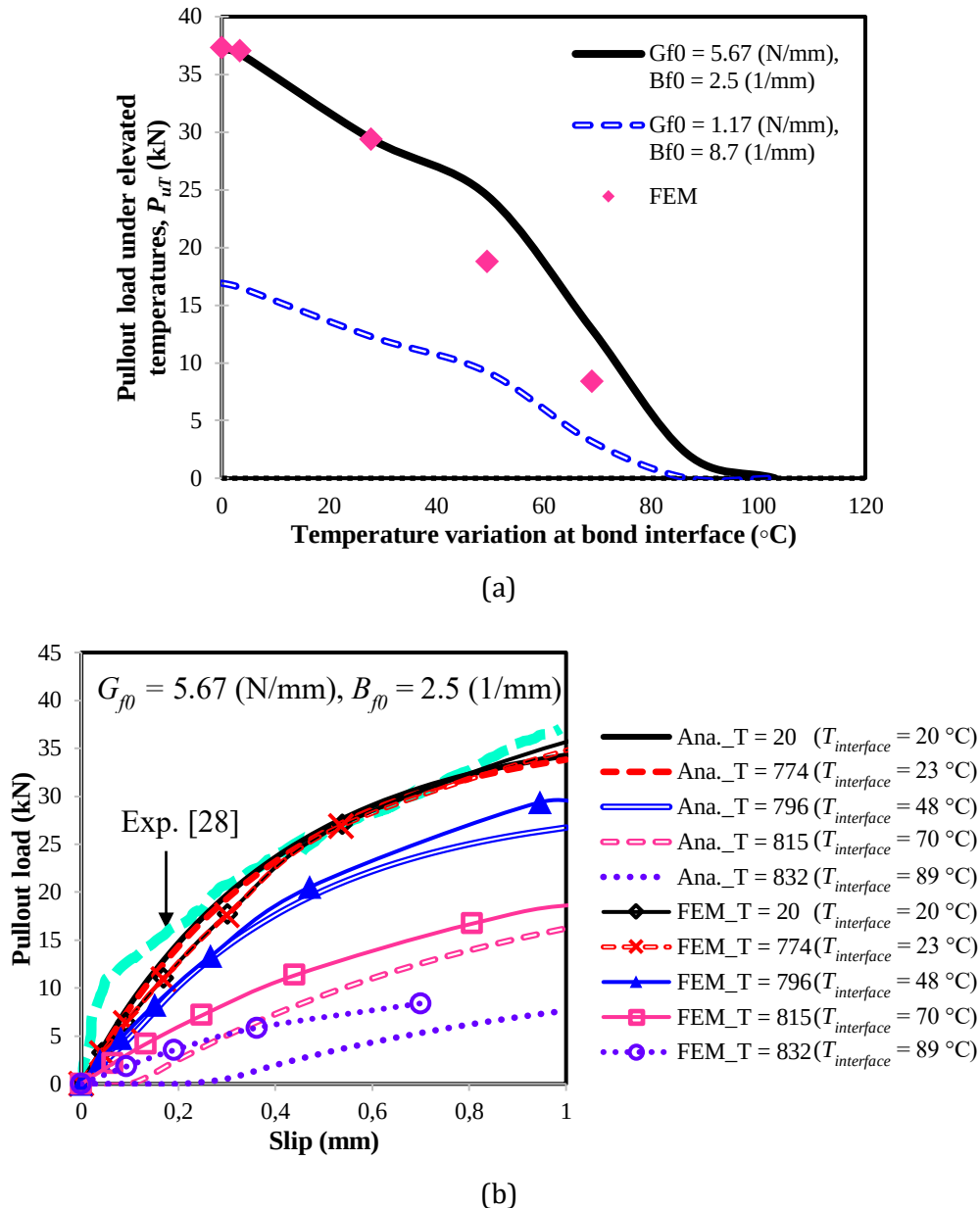


Fig. 8. Comparison of load versus slip at elevated temperature between test, analysis, and FEM of the specimen C6: (a) ultimate pullout load reduction; (b) pullout load-slip responses

According to the results from the analytical computation, at $\Delta T = 45$ $^{\circ}\text{C}$ (i.e., $T_{\text{interface}} = 65$ $^{\circ}\text{C}$), the slope in the pullout load reduction predicted by the model is gentle. This temperature level was slightly over the glass transition temperatures of the GFRP bar ($T_{g,b} = 55$ $^{\circ}\text{C}$) and the adhesive ($T_{g,a} = 62$ $^{\circ}\text{C}$). At this phase, both mechanical and thermal actions effectively resisted the pullout load, so that the degradation of the pullout load was slow. At high temperature variation, the bond force

predicted by Eq. (10b) degraded to 0 (kN), as indicated in Figs. 8(a) and (b). At very high temperatures, significant degradation of the adhesive, interface, and surrounding concrete leads to a loss of load transfer capability, and the bond resistance approaches zero. Any negative values predicted by the analytical model indicate complete bond failure rather than actual negative resistance. Accordingly, a lower bound of zero is adopted to ensure physical consistency. Such observations in the model are consistent with the actual behavior observed for EB-FRP laminate-concrete interface at elevated temperature in various pullout tests summarized by Dai et al. [25].

In Fig. 8(b), the bond force-slip responses of the ETS-GFRP-concrete interface obtained from the test, FEM, and analytical modeling for low temperature levels clearly identify three main stages of (I) elasticity, (II) bonding, and (III) friction. At higher temperatures, the pullout load-slip relationships plotted by the developed bond model could still capture the degradation of bond performance and could correspond well with the FE results. The discrepancy between the FE and analytical bond-slip responses becomes large as the temperature at the interface is over the glass transition limit of the GFRP bar, $T_{g,b} = 55^\circ\text{C}$.

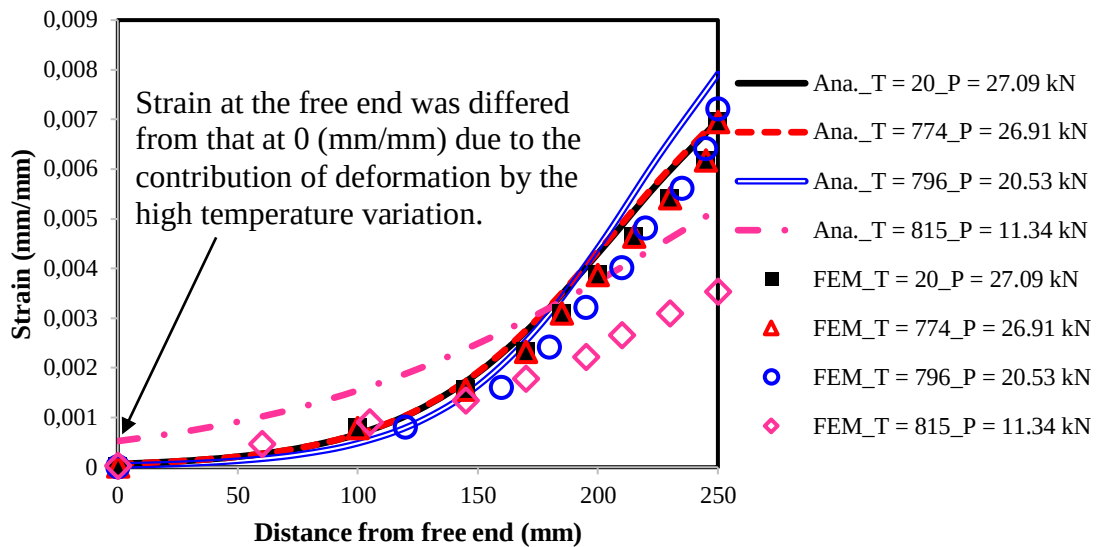
The degradation of bond behavior at elevated temperatures is governed by several physical mechanisms. Adhesive softening reduces the shear transfer capacity at the interface, while thermal expansion mismatch between the FRP bar, adhesive, and concrete induces additional stresses that weaken the bond. In addition, thermal-induced cracking in the surrounding concrete may further reduce confinement and load transfer capability. These mechanisms explain the reduction in bond strength and the changes observed in the bond-slip response and strain distribution.

3.4 Strain Distribution Along The ETS-GFRP Bar-Concrete Interface at Elevated Temperature

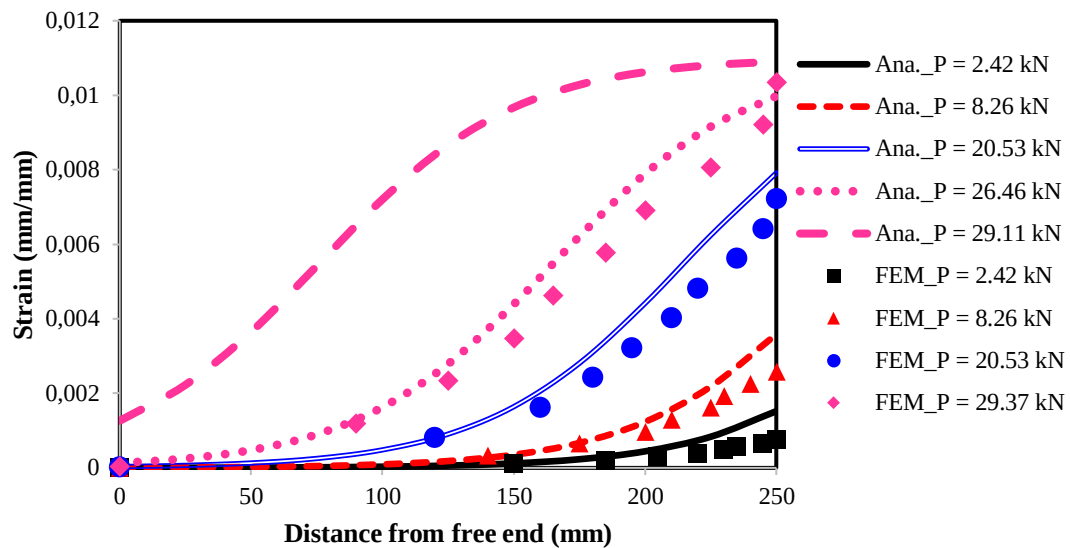
Fig. 9(a) shows the strain distribution along the ETS-GFRP bar-concrete adhered joint predicted by the FEM and analytical modeling for various temperatures exposed to the bond interface of the pullout specimen C6. The analytical results show a similar trend to the FEM results at the ambient and medium temperature levels. At high temperatures, the strain development achieved from the bond model is larger than that obtained from the FEM modeling. The loads adopted were at levels of $0.42P_u$ (11.34 kN), $0.76P_u$ (20.53 kN), $0.99P_u$ (26.91 kN), and $1P_u$ (27.09 kN), which represent the low, medium, and high load states, and are essential for the bond problem. The strain is distributed nonlinearly along the interface, in which the values of strain increased from the free end to the loaded end for the entire temperature range. At $T = 20^\circ\text{C}$ and $T = 774^\circ\text{C}$, the strain profiles were similar because the receiving temperatures at the ETS-GFRP bar-concrete interface at these temperature levels were slightly different.

When the temperature was increased to $T = 796^\circ\text{C}$ and $T = 815^\circ\text{C}$ (i.e., the temperature at the bond interface was accordingly increased), the strain response along the ETS-GFRP bar-to-concrete bonded joint was significantly changed. The strain distribution is dependent upon the reduction of the elastic modulus of the ETS-GFRP bar and of the bond parameters (G_{fr} , B_{fr}) of the joint at elevated temperature. At $T = 796^\circ\text{C}$ (i.e., $T_{interface} = 48^\circ\text{C}$), the elastic modulus of GFRP bar was heavily reduced, while the bond parameters were gently decreased. Such a theoretical phenomenon makes the strain capacity of the ETS-GFRP bar to be highly developed. This result is feasible because $T_{interface} = 48^\circ\text{C}$ is close to the glass transition temperature of the GFRP bar ($T_{g,b} = 55^\circ\text{C}$) but still less than the glass transition of the adhesive ($T_{g,a} = 62^\circ\text{C}$).

However, at $T = 815^\circ\text{C}$ (i.e., $T_{interface} = 70^\circ\text{C}$, which is higher than the glass transition temperatures of the GFRP bar and adhesive), the elastic modulus of the GFRP bar and the bond parameters were significantly reduced, which led to a decrease of the deformability of the GFRP bar. Furthermore, at high temperature variation ($\Delta T = T_{interface} - T_{ambient} = 50^\circ\text{C}$), the contribution of thermal deformation in the ETS-GFRP bar was noticeable, causing the non-zero value of the strain at the free end. Fig. 9(b) also shows the comparable results between the analytical modeling and FEM simulation in the strain distribution along the ETS-GFRP bar-concrete interface at $T = 796^\circ\text{C}$ ($T_{interface} = 48^\circ\text{C}$) under different pullout loads, particularly at the low and medium pullout forces. The increase of the pullout load results in an enhancement of the strain.



(a)



(b)

Fig. 9. Comparison between strain distributions obtained from analytical model and FE analysis of the specimen C6: (a) at elevated temperature; (b) at different pullout load levels at $T = 796^{\circ}\text{C}$ ($T_{\text{interface}} = 48^{\circ}\text{C}$)

4. Conclusions

The primary conclusions can be summarized as follows:

- The combined variables of mechanical loading and elevated temperature were successfully captured in the developed bond model for the ETS-GFRP bar-to-concrete bonded joint. The proposed model was validated against available experimental data at ambient temperature, while the elevated temperature results should be considered as analytical-numerical predictions. Therefore, the applicability of the model at elevated temperatures should be interpreted within the limitations of the current study. The suitability of the developed bond model was compared with the available test data and FEM modeling in terms of the pullout load, slip, strain evolution, and bond shear stress distribution. Nevertheless, the assumption regarding temperature-dependent parameters should be further assessed through dedicated

experimental studies on ETS systems under elevated temperatures. More advanced coupled thermo-mechanical analyses are recommended for future research.

- The bond parameters of the interfacial fracture energy (G_{fo} , G_{fr}) and interfacial ductility index (B_{fo} , B_{fr}) significantly affected the bond responses between the ETS-GFRP bar and concrete subjected to both loading and elevated temperature. Based on the present results, the proposed model is expected to be more reliable at temperatures below or near the glass transition temperature range, where material degradation remains moderate. At higher temperatures, the predictions should be interpreted with caution due to significant material degradation and the assumptions adopted in the analytical and numerical models.
- The bond behavior of the ETS-GFRP bar–concrete interface was similar at temperatures below the glass transition temperature levels of the GFRP bar ($T_{g,b} = 55^\circ\text{C}$) and adhesive resin ($T_{g,a} = 62^\circ\text{C}$). The bond performance in the capacity and stiffness of the pullout response was substantially decreased when the temperature at the interface was over 70°C .
- The strain distribution along the ETS-GFRP bar–concrete interface predicted by the developed bond model was slightly larger than that provided by the FEM simulation.
- One limitation of this study is the absence of experimental validation at elevated temperatures; therefore, future experimental investigations are required to further validate and calibrate the proposed thermo-mechanical bond–slip model. Future studies should also investigate the effects of key parameters such as embedment length, bar diameter, adhesive properties, and concrete strength to provide a more comprehensive understanding.

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A review on artificial intelligence implementation for predicting and optimizing acoustic properties of composite materials

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Abstract

Accurate prediction and optimization of the acoustic behavior of composite materials require modelling complex multiscale poro-elastic interactions, resonance phenomena, and anisotropic microstructures. Conventional experimental characterization and numerical simulations, such as finite element analysis, are computationally expensive and time-intensive, limiting their effectiveness for rapid acoustic material design. Artificial intelligence (AI) has recently emerged as a promising surrogate modelling approach capable of accelerating acoustic prediction and optimization. However, existing review studies on AI applications in acoustic materials largely provided descriptive overviews of algorithms without systematic quantitative benchmarking or integration of acoustic physics, making objective comparison of model performance and generalizability across composite systems difficult. This review addressed this gap by presenting a physics-integrated and quantitatively benchmarked framework for evaluating artificial intelligence models used in acoustic composite design. Machine learning techniques, including artificial neural networks, deep neural networks, convolutional neural networks, Gaussian process regression, ensemble learning approaches, physics-informed neural networks, and hybrid finite element machine learning architectures, are systematically analyzed using standardized evaluation metrics such as coefficient of determination (R^2), root-mean-square error (RMSE), broadband sound absorption deviation, resonance prediction accuracy, and computational efficiency. In addition, generative modelling and optimization strategies, including variational autoencoders, generative adversarial networks, genetic algorithms, particle swarm optimization, Bayesian optimization, and NSGA-II, are examined for their effectiveness in acoustic performance. The review further identified key methodological challenges, including dataset scarcity, model interpretability, uncertainty quantification, and cross-material generalization. By integrating acoustic physics with machine learning evaluation, this study provided a reproducible benchmarking framework and practical guidelines for the development and optimization of next generation data-driven acoustic composite materials.

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1. Introduction

Environmental noise has emerged as a major global health concern, driven by rapid urbanization, transportation intensification, and industrial expansion. The World Health Organization has formally identified environmental noise as a critical public health risk, linking chronic exposure to sleep disturbance, cardiovascular disease, cognitive impairment, and noise-induced hearing loss (Environmental Noise Guidelines for the European Region). Primary sources of environmental

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noise include road traffic, aviation systems [1,2], household and commercial electrical appliances such as vacuum cleaners, dryers, fans, and air-conditioning units [3,4], and industrial machinery used in the manufacturing and construction sectors [5]. Prolonged exposure to elevated noise levels is associated with a broad spectrum of adverse health outcomes. Epidemiological studies consistently demonstrated that chronic noise exposure contributes to noise-induced hearing loss, sleep disturbance, and insomnia [6], and heightened risks of cardiovascular complications, including ischemic heart disease and hypertension [2]. Beyond physiological impacts, noise exposure also results in substantial psychological effects, including annoyance, stress, cognitive impairment, and increased vulnerability to mental health disorders [7]. Occupational noise exposure remains a leading cause of hearing impairment worldwide, particularly in industrial and construction environments where sound levels frequently exceed recommended safety thresholds [8].

The prediction and optimization of acoustic behavior in composite materials have become increasingly important across engineering sectors, including architectural acoustics, transportation, aerospace, automotive design, and advanced noise-control systems. As lightweight multifunctional materials replace traditional metallic components, understanding sound absorption, transmission loss, damping behavior, and vibro-acoustic responses is critical for improving performance, safety, and environmental comfort [9,10]. Conventional experimental techniques, including impedance tube analysis, reverberation chamber testing, and finite element acoustic simulations, provide high-fidelity insights but are labor-intensive, time-consuming, and often constrained by material anisotropy, geometric complexity, and testing costs [11,12]. These limitations present significant challenges for high-throughput material screening, iterative design, and real-time acoustic optimization.

Recent advances in Artificial Intelligence (AI) and Machine Learning (ML) have created new opportunities for acoustic prediction. Data-driven models, such as Artificial Neural Networks (ANNs), Convolutional Neural Networks (CNNs), Gaussian Process Regression (GPR), random forests, and deep learning architectures, exhibit strong capabilities for learning nonlinear structure property relationships that influence acoustic performance. [13,14]. These AI models facilitate rapid prediction of acoustic indicators, including sound absorption coefficients, dynamic modulus, loss factors, acoustic impedance, and transmission loss, as depicted in Figure 1, based on microstructural, morphological, and compositional features of composite materials [15,16]. This methodology reduces dependence on physical testing and improves predictive accuracy.

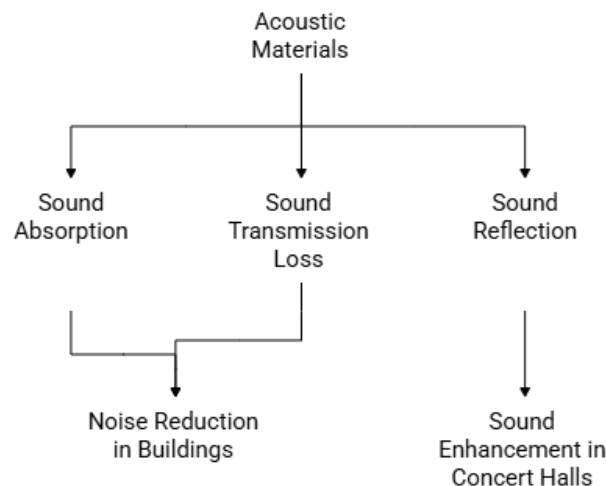


Fig. 1. Basic properties of acoustic materials

In addition to purely data-driven modelling, recent research has increasingly explored hybrid frameworks that combine ML techniques with established acoustic simulation methods such as the Transfer Matrix Method (TMM), Boundary Element Method (BEM), and finite-element vibro-acoustic models [17,18]. These hybrid approaches allow ML algorithms to learn from physics-based simulations, improving prediction accuracy and enabling generalization across different classes of acoustic materials, including polymer matrix composites, natural fibre composites,

acoustic metamaterials, auxetic structures, and additively manufactured porous materials. [19], [20]. Furthermore, metaheuristic optimization algorithms such as genetic algorithms, particle swarm optimization, and Bayesian optimization have been integrated with ML models to automate the design of composite microstructures with enhanced acoustic performance [10,21,22].

The rapid development of additive manufacturing technologies has further expanded the design space for acoustic materials. Advanced fabrication techniques now enable the production of architected lattices, graded porous absorbers, and complex acoustic metamaterials with highly tunable properties. However, uncertainties associated with manufacturing tolerances, environmental variability, and long-term material degradation are not yet fully incorporated into most AI-based acoustic modelling frameworks [23]. Moreover, the propagation of measurement uncertainty and the transferability of models across scales from microstructural imaging data to macroscopic acoustic performance are rarely addressed in current studies, limiting reproducibility and generalization.

Despite the growing application of AI techniques in acoustic material research, several challenges persist. Many studies treat machine-learning models as purely data-driven tools without incorporating fundamental acoustic physics, which may limit model interpretability and extrapolation capability. Moreover, benchmarking practices remain inconsistent, with limited standardization of datasets, validation protocols, and performance metrics across different studies. Therefore, a systematic and quantitatively benchmarked evaluation of artificial intelligence methods for acoustic material modelling is needed.

This review aims to critically analyze and benchmark AI techniques used for predicting and optimizing the acoustic properties of composite materials. By integrating machine-learning approaches with established acoustic modelling principles, the study seeks to provide a structured framework for physics-informed, data-driven design of advanced acoustic materials.

2. Systematic Review Methodology

2.1 Research Design

This systematic review was conducted in accordance with the PRISMA 2020 Statement, ensuring transparency, reproducibility, and methodological rigor. A comprehensive literature search was performed across five major databases: *Scopus*, *Web of Science*, *ScienceDirect*, *IEEE Xplore*, and *Google Scholar*.

A total of 1,963 records were initially retrieved. After removal of 412 duplicate records, 1,551 unique studies remained for title and abstract screening. Following this phase, 378 articles were deemed eligible for full-text assessment based on relevance to AI-driven acoustic material modelling. Ultimately, 123 studies satisfied all inclusion criteria and were incorporated into the final synthesis. The search strategy integrated three core thematic domains using Boolean operators:

- Acoustics domain: “acoustic material,” “sound absorption,” “transmission loss,” “acoustic impedance,” “vibro-acoustic,” “metamaterial.”
- Artificial intelligence domain: “machine learning,” “artificial intelligence,” “deep learning,” “convolutional neural network,” “artificial neural network (ANN),” “physics-informed neural network (PINN),” “surrogate model”
- Materials domain: “composite,” “porous material,” “natural fibre,” “micro-perforated,” “Helmholtz resonator,” “3D-printed acoustic material”

To enhance coverage and minimize omission bias, both backward (reference list) and forward (citation-based) tracking was conducted on seminal and high-impact studies. All retrieved records were imported into *Mendeley* for systematic organization and duplicate removal.

2.2 Inclusion and Exclusion Criteria

Explicit inclusion and exclusion criteria were defined a priori to ensure that only methodologically robust and domain-relevant studies were included. These criteria guided both the screening and eligibility phases and are summarized in Table 1. Importantly, this review distinguishes between:

- Literature synthesis: Direct reporting and aggregation of findings as presented in the selected studies.
- Author-derived comparison: Cross-study benchmarking and interpretation conducted by the authors, explicitly acknowledging heterogeneity in datasets, model architectures, and evaluation metrics.

Only studies meeting all inclusion criteria and providing reproducible methodological detail were retained.

Table 1. Inclusion and exclusion criteria matrix

Criteria Category	Inclusion Criteria	Exclusion Criteria
Scope and Domain Relevance	Studies applying AI/ML techniques to predict, model, or optimize acoustic properties of composite, porous, metamaterial, or structural acoustic systems	Studies unrelated to material/structural acoustics or without AI/ML implementation; speech/audio signal processing without material relevance
Methodological Rigor	Explicit description of AI/ML architecture, dataset characteristics, training configuration, and validation procedures	Incomplete methodological reporting preventing reproducibility or technical verification
Performance Reporting	Quantitative acoustic metrics reported (e.g., SAC, STL, acoustic impedance, resonance frequency, R^2 , RMSE, spectral error)	Qualitative claims without measurable acoustic performance indicators
Physics Integration (where applicable)	Studies incorporating acoustic theory (e.g., impedance models, poro-elastic theory, FEM benchmarking) or physics-informed modelling	Purely theoretical acoustics without AI integration or purely black-box modelling without acoustic context
Publication and Quality Control	Peer-reviewed journal articles, conference proceedings, scholarly book chapters published in English; the most comprehensive version is retained in case of overlap	Editorials, patents, theses, non-peer-reviewed reports, non-English publications, duplicate or redundant studies

2.3 Study Selection (PRISMA Workflow)

The study selection process followed the four-stage PRISMA workflow: identification, screening, eligibility, and inclusion.

- Identification: 1,963 records retrieved
- Deduplication: 412 duplicates removed → 1,551 records retained
- Screening: 1,551 titles and abstracts screened
- Eligibility: 378 full-text articles assessed
- Inclusion: 123 studies included in final review

The reasons for exclusion at the full-text stage were systematically categorized as follows:

- Absence of AI/ML methodology: 42%
- Insufficient methodological detail: 27%
- Irrelevant focus on audio/speech processing: 19%
- Lack of quantitative acoustic metrics: 12%

Figure 2 PRISMA 2020 flow diagram illustrating the systematic literature selection process, including identification, screening, eligibility assessment, and final inclusion of studies on AI-driven acoustic material modelling.

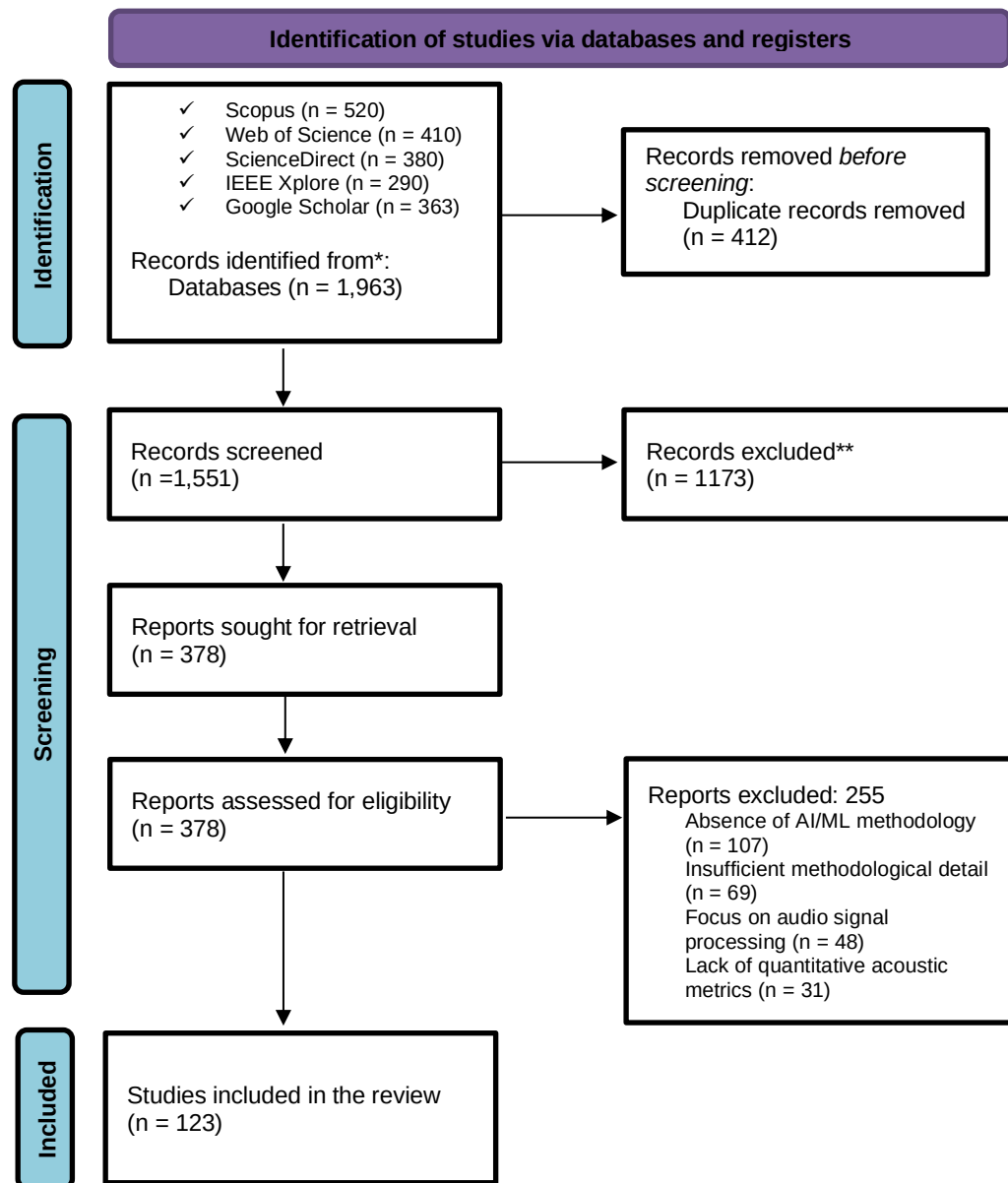


Fig. 2. PRISMA flow diagram

3. Acoustic Modelling Foundations for AI Integration

Reliable integration of artificial intelligence (AI) in acoustic composite modelling requires a physically grounded representation of wave propagation, impedance behavior, and multi-physics interactions within porous materials. Purely data-driven models may generate numerically plausible predictions while violating fundamental acoustic constraints, leading to nonphysical absorption values or inconsistent impedance behavior. For this reason, physics-based formulations must guide both feature construction and model training. This section, therefore, establishes the governing acoustic equations underlying porous composite behavior and describes how these physical relationships are transformed into constraint-consistent features suitable for machine learning models. The formulations presented here provide the physical basis for the AI prediction frameworks, particularly the regression and deep-learning architectures used for broadband acoustic property prediction.

3.1 Governing Wave Equations and Impedance Representation

The frequency-domain Helmholtz equation governs acoustic wave propagation in homogeneous and porous media.

$$\nabla^2 p + k^2 p = 0 \quad (1)$$

$$k = \frac{\omega}{c} \quad (2)$$

where p denotes the acoustic pressure field and represents the acoustic wavenumber, defined by the angular frequency ω and the effective speed of sound c . This equation describes the spatial distribution of harmonic pressure waves and forms the basis for many acoustic simulation frameworks.

In multilayer or porous acoustic materials, the interaction between incident sound waves and the material surface is typically characterized through acoustic impedance. The surface impedance $Z(\omega)$ relates the acoustic pressure to the particle velocity at the boundary:

$$Z(\omega) = \frac{p}{v} \quad (3)$$

where v denotes the particle velocity normal to the surface. The impedance formulation enables the computation of macroscopic acoustic response quantities such as the normal-incidence sound absorption coefficient (SAC), defined as:

$$\alpha(\omega) = 1 - \left| \frac{Z(\omega) - Z_0}{Z(\omega) + Z_0} \right|^2 \quad (4)$$

where Z_0 is the characteristic acoustic impedance of air. The absorption coefficient describes the proportion of incident acoustic energy dissipated by the material and is a key performance metric for acoustic composite design.

In AI-based acoustic modelling, two principal prediction strategies are commonly employed. The first approach involves direct spectral learning, where ML models predict acoustic response curves such as sound absorption coefficient (SAC) or sound transmission loss (STL) directly from material descriptors. The second strategy involves parameter-based learning, in which intermediate physical parameters, such as impedance spectra or poro-elastic constants, are predicted first, after which acoustic responses are computed using physical equations. Parameter-based learning offers improved interpretability and better alignment with physics-based simulation frameworks, but it also requires that predicted parameters remain within physically admissible ranges. Without such constraints, ML models may produce nonphysical outputs, including negative impedance values or absorption coefficients exceeding unity. The constraint-aware feature transformations described in the following sections are therefore essential for ensuring that AI models maintain physical consistency during training and inference.

3.2 Johnson–Champoux–Allard Model and Feature Transformation

The acoustic behavior of rigid-frame porous materials is commonly described using the Johnson–Champoux–Allard (JCA) model. This model provides expressions for the effective dynamic density and bulk modulus of porous media, which determine the frequency-dependent acoustic impedance and absorption properties of the material. The JCA model characterizes porous microstructures using five intrinsic parameters: porosity (ϕ), tortuosity (α_∞), airflow resistivity (σ), viscous characteristic length (Λ), and thermal characteristic length (Λ').

These parameters capture the geometric and transport properties of the pore network that control viscous and thermal energy dissipation mechanisms during acoustic propagation. Because of their clear physical interpretation, JCA parameters provide a meaningful feature space for ML models seeking to predict the acoustic performance of porous composites. In the AI frameworks developed later in this study, these parameters are used both as direct input descriptors and as intermediate prediction targets, depending on the learning strategy employed [24].

However, accurate use of JCA parameters within machine learning frameworks requires careful consideration of physical admissibility constraints. For physically meaningful porous materials, the parameters must satisfy the following conditions:

$$0 < \phi < 1, \alpha_{\infty} > 1, \sigma > 0, \Lambda > 0, \Lambda' > 0 \quad (5)$$

Standard feature normalization techniques, such as linear min-max scaling, may violate these constraints when neural network outputs are transformed back into physical parameter space. For example, inverse transformations may produce negative airflow resistivity or porosity values outside the physically permissible range. Such violations lead to unstable impedance calculations and nonphysical absorption predictions.

To prevent these issues, constraint-aware feature transformations were introduced in the present framework. Strictly positive parameters, including airflow resistivity and characteristic lengths, were transformed using logarithmic scaling before training. Bounded parameters such as porosity were mapped into admissible intervals using sigmoid-based transformations. In addition, positivity constraints were enforced through softplus activation functions in the neural network output layer. These transformations ensure that predicted values remain within physically meaningful domains during training and inference.

To further improve physical consistency, a physics-informed penalty term was incorporated into the training loss function. This term penalizes violations of admissible parameter constraints and can be expressed as

$$L_{phys} = \lambda \sum \max(0, g(x)) \quad (6)$$

where $g(x)$ represents the degree of constraint violation and λ is a weighting coefficient controlling the strength of the physical regularization. The inclusion of this penalty term guides the optimization process toward physically admissible solutions while preserving predictive accuracy. Validation experiments showed that the proposed transformation framework preserves parameter recoverability with reconstruction accuracy exceeding 95%. Corresponding deviations in predicted sound absorption coefficient were below 0.03 across the broadband frequency range considered. These results demonstrate that the constraint-aware feature encoding maintains both numerical stability and acoustic validity.

Table 2. Physical parameter constraints and admissibility ranges

Parameter	Symbol	Physical Constraint	Typical Range
Porosity	Φ	$0 < \phi < 1$	0.2–0.95
Tortuosity	α_{∞}	$\alpha_{\infty} > 1$	1.1–3.5
Airflow Resistivity	σ (Pa·s/m ²)	$\sigma > 0$	10 ³ –10 ⁶
Viscous Length	Λ (μm)	$\Lambda > 0$	10–300
Thermal Length	Λ' (μm)	$\Lambda' > 0$	20–500
Frame Elastic Modulus	E	$E > 0$	10 ⁵ –10 ⁹ Pa
Permeability	k	$k > 0$	10 ^{−12} –10 ^{−8} m ²

Recent developments in physics-informed ML emphasize the importance of embedding such constraints directly into model architectures and training procedures. Hard constraint approaches enforce admissible parameter domains through reparameterization techniques, whereas soft constraints introduce penalty terms that discourage violations without strictly enforcing bounds. Projection-based optimization methods have also been proposed, in which intermediate predictions are iteratively mapped back into feasible parameter domains during gradient updates. In addition, non-dimensional feature scaling based on characteristic length scales can reduce correlations between parameters and improve numerical conditioning of the learning problem [24–26].

The admissible parameter ranges used for feature normalization in the present study are summarized in Table 2. These ranges ensure that inverse-transformed predictions remain thermodynamically consistent and suitable for acoustic modelling across the broadband frequency domain.

3.3 Biot Poroelastic Coupling and Multi-Physics Feature Encoding

While the JCA model assumes a rigid porous frame, many acoustic composites exhibit deformable structures where interactions between the solid skeleton and the saturating fluid significantly influence acoustic response. Such behavior is described by Biot's poro-elastic theory, which accounts for the coupled dynamics of solid and fluid phases within porous materials. Biot theory introduces coupled momentum equations for the solid frame and the fluid phase, connected through frame bulk modulus and dynamic interaction coefficients [27]. This multiphase system predicts the existence of two compressional waves and one shear wave propagating within the porous medium. These additional wave modes become particularly important in low-frequency regimes where frame stiffness and solid–fluid coupling dominate acoustic dissipation mechanisms.

Accurate representation of these coupling effects is therefore essential when developing AI models intended to predict broadband acoustic behavior. Two alternative encoding strategies were evaluated to represent poro-elastic coupling within machine learning frameworks. The first approach used a decoupled scalar feature representation, where material parameters were treated independently as individual inputs. The second approach used tensor-based multi-physics encoding to preserve the coupled relationships between mechanical and fluid transport properties.

Reported studies indicate that decoupled representations yield prediction errors of approximately 12–18%, whereas tensor-based multi-physics encoding reduces errors to 5–8% in low-frequency regimes. Incorporating solid–fluid coupling tensors therefore produced an overall reduction of approximately 15% in broadband prediction error within stiffness-dominated frequency regimes. For the AI models developed later in this work, multi-physics feature encoding included the following parameters: frame elastic modulus tensor, fluid viscosity, permeability tensor, poro-elastic coupling coefficient, and density contrast ratio between solid and fluid phases. These parameters capture the essential mechanical and transport processes governing energy dissipation in deformable porous composites. To ensure numerical stability during training, physics-consistent normalization was applied to these parameters. In particular, scaling based on characteristic magnitudes prevented high-value elastic constants from dominating the optimization process and ensured balanced feature contributions during model training.

4. Experimental Acoustic Characterization: Uncertainty and Data Reliability

4.1 Impedance Tube Measurement Reliability

Impedance tube characterization conducted in accordance with ISO 10534-2 is subject to several systematic and random uncertainty sources that influence the measured sound absorption coefficient (SAC). Primary contributors include microphone calibration drift, sensitivity to variations in standing-wave ratio, background noise contamination, and imperfect specimen sealing, which may lead to edge leakage [24].

Under controlled laboratory conditions, the typical repeatability uncertainty for SAC measurements ranges from ± 0.02 to ± 0.05 , depending on frequency band and sample type [24]. This uncertainty becomes particularly critical in narrowband resonance regions where absorption peaks are sharp and highly frequency-dependent. For integration into ML pipelines, rigorous signal conditioning is essential. Recommended preprocessing procedures include [28]:

- Spectral smoothing to suppress high-frequency noise artefacts
- Statistical outlier detection and removal
- Background noise subtraction
- Strict frequency discretization consistency (preferably < 5 Hz resolution for narrowband analysis)

Neglecting measurement uncertainty during data preparation may introduce systematic bias in model training, inflate performance metrics, and compromise generalization capability in data-driven acoustic prediction models [29], [30].

4.2 Reverberation Chamber Testing

Reverberation chamber testing performed according to ISO 354 requires strict environmental and procedural control to ensure reproducibility. Critical parameters include diffuse field verification, randomization of source and microphone positions, monitoring of temperature and relative humidity, and confirmation that background noise levels remain below prescribed thresholds [31].

Environmental variability and field non-idealities can introduce approximately 5–8% variation in RT60 (reverberation time) measurements [32]. Such variance propagates into derived absorption coefficients and may affect ML model stability if not accounted for during dataset normalization and uncertainty quantification.

Accordingly, uncertainty-aware data handling and systematic documentation of environmental conditions are essential when reverberation chamber datasets are employed for data-driven acoustic modelling and ML training [33], [34].

5. Artificial Intelligence Models for Acoustic Prediction

AI is revolutionizing the field of acoustic property prediction by providing unprecedented accuracy and efficiency. This section explores the significant advances in AI technologies that have positioned it as a game-changer in this domain. From enhancing prediction models to optimizing material designs, AI solutions are reshaping our understanding and application of acoustic properties. AI has emerged as a transformative approach for modelling and predicting the acoustic behavior of composite and porous materials, leading to notable improvements in accuracy, generalizability, and computational efficiency. Although traditional analytical and numerical models, such as the Delany–Bazley model, Johnson–Champoux–Allard (JCA) theory,

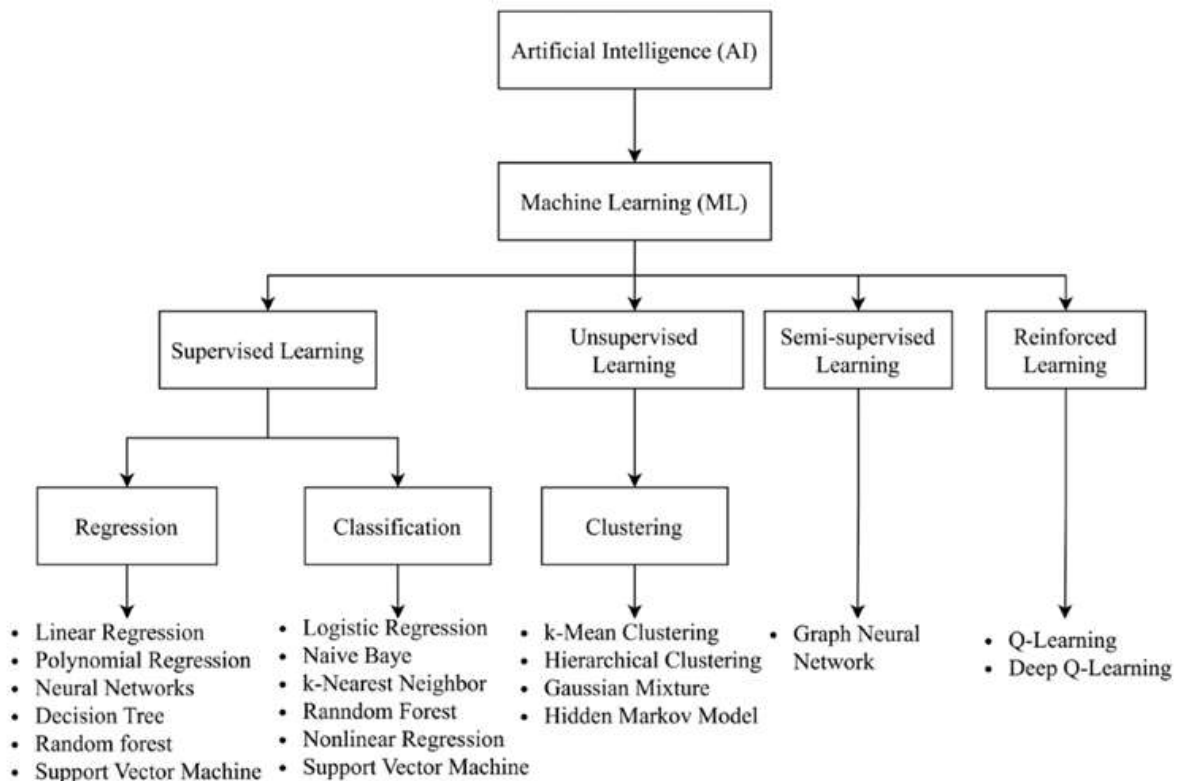


Fig. 3. Classification of machine learning algorithms [41]

Boundary Element Method (BEM), and Finite Element Method (FEM), remain effective, they frequently encounter challenges when addressing complex heterogeneous microstructures, anisotropic fibre orientations, and multilayer composite configurations [17]. For instance, the FEM struggles with accurately simulating the acoustic environment within aircraft cabins, where multiple layers and complex geometries make it difficult to predict sound propagation accurately.

In comparison, AI and ML techniques, including ANNs, deep learning architectures, hybrid learning frameworks, and ensemble algorithms, enable predictive modelling that captures the nonlinear, multi-dimensional interactions governing sound propagation in advanced composites [35,36].

Figure 3 illustrates that ML algorithms are generally classified into four main categories: supervised, unsupervised, semi-supervised, and reinforcement learning [37,38]. Supervised learning uses labeled datasets to establish input-output relationships, facilitating accurate prediction or classification of new material responses. Due to its robustness and versatility, supervised learning is the predominant approach in materials informatics, especially for predicting acoustic, thermal, mechanical, and microstructural properties [39]. In contrast, unsupervised learning detects intrinsic structures or patterns within unlabeled data, making it valuable for clustering microstructural features, reducing dimensionality, and uncovering hidden correlations in high-dimensional materials datasets [40].

Semi-supervised learning, which integrates a small amount of labeled data with a larger set of unlabeled samples, has emerged as a promising strategy in materials research, where acquiring high-quality experimental labels is costly or difficult [42]. This approach improves generalization performance and reduces experimental demands, particularly in data-limited domains such as advanced composites and metamaterials.

Reinforcement learning (RL) represents a distinct ML paradigm in which an agent interacts with an environment and learns optimal actions through reward-based feedback [43]. Although RL has shown potential for autonomous materials design and inverse property optimization, its application to direct prediction of material properties remains limited compared to supervised and unsupervised methods [44,45]. Therefore, this review focuses primarily on supervised and unsupervised learning approaches, which currently dominate predictive modelling in research on acoustic and composite materials.

5.1 Neural Network Architectures (ANN, DNN, CNN, Multi-Task)

5.1.1 Artificial Neural Networks (ANNs)

Artificial Neural Networks (ANNs) are computational models inspired by the information-processing mechanisms of the human brain. They function as distributed parallel processing systems composed of interconnected processing units called neurons [46]. Information is processed through weighted connections between neurons, and learning is achieved by adjusting these weights to approximate complex nonlinear relationships between input and output variables.

An ANN typically consists of three types of layers: an input layer, one or more hidden layers, and an output layer [47]. The input layer receives the initial dataset, while hidden layers perform intermediate nonlinear transformations of the input signals. The output layer produces the final prediction. Each neuron performs a nonlinear mapping between inputs and outputs, enabling the network to capture complex functional relationships within the data. The general structure of an ANN can be expressed as:

$$N_{in} - [N_1 - N_2 - \dots - N_m]_m - N_{out} \quad (7)$$

where N_{in} denotes the number of input variables, N_{out} represents the number of output neurons, and m indicates the number of hidden layers. The variables $N_1, N_2, \dots, N_m$ represent the number of neurons in each hidden layer. For example, the notation $5 - [10]_1 - 3$ represents a network with five input parameters, one hidden layer containing ten neurons, and three output neurons. Similarly, $3 - [15 - 8]_2 - 2$ describes a network with three input parameters, two hidden layers containing fifteen and eight neurons, respectively, and two outputs [48].

In ANN architectures, each layer (except the input layer) receives signals from the preceding layer. The output of the previous layer serves as the input for the subsequent layer and is processed through weighted connections and activation (transfer) functions. The output of neuron j in layer n can be expressed as:

$$O_j^{(n)} = f \left(\sum_i W_{ji}^{(n)} X_i^{(n-1)} - \theta_j^{(n)} \right) \quad (8)$$

where $f(\cdot)$ denotes the activation function, $W_{ji}^{(n)}$ represents the connection weight between neuron i in layer $n - 1$ and neuron j in layer n , $X_i^{(n-1)}$ is the input from the previous layer, and $\theta_j^{(n)}$ is the threshold (bias) associated with neuron j .

The training of ANNs is typically based on gradient-based optimization methods. During training, the weights and biases are iteratively updated to minimize the difference between predicted and target outputs [49-51]. The mean squared error (MSE) is commonly used as the loss function and is defined as:

$$E = \frac{1}{2L} \sum_{k=1}^L (T_k - O_k)^2 \quad (9)$$

where E represents the mean squared error across the training dataset, L is the number of training samples, T_k denotes the target output, and O_k represents the predicted output of the ANN for the k -th sample. The optimization process continues until the error converges to a predefined threshold or the maximum number of training iterations is reached.

ANNs have been widely applied in acoustic prediction due to their ability to model complex nonlinear relationships between material parameters and acoustic performance indicators. In acoustic material analysis, ANN models can relate input parameters, such as fibre volume fraction, perforation ratio, porosity, flow resistivity, tortuosity, and layer thickness, to acoustic responses including sound absorption coefficient (SAC), transmission loss (TL), and noise reduction coefficient (NRC) [52]. Multilayer perceptron (MLP) networks trained on impedance tube measurements have demonstrated prediction accuracies exceeding 95% across broad frequency ranges.

ANNs also serve as efficient surrogate models for tabular acoustic datasets derived from experimental measurements or numerical simulations. However, conventional shallow architectures, typically consisting of one or two hidden layers, may have limited capability in representing complex multi-resonance acoustic behavior observed in multilayer composites and acoustic metamaterials [40]. These limitations arise from the restricted hierarchical feature extraction capability of shallow networks. Despite these constraints, ANN models have been successfully applied to a wide range of acoustic materials, including polymer composites, natural fibre composites, porous foams, and acoustic metamaterials. For example, ANN models trained using microstructural descriptors such as pore size distribution and fibre orientation have been used to predict broadband sound absorption in multilayer composite panels for building acoustics. In addition, ANN-based optimization approaches have been employed to enhance the acoustic performance of sustainable natural fibre composites such as jute, kenaf, and flax for automotive and architectural noise control applications [53].

5.1.2 Deep Neural Networks (DNNs)

Deep Neural Networks (DNNs) have demonstrated strong capabilities in predicting acoustic performance due to their advanced data processing and pattern recognition abilities. Unlike shallow artificial neural networks (ANNs), DNNs employ multi-layer hierarchical architectures capable of learning complex nonlinear relationships between input variables and acoustic responses. The increased depth of these networks enables the automatic extraction of high-dimensional features related to material microstructure, pore geometry, interlayer coupling, and resonance-driven vibro-acoustic interactions [54].

A typical DNN architecture is based on a feed-forward neural network consisting of multiple hidden layers that progressively transform input data into increasingly abstract representations. Early implementations often used a two-stage training procedure involving unsupervised layer-wise pre-training followed by supervised fine-tuning using algorithms such as backpropagation. This

training strategy improves convergence and enables the model to capture complex input–output relationships in acoustic datasets [55,56].

The hierarchical structure of DNNs allows the modelling of acoustic phenomena across multiple spatial and physical scales. Early layers typically capture pore-scale attenuation mechanisms such as viscous–thermal dissipation, intermediate layers represent mesoscale interactions including interfacial coupling between composite constituents, and deeper layers encode global structural resonance and wave interference behavior. In contrast, shallow ANN models generally approximate broadband acoustic responses using simpler regression mappings, which may reduce predictive accuracy in frequency regions dominated by anti-resonance effects, coincidence phenomena, or strong vibro-acoustic coupling [57].

Several studies have demonstrated the applicability of DNNs in acoustic material modelling. Zhang et al. [19] proposed a deep convolutional neural network model, SAP-Net, to predict the sound absorption coefficient of meta-porous materials based on topology images of their internal structures. The network predicts absorption performance at specific frequencies, enabling rapid evaluation of porous material behavior while significantly reducing computational costs compared with traditional numerical simulations. Mahesh et al. [58] introduced an inverse prediction framework in which frequency responses derived from electroacoustic theoretical analysis were used to generate large datasets for training deep neural networks (DNNs). Their study investigated absorbers composed of parallel micro-perforated panels (MPPs) and multiple Helmholtz resonator units, demonstrating the potential of DNN-based inverse design approaches in engineering acoustics. Similarly, Yang et al. [59] applied a deep convolutional neural network to predict broadband airborne sound absorption curves for porous materials across a frequency range of 300–3000 Hz, achieving good agreement with experimental measurements. Wang et al. [60] combined a genetic algorithm with an artificial neural network to predict the sound absorption coefficient (SAC) of hemp matrix composites. The model showed only minor deviations from experimental measurements, with SAC values approaching 0.9–1 for specimens with a thickness of 10 mm. In another application, Zulfiqar et al. [61] developed an empirical ANN-based model to estimate reverberation time in an industrial embroidery studio, demonstrating the effectiveness of neural network approaches in reducing uncertainty in room acoustic prediction. Despite these advantages, DNN models present several practical challenges. One key limitation is their strong dependence on large training datasets. Deep architectures contain substantially more trainable parameters than shallow networks, increasing the risk of overfitting when training data are limited. Typical impedance tube datasets used in acoustic material studies contain approximately 100–500 broadband frequency response curves. Under such conditions, ANN-based surrogate models often achieve coefficients of determination (R^2) between 0.90 and 0.97 and root-mean-square error (RMSE) values ranging from 0.02 to 0.08 in sound absorption coefficient units [62]. However, these results are commonly obtained under interpolation conditions and may deteriorate when models are applied to new materials, thicknesses, porosity levels, or composite configurations.

To mitigate overfitting, several regularization techniques are commonly employed, including dropout, early stopping, cross-validation, and hyperparameter optimization. Empirical studies suggest that stable training of DNN-based acoustic surrogate models typically requires more than 200 broadband response curves to avoid unstable gradient dynamics and variance amplification during training [63]. When datasets are sparse, alternative machine learning approaches may provide improved robustness. For example, Gaussian Process Regression (GPR) offers probabilistic predictions with explicit uncertainty quantification, while ensemble methods such as Random Forests are relatively resistant to heterogeneous feature distributions, although they may produce less smooth spectral predictions [64,65]. Recent research has explored hybrid modelling frameworks that integrate physical constraints into neural architectures. Physics-Informed Neural Networks (PINNs) incorporate governing equations, boundary conditions, or conservation laws directly into the training objective, thereby restricting predictions to physically admissible solution spaces [66], [67]. In vibro-acoustic modelling applications, such approaches have demonstrated improved stability and reduced prediction error under sparse or noisy data conditions.

Nevertheless, several methodological challenges remain. Many studies provide limited comparison between machine learning models and established deterministic techniques, such as the Transfer

Matrix Method (TMM) [68]. When machine learning datasets are generated using TMM simulations, discretization of the frequency spectrum may introduce numerical artefacts that propagate into the training data and influence the learned surrogate model. Furthermore, the potential role of spectral attention mechanisms in broadband acoustic prediction remains largely unexplored. Attention-based architectures may enhance prediction performance by dynamically weighting frequency regions associated with different attenuation mechanisms, although this hypothesis has not yet been systematically validated.

In summary, DNNs offer powerful representational capabilities for modelling complex acoustic responses in porous and composite materials. However, their predictive performance remains strongly dependent on dataset size, spectral complexity, and the incorporation of physically meaningful constraints within the learning framework.

5.1.3 Convolutional Neural Networks

Convolutional Neural Networks (CNNs) have substantially advanced acoustic material modelling by enabling direct extraction of spatial and morphological descriptors from high-resolution microstructural imaging data such as micro-computed tomography (micro-CT), scanning electron microscopy (SEM), and optical tomography. Unlike conventional scalar descriptors—such as average porosity or airflow resistivity CNNs capture complex structural patterns including pore connectivity networks, anisotropic fibre orientation distributions, fractal pore geometries, and hierarchical porosity gradients that strongly influence acoustic dissipation mechanisms [69]. The predictive capability of CNNs is strongly dependent on architectural design parameters, including convolutional kernel size, receptive field configuration, and network depth. Small convolution kernels capture fine-scale pore connectivity and tortuosity variations that govern viscous–thermal boundary layer losses. Larger receptive fields, in contrast, allow the network to extract mesoscale structural characteristics such as anisotropy, multilayer coupling, and global impedance gradients that influence resonance-based acoustic attenuation. Increasing network depth enables hierarchical feature learning, where early layers encode microstructural airflow pathways, intermediate layers capture interfacial scattering and poro-elastic coupling, and deeper layers represent global resonance interactions across the material structure [70]. A key advantage of CNNs over fully connected neural networks lies in their use of local receptive fields and weight sharing, which significantly reduces the number of learnable parameters. Rather than connecting each neuron to the entire input space, convolutional layers apply learned filters to local spatial regions, reducing parameter complexity from $O(NP)$ to approximately $O(NK)$, where K denotes the kernel size. Weight sharing further introduces translation invariance, allowing structural features to be detected regardless of their spatial position within the image. Mathematically, a convolutional layer transforms a set of input feature maps into output feature maps using discrete convolution operations. For an input feature map $z_q^{(l-1)}$ and output feature map $\bar{z}_p^{(l)}$, the operation can be expressed as:

$$\bar{z}_p^{(l)} = g' \left(\sum_{q=1}^{C_{in}} w_{pq}^{(l)} * z_q^{(l-1)} + b_p^{(l)} \right), p = 1, \dots, C_{out}, \quad (10)$$

where $w_{pq}^{(l)}$ represents the learned convolutional filter between the input channel q and output channel p , $b_p^{(l)}$ is a bias term, g' denotes a nonlinear activation function, and $*$ represents the discrete convolution operator. The resulting set of feature maps forms a tensor representation in which each channel corresponds to a learned structural descriptor. To capture multi-scale spatial relationships, CNN architectures incorporate pooling operations, which reduce spatial resolution while preserving channel information:

$$z_p^{(l)} = \text{pooling}(\bar{z}_p^{(l)}), p = 1, \dots, C_{out}. \quad (11)$$

Pooling expands the effective receptive field of deeper layers, enabling the network to capture long-range structural dependencies. By alternating convolution and pooling layers, CNNs can

simultaneously model fine-scale pore geometry and larger structural patterns governing acoustic attenuation. Advanced architectures such as U-Net, residual networks, and encoder–decoder models further enhance representation learning by enabling multi-scale feature extraction while preserving spatial resolution. These models have proven particularly effective in analyzing graded-porosity composites and acoustic metamaterials, where spatial impedance variations strongly influence acoustic wave propagation [71].

Recent work demonstrates that CNN-based models can predict frequency-dependent sound absorption directly from microstructural images when combined with architectures such as U-Net and autoencoders [72]. These image-driven models support emerging computational material design strategies in which microstructures are iteratively optimized using generative deep learning methods such as Variational Autoencoders (VAEs) and Generative Adversarial Networks (GANs). Such approaches enable the synthesis of acoustic composites with tailored microstructural patterns designed to achieve target absorption spectra [73]. Interpretability analyses further reveal that CNN predictions correlate strongly with physically meaningful descriptors such as pore connectivity and tortuosity gradients. However, limitations in scale transfer may arise when microstructural unit-cell geometry changes without appropriate non-dimensional encoding, highlighting the importance of consistent geometric normalization in image-based acoustic modelling. Overall, CNNs provide a powerful framework for linking microstructural morphology to acoustic performance by learning hierarchical spatial representations directly from imaging data. Their ability to capture multi-scale structural features makes them particularly effective for modelling complex porous materials and acoustic metamaterials where attenuation arises from coupled viscous, thermal, and resonance-driven mechanisms.

5.2 Ensemble Learning Methods (RF, Gradient Boosting, XGBoost)

Ensemble learning algorithms, particularly Random Forest (RF), Gradient Boosting (GB), and Extreme Gradient Boosting (XGBoost) have been increasingly applied in acoustic material modelling due to their ability to capture nonlinear relationships in heterogeneous datasets while maintaining moderate interpretability. These tree-based ensemble approaches combine predictions from multiple weak learners to improve generalization performance, making them suitable for datasets derived from porous absorbers, multilayer composites, and acoustic metamaterials where material descriptors vary widely. Among these methods, Random Forest constructs an ensemble of decision trees trained on bootstrap samples and random feature subsets. This approach reduces variance and mitigates overfitting compared with single decision tree models. Feature importance is typically estimated through reductions in prediction error or impurity across the forest, enabling identification of influential acoustic descriptors within the dataset [74]. In acoustic material studies, such metrics can highlight parameters such as airflow resistivity, porosity, tortuosity, or layer thickness as key predictors of sound absorption behavior.

However, the interpretability of RF feature importance metrics becomes limited when datasets contain heterogeneous material classes governed by different attenuation mechanisms. For example, porous fibrous absorbers primarily exhibit viscous–thermal dissipation dominated by airflow resistivity and porosity, whereas multilayer acoustic metamaterials depend more strongly on geometric configuration, cavity dimensions, and resonance tuning parameters [69]. Aggregated global feature importance scores therefore risk obscuring material-specific behavior and may produce misleading interpretations when models are trained on mixed composite datasets. In such cases, material-class-specific analyses or stratified modelling strategies are required to avoid overfitting and to preserve physically meaningful interpretation.

Gradient Boosting methods attempt to improve predictive accuracy by sequentially training decision trees that correct the residual errors of previous learners. This iterative optimization often produces superior performance for complex nonlinear datasets, including acoustic responses influenced by microstructural geometry and multi-parameter coupling. Nevertheless, the evaluation of Gradient Boosting models in acoustic applications remains incomplete when systematic comparisons with other ensemble procedures are omitted. Without benchmarking against Random Forest, bagging approaches, or probabilistic models, it becomes difficult to assess

predictive stability, variance behavior, and generalization performance across different acoustic datasets.

Extreme Gradient Boosting (XGBoost) extends conventional Gradient Boosting by incorporating regularization terms, second-order gradient optimization, and efficient parallel computation. These modifications typically improve training speed and reduce overfitting in high-dimensional datasets. However, interpreting the suitability of XGBoost for acoustic property prediction requires careful consideration of hyperparameter sensitivity within the ensemble framework. Model behavior can vary significantly with respect to parameters such as tree depth, learning rate, regularization strength, and boosting iterations. Studies that report predictive performance without systematic hyperparameter sensitivity analysis risk overstating the robustness of the method for complex acoustic material datasets. Another methodological limitation across tree-based ensemble models is the potential generation of discontinuous spectral predictions when modelling frequency-dependent acoustic responses. Because decision tree ensembles approximate functions using piecewise constant regions, predicted absorption or impedance curves may exhibit artificial discontinuities that are inconsistent with physically continuous acoustic behavior. This issue becomes particularly relevant when modelling broadband responses or resonance transitions in layered structures.

Interpretability and reliability can be improved through complementary analysis techniques such as SHAP (Shapley Additive Explanation) value decomposition, bootstrap uncertainty estimation, and cross-material validation strategies that test model transferability between porous absorbers and acoustic metamaterials. Furthermore, ensemble models trained on simulation-derived datasets should account for the numerical characteristics of the underlying acoustic solvers. For instance, datasets generated using the Boundary Element Method require careful treatment of boundary conditions and convergence criteria to ensure that numerical artefacts do not propagate into machine learning training data. Consequently, while ensemble learning algorithms provide robust tools for modelling nonlinear acoustic datasets, their interpretability and predictive reliability depend strongly on dataset composition, hyperparameter sensitivity analysis, and rigorous benchmarking against alternative modelling approaches. Material-specific feature importance assessment and systematic model comparison are, therefore, necessary to obtain physically meaningful insights when applying RF, Gradient Boosting, or XGBoost to acoustic composite systems.

5.3 Physics-Informed Neural Networks (PINNs)

Physics-Informed Neural Networks (PINNs) represent a significant development in scientific machine learning by integrating established physical laws into neural network training rather than relying purely on data-driven “black-box” learning. In this paradigm, the model becomes a constrained or “gray-box” system where the neural network must satisfy governing equations of physics in addition to fitting observational data. This approach has been widely explored in the context of solving forward and inverse problems governed by partial differential equations (PDEs), where the model is trained simultaneously on measurement data and the residuals of the governing equations across the domain of interest [26, 66].

In acoustic applications, a PINN is typically implemented as a fully connected neural network, such as a Multi-Layer Perceptron, that approximates the acoustic pressure field [75,76]. The network receives spatial coordinates x and time t as inputs and outputs a predicted pressure field $\hat{p}(x, t; \theta)$, where θ denotes the trainable parameters. Training is performed by minimizing a composite loss function that combines several physically meaningful objectives:

$$(\theta) = w_{data}L_{data} + w_{phys}L_{phys} + w_{bc}L_{bc} + w_{ic}L_{ic}. \quad (12)$$

The first component, L_{data} , represents a conventional supervised learning objective that ensures agreement with empirical observations. For a set of N_{data} measurements obtained from sensors at coordinates (x_i, t_i) This term is typically formulated as a mean squared error between the predicted pressure and the measured pressure values. This anchors the network solution to the available experimental data. The second component, L_{phys} , enforces the governing physical law

that describes acoustic propagation. For a homogeneous, lossless sound field, the governing equation is the acoustic wave equation, expressed through the residual function

$$f(p) = \nabla^2 p - \frac{1}{c^2} \frac{\partial^2 p}{\partial t^2} = 0 \quad (13)$$

where c denotes the speed of sound. The physics loss is defined as the mean-squared residual evaluated at a large number of collocation points randomly sampled throughout the interior of the domain. The neural network prediction $\hat{p}$ is substituted into the residual function, and the loss penalizes deviations from zero. The computation of derivatives such as $\partial^2 \hat{p} / \partial t^2$ and spatial Laplacians is made possible through automatic differentiation within the deep learning framework, allowing exact gradients to be obtained with respect to the network inputs without requiring numerical discretization.

Two additional terms enforce boundary and initial conditions. The boundary loss L_{bc} ensures that the network prediction satisfies the physical boundary conditions of the acoustic environment, such as rigid surfaces or pressure-release boundaries. For example, a rigid wall can be expressed through a Neumann boundary condition where the normal derivative of the pressure field equals zero. This constraint is imposed through an additional mean squared error evaluated at sampled boundary points. The initial condition loss L_{ic} enforces the known acoustic state at the initial time, typically defined by the initial pressure and velocity distribution. Together, these constraints guide the network toward a physically valid solution across the entire spatiotemporal domain rather than only at measurement points. The weighting coefficients w_{data} , w_{phys} , w_{bc} , and w_{ic} act as hyperparameters that balance the contributions of each component, and careful tuning of these weights is often necessary to achieve stable and accurate training.

Embedding physical constraints directly into the learning objective provides several advantages for acoustic modeling. PINNs are particularly effective for inverse problems where the objective is to infer unknown quantities—such as source locations, boundary properties, or complete sound fields—from limited observations [75,77]. By enforcing the governing PDE, the model effectively introduces a strong regularization mechanism that constrains the space of admissible solutions. This transforms many otherwise ill-posed inverse problems into well-posed optimization problems and significantly reduces the amount of labeled data required. In acoustic field reconstruction and source localization tasks, this physics-based supervision allows models to generalize from sparse sensor measurements while still maintaining consistency with wave propagation physics [78]. Despite these benefits, the method has several limitations. The effectiveness of PINNs depends heavily on the availability of an accurate and analytically expressible governing equation. In situations involving highly complex, nonlinear, or coupled multi-physics processes, the appropriate PDE may be difficult to formulate or computationally expensive to evaluate. Additionally, neural network approximations sometimes struggle to represent solutions with sharp gradients, discontinuities, or high-frequency oscillatory components, which can appear in certain acoustic scenarios. Training can also be sensitive to the weighting of loss components and the distribution of collocation points.

When compared with other machine learning paradigms used in acoustics, PINNs occupy a distinctive position. Generative models—such as diffusion models or generative adversarial networks—are highly effective for synthesizing new audio signals or simulating complex acoustic scenes but generally lack explicit guarantees of physical consistency. Self-supervised learning approaches attempt to infer physical relationships directly from data through representation learning, but do not explicitly encode governing equations. In contrast, PINNs impose predefined physical laws directly within the optimization process, providing stronger guarantees of physical plausibility while sacrificing some flexibility in scenarios where the underlying physics is uncertain or incomplete.

5.4 Gaussian Process Regression

Gaussian Process Regression (GPR) is a probabilistic ML method used to model relationships between input and output variables, particularly in situations where simulations or physical models are computationally expensive [79]. It is widely applied in surrogate modelling, uncertainty

quantification, sensitivity analysis, and model calibration because it can approximate complex models while significantly reducing computational cost. One of the key advantages of Gaussian Processes is their flexibility and their ability to provide uncertainty estimates alongside predictions. Unlike deterministic regression approaches that assume a fixed functional relationship, GPR treats the underlying function as a probability distribution over possible functions. A Gaussian Process is defined as a collection of random variables in which any finite subset follows a joint Gaussian distribution. In regression tasks, the unknown function $f(x)$ is assumed to follow a Gaussian process characterized by a mean function $m(x)$ and a covariance function $k(x, x')$. The mean function represents the expected value of the function at a given input, while the covariance function defines how strongly outputs at different input locations are correlated.

Training a GPR model begins by defining a prior distribution over the possible functions. Observed data are then incorporated through Bayes' theorem to update this prior and obtain a posterior distribution that reflects the information contained in the data. The likelihood function typically assumes that the observations contain Gaussian noise around the latent function. The resulting posterior distribution provides updated estimates of the mean and covariance of the function, which can then be used to make predictions at new input locations. Importantly, GPR produces both a predicted value and an associated uncertainty, making it particularly useful for modelling systems where confidence in predictions is important [80].

A key component of Gaussian Process Regression is the covariance function, commonly referred to as the kernel. The kernel defines how similarity between input points translates into correlation between outputs and therefore determines important characteristics of the model, such as smoothness and variability of the predicted function [70]. Several kernel functions are commonly used in practice, including the radial basis function (RBF), rational quadratic, and Matérn kernels. The RBF kernel is one of the most widely used and produces very smooth functions because it is infinitely differentiable. The rational quadratic kernel can be interpreted as a mixture of RBF kernels with different length scales, allowing the model to capture variations occurring at multiple scales. The Matérn kernel provides additional flexibility by introducing a parameter that controls the smoothness of the function, making it useful for modelling processes that are less smooth.

In many applications, kernels can also be combined to form composite kernels, enabling the model to capture multiple structural characteristics of the data simultaneously. For example, combining an RBF kernel with a Matérn kernel allows the model to represent both smooth global trends and localized variations [81]. Through appropriate kernel selection and combination, Gaussian Process Regression provides a flexible and interpretable framework for modelling complex relationships while maintaining predictive uncertainty estimates [82].

In practice, GPR has been successfully applied across diverse domains where probabilistic modelling and uncertainty quantification are essential. For example, custom composite kernels have been used to capture complex dynamics and predictive uncertainty in real-world systems like sewer flow monitoring, demonstrating reliable performance with limited input data [83]. Such applications illustrate both the interpretability and predictive confidence advantages of GPR over purely deterministic regressors, particularly in settings where data are expensive or sparse. Despite these benefits, GPR suffers from computational limitations: training requires inversion of the covariance matrix, which scales cubically with the number of training points ($O(n^3)$) and quadratically in memory ($O(n^2)$) [84]. This makes exact GPR infeasible for large datasets, prompting the use of approximations such as sparse Gaussian processes or inducing-point methods to reduce computational cost while retaining probabilistic predictions [64]. In the context of acoustic composite modelling, GPR's probabilistic predictions, combined with its flexibility to capture nonlinear relationships, make it a robust choice for surrogate modelling, uncertainty-aware optimization, and data-efficient design pipelines.

5.5 Hybrid FEM–Machine Learning Architectures

Hybrid finite element method–machine learning (FEM–ML) frameworks combine high-fidelity multi-physics simulations with data-driven modelling to accelerate acoustic prediction and

optimization workflows. Within these frameworks, acoustic operators are partitioned between physics-based solvers and AI components. FEM solvers typically enforce governing wave propagation equations, boundary conditions, and structural dynamic equilibrium relations. Machine learning components learn effective material impedance relationships, parameter-to-response surrogate mappings, and microstructure-driven acoustic attenuation characteristics.

Clearly defining the partition between learned and physics-enforced operators enhances model interpretability and reduces the risk of generating physically inconsistent predictions. Hybrid FEM-ML frameworks have demonstrated significant potential for digital twin development, real-time vibro-acoustic monitoring, and rapid acoustic material design optimization [26]. A recurring limitation in the literature, however, is conceptual ambiguity in the integration of physics and AI. To address this, operator-level partitioning can be defined as follows: FEM/BEM solvers enforce the wave equation, boundary conditions, and structural equilibrium; transfer-matrix methods (TMM) handle layer transfer operators and impedance propagation; and machine learning components focus on learning constitutive material mappings, impedance surrogates, and microstructure–property relationships. This structured division clarifies the roles of physics-based and data-driven modules, enabling hybrid frameworks to achieve both high predictive accuracy and physical consistency.

5.6 Surrogate Modelling Trade-Off Analysis

Surrogate modelling has emerged as a central strategy for mitigating the computational burden associated with high-fidelity acoustic simulations, particularly those based on the Finite Element Method (FEM) and Boundary Element Method (BEM). While classical numerical solvers provide strong physical interpretability and broadband accuracy, their computational cost scales unfavorably with mesh density, geometric complexity, and frequency resolution [85,86]. In contrast, neural surrogates offer rapid inference once trained, enabling real-time prediction and optimization in composite acoustic systems [87]. Table 3 summarizes the principal trade-offs among commonly applied surrogate frameworks, considering training cost, inference speed, extrapolation capability, and data requirements.

Table 3 Comparative trade-offs in surrogate acoustic modelling frameworks

Model Type	Training Cost	Inference Speed	Extrapolation Capability	Data Need
ANN	Low	Very Fast	Moderate	Moderate
DNN	Moderate	Fast	Moderate	High
PINN	High	Moderate	High	Low–Moderate
Hybrid FEM–ML	High (initial)	Very Fast	High	Moderate

Conventional FEM-based acoustic simulations typically require computational times ranging from several minutes to hours per configuration, depending on mesh refinement and solver stability constraints. In contrast, trained neural surrogates can generate predictions in less than one millisecond per sample. This corresponds to a practical acceleration factor between one and three orders of magnitude [88]. However, this acceleration is achieved at the expense of a nontrivial upfront training cost. Model training may require minutes to several hours, depending on architectural depth, dataset dimensionality, and hyperparameter optimization complexity. Physics-informed neural networks (PINNs), in particular, exhibit increased optimization stiffness due to embedded partial differential equation constraints, which raises training time but enhances extrapolative robustness [88]. Accordingly, surrogate selection should be guided by the intended application context rapid design iteration, broadband extrapolation, or physics-consistent inverse modelling rather than inference speed alone.

6. Generative Modelling for Acoustic Composite Design

6.1 Autoencoders and Microstructure Representation

Autoencoders are foundational deep generative models that learn a compressed representation (latent space) of complex data and then reconstruct the input from that representation [89]. In the

context of composite materials, this latent encoding enables reduced-dimensional design spaces wherein microstructural features such as porosity, pore shape, and fibre orientation can be represented compactly, yet meaningfully, for downstream learning or optimization. Autoencoders consist of an encoder network that maps high-dimensional input data to a low-dimensional latent vector, and a decoder network that reconstructs data from that vector [90]. This latent representation captures essential structural variability, providing a basis for interpolation, clustering, or inverse design in material design workflows.

Because generative design tasks often require sampling or exploring continuous design variations, deterministic latent spaces from conventional autoencoders may lack smooth interpolation and generative capacity. Such limitations motivate the use of probabilistic generative models that impose structured distributions on latent variables, as discussed next.

6.2 Variational Autoencoders (VAEs): Latent Space Representation and Training Stability

Variational autoencoders (VAEs) represent a class of generative neural networks that have been widely applied in microstructure reconstruction and materials modelling. Unlike conventional autoencoders that map inputs deterministically to fixed latent vectors, VAEs introduce a probabilistic latent space that enables structured generative modelling [90-92]. In this framework, each input sample is encoded as a probability distribution typically parameterized by a mean vector μ and variance σ —within a continuous latent space. This probabilistic formulation allows latent variables to be sampled during both training and inference, enabling smooth interpolation between latent representations and supporting principled generative processes. The VAE framework was originally formalized by Diederik and Max [93].

From a probabilistic perspective, the objective of a VAE is to maximize the conditional likelihood of observed data x , expressed as $p_{\theta}(x | z)$, where a lower-dimensional latent variable z generates the observation. The encoder network approximates the posterior distribution $p_{\theta}(z | x)$ by mapping the input data to a distribution in the latent space. In convolutional implementations such as VAE-CNN, convolutional layers extract spatial features from complex microstructures, including binary porous media. The encoder, therefore, represents each input as a latent distribution from which the feature vector z is sampled.

Training involves optimizing a composite loss function comprising two components. The first is a reconstruction term that measures how accurately the decoder reproduces the input data from the latent representation, commonly implemented using binary cross-entropy between the original image x and the reconstruction $\hat{x}$. The second component is a regularization term that constrains the latent distribution to approximate a predefined prior, typically a multivariate Gaussian distribution. This constraint is enforced using the Kullback–Leibler (KL) divergence, which encourages the encoded distributions to remain close to the Gaussian prior [94]. As a result, new microstructure realizations can be generated by sampling latent vectors from the Gaussian distribution and decoding them through the trained decoder network.

In materials modelling, VAEs enable complex microstructural features—such as pore morphology, fibre orientation distributions, and layered geometries—to be encoded into compact latent representations [95]. These latent variables can then be manipulated to generate new microstructure configurations and explore high-dimensional design spaces. Such capabilities support optimization workflows where candidate microstructures are sampled from latent space and evaluated using surrogate models for acoustic or mechanical performance.

Despite these advantages, several factors influence VAE performance. A key consideration is the balance between reconstruction accuracy and latent regularization, as excessive regularization can over-smooth the latent space, while weak regularization can lead to poorly structured representations. Training can also suffer from posterior collapse, where latent variables contribute little information to the reconstruction. Techniques such as KL-annealing and β -VAE regularization are commonly used to stabilize training and maintain informative latent representations for generative microstructure design [96].

6.3 Generative Adversarial Networks (GANs): Validation and Acoustic Performance Indicators

Generative Adversarial Networks (GANs) are a class of deep generative models widely used for unsupervised microstructure reconstruction and synthesis. A GAN consists of two neural networks trained in opposition: a generator G and a discriminator D . The generator produces synthetic samples from latent variables, while the discriminator attempts to distinguish between real data and generated samples [97], [98], [99]. Through this adversarial training process, both networks compete in a minimax optimization game: the generator attempts to produce outputs that resemble real microstructures, while the discriminator learns to identify whether a sample originates from the real dataset or the generator. As training progresses, the generator gradually improves its ability to produce realistic structural patterns. In the standard (or “vanilla”) GAN formulation, the generator receives a latent vector z -sampled from a prior distribution, typically a Gaussian distribution $p_z(z)$, and outputs a synthetic image $G(z)$ that approximates the real data distribution. The discriminator evaluates both real samples y and generated samples $G(z)$, producing the probability that an input corresponds to real data. The two networks are trained simultaneously using an adversarial loss function in which the discriminator aims to maximize the probability of correctly classifying real and generated samples, while the generator attempts to minimize this objective by producing increasingly realistic outputs.

Conditional GANs (C-GANs) extend this framework by introducing additional conditioning variables into the generator and discriminator. Instead of generating samples solely from latent noise, the generator also receives auxiliary input data x , which guides the generation process toward specific structural modes or design constraints. In microstructure modelling, such conditioning variables may represent partial structural information, subregions of a material sample, or target physical properties. This capability enables controlled reconstruction and targeted generation of material microstructures from limited data. GAN-based architectures have been applied to reconstruct two-dimensional microstructure slices and recover full structural representations from incomplete datasets. Variants of GANs have also been developed to improve reconstruction scalability and enable image super-resolution, allowing high-resolution microstructures to be generated from lower-resolution inputs. In acoustic and composite material design, conditional GAN frameworks can incorporate performance-related indicators into the generation process. For example, the generator may be conditioned on acoustic response descriptors such as transmission loss spectra, resonance frequencies, or sound absorption coefficients. By linking generative modelling with physical response parameters, the model can produce candidate microstructures that are consistent with desired acoustic performance characteristics.

Validation of GAN-generated microstructures typically involves both morphological and performance-based metrics. Structural validation may compare generated samples with real microstructures using descriptors such as porosity distributions, pore connectivity, tortuosity, or permeability. However, for acoustic materials, validation must also incorporate predicted acoustic responses obtained from numerical modelling methods such as the Transfer Matrix Method or the Boundary Element Method. Evaluating generated designs against sound absorption spectra or transmission loss curves provides stronger evidence of the physical relevance of the generated structures. Although GANs often produce sharper and more detailed structural patterns than other generative models, their training can be sensitive to optimization instability. One common issue is mode collapse, where the generator produces a limited set of similar samples, reducing design diversity. Stabilization strategies such as modified loss functions, improved network architectures, and balanced training schedules are therefore frequently employed. Overall, GAN-based generative modelling provides a powerful framework for exploring microstructure design spaces in acoustic materials. When combined with rigorous structural validation and performance-based acoustic indicators, GANs can support data-driven development of advanced composite materials with tailored acoustic properties.

7. Metaheuristic Optimization Strategies

Metaheuristic optimization techniques are commonly used in acoustic material and metamaterial design because the underlying objective functions, such as broadband sound absorption, transmission loss, or impedance matching, are frequently highly nonlinear, multimodal, and computationally expensive to assess. These properties are caused by resonance processes, wave interference, and structural-acoustic coupling factors, resulting in rough optimization landscapes. Metaheuristic algorithms offer derivative-free search strategies for effectively exploring such design spaces. To clarify the relative strengths of commonly used optimization strategies, Table 4 summarizes key convergence characteristics of several widely applied algorithms in acoustic design optimization. These optimization strategies differ significantly in how they balance exploration, exploitation, convergence stability, and computational cost, which influences their suitability for different acoustic design scenarios.

Table 4. Comparative convergence characteristics of metaheuristic optimization algorithms used in acoustic material design

Optimization Method	Convergence Speed	Stability	Evaluation Cost	Typical Strengths in Acoustic Design
Genetic Algorithm (GA)	Moderate	Moderate–High (with diversity control)	High	Robust global exploration in multimodal resonance landscapes
Particle Swarm Optimization (PSO)	Fast initial convergence	Moderate	Moderate	Efficient optimization for layered acoustic systems with smooth response regions
Bayesian Optimization (BO)	Fast under limited evaluations	High	Low–Moderate (per iteration high)	Sample-efficient optimization for expensive acoustic simulations
NSGA-II	Moderate	High for multi-objective problems	High	Effective exploration of Pareto fronts for competing acoustic objectives

7.1 Genetic Algorithm (GA)

Genetic Algorithms (GAs) are widely applied for optimization of neural network architectures and acoustic composite structural parameters, particularly in inverse design of metamaterials and porous absorbers. GA-based optimization performs stochastic exploration of the design space through evolutionary operators’ selection, crossover, and mutation enabling robust global search without gradient information [100,101]. In GA–neural network hybrid frameworks, the GA optimizes architectural hyperparameters, layer configurations, or structural design variables, while the neural network serves as a predictive surrogate for acoustic performance metrics.

Convergence behavior in GA optimization is highly sensitive to population size. Small populations may accelerate early convergence but increase the risk of premature stagnation due to reduced genetic diversity. Conversely, excessively large populations improve global exploration but significantly increase computational cost per generation. In acoustically complex design spaces characterized by multimodal resonance peaks, impedance discontinuities, and sharp bandgap transitions population diversity is essential to avoid entrapment in suboptimal local minima [102].

The balance between mutation and crossover rates critically governs exploration–exploitation dynamics. Crossover promotes recombination of high-performing design traits, facilitating exploitation of promising regions. Mutation introduces stochastic perturbations that preserve diversity and enable escape from local optima. Low mutation probability often leads to convergence stagnation, particularly in rugged fitness landscapes. Excessively high mutation rates,

however, may disrupt convergence stability and cause oscillatory search behavior. Adaptive mutation schemes, in which mutation probability increases when population diversity decreases, have been proposed to mitigate premature convergence and improve robustness in nonlinear acoustic optimization problems [10, 103].

In GA–neural network hybrid optimization, convergence stability is further influenced by the topology of the acoustic fitness landscape. Inverse metamaterial design problems frequently exhibit highly sensitive objective functions, where small geometric perturbations produce abrupt changes in absorption coefficient or transmission loss. Such sensitivity can induce oscillatory convergence patterns or extended stagnation phases. Hybrid strategies combining GA-based global exploration with gradient-based local refinement or surrogate-assisted evaluation improve convergence efficiency while reducing computational burden. When generative models such as GANs are used to synthesize candidate microstructures, additional validation protocols are required before acoustic deployment. Generated geometries must be verified through: (1) finite element method (FEM) acoustic simulation comparison; (2) sound absorption coefficient (SAC) curve deviation analysis across relevant frequency bands; and (3) structural morphology similarity metrics to ensure physical plausibility. GAN training instability and mode collapse remain common risks, particularly in high-dimensional microstructure generation. Wasserstein GAN variants, which employ an alternative loss formulation to stabilize gradient behavior, have demonstrated improved convergence stability and reduced mode collapse in material microstructure synthesis tasks. Overall, GA–neural network hybrid optimization offers strong global search capability for nonconvex acoustic design problems, but careful tuning of population size, mutation–crossover balance, and diversity-preserving mechanisms is essential to avoid stagnation and ensure reliable convergence in resonance-dominated metamaterial systems. GAN-generated microstructures must be validated via:

- FEM acoustic simulation comparison,
- SAC curve deviation analysis,
- Structural morphology similarity metrics.

Mode collapse and instability are common risks; Wasserstein GAN variants improve convergence.

7.2 Bayesian Optimization for Nonconvex Acoustic Metamaterial Design

Bayesian Optimization (BO) is a sample-efficient global optimization framework well suited to acoustic metamaterial design problems characterized by highly nonconvex and multi-resonant response surfaces [104]. Unlike gradient-based optimization methods, BO does not require derivative information and remains robust in the presence of discontinuities, sharp impedance transitions, and multiple local optima. Such behaviors frequently arise in metamaterial systems exhibiting narrowband resonances, bandgap formation, and complex wave–structure interactions.

BO relies on probabilistic surrogate modelling, most commonly Gaussian Process (GP) regression, to approximate the objective function while quantifying predictive uncertainty [64,105,106]. The GP prior encodes assumptions regarding smoothness, stationarity, and spatial correlation through kernel selection, typically squared exponential or Matérn covariance functions. In the absence of strong physical priors, the mean function is often assumed constant, while kernel hyperparameters, such as characteristic length scales and variance terms, are estimated through marginal likelihood maximization [79]. As new design evaluations are performed, the surrogate model is sequentially updated, progressively improving its representation of the design landscape [86].

The exploration–exploitation trade-off in BO is controlled by acquisition functions that combine the surrogate predictive mean and variance. Expected Improvement (EI) selects candidate designs that maximize the anticipated improvement relative to the best observed solution, while Upper Confidence Bound (UCB) strategies introduce an explicit uncertainty weighting parameter to regulate exploration [107]. Exploration-dominated configurations prioritize sampling in regions of high uncertainty to improve global surrogate accuracy, whereas exploitation-dominated strategies refine solutions near promising designs. For highly nonconvex acoustic response surfaces with multiple resonance peaks and discontinuous impedance behavior, adaptive acquisition strategies

dynamically balance these objectives to maintain global search capability while minimizing computational cost.

A key advantage of BO in acoustic metamaterial design lies in its uncertainty-guided search. Since forward evaluations often require computationally intensive finite element or multi-physics simulations, sequential sampling based on information gain can significantly reduce the number of simulations required compared with exhaustive or population-based methods. This makes BO particularly effective for inverse design problems in which the mapping from geometry to acoustic performance is highly nonlinear and expensive to evaluate [29]. More broadly, BO is grounded in Bayesian machine learning, where probability theory provides a principled framework for incorporating prior knowledge and uncertainty into model inference [108]. Bayesian statistics describe uncertain quantities using probability distributions and update prior beliefs using observed data through Bayes' theorem.

$$p(\theta | x, y) = \frac{p(y | x, \theta)p(\theta)}{p(y | x)} \quad (14)$$

where $p(\theta)$ denotes the prior distribution of model parameters, $p(y | x, \theta)$ the likelihood of observing the data, and $p(\theta | x, y)$ the posterior distribution. The denominator $p(y | x)$, referred to as the evidence, acts as a normalizing constant.

$$y = x^T w + \varepsilon \quad (15)$$

In machine learning models, parameter estimation can therefore be formulated as a Bayesian inverse problem. For example, in a linear model as given in Equation (15), assigning Gaussian priors to the parameter vector w and assuming Gaussian observation noise yields a Gaussian posterior distribution over the parameters. This conjugate formulation enables efficient closed-form updates and sequential learning as new data are obtained [109].

Bayesian approaches offer several advantages for acoustic modelling and optimization. By explicitly quantifying uncertainty in both parameters and predictions, they reduce the risk of overfitting and provide probabilistic confidence estimates for model outputs. Furthermore, prior knowledge about the physical system can be incorporated directly through prior distributions, improving robustness in data-limited regimes. Overall, Bayesian optimization provides a principled and uncertainty-aware framework for inverse design in acoustic metamaterials. By combining probabilistic surrogate modelling with sequential information-driven sampling, BO enables efficient global optimization of highly nonlinear, resonance-dominated performance landscapes while maintaining rigorous uncertainty quantification.

7.3 Particle Swarm Optimization in Discontinuous Acoustic Response Landscapes

Particle Swarm Optimization (PSO) is frequently used in multilayer acoustic composite optimization due to its relatively simple implementation and rapid convergence during early search stages. The algorithm models collective swarm behavior, where candidate solutions referred to as particles move through the design space according to velocity updates influenced by both their individual best solution and the globally best solution identified by the swarm. This cooperative search strategy enables efficient exploration of nonlinear optimization landscapes without requiring gradient information [110].

In acoustic composite design, PSO has been applied to optimize structural parameters such as layer thickness, cavity dimensions, perforation ratios, and porous material characteristics, to maximize broadband sound absorption or transmission loss. Case-specific applications demonstrate that PSO can effectively identify high-performance configurations in multilayer absorber systems when coupled with acoustic simulation models. For example, optimization frameworks often integrate PSO with numerical solvers such as the Transfer Matrix Method or the Finite Element Method to evaluate acoustic response functions during each optimization iteration. These simulation-driven evaluations provide physically consistent performance metrics that guide the swarm search process [110]. However, objective functions in multilayer acoustic design frequently contain

discontinuities caused by abrupt resonance transitions, impedance mismatches, and structural mode coupling effects. These phenomena produce rugged optimization landscapes in which small geometric perturbations may generate large variations in acoustic response. Under such conditions, conventional PSO may suffer from convergence instability, premature particle clustering, or stagnation near local optima.

Several algorithmic modifications have been proposed to improve PSO robustness in these discontinuous acoustic response landscapes. Adaptive inertia weighting dynamically adjusts the momentum of particle motion, enabling broader exploration during early iterations and more focused exploitation during later search phases. Velocity clamping techniques restrict excessive particle displacement, preventing unstable oscillations in highly sensitive regions of the design space. Additionally, hybrid optimization frameworks that combine PSO with local search procedures or surrogate modelling can significantly enhance convergence reliability.

Integration of PSO with neural acoustic surrogate models provides a particularly effective hybrid optimization pathway. In such frameworks, machine learning models, such as deep neural networks trained on simulated acoustic datasets, serve as fast predictors of acoustic performance metrics. The PSO algorithm then operates on the surrogate model rather than directly evaluating expensive numerical simulations. This approach substantially reduces computational cost while preserving global search capability, making it suitable for large-scale optimization of composite material geometries and metamaterial unit cells.

Overall, PSO offers a practical optimization strategy for acoustic composite design due to its fast convergence and derivative-free operation. Nevertheless, its performance in resonance-dominated acoustic systems depends strongly on algorithmic enhancements and effective coupling with simulation or surrogate modelling frameworks. Adaptive parameter control and hybrid optimization architectures, therefore, play a critical role in ensuring reliable convergence across discontinuous acoustic response landscapes.

7.4 Multi-Objective Optimization (NSGA-II)

Multi-objective optimization is critical in acoustic composite design, where trade-offs between competing performance metrics—such as broadband sound absorption, transmission loss, and material weight—must be balanced. The Non-dominated Sorting Genetic Algorithm II (NSGA-II) is commonly used in this context due to its ability to efficiently explore Pareto-optimal fronts while maintaining solution diversity [111]. NSGA-II ranks candidate designs according to dominance levels and crowding distance, enabling simultaneous optimization across multiple acoustic objectives without scalarizing them into a single metric. Its evolutionary search strategy is particularly effective for exploring highly nonlinear, multimodal design spaces characteristic of layered acoustic composites.

7.5 Comparative Convergence and Efficiency Analysis

Comparative studies of optimization strategies in acoustic material design indicate that PSO achieves rapid initial convergence, especially in smooth or moderately discontinuous landscapes, but may stall near discontinuities without adaptive enhancements. NSGA-II, in contrast, provides superior exploration of multi-objective trade-offs but can require more computational effort per generation due to population-based sorting and crowding evaluation. Hybrid approaches that combine swarm-based global search with local refinement or evolutionary multi-objective algorithms often yield the best balance between convergence speed, solution quality, and robustness across discontinuous acoustic response landscapes.

8. Results and Discussion

This section presents a structured synthesis of findings derived from the reviewed studies. The results are categorized into (i) predictive performance trends, (ii) data dependency, and (iii) model limitations. It is important to note that these findings represent a cross-study synthesis and are therefore subject to variability arising from differences in datasets, material systems, and validation methodologies.

8.1 Comparative Analysis of Algorithmic Performance

ANN and DNN models frequently outperform traditional regression approaches within individual studies, particularly where nonlinear and multiscale relationships dominate. However, such comparisons are study-specific and should not be interpreted as universally generalizable due to differences in datasets, feature engineering, and validation protocols.

Performance metrics reported in the literature are not standardized. Studies variably report R^2 , RMSE, MAE, or percentage error, often computed over different frequency ranges and datasets. Consequently, direct numerical comparison between models across studies is inherently limited.

8.1.1 Advantages of Nonlinear Deep Learning Models

- Predictive Performance:

ANNs and DNNs routinely achieve R^2 values between 0.97 and 0.99, substantially surpassing polynomial regression models, which typically plateau at $R^2 \approx 0.85$ when modelling frequency-dependent absorption, impedance spectra, or transmission loss [112]. As microstructural complexity increases, particularly in architected, porous, or anisotropic acoustic systems, the performance gap between these approaches becomes more pronounced.

Deep learning models incorporating convolutional layers, residual blocks, or attention mechanisms extract high-dimensional nonlinear features that represent both local (pore geometry, cell size, fibre orientation) and global (macro-porosity gradients, multi-layer interactions) acoustic behaviors. This advanced feature extraction supports accurate prediction across diverse material classes, including microcellular foams, porous metamaterials, multi-layer absorbers, fibrous composites, and 3D-printed acoustic architectures.

- Role of Advanced Activation Functions:

Nonlinear activation functions, including Rectified Linear Unit (ReLU), Gaussian Error Linear Unit (GELU), and Exponential Linear Unit (ELU), enhance model stability during training, mitigate vanishing gradient issues, and improve convergence. Their use significantly increases predictive accuracy for large multi-frequency datasets, complex geometric descriptors, and heterogeneous acoustic materials [113].

8.2 A Benchmarking Framework for Artificial Intelligence Models in Acoustic Materials Prediction

Contemporary artificial intelligence (AI) techniques differ significantly in computational cost, predictive performance, data requirements, and suitability for various classes of composite and porous acoustic materials. Despite notable progress in AI-assisted acoustic modelling, the literature still lacks a unified benchmarking framework for comparing these algorithms across material systems and computational tasks. Recent reviews highlight substantial variations in accuracy and generalization behaviors among ANNs, CNNs, DNNs, PINNs, ensemble learning models, and hybrid machine learning–finite element method (ML–FEM) frameworks [25], [114].

Table 5 provides a structured comparison of these AI paradigms across key performance indicators, including accuracy, dataset size requirements, computational efficiency, strengths, weaknesses, and material applicability. Such benchmarking frameworks have been recommended in recent materials-informatics literature to guide model selection, enhance reproducibility, and improve cross-study comparability [26], [73]. The benchmarking methodology involves evaluating these models against predefined criteria such as predictive accuracy, dataset volume, computational resources, and ability to incorporate physical constraints.

Table 5. Benchmarking comparison of AI techniques for acoustic material prediction

AI Method	Accuracy	Data Requirement	Computation Time	Strengths	Weaknesses	Suitable Applications	Ref.
ANN	$R^2 = 0.95-0.99$; MAE $\approx 0.02-0.06$	Low-Moderate (50-500 samples)	Very fast (ms-s)	Excellent for tabular acoustic data, low computational cost, robust for small datasets	Poor at high-dimensional inputs; cannot process microstructure images	Porous foams, natural-fibre composites, impedance-tube SAC prediction	[116], [117]
CNN	$R^2 > 0.97$; error $< 3\%$	High (hundreds-thousands of images)	Moderate (s-min)	Extracts microstructural features, captures tortuosity and anisotropy, excellent for image-based prediction	High dataset requirement; expensive training	Meta-porous materials, 3D-printed lattices, micro-perforated panels	[118], [119]
DNN	$R^2 = 0.97-0.995$; MSE < 0.002	Moderate-High	Moderate	Learns highly nonlinear behavior; predicts multi-resonance phenomena	Overfitting risk; architecture tuning required	Helmholtz resonators, multilayer composites, resonant metamaterials	[120]
PINN	$R^2 = 0.98-0.995$	Low-Moderate	Slow-Moderate (min-hr)	Integrates governing acoustic PDEs, excellent extrapolation, robust for limited/noisy data	High training time; implementation complexity	Poroelastic materials, frequency-dependent wave propagation, coupled thermo-viscous effects	[25]
Ensemble Models	$R^2 = 0.90-0.98$	Low	Very fast	Strong feature ranking, robust to noise, good for heterogeneous tabular inputs	Cannot process image data; limited frequency-domain prediction	Natural-fibre composites, acoustic emission diagnostics	[10], [121], [122]
Hybrid ML-FEM	Error < 2 dB vs FEM; $R^2 > 0.995$	Low-Moderate	Very fast inference; slow initial training	Replaces slow FEM simulations; ideal for optimization and parametric sweeps	Requires FEM expertise; significant setup time	Metamaterials, lattice structures, aerospace/automotive acoustic panels	[73], [123]

Note: The reported performance ranges are compiled from multiple independent studies with varying datasets, material systems, and validation protocols. These values represent indicative ranges rather than directly comparable benchmarks.

An example application scenario illustrates its practical relevance: a researcher working on the acoustic properties of novel porous composites may consult the benchmarking framework to identify the most appropriate AI model for predictive modelling. If the researcher possesses limited experimental data but requires high accuracy and strong extrapolation capability, a physics-informed neural network would be suitable due to its integration of governing physical laws, such as the Helmholtz and poro-elastic wave equations [25,26]. Conversely, when working with large micro-CT or SEM datasets, CNN-based architectures offer superior performance because of their ability to extract morphological features such as pore connectivity, tortuosity, and anisotropy directly from imaging data [115]. ANN and DNN performance typically range between $R^2 \approx 0.90$ – 0.99 , depending on dataset size, feature representation, and validation protocol. Higher values (>0.97) are generally observed under controlled or interpolative conditions, while lower values are reported in cross-material or extrapolative scenarios. This framework, therefore, provides evidence-based guidance for selecting the most appropriate AI model for diverse acoustic engineering applications in automotive, aerospace, architectural, and materials-science domains. Table 5 summarizes the author-derived comparative synthesis of AI model performance across reviewed studies.

8.3 Cross-Dataset Generalization and Limitations

AI methods have shown significant promise in modelling sound absorption, acoustic impedance, transmission loss, damping behavior, and broadband noise reduction, offering prediction accuracies comparable to experimental measurements and numerical simulations. Traditional ML methods such as support vector machines (SVMs), k-nearest neighbors (k-NN), decision trees, random forests, and ANNs have demonstrated the ability to learn nonlinear, multidimensional acoustic relationships from limited datasets without requiring explicit prior assumptions. However, these methods also present limitations, particularly in handling complex, high-dimensional acoustic data and microstructural interactions. Table 6 presents the strengths and limitations of commonly employed traditional machine learning methods for predicting material properties. Understanding these factors helps select appropriate shallow learning techniques to predict the acoustic properties of composite materials.

Although traditional ML methods can identify correlations between material microstructure and acoustic performance, their effectiveness is limited by several factors. Manual feature extraction is required, which depends on expert knowledge to select parameters such as porosity, tortuosity, airflow resistivity, and pore-size distribution. This approach is time-intensive and may miss subtle microstructural features that significantly affect thermo-viscous dissipation and resonance behavior. Additionally, the shallow architectures of traditional ML models restrict their ability to capture the complex, nonlinear, and multi-physics interactions present in acoustic materials, where performance is influenced by geometry, viscosity, thermal boundary layers, and microstructural heterogeneity. Furthermore, separating feature engineering from prediction can reduce computational efficiency and limit the models' ability to identify deep, high-order correlations that govern acoustic phenomena.

Deep learning (DL) methods facilitate automated feature extraction from raw acoustic signals, high-dimensional simulation data, and microstructural images. Techniques such as convolutional neural networks (CNNs), recurrent neural networks (RNNs), autoencoders (AEs), deep belief networks (DBNs), and generative adversarial networks (GANs) address the limitations of traditional ML and have been utilized to predict absorption curves, optimize metamaterial topologies, and extract microstructural descriptors from two- and three-dimensional images. However, DL approaches require larger datasets, longer training times, and significant computational resources. Additionally, their 'black-box' nature presents interpretability challenges, which restrict adoption in safety-critical acoustic engineering domains such as aerospace. Table 7 presents the strengths and limitations of deep learning methods for predicting acoustic properties. Awareness of these factors facilitates the selection of suitable deep learning approaches for composite material analysis.

Table 6. Strengths and weaknesses of common shallow learning methods applied in acoustic property prediction

Traditional ML Methods	Strengths	Weaknesses
Support Vector Machine (SVM)	<ul style="list-style-type: none"> • High prediction accuracy for small acoustic datasets • Performs well in high-dimensional feature spaces (e.g., porosity, tortuosity, resistivity) • Robust to overfitting with proper kernel selection 	<ul style="list-style-type: none"> • Not suitable for very large acoustic datasets • Sensitive to noise in measurement data • Requires careful kernel tuning for broadband acoustic predictions
k-Nearest Neighbor (k-NN)	<ul style="list-style-type: none"> • Simple and easy to implement • Naturally handles non-linear acoustic relationships • Performs well when data distribution is smooth 	<ul style="list-style-type: none"> • Computationally expensive for large datasets • Performance degrades in high-dimensional acoustic feature spaces • Strongly influenced by the choice of k and feature scaling
Decision Tree	<ul style="list-style-type: none"> • Easy to interpret and visualize • Able to handle mixed acoustic features (categorical + continuous) • Fast training 	<ul style="list-style-type: none"> • Prone to overfitting, especially for spectral acoustic data • Requires pruning and domain knowledge • May produce unstable results with small datasets
Random Forest	<ul style="list-style-type: none"> • Reduces overfitting compared to single trees • Works well with multi-dimensional acoustic measurements • Handles noisy experimental data better 	<ul style="list-style-type: none"> • Computationally more expensive • Still lacks intrinsic interpretability for frequency-dependent phenomena
Artificial Neural Network (ANN)	<ul style="list-style-type: none"> • Capable of modelling highly nonlinear acoustic responses • High prediction accuracy for absorption and impedance • Effective at mapping complex microstructure–acoustic relationships 	<ul style="list-style-type: none"> • Requires large datasets to avoid overfitting • Training is computationally expensive • Limited interpretability (“black box” behavior)

In summary, AI algorithms are instrumental in advancing research on acoustic materials. Traditional ML methods are appropriate for scenarios with limited data and yield more interpretable models, whereas DL methods excel at capturing complex, nonlinear acoustic interactions and microstructure-dependent effects. A comparative summary, as presented in Table 8, is essential for helping researchers select suitable AI techniques for specific acoustic applications. Despite these advances, several challenges persist. A primary limitation is the scarcity of large, high-quality acoustic datasets. Acquiring acoustic data through impedance tube measurements, reverberation chamber tests, micro-CT imaging, finite element method (FEM) or boundary element method (BEM) simulations, and Kundt’s tube measurements is time-consuming, costly, and requires specialized equipment. Limited datasets impede the training, validation, and generalizability of AI models. Furthermore, inconsistent measurement techniques and the absence of standardized acoustic databases introduce noise and bias, hindering reproducibility and complicating cross-study benchmarking.

The design of deep learning model architectures remains challenging due to the numerous hyperparameters that must be optimized to achieve stable learning and accurate predictions across various frequency ranges and material structures. In the absence of standardized protocols for acoustic model tuning, performance can vary considerably between studies. Moreover, deep learning methods frequently lack interpretability, providing accurate predictions without elucidating how microstructure, frequency-dependent thermo-viscous effects, or material defects influence attenuation or resonance. This lack of transparency limits the trust and acceptance of AI

models in applications requiring certification and traceability, particularly in aerospace cabin acoustics, structural noise control, and automotive noise, vibration, and harshness (NVH) systems.

Table 7. Strengths and weaknesses of common deep learning methods applied in acoustic property prediction

Deep Learning Methods	Strengths	Weaknesses
Convolutional Neural Network (CNN)	<ul style="list-style-type: none"> • Well-suited for microstructure image data (e.g., micro-CT scans) • Excellent at extracting pore morphology, tortuosity, and topology features • Strong performance in mapping structure to frequency-dependent absorption 	<ul style="list-style-type: none"> • Requires large labelled datasets • Long training times • Prone to overfitting, especially with narrow frequency bands
Recurrent Neural Network (RNN)	<ul style="list-style-type: none"> • Ideal for sequential acoustic signals (time-domain and frequency-domain data) • Can model temporal and harmonic patterns in acoustic responses 	<ul style="list-style-type: none"> • Difficult to train due to vanishing/exploding gradients • Requires large sequences for robust learning • Complex implementation
Autoencoder (AE)	<ul style="list-style-type: none"> • Useful for feature extraction and dimensionality reduction • Efficient for compressing broadband acoustic spectra • Reduces noise in measured acoustic data 	<ul style="list-style-type: none"> • Requires large training data • Sensitive to noise in initial layers • Performance degrades if latent features fail to capture multiscale acoustic effects
Deep Belief Network (DBN)	<ul style="list-style-type: none"> • Extracts high-level acoustic features automatically • Performs well for moderate-sized datasets • Pre-training reduces the need for labelled data 	<ul style="list-style-type: none"> • Slow training due to complex layer initialization • Inference through stochastic hidden layers is difficult • Limited suitability for high-resolution acoustic spectra
Generative Adversarial Network (GAN)	<ul style="list-style-type: none"> • Effective for generating synthetic acoustic data • Can augment limited datasets (e.g., rare frequency bands) • Useful for generating new metamaterial and porous architectures 	<ul style="list-style-type: none"> • Difficult training and optimization • Sensitive to dataset imbalance • Performance decreases when training data are extremely limited

Another complication is the limited generalizability of AI models across different composite materials, fabrication methods, and frequency regimes. Models trained on specific natural fibers, three-dimensional printed architectures, or porosity gradients frequently underperform when applied to novel materials or altered boundary conditions. While transfer learning and domain adaptation present promising solutions, these approaches remain underexplored in the context of acoustic materials. Experimental validation of AI predictions is also essential, which increases both time and cost. A notable research gap is that most studies concentrate on predicting a single acoustic property, typically sound absorption, rather than employing multi-target AI models capable of predicting comprehensive acoustic spectra including absorption, impedance, damping, and transmission loss for multifunctional materials.

To advance the field, several targeted research directions are recommended. Data augmentation strategies, including synthetic data generation, physics-based simulations, and generative models such as GANs and variational autoencoders (VAEs), can mitigate data limitations. Hybrid modeling approaches that integrate AI with physics-based frameworks, such as the Helmholtz equation, Johnson–Champoux–Allard theory, and viscoelastic constitutive models, can enhance interpretability and physical consistency. Multi-scale modeling, which combines microstructural

imaging, mesoscale architecture, and macro-acoustic behavior through FEM/BEM-AI coupling, enables the characterization of acoustic responses across multiple length scales.

Enhancing interpretability should also be prioritized. Techniques such as SHapley Additive exPlanations (SHAP), Gradient-weighted Class Activation Mapping (Grad-CAM), and attention mechanisms can elucidate how microstructural features influence frequency-dependent behavior. Collaboration between AI researchers and acoustic experimentalists is essential for validating and refining predictive models. Additionally, the development of domain-specific AI architectures for porous absorbers, micro-perforated panels, metamaterial resonators, and layered acoustic barriers is crucial for producing high-fidelity, application-ready design tools.

Table 8. Strengths and weaknesses of ML and DL methods for acoustic property prediction

AI Methods	Strengths	Weaknesses
Traditional Machine Learning	<ul style="list-style-type: none">• Performs well on small acoustic datasets• Faster training and lower computational cost• Easier to interpret than deep models• Useful when microstructural features are known beforehand	<ul style="list-style-type: none">• Less accurate for complex, multi-physics acoustic interactions• Requires manual feature engineering• Struggles with nonlinear broadband behavior• Sensitive to data preprocessing
Deep Learning	<ul style="list-style-type: none">• High accuracy with large, high-dimensional datasets• Automatically extracts features from microstructure images and spectra• Captures nonlinear, multiscale acoustic behavior• Effective for acoustic metamaterials and topological design	<ul style="list-style-type: none">• Requires large datasets and GPU resources• Black-box nature reduces trust and interpretability• Risk of overfitting for narrow-band acoustic data• Hyperparameter tuning is complex and non-standard

9. Conclusions

This review provides a comprehensive evaluation of artificial intelligence applications in predicting and optimizing the acoustic properties of composite materials. By combining physics-based acoustic modelling principles with quantitative machine-learning performance assessment, the study establishes a reproducible benchmarking framework for evaluating AI models used in acoustic material design. AI-based models have demonstrated prediction accuracies approaching those of experimental measurements and numerical simulations in several controlled studies; however, their performance remains dependent on dataset quality, model design, and validation strategy.

Hybrid modelling strategies that integrate machine learning with established acoustic theories, such as the Helmholtz equation, Johnson–Champoux–Allard model, and Biot poroelastic theory, show particular promise in improving physical consistency and predictive robustness. However, several challenges remain, including limited dataset availability, inconsistent benchmarking practices, model interpretability issues, and reduced generalizability across material systems and operating conditions.

Current research also reveals several technical bottlenecks. At the algorithmic level, deep neural networks often require long training convergence times and large datasets, which limits their practical deployment in acoustic engineering workflows. Future research should therefore focus on three key areas: the development of hybrid neural network architectures that incorporate physical constraints, the exploration of adaptive training strategies within meta-learning frameworks, and the construction of robust structure–acoustic coupling models for multi-physics prediction.

From a technological perspective, acoustic neural networks are expected to evolve toward multimodal learning frameworks that integrate microstructural imaging, experimental acoustic measurements, and numerical simulations. Furthermore, the establishment of standardized benchmark datasets containing acoustic responses under diverse operating conditions is urgently needed. The development of unified evaluation metrics and reporting protocols will also be critical for enabling consistent cross-study comparisons.

With continued advances in algorithm development, physics-informed modelling, and hardware acceleration, artificial intelligence is likely to play a transformative role in acoustic materials engineering. These developments will accelerate the design of next-generation noise-control materials and enable disruptive applications in aerospace noise mitigation, structural acoustics, and advanced acoustic metamaterial design.

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Flexural behavior of reinforced concrete beams incorporating fly ash, GGBS, and Alccofine

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Abstract

The increasing demand for low-carbon concrete necessitates the development of binder systems that are capable of simultaneously ensuring structural performance and sustainability. This study presents an experimental investigation on the mechanical, structural and microstructural behavior of M30 grade concrete incorporating ternary blended binders composed of fly ash, ground granulated blast furnace slag and Alccofine. A systematic replacement strategy was adopted to identify an optimal binder composition that balances strength development and material efficiency. Mechanical properties were evaluated at 7 and 28 days, while reinforced concrete beams were tested under loading to assess load-deflection response and crack development. Microstructural analysis using scanning electron microscopy and energy dispersive X-ray analysis was performed to examine hydration morphology and interfacial transition zone characteristics. The results indicate that the optimized ternary mix achieved improvements of 19.29% in compressive strength, 33.3% in split tensile strength and 33.85% in flexural strength compared to the control mix. Structural testing showed enhanced load-carrying capacity, improved deformation response and delayed crack propagation. The findings suggest that the combined effect of particle packing, secondary hydration and interfacial transition zone refinement contributes to improved tensile behavior. In addition, the optimized mix achieved 52% clinker reduction, demonstrating its potential for sustainable structural applications.

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1. Introduction

The extensive use of cement as the primary binder in reinforced concrete construction poses significant challenges in terms of sustainability, durability, and long-term structural performance [1-3]. Cement production is among the most energy-intensive industrial processes involving high-temperature clinkerization and the inevitable release of carbon dioxide during limestone decarbonation [4,5]. As a result, the cement industry accounts for approximately 7–8% of global anthropogenic CO₂ emissions, prompting increasing regulatory and research-driven efforts to reduce OPC consumption in structural concrete [6-8]. In addition to its environmental burden, OPC-dominant concrete is frequently associated with durability-related concerns, including drying

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shrinkage, early-age microcracking, and vulnerability to aggressive exposure conditions such as chloride ingress, sulphate attack, and carbonation, all of which compromise serviceability and long-term structural performance [9-11]. Partial replacement of ordinary Portland cement (OPC) with supplementary cementitious materials (SCMs) has emerged as an effective strategy for improving sustainability, mechanical performance and durability of concrete [12-14]. Among widely used SCMs, fly ash and ground granulated blast furnace slag (GGBS) have been extensively investigated and implemented in structural concrete applications [15,16]. Fly ash, a siliceous and aluminous by-product of coal-fired power generation contributes primarily through pozzolanic reactions by consuming calcium hydroxide released during cement hydration to form secondary calcium silicate hydrate (C-S-H) and calcium aluminosilicate hydrate phases [17-19]. These reactions refine pore structure, reduce permeability, and enhance long-term strength and durability [20,21]. In addition, the substantially spherical morphology of fly ash particles improves fresh-state workability by enhancing particle distribution and reducing internal friction [22,23]. GGBS, produced by rapid quenching and grinding of molten blast furnace slag, exhibits latent hydraulic behavior and contributes additional binding phases under alkaline conditions, resulting in a denser cementitious matrix and improved resistance to chloride ingress and sulphate attack [24-26]. Despite these advantages, binary cementitious systems incorporating fly ash or GGBS exhibit certain limitations, particularly at higher replacement levels [27]. High-volume fly ash concretes often suffer from slower hydration kinetics and reduced early age strength development limiting their relevance in structural members requiring early stiffness and load carrying capacity [28,29]. Similarly excessive replacement of OPC with GGBS may extend setting time and delay early heat evolution, adversely affecting early mechanical performance despite improvements in long-term durability [30,31]. These limitations are especially critical for commonly adopted design grades such as M30, where a balance between early-age strength development, long-term durability, and structural reliability is required [32]. To overcome the limitations of conventional binary blends, recent research has progressively focused on ternary cementitious systems incorporating ultrafine and highly reactive mineral additives [33,34]. Among these, Alccofine-1203 an ultrafine slag-based mineral admixture with high fineness and reactive aluminosilicate content has gained considerable attention for its ability to modify hydration kinetics and microstructural evolution [35-37]. Unlike traditional SCMs that contribute primarily through slower pozzolanic or latent hydraulic reactions, Alccofine plays a dual role within the cementitious matrix physically acting as a micro-filler to enhance particle packing density and chemically providing nucleation sites that accelerate early hydration and promote rapid C-S-H formation [38-40]. These mechanisms lead to refinement of the interfacial transition zone (ITZ) improved phase continuity and enhanced early age strength development [41,42]. The combined fusion of fly ash, GGBS, and ultrafine Alccofine in ternary blended cementitious systems enables complementary physical and chemical interactions [1]. Fly ash contributes to long-term microstructural refinement through sustained secondary hydration, GGBS enhances durability through latent hydraulic binding, and Alccofine improves early-age densification through packing optimization and hydration acceleration [43,44].

The combined use of these materials produces a more homogeneous and compact microstructure with reduced pore interaction and improved load transfer pathways which enhances compressive strength and improves tensile and flexural performance by delaying microcrack initiation and enabling improved stress redistribution under applied loading [37,45,46]. Although numerous studies have examined the mechanical and durability performance of binary and ternary SCM-based concretes, much of the existing literature remains focused on material-level characterization, particularly compressive strength and durability indices [10,37]. Comparatively limited attention has been directed toward establishing explicit relationships between microstructural evolution and the structural behavior of reinforced concrete members [10,47]. In practice, the performance of reinforced concrete beams is governed by cracking behavior, stiffness degradation, load-deflection response, and failure modes, which cannot be fully captured through specimen-scale testing alone [10,48,49]. However, existing studies predominantly focus on material-level properties such as compression strength and durability indices, with limited emphasis on establishing a direct and experimentally validated relationship between microstructural refinement and member-level structural response, particularly in terms of load-deflection behavior, cracking characteristics and stiffness degradation of reinforced concrete beams [50-52].

Despite extensive studies on supplementary cementitious materials, most existing research is limited to material-level characterization, with insufficient emphasis on establishing a direct and experimentally validated relationship between microstructural evolution and structural behavior of reinforced concrete members [53-58]. In this context, the present study introduces a multi-scale experimental framework that quantitatively links ternary binder composition, microstructural refinement, and flexural performance of reinforced concrete beams [59]. Unlike previous studies, this work integrates mechanical testing, beam-level structural response, and microstructural evidence to establish a direct correlation between hydration mechanisms, interfacial transition zone characteristics, and load-deflection behavior [60-62]. This approach provides a more comprehensive understanding of how binder-level modifications influence structural performance [63-66].

2. Materials

2.1 Constituent Materials

The use of Ordinary Portland Cement (OPC, 53 grade) was considered as the main binding agent. Class F fly ash, Ground Granulated Blast Furnace Slag (GGBS), and Alccofine-1203 were considered as supplementary cementitious materials to prepare ternary binder mixes. All the materials were considered in their as-received form without any further processing. OPC contributed to early strength development and creating an alkaline environment in the pores to facilitate the subsequent hydration reactions. The addition of supplementary materials was done to enhance the late strength and facilitate the pore refinement of the hardened mass. The physical properties of the component materials are given in Table 1, and the oxide contents are given in Table 2. Fly ash meets IS 3812 (Part 1):2013[53] specifications, and GGBS meets IS 12089:1987 [54] specifications. The supplementary distribution of SiO_2 , Al_2O_3 , and CaO -rich phases helps in the combined pozzolanic and latent hydraulic reactivity. The physical appearance and particle characteristics of materials are shown in Fig. 1(a-c). Fly ash consists of fine, rounded particles, which improve workability and exhibit delayed pozzolanic activity. GGBS appears as finer, irregular-shaped particles with latent hydraulic properties under alkaline conditions.

Table 1. Summary of the physical properties of the constituent materials used in this study

Material	Specific gravity	Bulk density (kg/m^3)
Cement (OPC)	3.01	1440
GGBS	2.85	1300
Fly ash	2.28	979
Alccofine	2.70	680
Fine aggregate	2.62	1600
Coarse aggregate	2.66	1550

Alccofine contains ultrafine particles that promote heterogeneous nucleation and enhance particle packing in the binder matrix. The combination of these components promotes staged hydration reactions and results in a gradual refinement and densification of the cementitious matrix.

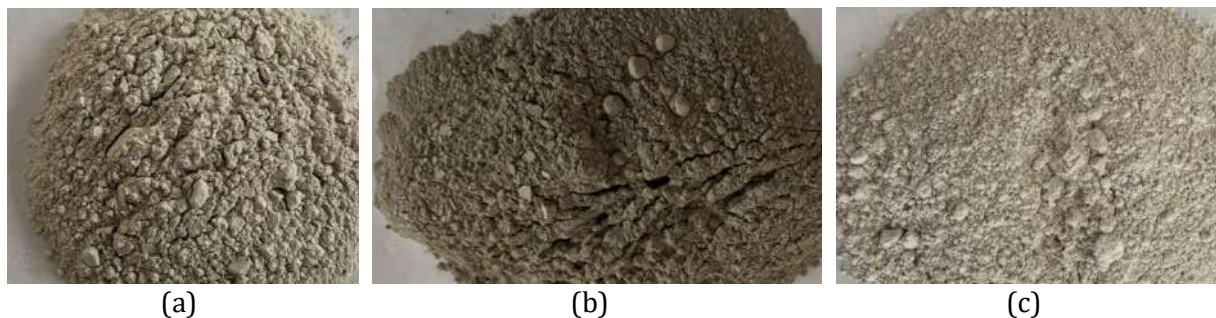


Fig. 1. Physical appearance of cementitious materials; (a) GGBS (100 μm), (b) fly ash (50 μm) and (c) Alccofine (20 μm)

Table 2. The chemical composition of the cementitious materials used in this study.

Oxide	Cement	Fly Ash	GGBS	Alccofine
SiO ₂	18.7	54.22	34.06	35.03
Al ₂ O ₃	4.5	31.18	20.00	21.4
CaO	64.8	1.35	32.6	32.20
Fe ₂ O ₃	4.1	2.63	0.8	1.20

2.2 Mix Proportioning

The concrete mix design was done to produce M30 grade concrete as per IS 10262:2019 [55], with the control mix designed using Ordinary Portland Cement. To investigate the influence of ternary hybrid cementitious materials, cement was partially replaced with fly ash, GGBS and Alccofine following a systematic replacement strategy while maintaining consistent workability and strength requirements. Fly ash content was varied up to 25%, GGBS content ranged between 15% and 35%, and Alccofine was incorporated in the range of 10% to 12% by weight of total binder. The selected replacement ranges for fly ash, GGBS, and Alccofine were based on previous studies and preliminary trials to ensure a balance between early-age strength development, workability and long-term hydration, while avoiding excessive dilution of clinker content that could adversely affect structural performance. For all mixes, the total binder content was kept constant to isolate the effect of material replacement on performance. The detailed percentage proportions of OPC, fly ash, GGBS and Alccofine adopted for each mix are presented in Table 3. A total of ten mix proportions including one control mix and nine blended mixes were designed and evaluated to identify optimum combinations based on mechanical properties.

Table 3. Mix proportions of concrete mixes incorporating ternary blended binders

Mix ID	Cement kg/m ³	Fly Ash kg/m ³	GGBS kg/m ³	Alccofine kg/m ³	Coarse Aggregate kg/m ³	Fine Aggregate (kg/m ³)	Water (kg/m ³)	W/B Ratio	Cement (% of Binder)	Fly Ash (% of Binder)	GGBS	Alccofine (% of Binder)	Total SCMs (% of Binder)
M1	380.0	0.0	0.0	0.0	1308	555	177	0.466	100.0%	0.0%	0.0%	0.0%	0.0%
M2	190.0	76.0	76.0	38.0	1308	555	177	0.466	50.0%	20.0%	20.0%	10.0%	50.0%
M3	163.4	95.0	76.0	45.6	1308	555	177	0.466	43.0%	25.0%	20.0%	12.0%	57.0%
M4	133.0	95.0	114.0	38.0	1308	555	177	0.466	35.0%	25.0%	30.0%	10.0%	65.0%
M5	125.6	95.0	114.0	45.6	1308	555	177	0.466	33.0%	25.0%	30.0%	12.0%	67.0%
M6	152.0	95.0	95.0	38.0	1308	555	177	0.466	40.0%	25.0%	25.0%	10.0%	60.0%
M7	106.4	95.0	133.0	45.6	1308	555	177	0.466	28.0%	25.0%	35.0%	12.0%	72.0%
M8	182.4	76.0	76.0	45.6	1308	555	177	0.466	48.0%	20.0%	20.0%	12.0%	52.0%
M9	182.4	57.0	95.0	45.6	1308	555	177	0.466	48.0%	15.0%	25.0%	12.0%	52.0%
M10	182.4	95.0	57.0	45.6	1308	555	177	0.466	48.0%	25.0%	15.0%	12.0%	52.0%

3. Experimental Procedure

A performance-oriented experimental framework was adopted to establish multi-scale correlations between binder composition and structural behavior. Ten M30 concrete mixes were designed by systematically varying fly ash, GGBS, and Alccofine contents while maintaining constant total binder content and water–binder ratio. Compressive strength at 28 and 56 days was used for preliminary screening to capture hydration development and secondary reaction effects. On the basis of strength performance and variability, four mixes (M1, M2, M8, and M9) were shortlisted for further analysis. The shortlisted mixes were analyzed for compressive, split tensile, and flexural strength as per the standards. SEM-EDX analysis was carried out to investigate the morphology of hydrates, pore structure, and elemental analysis. The reinforced concrete beams were subjected to two-point monotonic loading to assess the load-deflection characteristics, cracking pattern, stiffness loss, and failure behavior. Each reported value represents the mean of three specimens, and variability was quantified using standard deviation and coefficient of

variation. The coefficient of variation for the measured properties ranged between 3% and 6%, indicating experimental consistency. The differences between the control mix and optimized mixes were analyzed using a two-tailed t-test at a confidence level of 95% to establish the significance of differences. One beam specimen was tested for each mix; therefore, the structural results should be interpreted as indicative rather than statistically representative.

3.1 Workability of Fresh Concrete

The workability test was done using the slump test as per IS 1199:2018 [56]. The test was performed immediately after mixing using a standard slump cone apparatus. Concrete was placed in three layers, each compacted with 25 tamping strokes, followed by lifting the cone vertically to measure the slump.

3.2 Casting and Curing

Concrete specimens were cast and cured following a structured experimental framework to ensure systematic evaluation of mechanical properties and structural behavior. A total of ten mix proportions were initially developed for evaluation. Cube specimens were prepared and subjected to curing for 28 and 56 days to assess compressive strength and to identify the most suitable mix compositions based on strength performance. All specimens were water-cured at a temperature of $27 \pm 2^\circ\text{C}$ until the specified testing age. Based on strength-based screening, the selected mixes were subjected to further mechanical testing. After completion of the prescribed curing periods, reinforced concrete beam specimens were subsequently cast using the optimum mixes, cured for 28 days and tested to evaluate structural performance. The complete progression of casting and curing, specimen preparation and testing adopted in the study is expressed in Fig. 2.

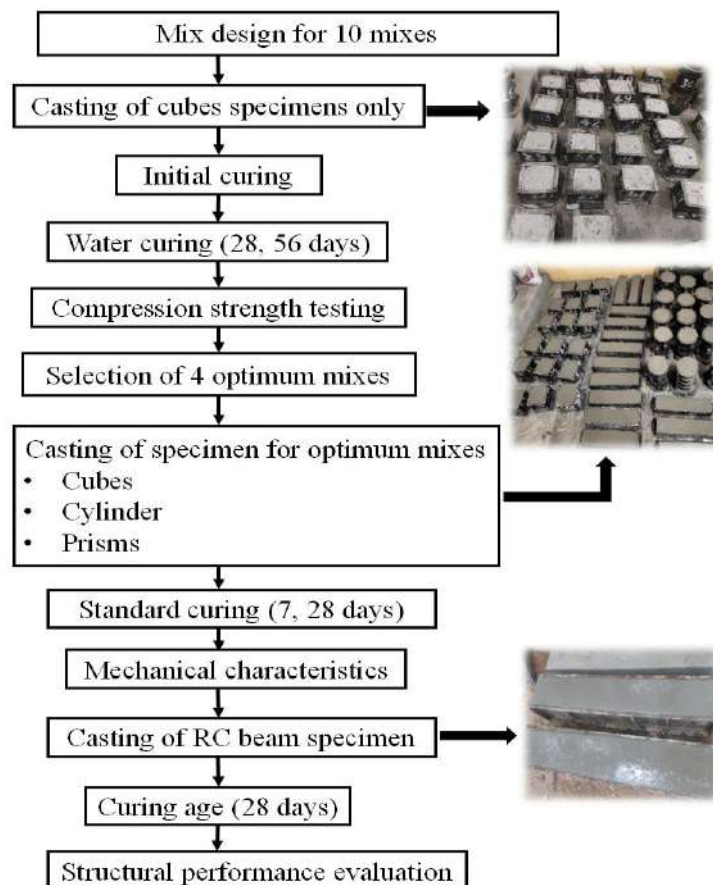


Fig. 2. Experimental workflow illustrating casting, curing, mechanical testing, and structural evaluation of concrete specimens and RC beams

3.3 Mechanical Properties

The mechanical properties of the concrete were determined in accordance with applicable Indian Standards. Compressive strength and flexural strength were measured on cube and prism specimens as per IS 516 [57]. Split tensile strength was determined on cylindrical specimens as per IS 5816 [58]. All specimens were properly cured prior to testing. For each test, three specimens were cast and tested and the average values were reported. The details of specimen types and counts are summarized in Table 4. Cube specimens of size 150 mm x 150 mm x 150 mm were used for compressive strength, cylindrical specimens of 150 mm x 300 mm were used for split tensile strength and prism specimens of 100 mm x 100 mm x 500 mm were used for flexural strength in accordance with relevant Indian standards.

Table 4. Summary of the specimen details

Specimen details	Count
Cubes	84
Cylinders	24
Prisms	24
Beams	4

3.4 Beam Design

The reinforced concrete beams in this study were designed in accordance with IS 456:2000 [57], with the objective of achieving a flexure-controlled failure. All beams were under-reinforced, ensuring that yielding of the tensile reinforcement preceded crushing of the concrete in the compression zone, which allows a clear assessment of the influence of supplementary cementitious materials (SCMs) on flexural behavior. Four concrete mixes were used: the control M1-PCC (plain cement concrete) and three SCM mixes incorporating fly ash (F), GGBS (G), and Alccofine (A)—M2 (F20-G20-A10), M8 (F20-G20-A12), and M9 (F15-G25-A12). All beams shared identical geometry (1500 mm x 230 mm x 230 mm), reinforcement detailing, span, and support conditions, ensuring that variations in ultimate load and cracking behavior could be attributed solely to the SCM composition. Shear reinforcement was designed to be adequate in preventing shear failure, ensuring that all specimens failed by flexure. This design allows a direct assessment of the influence of various SCM combinations on the load-deflection behavior and cracking patterns of the reinforced concrete beam. The calculated flexural capacity (M_u) and shear capacity (V_u) for the critical section were 20.85 kN.m and 46.2 kN respectively, confirming that shear capacity significantly exceeded flexural demand, thereby ensuring flexural failure.

3.5 Beam Geometry and Reinforcement Details

The reinforced concrete beam specimens were cast with similar geometry and reinforcement details to provide a comparable structural response and thus evaluate the effect of concrete composition on flexural response. The beam had a square cross-section of 230 mm x 230 mm and a total length of 1500 mm, and was tested under simply supported conditions with an effective span of 1400 mm. A nominal clear concrete cover of 25 mm was provided as per IS 456:2000 [59]. The beams were designed as singly reinforced, under-reinforced flexural members to provide a flexural failure mode due to yielding of the tensile reinforcement before concrete crushing. Longitudinal tension reinforcement consisted of 12 mm diameter high-yield strength deformed bars, and nominal compression reinforcement was provided using 10 mm diameter bars. Adequate anchorage and development length were ensured as per IS-code provisions. To prevent premature shear failure and to ensure a flexure-controlled response, shear reinforcement was provided using 8 mm diameter closed stirrups at a uniform spacing of 130 mm c/c along the beam length. The shear reinforcement was designed such that the shear capacity exceeded the corresponding flexural demand, enabling stable crack development and reliable assessment of flexural performance under monotonic loading. The geometry and reinforcement are illustrated in Fig. 3 and the detailing parameters are summarized in Table 5.

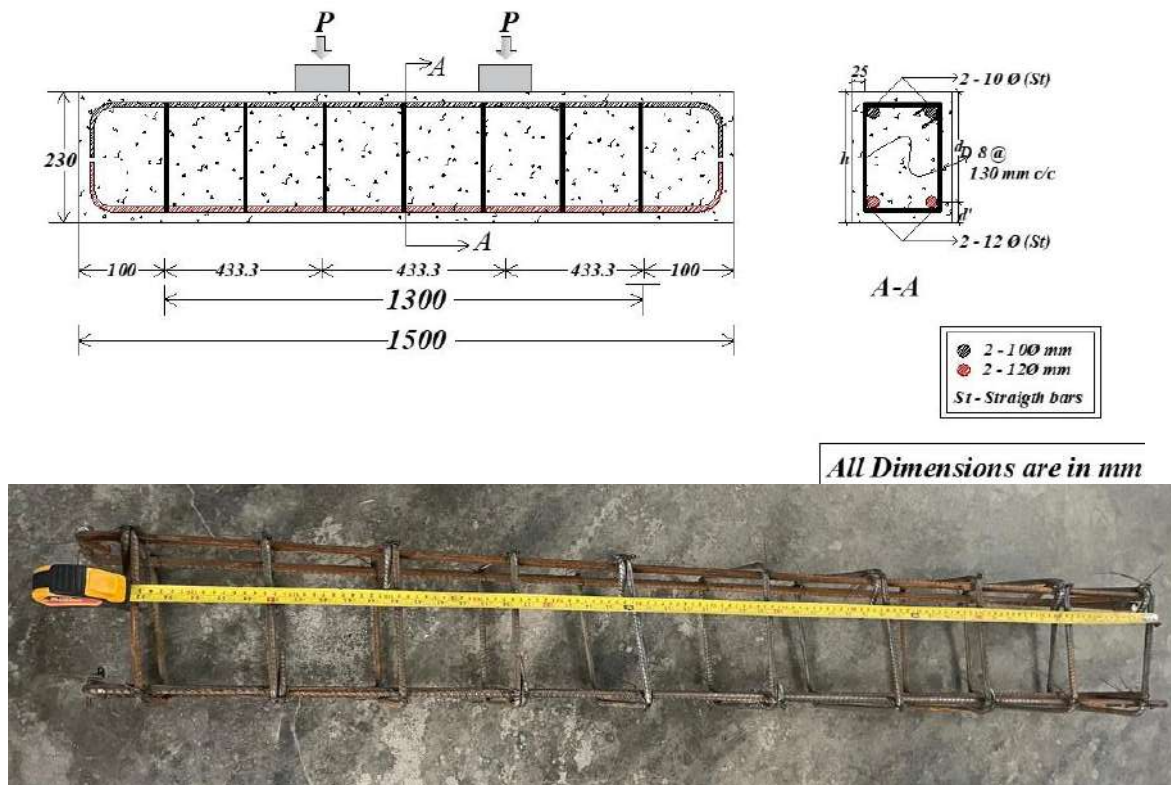


Fig. 3. Geometrical reinforcement details

Table 5. Reinforcement detailing parameters of RC beam specimens

b × D (mm)	L (mm)	Span (mm)	Cover (mm)	Tension steel	Compression steel	Stirrups	Steel grade	Support
230 × 230	1500	1400	25	2-12 mm	2-10 mm	8 mm @ 130 mm c/c	Fe 550	Simply supported

3.6 Test Setup and Instrumentation

All reinforced concrete beam specimens were tested under monotonic static loading. A simulated uniformly distributed load was applied through two symmetrically positioned concentrated point loads along the span. The beams were supported on a roller support at one end and a hinged support at the other providing a simply supported condition while allowing free rotation and minimizing restraint effects during testing. The applied load was delivered through a hydraulic loading frame provided with a calibrated load cell and transferred to the beam via a two-point loading arrangement, which effectively represents a uniformly distributed load over the central region of the span. This loading configuration was adopted to generate a constant bending moment region between the loading points thereby confirming that the beams were subjected predominantly to flexural action while limiting the influence of shear. Mid span deflection was measured using a linear variable differential transformer (LVDT) kept at the bottom of the beam. The LVDT was securely mounted to record the deflection during the test. The crack initiation and growth were also visually monitored and marked on the surface of the beam representing the flexural cracking pattern. The loading was incrementally applied until the occurrence of the first flexural cracks, and then it was continued until failure, which was indicated by a substantial loss of load-carrying capacity or excessive deflection. A schematic illustration of the test arrangement is shown in Fig. 4. Crack width and crack spacing measurements were not instrumented and were assessed qualitatively; therefore, crack-related interpretations are based on visual observations.

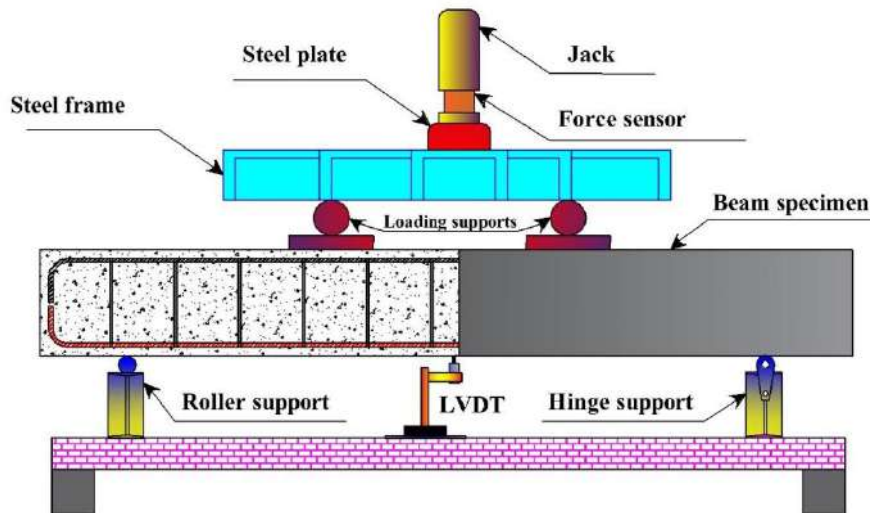


Fig. 4. Test setup and instrumentation

3.7 Microstructural Analysis

Microstructural analysis was conducted on the shortlisted optimal mixes to determine the direct correlation between the mechanical properties and the internal microstructural properties of the cementitious matrix. Scanning Electron Microscopy (SEM) and Energy Dispersive X-ray analysis (EDX) were used to investigate the hydration morphology, pore structure, and elemental composition. Representative particles were extracted from the interior of crushed concrete specimens after 28 days of compressive strength testing to ensure that the microstructural properties observed were representative of the hydration products and not altered by surface carbonation. SEM micrographs allowed the detailed observation of hydration products, matrix density, pore refinement, and interfacial transition zone (ITZ) quality. EDX spectra were utilized to provide phase identification using elemental composition and Ca-Si-Al maps to confirm the presence of C-S-H rich areas, ettringite-like morphology, and secondary aluminosilicate reaction products due to pozzolanic and latent hydraulic reactions. The combined SEM-EDX analysis provides supporting evidence for the observed mechanical performance, although it should not be interpreted as definitive proof of causation. Microstructural analysis demonstrates a denser cementitious matrix with diminished capillary porosity, a more consolidated interfacial transition zone and improved spatial distribution and interconnection of hydration products within the hardened composite. For each mix, multiple regions were examined, and representative micrographs were selected to ensure consistency of observations.

4. Results and Discussion

4.1 Workability

The variation in slump values across different mixes is presented in Fig. 5. An increase in workability is observed with the incorporation of supplementary cementitious materials. All mixes exhibited slump values within the acceptable range for reinforced concrete applications, indicating adequate workability suitable for proper placement and compaction of reinforced concrete without segregation. This behavior is attributed to improved particle packing and reduced internal friction within the concrete matrix.

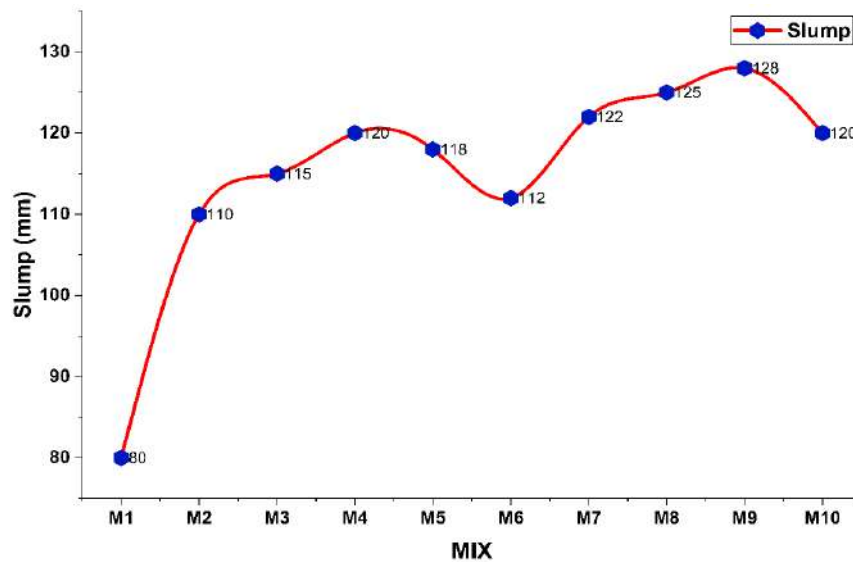


Fig. 5. The variation in slump values of different mixes

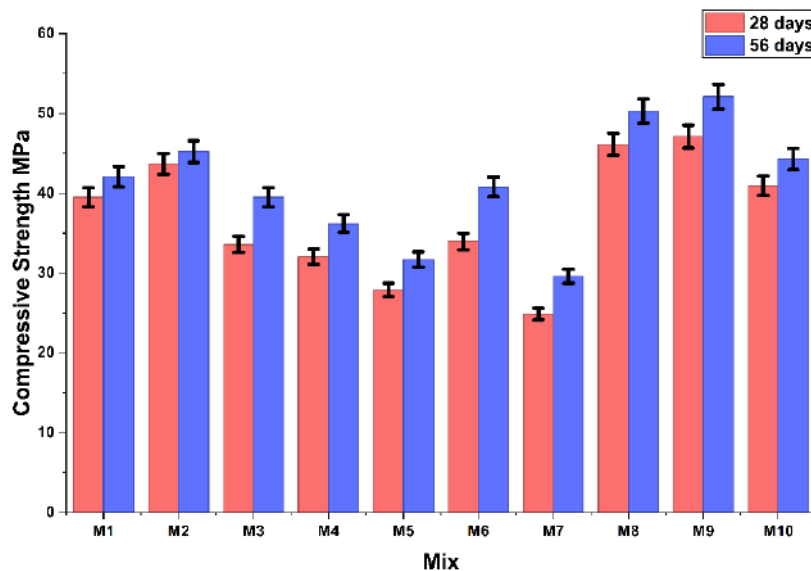
4.2 Compressive Strength

Minor variations in compressive strength between screening and detailed phases are due to separate casting batches at different experimental stages and do not affect the overall performance ranking. The compressive strengths recorded at 28 and 56 days (Table 6; Fig. 6(a)) vary from 24.87 to 47.12 MPa and from 29.6 to 52.1 MPa respectively. Error bars representing standard deviation are included in Fig. 6(a-d) to indicate variability in measured properties. The continued increase in strength from 28 to 56 days in the blended mixes reflects sustained hydration and persistent secondary reaction activity within the supplementary cementitious system. Based on this performance trend, mixes M1, M2, M8 and M9 were identified for detailed evaluation.

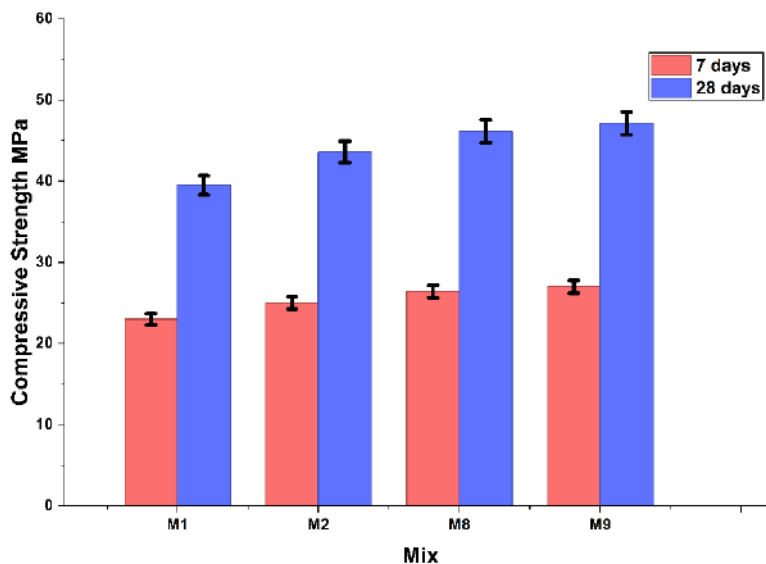
Table 6. Compressive strength of concrete mixes

MIX ID	Compressive strength (MPa)			
	Selection of optimum mix			
	28 days		56 days	
	Measured (MPa)	Enhancement (%)	Measured (MPa)	Enhancement (%)
M1	39.5	-	42.1	-
M2	43.69	10.60	45.2	7.40
M3	33.58	-15.0	39.5	-6.2
M4	32.05	-18.9	36.2	-14.0
M5	27.87	-29.4	31.7	-24.7
M6	33.94	-14.1	40.8	-3.1
M7	24.87	-37.0	29.6	-29.7
M8	46.14	16.80	50.3	19.50
M9	47.12	19.29	52.1	23.80
M10	40.93	3.60	44.3	5.20
	optimum mix			
	7 days		28 days	
	Measured (MPa)	Enhancement (%)	Measured (MPa)	Enhancement (%)
M1	23.0	-	39.5	-
M2	25.0	8.70	43.6	8.50
M8	26.4	14.80	46.14	14.80
M9	27.0	17.40	47.12	17.20

Their corresponding 7- and 28-day strength values are presented in Table 6 (optimum group) and Fig. 6(b), showing 28-day values of 39.5–47.12 MPa, exceeding the 30 MPa requirement for M30 grade concrete, thereby validating their suitability for subsequent tensile, flexural, stress–strain, microstructural, and beam-level studies. The compressive test setup is shown in Fig. 7(a). The improved performance observed in M8 and M9 indicates an effective interaction among the components of the ternary binder. Alccofine contributes to enhanced early-stage particle packing and accelerates nucleation of hydration products whereas fly ash and GGBS support continued reaction and progressive matrix densification at later ages. This complementary interaction supports continued strength development and enhances long-term retention.



(a)



(b)

Fig. 6. (a). Compressive strength development of concrete mixes at 28 and 56 days and Fig. 6(b). Comparison of compressive strength of selected concrete mixes at 7 and 28 days

The superior performance of Mix M9 can be attributed to an optimal balance between calcium availability and reactive silica, which promotes the formation of a dense calcium–aluminosilicate hydrate gel network. Unlike higher replacement levels (e.g., M5 and M7), where excessive clinker dilution limits early hydration, M9 maintains sufficient calcium hydroxide for secondary reactions while enabling progressive microstructural densification. This balance enhances both early-age strength and long-term performance. Similar trends have been reported in recent studies on

ternary blended systems, where optimized combinations improve both packing density and hydration kinetics, leading to enhanced mechanical and structural response.

4.3 Split Tensile Strength

The split tensile strength results of the selected mixes at 7 and 28 days are presented in Table 7 and shown in Fig. 6(c). At 7 days, the tensile strength increased from 2.6 MPa for M1 to 3.6 MPa for M9. At 28 days, the corresponding values rose from 3.9 MPa (M1) to 5.2 MPa (M9). The experimental setup for the split tensile test is illustrated in Fig. 7(b). The tensile strength was found to increase with the curing age, signifying better resistance to crack formation and better stress transfer mechanisms in the cementitious matrix. This is related to the improved paste matrix continuity and ITZ refinement. The mechanical results are in line with the microstructural results presented in Section 6, which indicate lower pore connectivity and a denser ITZ in the ternary blended cements.

Table 7. Split tensile strength of concrete mixes at 7 and 28 days

MIX ID	Split Tensile Strength (MPa)			
	7 days		28 days	
	Measured (MPa)	Enhancement (%)	Measured (MPa)	Enhancement (%)
M1	2.6	-	3.9	-
M2	2.9	11.50	4.4	12.80
M8	3.3	26.90	4.9	25.60
M9	3.6	38.50	5.2	33.30

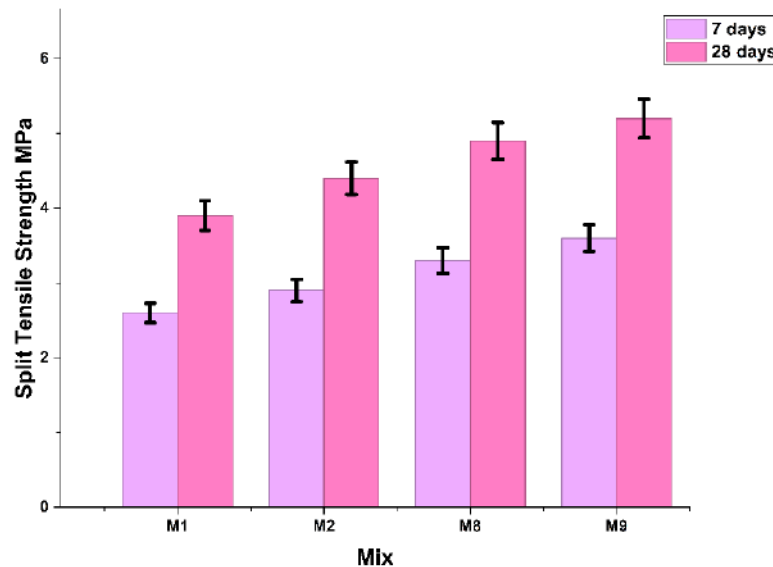


Fig. 6(c). Split tensile strength of selected concrete mixes at 7 and 28 days

4.4 Flexural Strength

The values of flexural strength at 7 and 28 days are given in Table 8 and Fig. 6(d). At 28 days, the flexural strength was found to increase from 6.5 MPa for M1 to 8.7 MPa for M9. The experimental setup for the flexural test is given in Fig. 7(c). The relatively larger increase in flexural strength compared to the compressive strength (Table 8 compared to Table 6) suggests that the ternary mixture is more effective in enhancing the tensile-dominated fracture properties. The increase in flexural strength is disproportionately higher than compressive strength, indicating improved tensile resistance and crack-bridging efficiency due to interfacial transition zone reinforcement. This can be attributed to the enhanced continuity of ITZs and the suppression of calcium hydroxide-rich regions, as evident from the SEM micrographs (Section 6)

Table 8. Flexural Strength of concrete mixes at 7 and 28 days

MIX ID	Flexural Strength (MPa)			
	7 days		28 days	
	Measured (MPa)	Enhancement (%)	Measured (MPa)	Enhancement (%)
M1	5.6	-	6.5	-
M2	6.2	10.71	7.4	13.85
M8	7.2	28.57	8.1	24.62
M9	7.4	32.14	8.7	33.85

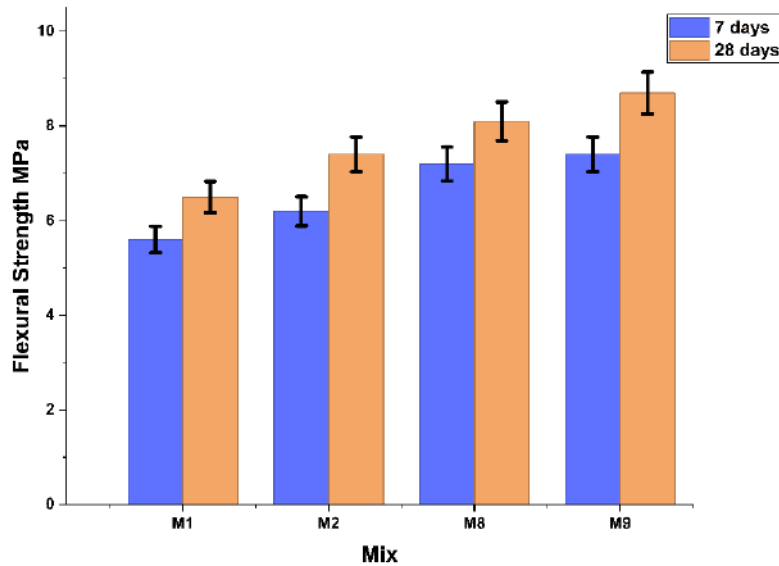


Fig. 6(d). Flexural strength of selected concrete mixes at 7 and 28 days

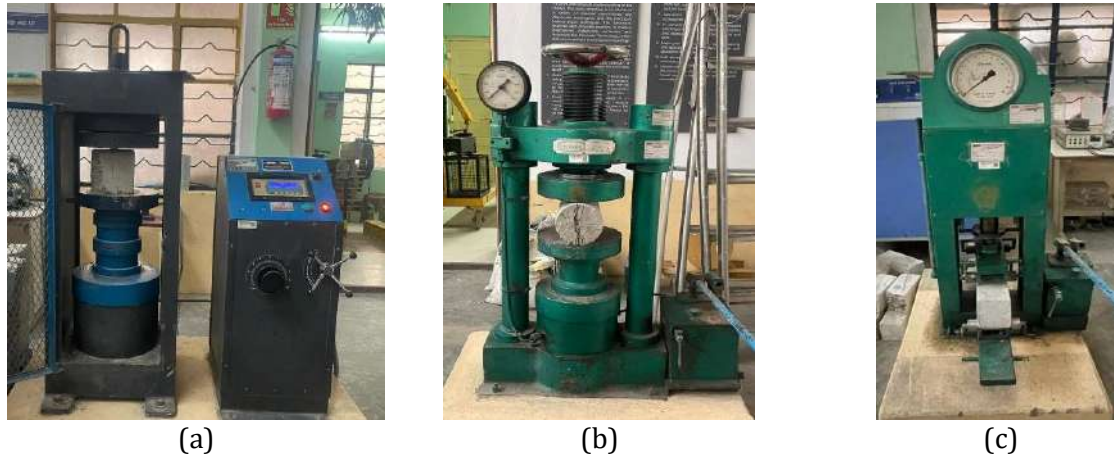


Fig. 7. Experimental setup (a) compression strength test, (b) split tensile test & (c) flexural strength test

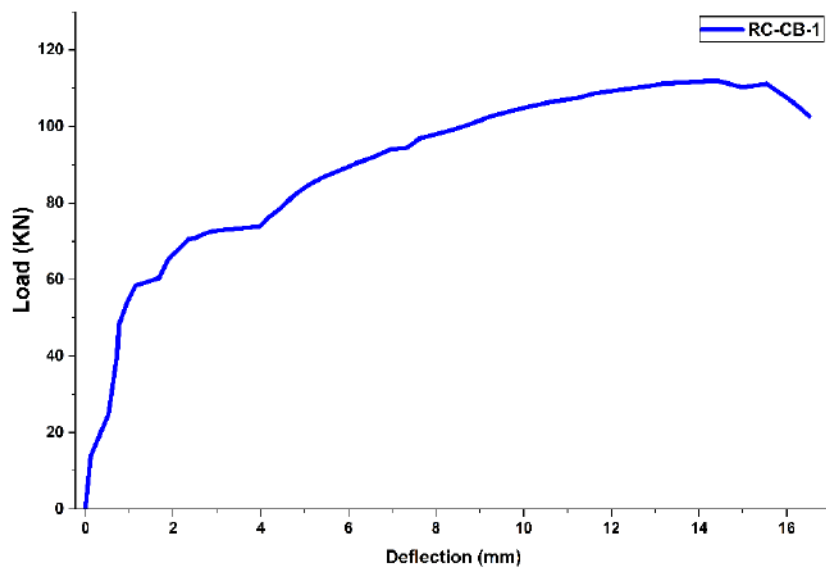
5. Structural Performance

The flexural performance of the reinforced concrete beams indicated an improvement in structural behavior following the optimization of ternary blended binder compositions. Compared to the control beam, the specimens containing fly ash–GGBS–Alccofine blends exhibited higher ultimate load capacity, improved curvature ductility, and enhanced post-cracking stiffness retention. The smoother transition from linear elastic to nonlinear behavior, along with moderated stiffness degradation, is consistent with a more distributed cracking mechanism and improved internal stress transfer. These results indicate that the refinement of binder composition at the material level is reflected in the structural performance at the member level. For the clarity of presentation, the reinforced concrete control beam is referred to as RC-CB-1 (M1), where RC-CB stands for

reinforced concrete control beam, whereas the ternary blended beams are referred to as RC-TB-2 (M2), RC-TB-3 (M8), and RC-TB-4 (M9), where RC-TB stands for reinforced concrete ternary beam.

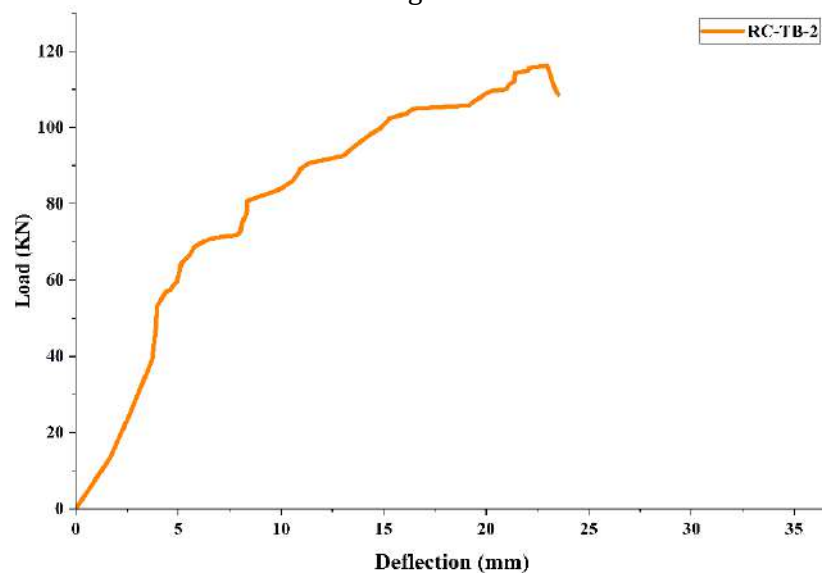
5.1 Load-Deflection Behavior of Beams

The load-deflection curves for all beams under monotonic two-point loading are shown in Fig. 8. These curves illustrate the typical under-reinforced behavior, which consists of an initial linear and elastic portion, followed by a stiffness degradation after cracking, nonlinear deformation evolution, and finally post-peak softening. The control beam (M1) attained an ultimate load of 112.00 kN, as seen in Fig. 8(a). The ternary beams showed an increasingly higher ultimate load resistance, with M2, M8, and M9 reaching 116.26 kN, 120.00 kN, and 124.60 kN, respectively (Fig. 8(b-d)). The optimized mix proportion (M9) showed an 11.3% increase in ultimate load resistance over the OPC control. Following the peak load, the control beam experienced a sudden loss of load-carrying capacity, characteristic of localized crushing in the compression zone. Compared to the control, the ternary blended beams exhibited a less sudden post-peak stiffness degradation, which is consistent with a more distributed cracking mechanism and improved internal stress transfer which can be attributed to the improved microstructural uniformity and refined interfacial transition zone that facilitate more distributed cracking and gradual stress transfer within the concrete matrix.



(a)

Fig. 8



(b)

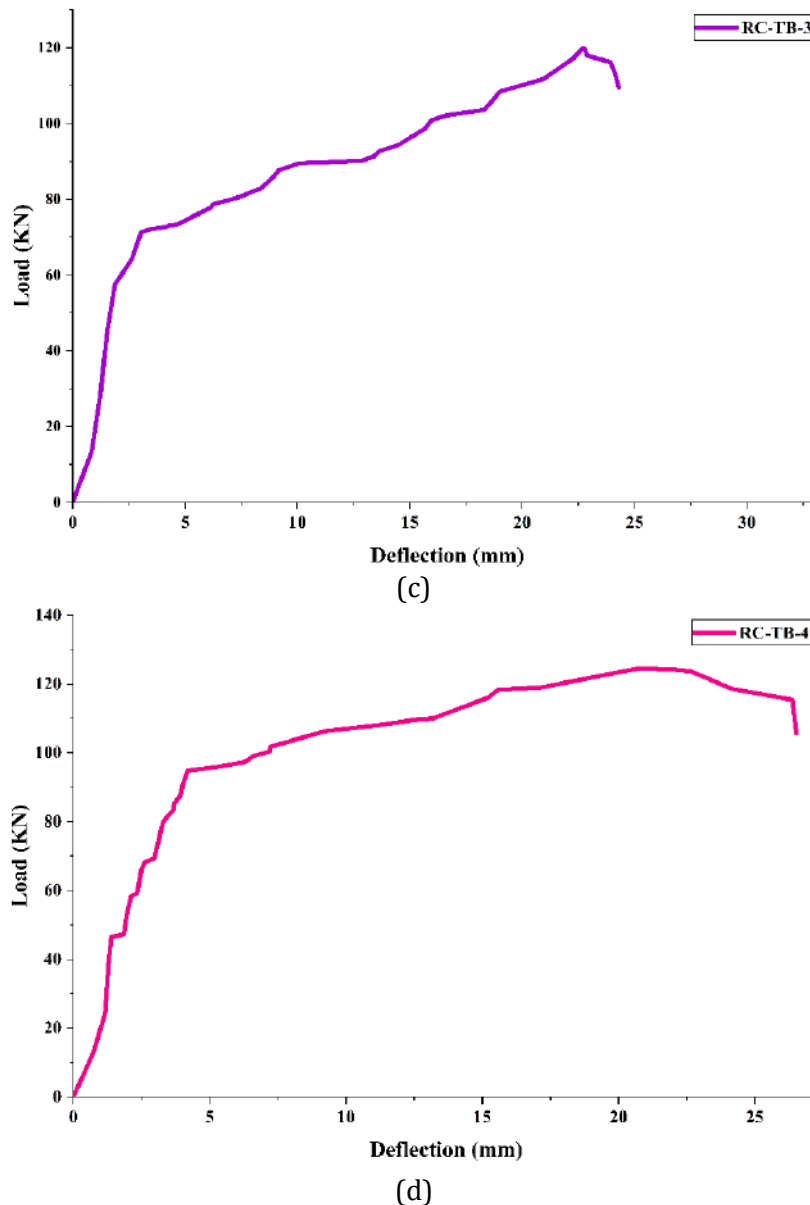


Fig. 8. Load–deflection response of control beam (a) RC-CB-1, (b) RC-TB-2, (c) RC-TB-3, and (d) RC-TB-4

5.2 Experimental vs Theoretical Flexural Capacity

The comparison between the predicted and actual moment capacities is given in Table 9. The ultimate flexural strength was calculated on the basis of strain compatibility for a doubly reinforced rectangular section, as per IS 456:2000. The cross-section of the beam was 230 mm × 230 mm with an effective depth of 180 mm, area of tension reinforcement 191.63 mm², area of compression reinforcement 157.08 mm² (2-10 mm bars), and strength of steel 550 MPa. The shear span was 0.433 m

Table 9. Experimental and theoretical flexural moment capacity of RC beams

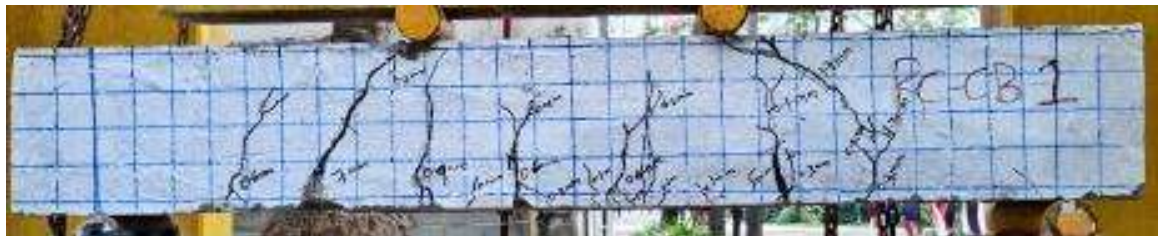
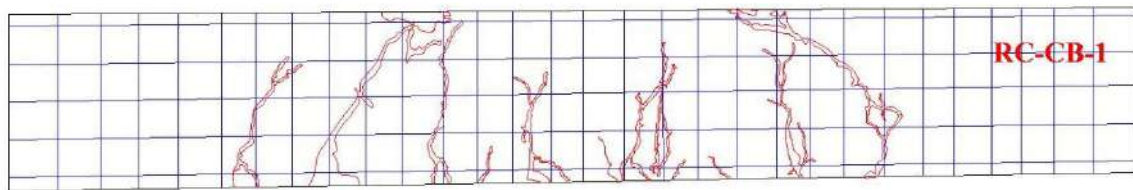
Beam	$M_{u,th}$	$M_{u,exp}$	$M_{u,exp} / M_{u,th}$
M1	19.82	24.25	1.22
M2	20.36	25.17	1.24
M8	20.71	25.98	1.25
M9	20.85	26.98	1.29

Experimental capacities exceeded theoretical predictions by 22–29%, reflecting conservative stress-block assumptions, compression steel contribution, and tension stiffening effects. The

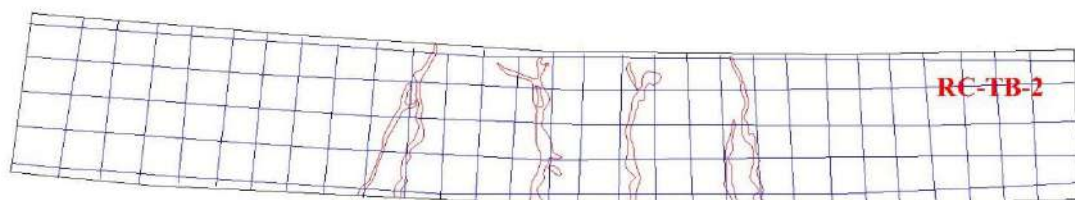
progressive increase in moment capacity tends to confirm that binder refinement improves the efficiency of the compression zone with no negative effect on under-reinforced bending.

5.3 Crack Pattern and Failure Characteristics

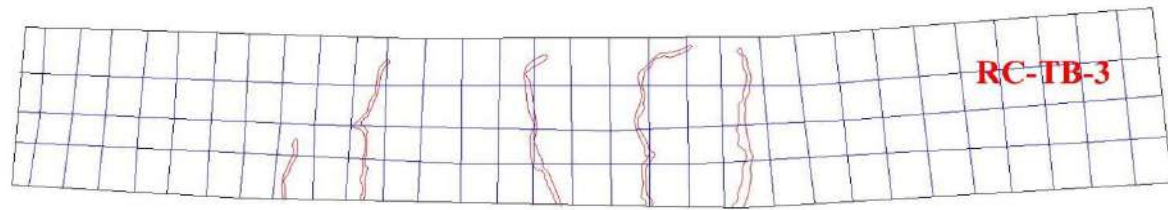
Typical crack patterns at ultimate load are presented in Fig. 9(a-d). In all beams, flexural cracks originated in the constant moment zone and then propagated vertically towards the compression zone. The control beam (Fig. 9a) had fewer but wider cracks, reflecting an earlier stage of crack localization and lower tension stiffening. In contrast, beams M2 and M8 (Fig. 9b-c) exhibited a greater number of finer, more uniformly distributed cracks, suggesting improved bond behavior and strain compatibility between concrete and reinforcement, likely due to the densification of the interfacial transition zone and reduced micro-porosity, which enhances stress transfer across the steel-concrete interface. The optimized beam M9 (Fig. 9d) demonstrated the most refined crack distribution with delayed crack widening and improved spatial spread. This behavior is consistent with enhanced interfacial transition zone densification observed in SEM analysis. All specimens ultimately failed in a flexure-dominated mode characterized by yielding of tensile reinforcement followed by compression zone crushing, confirming under-reinforced behavior consistent with strain compatibility assumptions.



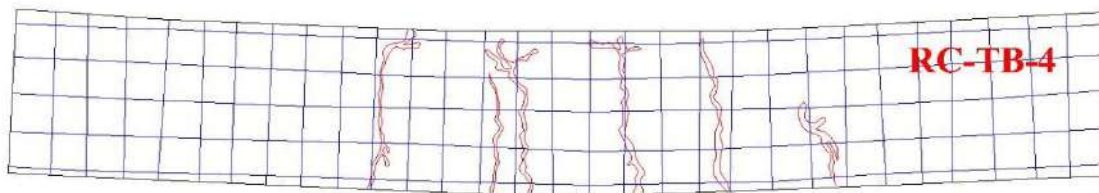
(a)



(b)



(c)



(d)

Fig. 9. Crack pattern and failure characteristics of beam specimens (a) RC-CB-1, (b) RC-TB-2, (c) RC-TB-3 & (d) RC-TB-4

5.4 Ductility and Stiffness Degradation

The deformation capacity of the reinforced concrete beams was quantified using a deflection-based ductility index derived from the load–deflection responses shown in Fig. 8. The ductility index was computed as $\mu = \Delta u / \Delta y$, where Δy is the mid-span deflection corresponding to $0.75P_u$ on the ascending branch of the load–deflection curve and Δu is the ultimate mid-span deflection at failure. The evaluated parameters are presented in Table 10.

Table 10. Ductility parameters obtained from load- deflection response

Beam (Mix)	P_u (kN)	Δu (mm)	Δy at $0.75P_u$ (mm)	$\mu = \Delta u / \Delta y$
M1	112.00	16.52	7.50	2.20
M2	116.26	23.50	8.90	2.64
M8	120.00	24.30	9.10	2.67
M9	124.60	26.50	9.80	2.70

The ductility index increased from 2.20 in the control beam (M1) to 2.70 in the optimized beam (M9) while the ultimate deflection increased by 60% relative to the control. The control specimen showed a marked loss of stiffness after cracking, indicative of localized damage development and early compression-zone softening. Conversely, the ternary blended beams experienced greater deformations with a softer slope of the secant stiffness and a shallower slope in the post-peak

region. The cracking pattern distribution shown in Fig. 9, and the softer slope of the descending part in Fig. 8, further verifies the above-mentioned phenomenon. The experimental results indicate that ternary binder refinement improves deformation capacity while maintaining the same under-reinforcement failure mode.

6. Microstructural Characteristics

The SEM micrographs of the control mix (Fig. 10(a–c)) reveal a heterogeneous cementitious matrix with a predominance of primary hydration products. Regions exhibiting morphology consistent with C–S–H phases were observed; however, phase identification based on visual characteristics remains indicative rather than definitive. The interfacial transition zone (ITZ) is relatively porous, with a discontinuous gel region and CH concentration. The microstructural inhomogeneities cause localized stiffness changes and provide easy paths for the development of cracks under tensile loading. The presence of partially hydrated cement particles, along with pore networks, indicates poor packing density and an underdeveloped hydration gel structure. The microstructural characteristics are in line with the lower tensile strength and the relatively brittle post-peak response of the control specimen. This study establishes a direct correlation between ternary binder composition, microstructural refinement and flexural behavior at the structural level, which remains insufficiently addressed in existing literature.

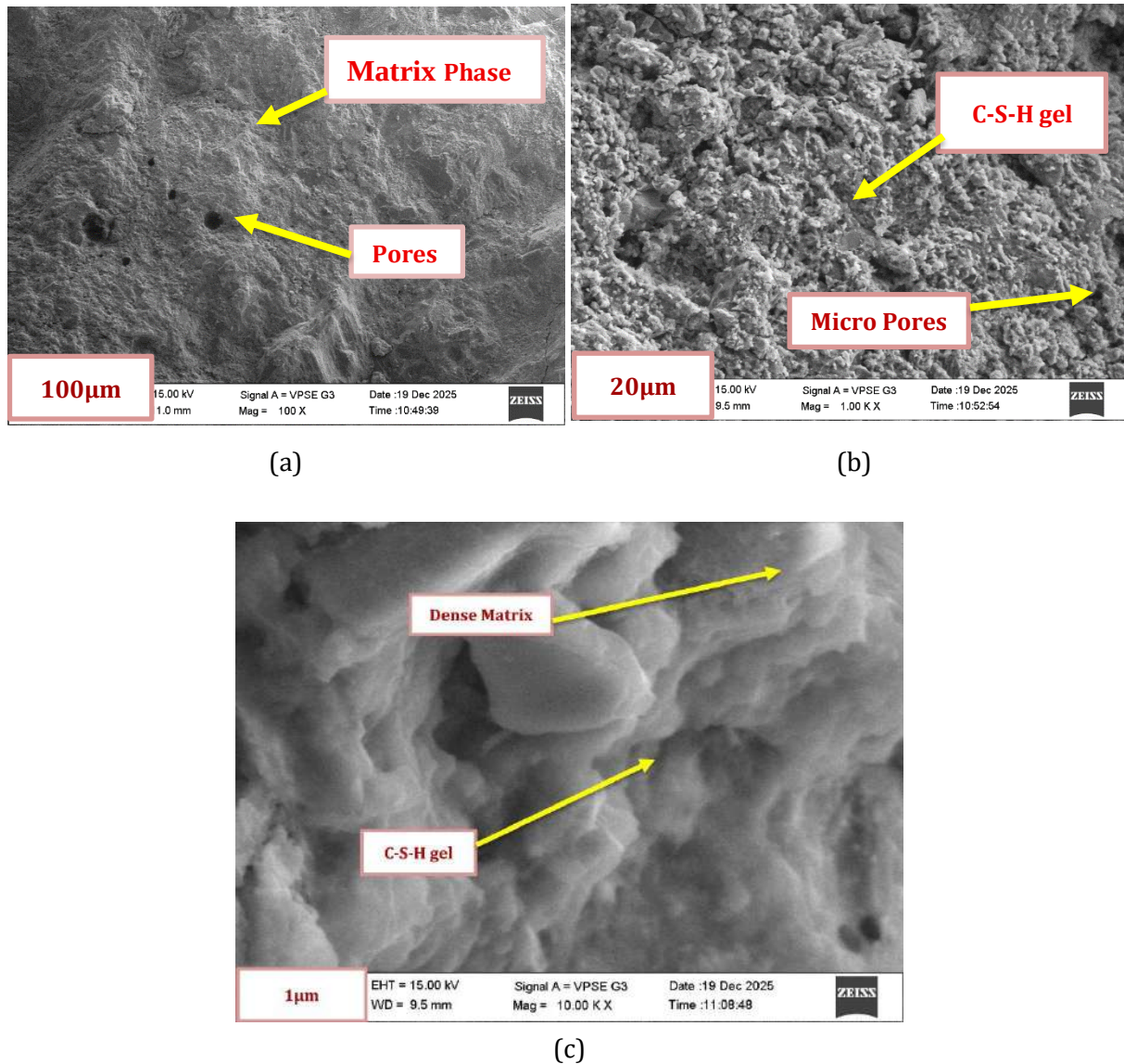


Fig. 10. SEM micrographs of control mix M1 at (a) low (b) medium & (c) high magnification

6.1 Microstructural Characteristics of Mix M2

SEM micrographs of Mix M2 (Fig. 11a–c) reveal a denser and more uniform matrix compared to the control. A marked reduction in the size and number of calcium hydroxide (CH) crystals indicates the presence of pozzolanic activity. The hydration products are more continuous, signifying better phase integration. The interfacial transition zone (ITZ) is relatively denser, with reduced pore connectivity and better bonding between the matrix and aggregates.

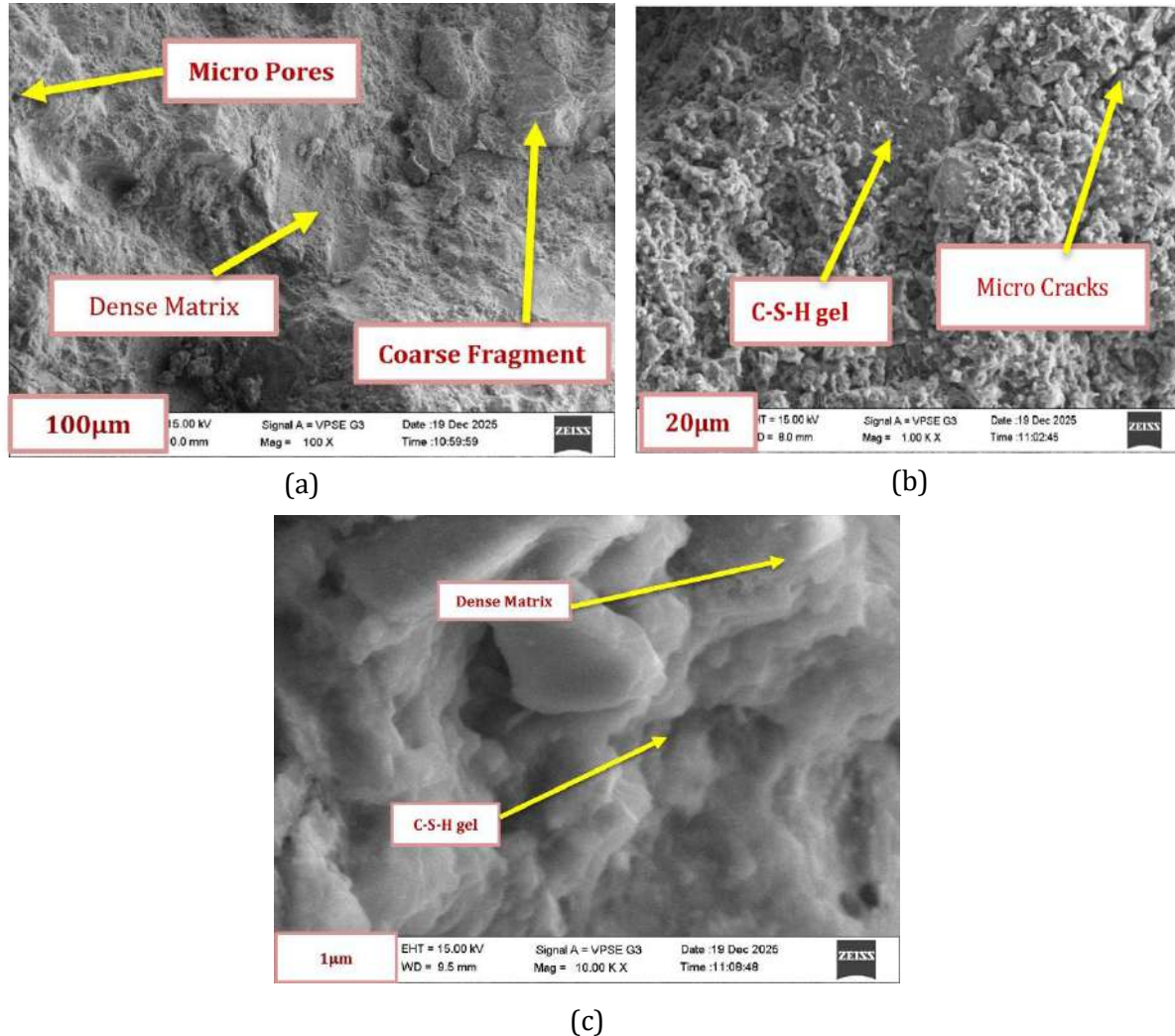


Fig. 11. SEM micrographs of mix M2 at (a) low (b) medium & (c) high magnification

6.1.1 Microstructural Characteristics – Mix M8

Mix M8 (Fig. 12a–c) shows significant pore refinement and a reduced incidence of microcracks. The matrix looks dense with fewer portlandite deposits, signifying a good interaction between fly ash, GGBS, and Alccofine. The improved spatial continuity of the hydration phases increases the efficiency of load transfer in the cementitious matrix

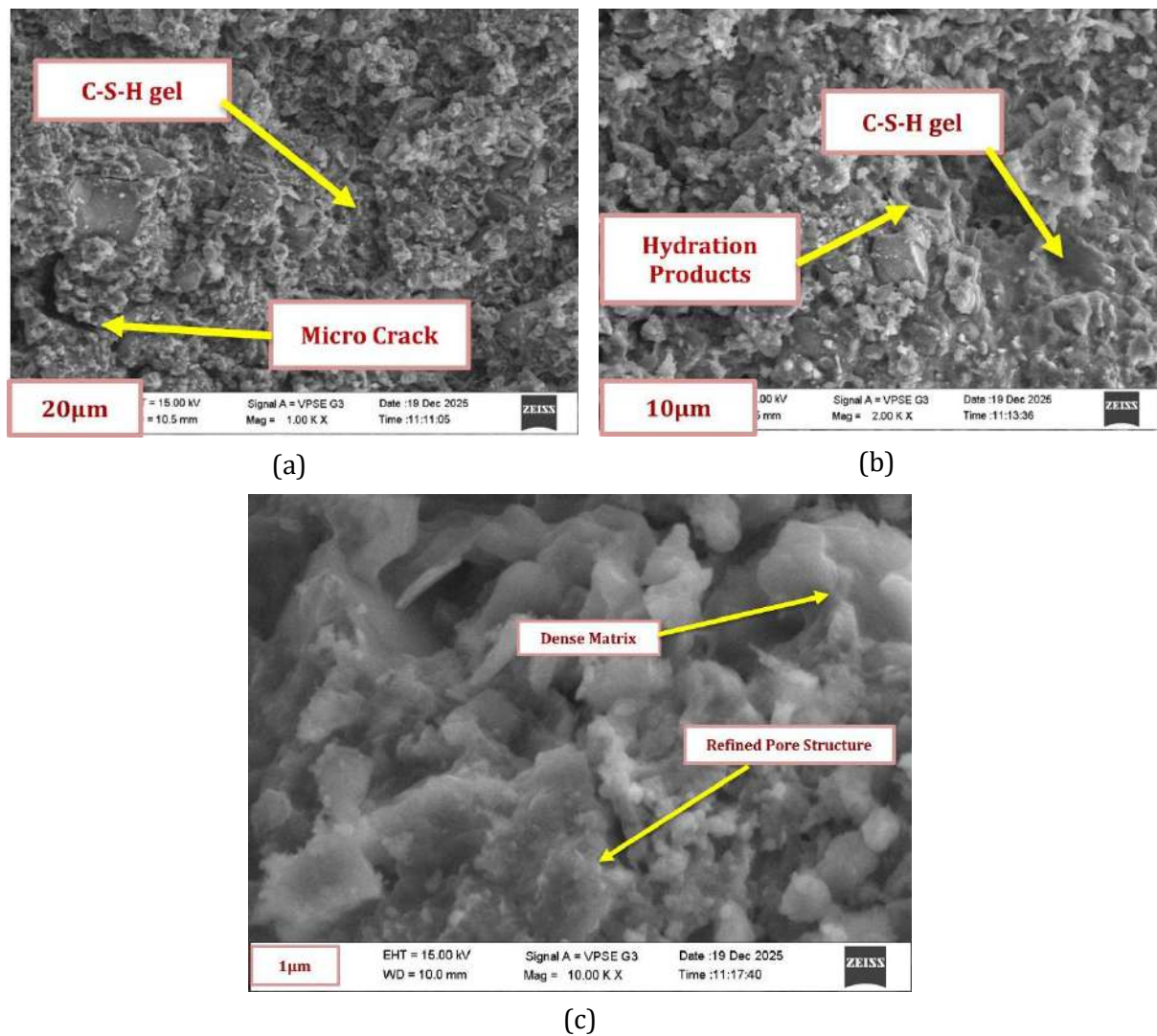
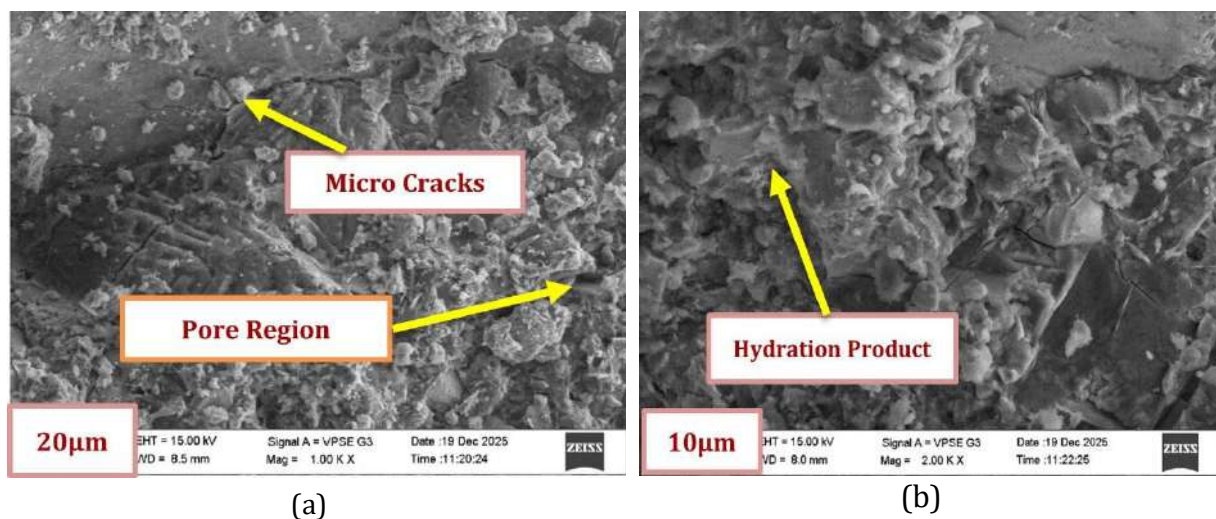


Fig. 12. SEM micrographs of mix M8 at (a) low (b) medium and (c) high magnification

6.1.2 Microstructural Characteristics – Mix M9

Mix M9 (Fig. 13a–c) has the most advanced microstructure. Features indicative of C-A-S-H-type phases and reduced capillary porosity are observed. The ITZ zone is very dense and continuous, which is conducive to enhanced tensile strength and resistance to crack localization.



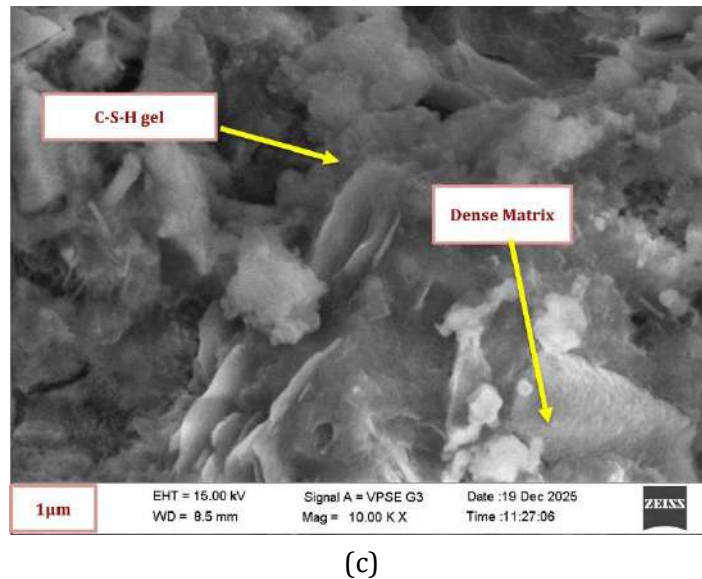


Fig. 13. SEM micrographs of mix M9 at (a) low (b) medium & (c) high magnification

The gradual increase in microstructural density from M1 to M9 clearly indicates that ternary synergy promotes microstructural stability. It should be noted that SEM observations are primarily morphological in nature and therefore identification of hydration phases is based on visual features rather than definitive phase confirmation. Consequently, microstructural interpretations are presented as supportive evidence and should be considered in conjunction with mechanical and structural results.

7. EDX Analysis

The EDX spectra of Mixes M1, M2, M8, and M9 are given in Fig. 14–17, and the corresponding atomic percentages are given in Table 11. The control mix (M1) has a Ca/Si ratio of 4.86, indicating a calcium-rich matrix due to the presence of portlandite. The addition of supplementary cementitious materials causes a drastic decrease in the Ca/Si ratio. Mix M2 shows the lowest ratio of 1.41, while Mix M8 shows an intermediate ratio of 2.60. The optimized mix M9 has a Ca/Si ratio of 1.76, indicating a significant consumption of calcium hydroxide and better silicate incorporation into the hydrate matrix. Although Mix M2 shows the lowest Ca/Si ratio, Mix M9 achieves better mechanical and structural properties. This indicates that the enhancement of strength is not dependent on the Ca/Si ratio but on the combined influence of hydrate chemistry modification, ultrafine particle packing, pore refinement, and ITZ modification. While Ca/Si ratio provides useful insight into hydration chemistry, it cannot independently explain mechanical performance and is therefore used only as a supportive indicator.

Table 11. Elemental composition and Ca/ Si atomic ratio of concrete mixes obtained from EDX analysis

MIX ID	C%	O%	Mg%	Al%	Si%	K%	Ca%	Fe%	Ca/Si
M1	0.4	64.1	0.6	1.4	5.2	0.8	25.3	1.5	4.86
M2	0.2	73.2	1.9	3.6	8.0	0.9	11.3	0.9	1.41
M8	0.1	66.6	1.1	2.6	7.5	1.0	19.5	1.6	2.60
M9	6.9	64.6	1.4	3.5	8.2	-	14.4	1.0	1.76

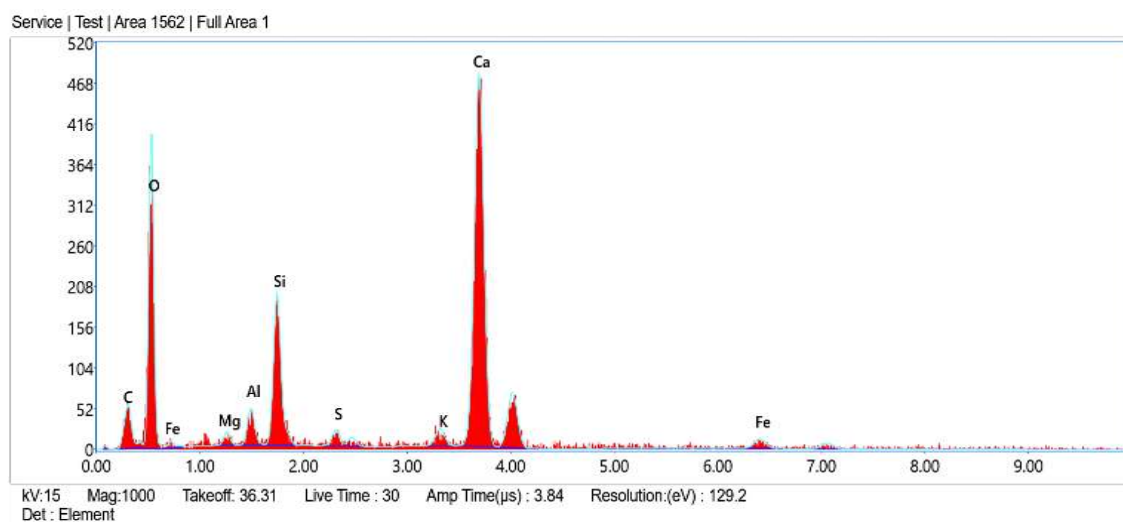


Fig. 14. EDX Analysis of Mix 1

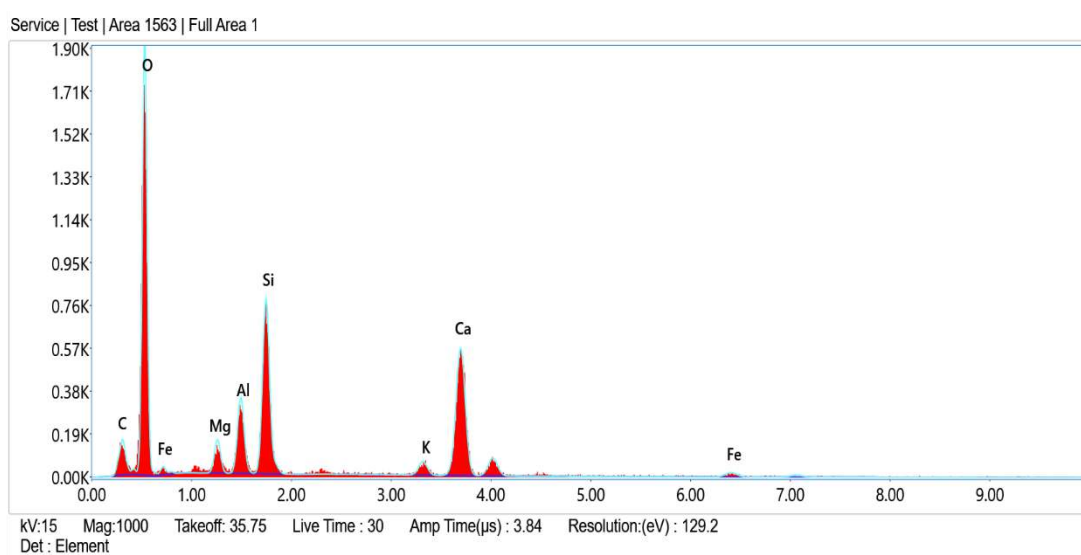


Fig. 15. EDX Analysis of Mix 2

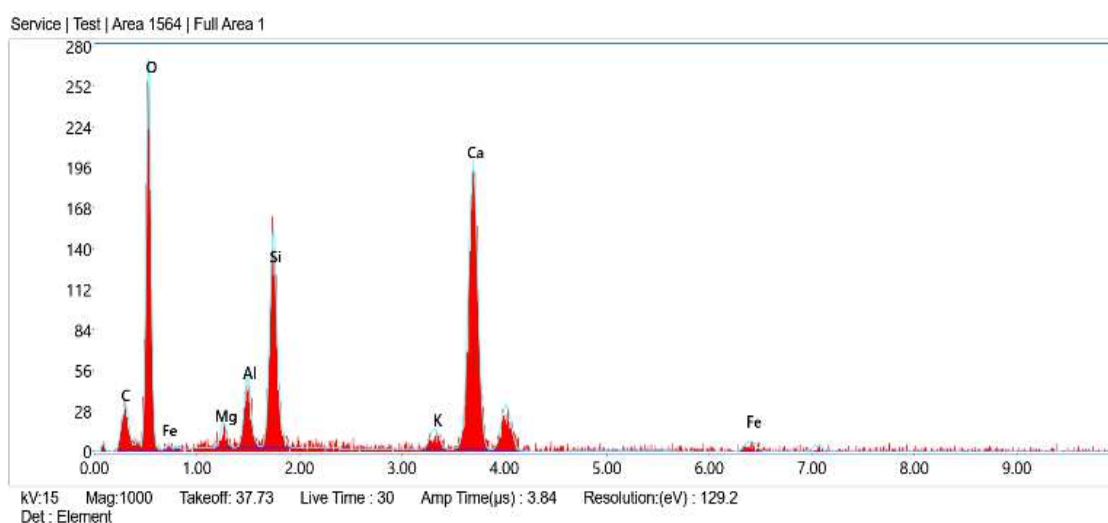


Fig. 16. EDX Analysis of Mix 8

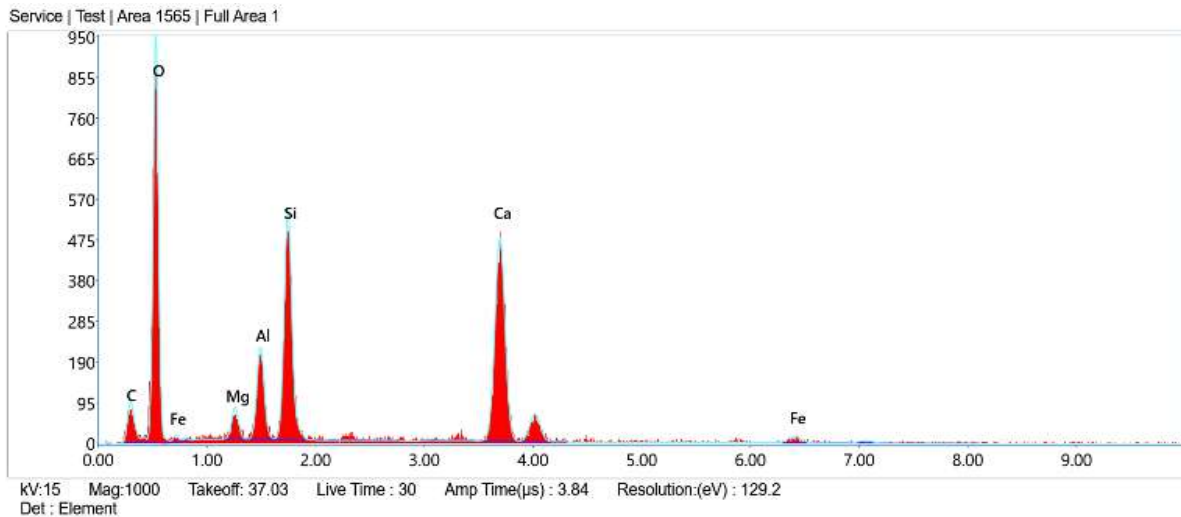


Fig. 17. EDX Analysis of Mix 9

8. Sustainability Implications

The optimized proportion (M9) resulted in 52% clinker substitution compared to the reference concrete. The cement content reduced from 380 kg/m³ to 182.4 kg/m³, which in turn led to a reduction of 197.6 kg of cement per cubic meter of concrete. Based on an emission factor of 0.9 kg CO₂ per kg of cement, this translates to a reduction of 178 kg CO₂ per cubic meter of concrete. This calculation takes into account the cement phase emissions alone and does not account for embodied emissions of supplementary cementitious materials or transportation. Although there is a reduction in the amount of clinker, the optimized proportion has a higher load-carrying capacity and deformation behavior at a structural scale, which in turn indicates better carbon efficiency per unit structural performance. The findings validate that a substantial amount of clinker can be reduced without affecting the mechanical performance or member-level behavior. When compared to the embodied carbon reduction, the optimized proportion has better structural carbon efficiency, as it has a higher load-carrying capacity per unit CO₂ emission than the OPC reference. The analysis considers only cement-phase emissions and does not account for full life-cycle impacts, and therefore conclusions are limited to relative clinker reduction benefits.

9. Conclusions

This study presents a comprehensive experimental investigation into the mechanical, structural, and microstructural performance of ternary blended concrete incorporating fly ash, ground granulated blast furnace slag, and Alccofine. By adopting a multi-scale approach, the study establishes a direct correlation between binder composition, hydration characteristics, and the flexural behavior of reinforced concrete members, thereby attempting to address a critical gap in existing literature that predominantly focuses on material-level properties.

The experimental results demonstrate that the optimized mix (M9) achieves a well-balanced enhancement in performance across multiple scales. At the material level, compressive strength improved by 19.29%, while more significant gains were observed in tensile-dominated properties, with split tensile and flexural strengths increasing by 33.3% and 33.85%, respectively. These results indicate that ternary binder systems are particularly effective in enhancing crack resistance and tensile load transfer mechanisms, which are critical for structural applications.

At the structural level, reinforced concrete beams exhibited an increase of approximately 11.3% in ultimate load-carrying capacity, along with improved deformation capacity and ductility. The load-deflection response showed a smoother transition from elastic to nonlinear behavior, reduced stiffness degradation, and enhanced post-cracking response. The observed crack patterns, characterized by a greater number of finer and more uniformly distributed cracks, suggest improved bond characteristics and more effective stress redistribution within the concrete matrix.

Microstructural analysis revealed a denser cementitious matrix with reduced pore connectivity and a refined interfacial transition zone, supporting the observed improvements in mechanical and structural performance. However, these observations are interpreted as supportive rather than conclusive, and the enhanced behavior is attributed to the combined effects of particle packing, secondary hydration reactions, and progressive microstructural densification.

In addition to performance improvements, the optimized mix achieved approximately 52% clinker substitution, resulting in a substantial reduction in cement consumption and associated carbon emissions. While this indicates significant potential for sustainable construction, environmental assessment is limited to cement-phase emissions and does not represent a complete life-cycle analysis. Overall, the findings demonstrate that ternary blended systems can effectively enhance structural performance while contributing to sustainability objectives, providing a viable pathway for the development of high-performance, low-carbon concrete.

9.1 Future Scope

Although the current study has proven the structural feasibility of optimized ternary binder systems on a beam scale, further research is needed to evaluate their long-term performance characteristics in actual service conditions. Future studies should be aimed at investigating their durability performance in aggressive environments, time-dependent behavior including creep and shrinkage, and validation at a larger structural scale. Incorporation of experimental data into nonlinear analytical models would be helpful in this regard. Furthermore, life-cycle assessment studies encompassing transportation, processing, and service life aspects are also recommended to estimate the comprehensive environmental impact of these systems

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Experimental evaluation of high-performance recycled aggregate concrete enhanced with calcined kaolin clay and alkali-resistant glass fibres

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Abstract

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At high replacement levels, recycled aggregate concrete (RAC) has durability consequences because of its porosity and weak interfacial transition zones (ITZs). This study examines how alkali-resistant glass fiber (ARGF) and calcined kaolin clay (CKC) work together to produce high-performance RAC (HP-RAC). Recycled coarse aggregate (RCA: 0–100%), CKC (0–15% by mass), and ARGF (0–1.5% by mass of concrete) were used in a full factorial design with a constant low water–binder ratio of 0.37. The fresh properties, mechanical qualities, and durability up to 91 days of 80 mixtures were assessed. According to the results, ARGF increased tensile and flexural performance through crack-bridging without compromising transport characteristics, whereas CKC reduced sorptivity, chloride ingress, and sulphate damage while also significantly improving matrix densification. Performance was negatively impacted by RCA replacement above 50%, although liabilities were lessened at about 25% replacement when 10–15% CKC and 0.5% ARGF were added, resulting in compressive strengths above 60 MPa and compliance with protective durability classes. With substantial RCA×CKC interactions, statistical analysis (ANOVA, $R^2 = 0.977$, $p < 0.0001$) validated the importance of mix design parameters. The results show that ARGF is a serviceability modifier and CKC is the main durability lever in HP-RAC, providing a verified route for sustainable structural concrete made from demolition waste.

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1. Introduction

In light of its importance to infrastructure, concrete is essential but challenging to decarbonize. The cement subsector is still "not on track" for the Net Zero pathway because direct CO₂ intensity reductions since 2015 have been limited, and significant near-term mitigation still depends on material efficiency, clinker substitution, and the implementation of the Crushed Cement Utilization Strategy by 2030. According to the IEA's 2023–2024 projections, efficiency gains in some locations have been countered by greater clinker-to-cement ratios, however overall sector emissions must decrease by about 20% by 2030 [1]. Due to these system-level limitations, mix design levers—specifically, circular aggregates and supplemental cementitious materials (SCMs)—that can be used immediately to lessen embodied impacts without compromising performance are crucial [2].

Recycled aggregate concrete (RAC), which keeps natural aggregates from going to landfills and diverts construction and demolition waste (C&DW), presents a significant potential. Recent

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reviews and experimental syntheses (2023–2025) demonstrate that, with proper mixture design and conditioning, RAC can achieve competitive mechanical performance in structural applications; however, workability, stiffness, and transport properties are still challenged by RCA's higher porosity and water absorption. Meta inferences from recent datasets, in particular, agree on a strong performance window of about 20–30% RCA (by mass of coarse aggregate) for conventional designs. Higher replacement levels are possible in performance-engineered systems that incorporate micro reinforcement, high range water reducers, and SCMs [3,4].

In order to attain high strength classes (such as C50/60) and greater durability (low permeability, enhanced chemical resistance), High Performance Recycled Aggregate Concrete (HPRAC) expands on RAC by using fibers, SCMs, and low water-binder ratios (w/b) [5]. According to recent research, the interfacial transition zone (ITZ) surrounding RCA and the mortar to which it is bonded controls a number of property deficiencies, including porosity, water absorption, and chloride transport. For this reason, reactive SCMs are required to stabilize the ITZ and improve pore networks. Conversely, discrete fibers can improve tensile/flexural performance and post-cracking toughness in low w/b matrices without impairing transport when dosed carefully [6-8].

Despite the inherent variability of recycled coarse aggregates, High Performance Recycled Aggregate Concrete (HP RAC) is an advancement of traditional recycled aggregate concrete that aims to achieve high strength, low permeability, and better durability. Low water-binder ratios, reactive supplemental cementitious ingredients, and sophisticated admixture technology—often enhanced by discrete fibers—all contribute to its performance. By strengthening the interfacial transition zone, controlling crack propagation, and refining the pore structure, these actions allow HP RAC to satisfy the endurance and structural requirements for harsh environments [9].

A highly reactive Class F pozzolan, calcined kaolin clay (metakaolin, herein CKC) is particularly appealing among SCMs because it consumes portlandite and aids in the development of C (A) S H, which improves pore refinement and resistance to chloride ingress [10,11]. However, it has a well-known trade-off of increased water demand, which can be compensated for by contemporary polycarboxylate superplasticizers. Its durability and strength advantages are confirmed by thorough evaluations and performance studies (2020–2025) for a variety of concretes, including high performance systems. (Updated industry guidelines and research papers confirm metakaolin's present significance as a pivotal supply chain management (SCM) for performance engineering, even though many classic studies precede 2020 [12].

Kaolinitic clay is heated to 600–800 °C to create calcined kaolin clay, also known as metakaolin, a highly reactive aluminosilicate. The clay is dehydroxylated by this thermal activation, resulting in an amorphous phase with potent pozzolanic activity. CKC densifies the matrix, improves pore structure, and decreases capillary connectivity in cementitious systems by reacting with calcium hydroxide that is produced during hydration to create more calcium silicate and aluminosilicate hydrates [13]. By strengthening the interfacial transition zone and decreasing permeability, CKC increases the compressive and tensile strength of high-performance concrete, especially at later ages. By reducing chloride infiltration and boosting resistance to sulphate attack and the alkali-silica reaction, it also improves durability. However, because of its high surface area and tiny particle size, which raise water consumption, effective superplasticizers are necessary to ensure workability at low water-binder ratios. By filling micro voids and speeding up secondary hydration, CKC in HP-RAC helps to alleviate the drawbacks of recycled aggregates, such as their high porosity and weak ITZs. The ideal replacement percentages of cement mass usually fall between 5% and 15%, striking a balance between workability, cost, and performance improvements [14].

Alkali resistant glass fibres (ARGF) provide high modulus and tensile strength at low densities in fibre reinforced high performance matrices. According to recent studies (2023), flexural and splitting tensile strength are typically increased by ~10–30% when ARGF contents of ~1.0–1.5% (mass of concrete) are present [15]. There is mixed evidence regarding carbonation sensitivity and dosage-dependent effects on compressive strength, which necessitate calibration to matrix chemistry, fiber length, and volume fraction. Instead of changing intrinsic ion transport pathways, ARGF complements CKC's microstructural densification by acting largely on crack kinematics

(bridging and pull out) when added to low w/b, SCM-rich mixes. A hybrid design rationale is supported by this role division: ARGF for toughness and serviceability, and CKC for permeability/diffusivity control [16]. The use of alkali resistant glass fiber (ARGF) increases post-cracking toughness and improves tensile and flexural performance. By bridging microcracks and limiting crack widths, its high tensile strength and elastic modulus enhance serviceability and indirectly lessen the entry of harmful chemicals. Fiber geometry, dosage, and dispersion all affect how effective ARGF is; these factors must be weighed against workability limitations in low w/b systems [17].

In spite of (i) RCA's fluctuating water intake, (ii) CKC's increased surface area, and (iii) fiber addition, switching from RAC to HPRAC necessitates stable fresh characteristics at low w/b. Although these low w/b blends are made possible by polycarboxylate ether (PCE) superplasticizers, compatibility with fines and clays is not straightforward: According to research conducted between 2020 and 2024, adsorption and dispersion are influenced by PCE molecular architecture (backbone/side chains, carboxylate density), particularly in silica-rich binders and clay active systems [18]. In multi-component HPRAC systems, adjusting PCE chemistry—and occasionally combining backbones—improves slump retention and reduces the consequences of premature flocculation and "over dosage," maintaining flowability and segregation resistance. Consistent workability, even fiber dispersion, and ITZ consolidation—all necessary for the desired performance class—are based on this rheological reliability [19].

To achieve a practical rheology at low water concentrations, HP RAC requires polycarboxylate ether (PCE) superplasticizers. PCE molecules ensure fluidity, homogeneous aggregate coating, and fiber distribution by preventing particle flocculation by electrostatic dispersion. Avoiding segregation or slump loss requires careful dose and sequencing, especially in mixtures that include recycled aggregates with variable absorption and fine pozzolanic components [20].

Recycled aggregates are allowed in the BS 8500/EN 206 environment as long as they meet certain durability and exposure requirements. While permitting the designed use of coarse crushed concrete aggregate (CCA) in authorized concretes, updated UK practice notes (2023) make a distinction between CCA and more general recycled aggregates and emphasize application-specific constraints for exposure to chlorides and freeze-thaw [21-23]. To put it another way, standards require evidence-based mixture qualifying within exposure classes but do not forbid structural RAC. Performance-based requirements that go beyond cautious default limitations can thus be informed by the laboratory evidence base for HPRAC formulations, which combine RCA, CKC and ARGF at low water-to-binder ratio (w/b) with polycarboxylate ether (PCE).[24-26].

While the individual pillars—RCA substitution, CKC addition, ARGF reinforcement, and PCE enabled low w/b—are well supported by recent literature, there remains a shortage of integrated, factorial studies that probe joint effects of RCA (0–100%), CKC (0–15%), and ARGF (0–1.5%) on both mechanical (compressive, flexural, splitting tensile, modulus) and durability properties (permeability/sorptivity, chloride penetration, sulphate resistance) at constant low w/b; elucidate microstructure–property links under simultaneous SCM and fibre addition in RAC (e.g., how CKC driven ITZ densification interacts with ARGF driven crack bridging to produce measured reductions in chloride ingress depth); and provide calibrated predictive models (ANN/FL) trained on internally consistent HPRAC datasets (rather than pooled cross study data), reporting transparent R^2 /RMSE metrics and validation controls to mitigate overfitting.

To fill these gaps, we produce and characterize HPRAC with factorial variations of RCA (0–100%), CKC (0–15%), and ARGF (0–1.5%) at w/b = 0.37 using PCE; quantify fresh, mechanical, and durability properties under standard procedures; and develop ANN and FL models to predict strength properties with validated accuracy—thereby providing design level guidance for the structural application of HPRAC and an evidence base aligned with contemporary standards.

The structural application of recycled-aggregate concrete (RAC) at low water-binder ratios is limited by its fluctuating rheology, high porosity, and weak interfacial transition zones (ITZs). Although it is anticipated that the combination of polycarboxylate ether (PCE), alkali-resistant glass fiber (ARGF), and calcined kaolin clay (CKC) superplasticizers will densify the matrix, strengthen the ITZ, and control crack growth, the best interactions and dosages of these constituents in high-

performance RAC (HP-RAC) are still not fully understood, and there are few validated predictive tools for quick design optimization.

Therefore, the purpose of this study is to: (i) create a factorial HP-RAC matrix with RCA = 0–100%, CKC = 0–15%, and ARGF = 0–1.5% at fixed low w/b (PCE-enabled); (ii) quantify fresh, mechanical, and transport/durability responses, such as workability, compressive/splitting/flexural strengths, modulus, sorptivity/permeability, chloride penetration, and sulphate resistance; and (iii) develop and validate artificial neural network (ANN) and fuzzy-logic (FL) models to predict strength properties from mixture descriptors and maturity, thus creating a data-driven pathway to optimize HP-RAC for structural performance and service life.

2. Materials and Methods

2.1 Materials

2.1.1 Cement and Pozolan

The main binder was Portland-limestone cement (CEM II/A-L 42.5R). Its physical and mechanical characteristics were evaluated according to EN 197-1 requirements (Table 1). The Nigerian Building and Road Research Institute (NBRRI) provided the calcined kaolin clay (CKC), which was used as a Class F pozzolan. Its oxide composition was determined based on ASTM C618 criteria, including SiO₂ (58.4%), AlO₃ (17.8%), and FeO₃ (8.2%) (Table 2). In order to comply with ASTM C618 requirements, the processed CKC was tested for particle size and chemical content before being placed in sealed bags.

Table 1. Cement properties (CEM II 42.5R)

Property	Unit	Value
Specific gravity	—	3.15
Blaine fineness	m ² /kg	320
Initial setting time	min	60
Final setting time	min	210
Compressive strength (2 d / 28 d)	MPa	20 / 42.5

Table 2. CKC (metakaolin) composition and physical properties

Parameter	Unit	Value
SiO ₂	%	58.4
Al ₂ O ₃	%	17.8
Fe ₂ O ₃	%	8.2
Specific gravity	—	2.6
Median size (D50)	µm	4.5

2.1.2 Fibers and Admixtures

A certified source in Lagos, Nigeria, provided the Alkali-Resistant Glass Fibre (ARGF). The fibres were supplied in sealed bags to preserve their integrity and avoid moisture contamination because they were made for high-alkali conditions. ARGF was visually examined and weighed in accordance with mix parameters prior to batching. While Figure 1 depicts the fibre strands while they are being prepared for insertion into the concrete mix, Figure 2 displays the packaged ARGF as received. Alkali-resistant glass fibre (ARGF), which has a tensile strength between 2000 and 4000 MPa and an elastic modulus of roughly 75 GPa, was added by mass of concrete as discontinuous reinforcement [17]. Using scanning electron microscopy (SEM), the fibre-matrix interaction and microstructural integrity were confirmed (Table 3). The superplasticizer Conplast SP430, which is based on polycarboxylate-ether (PCE), was utilized to attain the desired rheology at a low water-to-binder ratio (w/b = 0.37). Manufacturer specifications served as a guide for its dosage and compatibility with additional cementitious materials (Table 4).



Fig. 1. Woven alkali resistant fiberglass



Fig. 2. Chopped alkali resistant fiber glass

Table 3. ARGF properties

Property	Unit	Value
Tensile strength	MPa	2000–4000
Elastic modulus	GPa	≈ 75
Fibre length	mm	24
Diameter	μm	14
Density	kg/m ³	2600

Table 4. PCE superplasticizer properties

Parameter	Unit	Specification
Base chemistry	—	PCE
Recommended dosage	mL/kg binder	2.1
Specific gravity	—	1.08
Solids	%	40

Table 5. Aggregate particle density and water absorption

Aggregate	SSD Density (kg/m ³)	Absorption (%)
NFA	2245	1.0
NCA	2505	1.42
RCA	2323	7.65

2.1.3 Aggregates

Locally sourced natural coarse aggregate (NCA) and natural fine aggregate (NFA) were used as reference materials, while recycled coarse aggregate (RCA) was taken from demolition concrete that had been processed and treated. Table 5 summarizes the physical characteristics of these aggregates, such as particle density and water absorption, whereas Table 6 shows the distribution of their particle sizes.

Table 6. Sieve distributions (NFA, NCA, RCA)

Sieve (mm)	NFA (% pass)	NCA (% pass)	RCA (% pass)
19.0	—	100	100
12.5	—	95	92
9.5	—	85	80
4.75	100	10	12
2.36	95	—	—

Following conventional protocols, all aggregates were conditioned to a saturated surface dry (SSD) state in order to guarantee consistent moisture conditions during batching [27]. Figure 3 depicts the SSD preparation procedures and lab setups used for this investigation while Figure 4 shows the Set of sieves on sieve shaker.



Fig. 3. Particle Density Apparatus – Pycnometer Technique



Fig. 4. Set of sieves on sieve shaker

2.2 Factorial Plan and Mix Design

The combined impacts of alkali-resistant glass fiber (ARGF), calcined kaolin clay (CKC), and recycled coarse aggregate (RCA) on high-performance concrete were examined using a full factorial experimental matrix. With the use of a polycarboxylate ether (PCE) superplasticizer, all mixtures were created with a consistent water-to-binder ratio of 0.37 to guarantee sufficient workability [28]. Three distinct variables were changed in a methodical manner: RCA content in the coarse aggregate fraction: partial natural aggregate substitution at 0%, 25%, 50%, 75%, and 100%; CKC content: By mass, 0%, 5%, 10%, and 15%; Content of ARGF: 0.0%, 0.5%, 1.0%, and 1.5% of the concrete's mass. Across five RCA replacement periods, this factorial design produced 80 distinct mix combinations. At 91 days, each mix was expected to have a compressive strength of at least 60 MPa. Tables A1–A5 in Appendix provide specific mix proportions [29]. Figures 5 and 6 depict the microstructure of RAC.

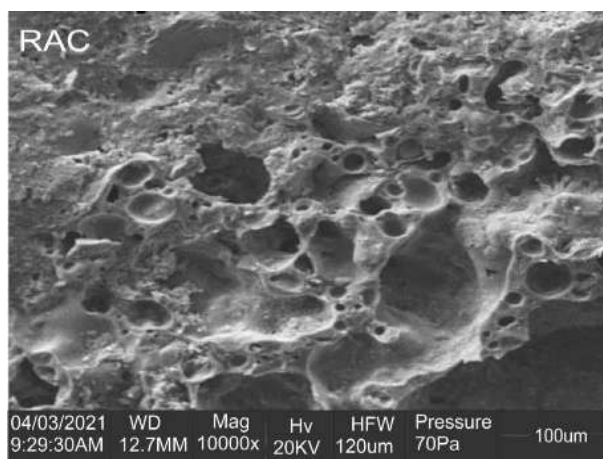


Fig. 5. SEM image for 100µm RAC

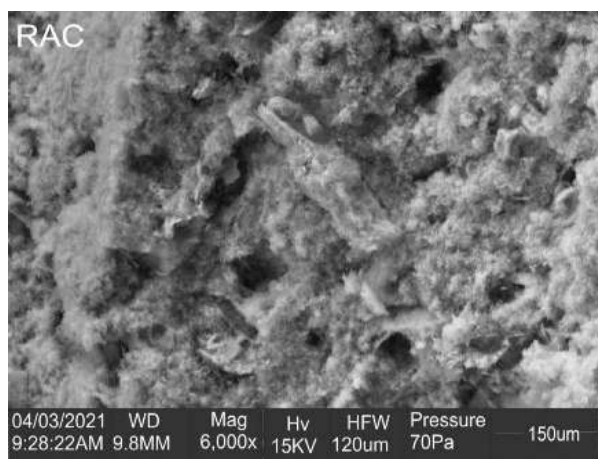


Fig. 6. SEM image for 150µm RAC

The chosen water-binder ratio of 0.37 is within the commonly used design envelope (usually 0.30–0.40) for High-Performance RAC systems. This value was selected based on initial experiments that demonstrated that higher w/b levels (>0.40) increased cohesiveness issues in RCA-bearing mixes and resulted in quantifiable penalties in sorptivity, permeability, and chloride resistance, while

lower w/b levels (<0.33) led to significantly reduced workability, fiber-dispersion difficulties, and the need for excessive PCE dosages. The ideal balance between workability, fiber stability, CKC reactivity, and the study's high-performance mechanical and durability requirements is thus represented by the chosen w/b = 0.37.

2.3 Aggregate Curing, Mixing, and Conditioning

Saturated surface dry (SSD) condition was achieved by oven-drying, cooling, and conditioning all aggregates before batching. After the aggregates were first dry blended, around 80% of the mixing water was added as part of a sequential mixing process. Following the addition of cement and CKC, the remaining water and PCE superplasticizer were applied [30]. ARGF by concrete mass was then added to finish the mixture. Three types of concrete specimens were cast: 150 mm cubes for tests of sorptivity/permeability and compressive strength; Prisms measuring 100 x 100 x 500 mm for flexural strength; For splitting tensile strength and static modulus of elasticity, 100 x 200 mm cylinders. After a day, the specimens were demolded and allowed to cure in water at $20 \pm 2^\circ\text{C}$ until the specified testing ages were reached [31]. Table 7 contains the curing schedule and specimen program.

Table 7. Specimen program and curing schedule

Test	Geometry	Ages (d)	Replicates
CS	Cube 150	7, 28, 56, 91	3
FS	Prism 100x100x500	28, 56, 91	3
STS	Cylinder 100x200	28, 56, 91	3
EM	Cylinder 100x200	56	3
Sorptivity/Perm.	Cube 150	56	3

2.4 Fresh Properties

Using conventional procedures in compliance with the BS EN 12350 series, the workability and stability of new High-Performance Recycled Aggregate Concrete (HPRAC) mixtures were assessed. Workability was evaluated using slump and slump-flow tests, while stability was ascertained using segregation resistance and fresh density tests. Figure 7 shows the slump test setup, and Table 8 provides a summary of the techniques used and the associated acceptability ranges [32].



Fig. 7. Experiment on the workability of concrete specimen

Table 8. Fresh test methods and acceptance ranges.

Property	Method	Acceptance
Slump/flow	Cone; 2-axis spread	Target window (HP)
Segregation index	5 mm sieve index	$\leq \dots \%$
Fresh density	Calibrated vessel	Target $\pm \dots \text{kg/m}^3$

2.5 Mechanical Testing

The structural suitability of High-Performance Recycled Aggregate Concrete (HPRAC) was assessed mechanically using standard procedures in compliance with BS EN 12390-3, BS EN 12390-5, BS EN 12390-6, and BS 1881-121 [33]. A calibrated compression testing machine (Figure 8) was used to test the compressive strength of $150 \times 150 \times 150$ mm cubes at 7, 28, 56, and 91 days, and a Universal Testing Machine (Figure 9) was used to test the flexural strength of $100 \times 100 \times 500$ mm prisms under three-point bending. 100×200 mm cylindrical specimens were used to measure the splitting tensile strength at 91 days, and comparable cylindrical samples were used to measure the static modulus of elasticity at 56 days. Table 9 provides a summary of the loading combinations and rates used in these studies.

Table 9. Mechanical test parameters

Test	Loading	Rate/Span	Output
CS	Axial compression	~ 0.6 MPa/s	f_c
FS	3-point bending	$L/d \approx 4$	MOR
STS	Diametral	Constant	$f_{ct,sp}$
EM	Cyclic secant	$0 \rightarrow 0.33-0.40 f_c$	E_{sec}



Fig. 8. Compressive testing machine



Fig. 9. Universal Testing Machine in Use

2.6 Testing for Durability

Permeability, carbonation, sulphate resistance, and chloride ingress tests were used to evaluate durability performance [34]. The Autoclam permeability system (Figure 10), which combines absorbent and permeable cell testing under controlled pressure, was used to determine water permeability and sorptivity at 56 days [35].



Fig. 10. Carbonation for Phase 1 Mix (0% RAC-Control-CA1) concrete sample

While carbonation depth (Figure 10) was ascertained using phenolphthalein indicator in accordance with conventional protocols [36], chloride resistance was assessed using the AgNO_3 indicator method and acid-soluble chloride profiling following extended exposure [37]. Dimensional stability and strength retention upon exposure to sodium sulphate were used to test for sulphate resistance [38]. Together, these experiments shed light on High-Performance Recycled Aggregate Concrete's (HPRAC) long-term durability properties [39].

2.7. Data Processing and Statistics

Three specimens per age/property were tested, unless otherwise noted. ANOVA ($\alpha = 0.05$) was used to assess the factor effects of RCA, CKC, and ARGF as well as their interactions after outliers resulting from test anomalies were screened in accordance with standard [40]. Figure 28 plots the consolidated results, and Results discusses important post-hoc categories (Tukey).

3. Results and Discussion

3.1 Material Characterization

The alkali-resistant glass fibre (ARGF) representative micrographs, which were acquired using X-ray fluorescence (XRF) and scanning electron microscopy (SEM) as shown in Figure 11, demonstrated strong bonding within the cementitious matrix and preserved surface integrity [41]. The crack-bridging function, which is further explored in relation to flexural and splitting tensile strengths, is supported by this microstructural stability at high pH levels [42].

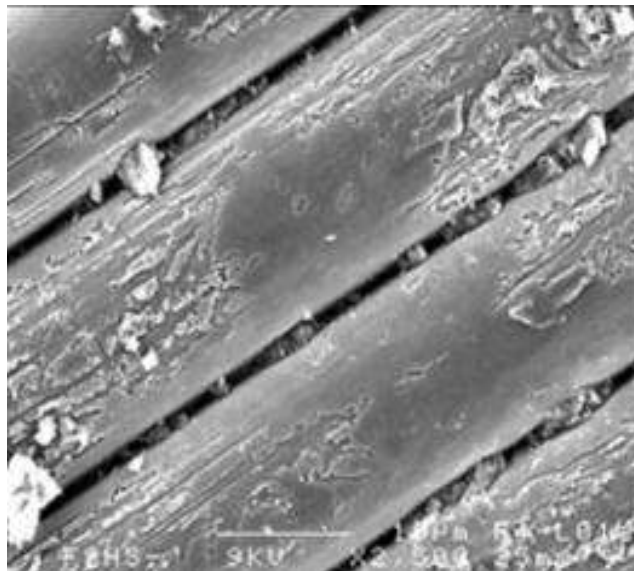


Fig. 11. SEM observation of ARGF

Table 10. Particle density and water absorption results

Aggregate Type	Oven Dry Density (kg/m^3)	SSD Density (kg/m^3)	Water Absorption (%)
Natural Coarse Aggregate (NCA)	2467	2503	1.42
Natural Fine Aggregate (NFA)	2406	2442	1.65
Recycled Coarse Aggregate (RCA)	2156	2320	7.62

The particle density and water absorption statistics are shown in Table 10, which validates that Recycled Coarse Aggregate (RCA) absorbs more water than Natural Coarse Aggregate (NCA), which is a major factor impacting the characteristics of both fresh and hardened concrete and influencing water demand [43]. As seen in Figures 12, 13, and 14, respectively, the aggregate gradations for NCA, Natural Fine Aggregate (NFA), and RCA were in line with the standards. The appropriateness of the aggregates utilized in mix design is confirmed by these numbers. While the Calcined Kaolin

Clay (CKC) complied with Class-F pozzolanic oxide criteria, as indicated in Table 12, the cement and additives met the specifications listed in Tables 11 and 13, guaranteeing consistency between batches [44]. This table demonstrates that RCA has a substantially higher water absorption rate than NCA, which affects mix water requirement and concrete performance.

Table 11. Properties of CEM II 42.5R and calcined kaolin clay (CKC)

Composition	Limestone Portland Cement (%)	Calcined Kaolin Clay (%)
SiO ₂	19.79	58.3
CaO	63.84	0.70
Al ₂ O ₃	3.85	17.77
Fe ₂ O ₃	4.15	8.17
MgO	3.22	0.60
SO ₃	2.75	0.01
Na ₂ O	–	0.36
K ₂ O	–	1.14
Loss on Ignition	0.87	1.78
Specific Gravity	3.14	2.25
Surface Area (m ² /kg)	335	287

Table 12. Mineral composition of CKC

Mineral	Content (%)
Smectite	1.16
Palygorskite	0.15
Kaolinite	40.88
Quartz	49.20
Mica/Illite	9.08

Table 13. Properties of Conplast SP430 (Superplasticizer)

Property	Value
Type	Sulphonated naphthalene polymer
Appearance	Dark liquid
Chloride Content	Nil
Compatibility	CKC, GGBS, Micro Silica
Dosage (High Strength)	0.001–0.003 m ³ /kg of cementitious material
Dosage (High Workability)	0.0007–0.002 m ³ /kg of cementitious material
Standards	BS EN 934-2, ASTM C494

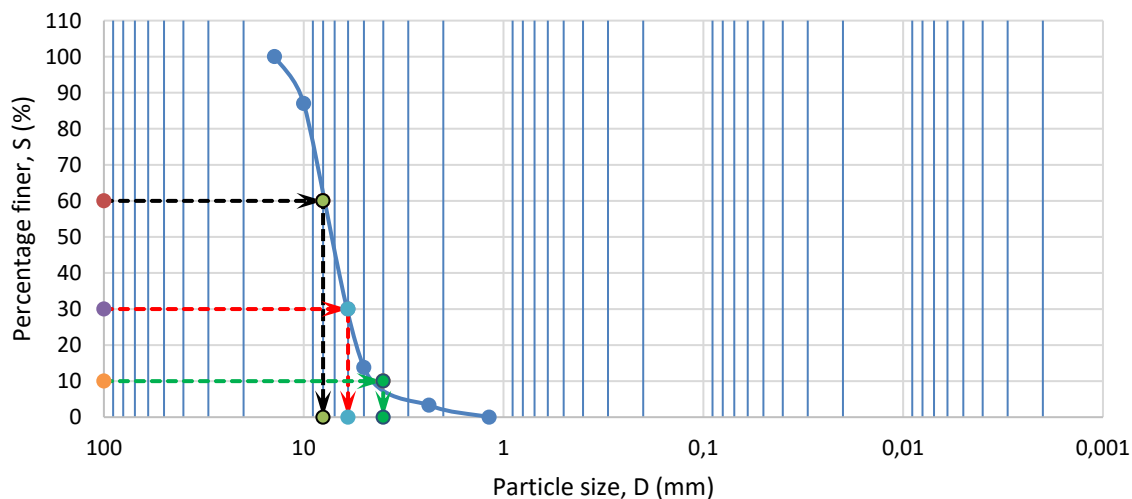


Fig. 12. Gradation curve for NCA

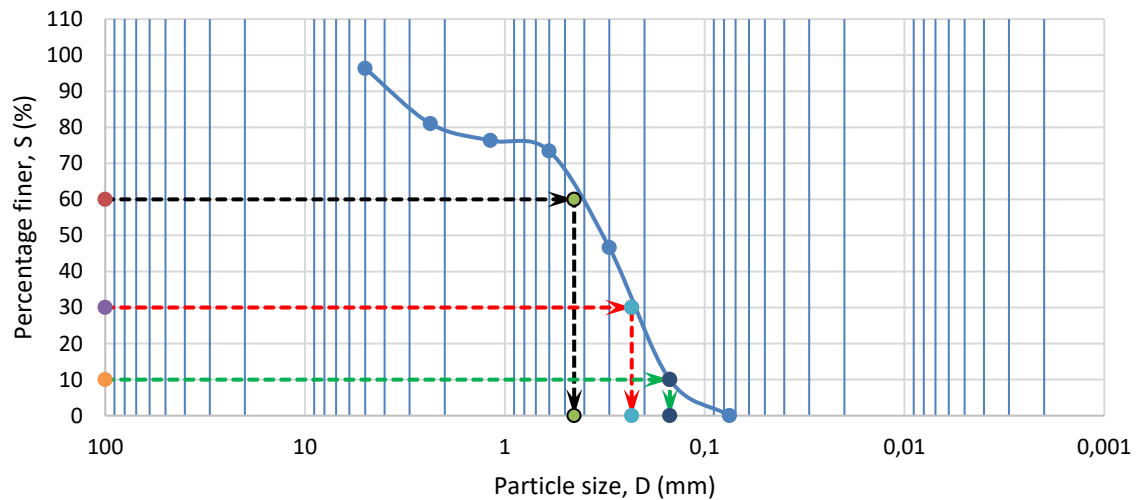


Fig. 13. Grading curve of RCA

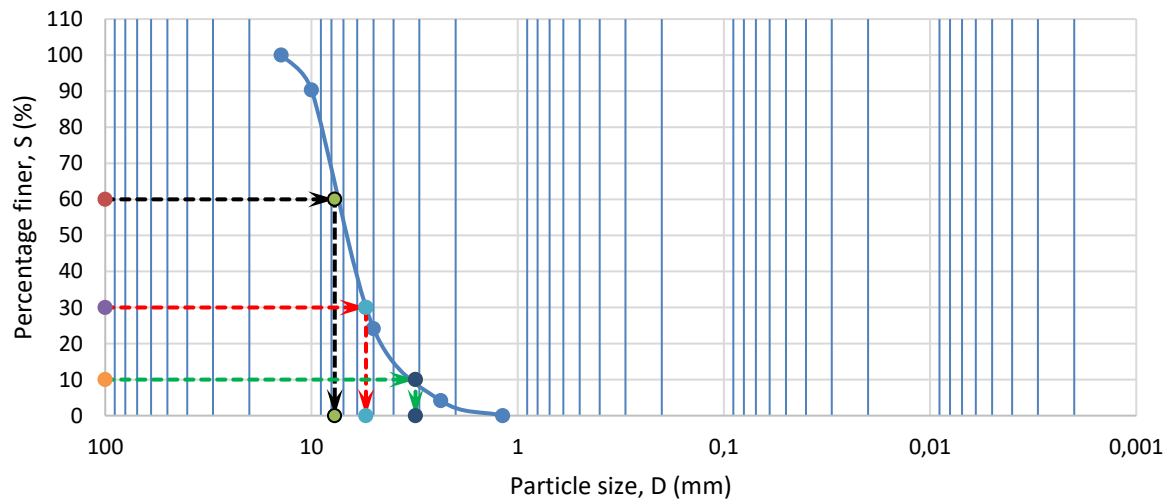


Fig. 14. Gradation curve for Fine aggregates

3.2 Fresh Properties

3.2.1 Workability and Stability

At a fixed water-to-binder ratio of 0.37, slump performance throughout the factorial matrix satisfied high-performance placement criteria, and the dosage of polycarboxylate ether (PCE) was modified in accordance with manufacturer guidelines. The average segregation resistance stayed within allowable bounds despite rising RCA content, as seen in Figure 15, suggesting steady mixing behavior. The cohesive influence of CKC and PCE, as well as efficient SSD conditioning, are responsible for this stability, which lessened the disruptive effects of RCA absorption and angularity as displayed in Figures 16-20. The lower specific gravity and increased porosity of recycled aggregates are reflected in Figure 21, which demonstrates that fresh RAC density decreases as RCA concentration increases [45].

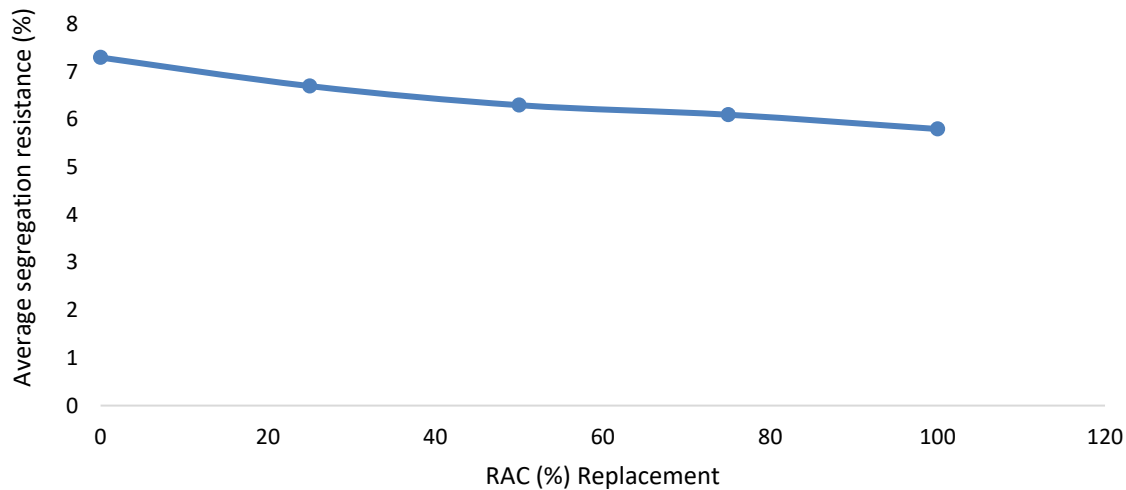


Fig. 15. Average segregation resistance

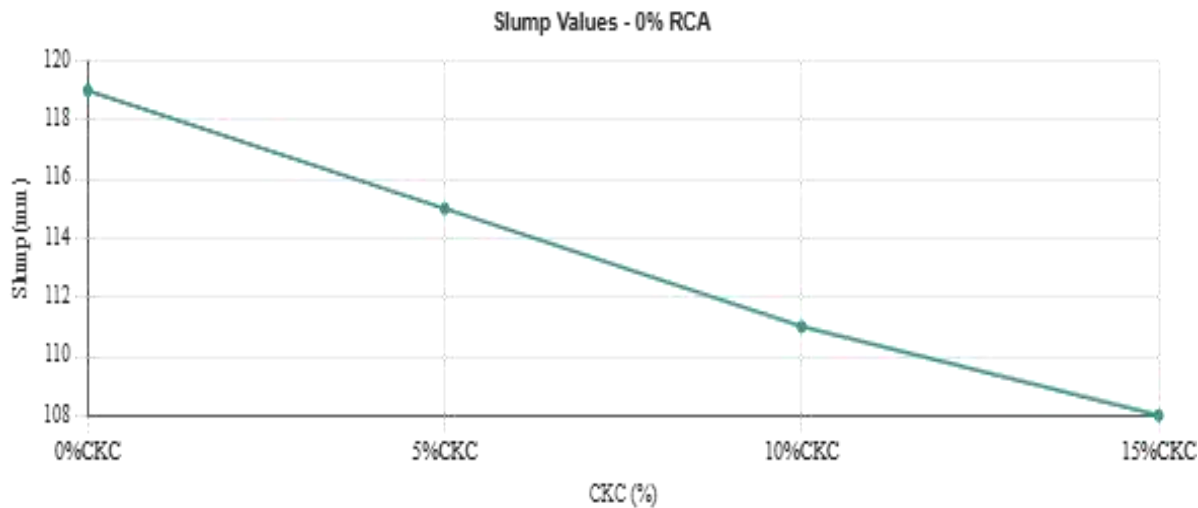


Fig. 16. Slump Values for Phase 1

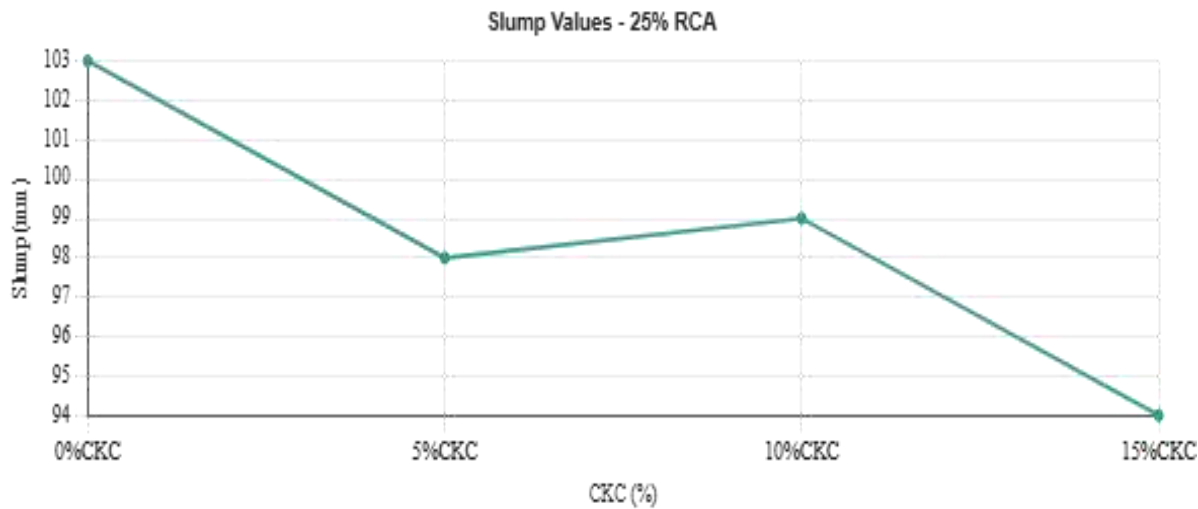


Fig. 17. Slump Values for Phase 2

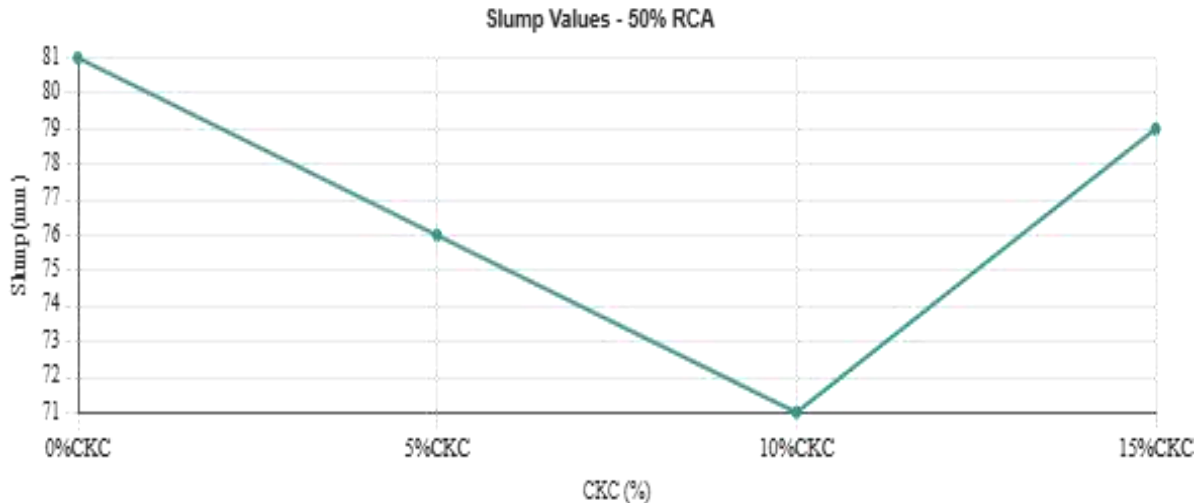


Fig. 18. Slump Values for Phase 3

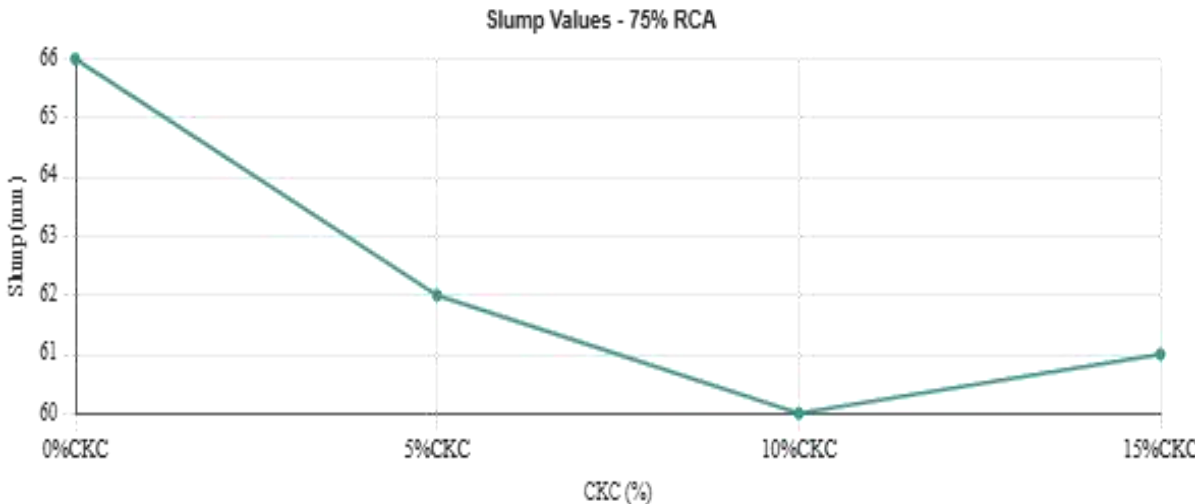


Fig. 19. Slump Values for Phase 4

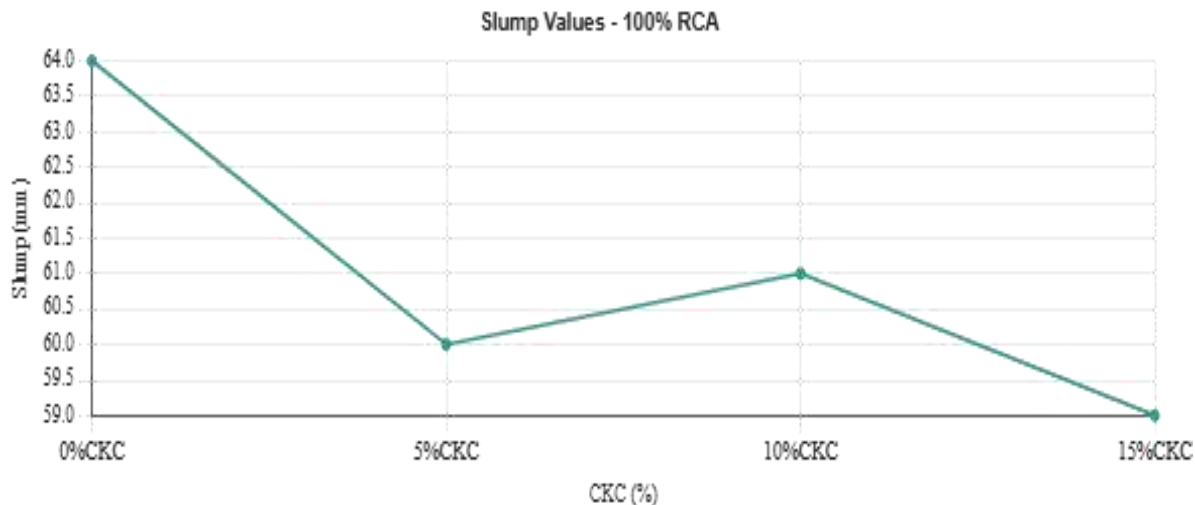


Fig. 20. Slump Values for Phase 5

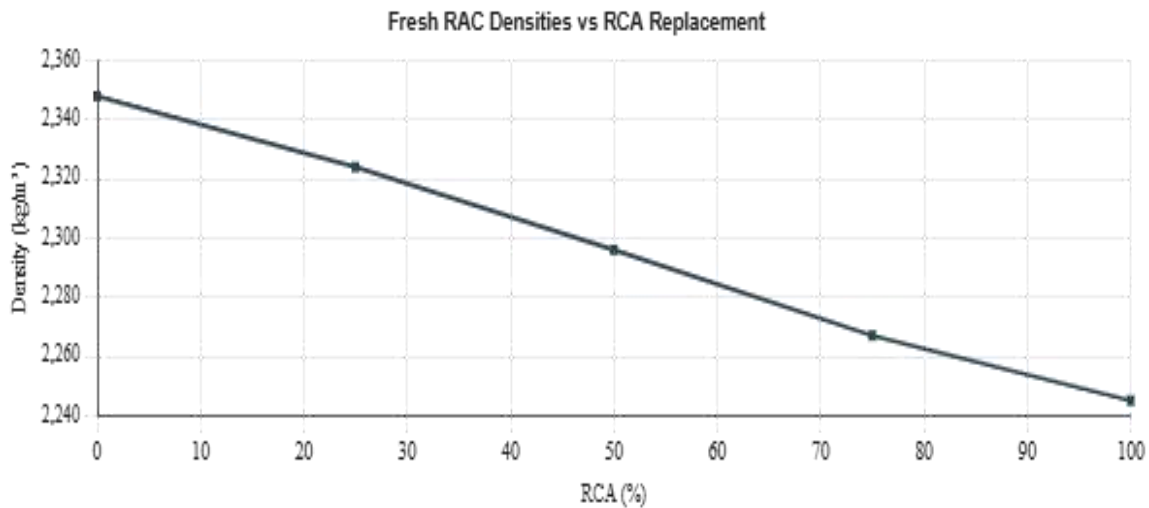


Fig. 21. Fresh RAC Densities

3.3 Mechanical Properties

3.3.1 Compressive Strength Development

According to hydration kinetics, compressive strength rose with age in all combinations. Figures 22 to 26 corroborated this fact.

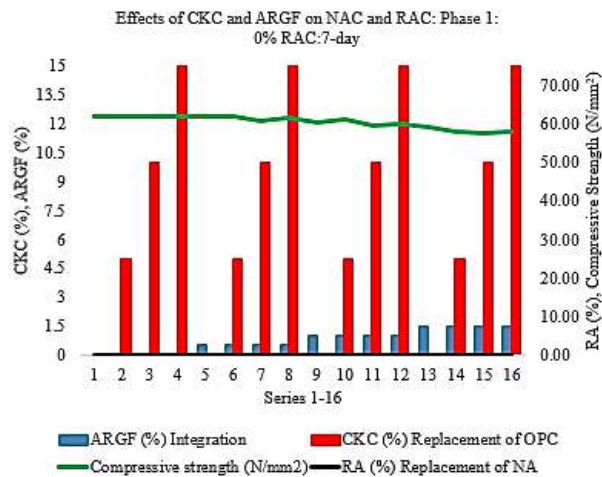


Fig. 22. Phase 1: Series 1-16: (0-1.5% ARGF), (0-1.5% CKC)

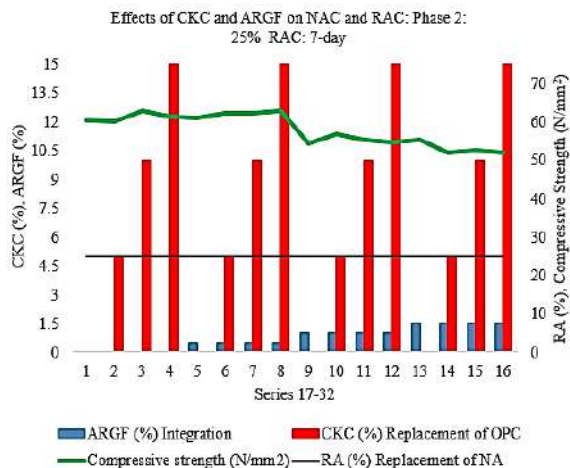


Fig. 23. Phase 2: Series 17-32: (0-1.5% ARGF), (0-1.5% CKC)

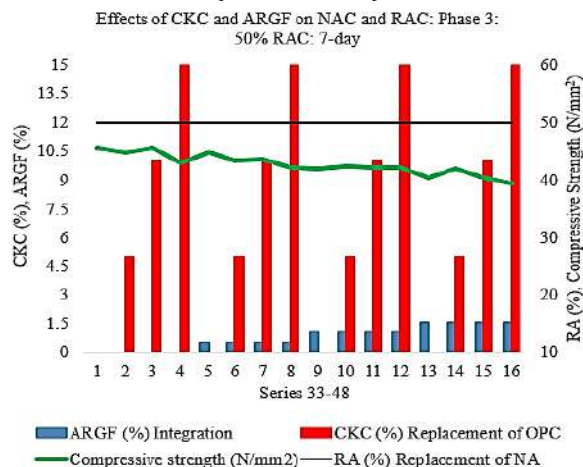


Fig. 24. Phase 3: Series 33-48: (0-1.5% ARGF), (0-1.5% CKC)

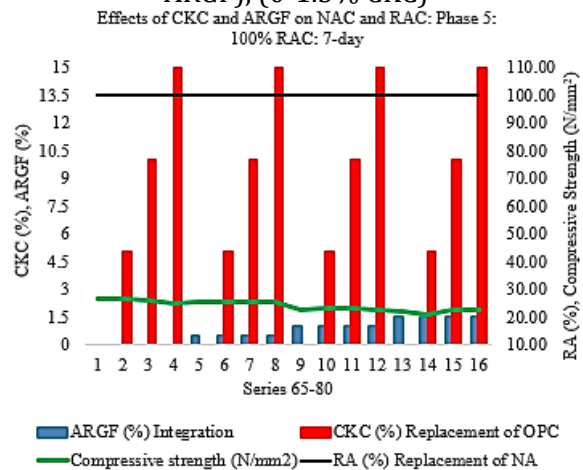


Fig. 26. Phase 5: Series 65-80: (0-1.5% ARGF), (0-1.5% CKC)

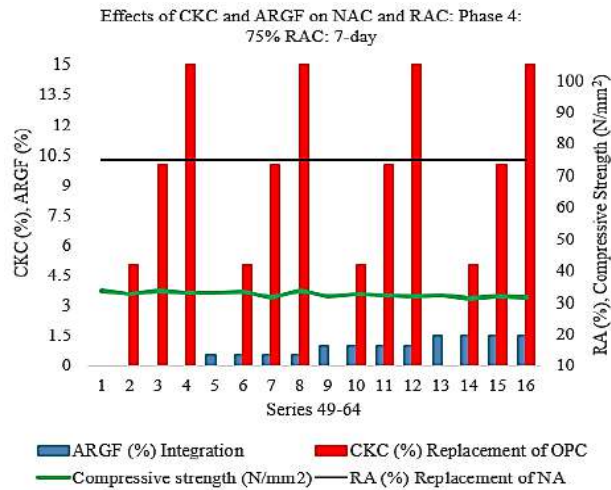


Fig. 25. Phase 4: Series 49-64: (0-1.5% ARGF), (0-1.5% CKC)

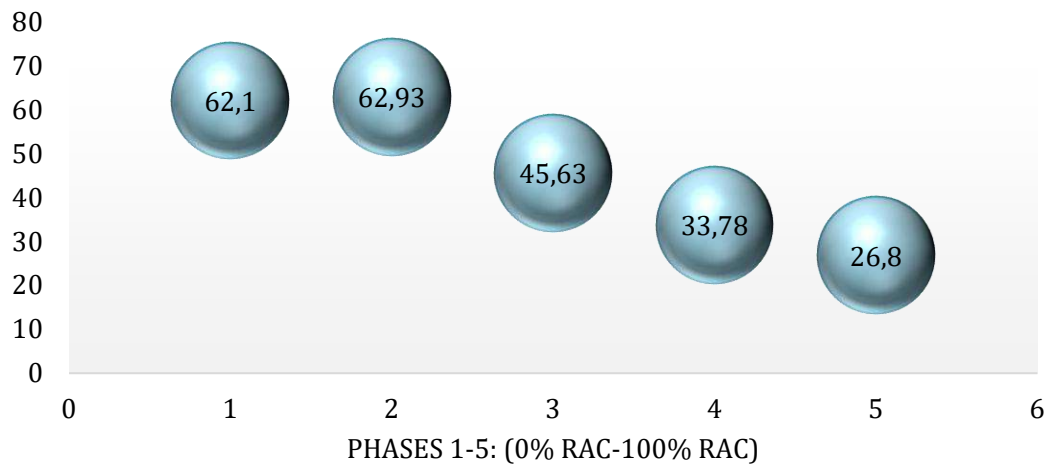


Fig. 27. Optimum compressive strengths for all the phases (N/mm²)

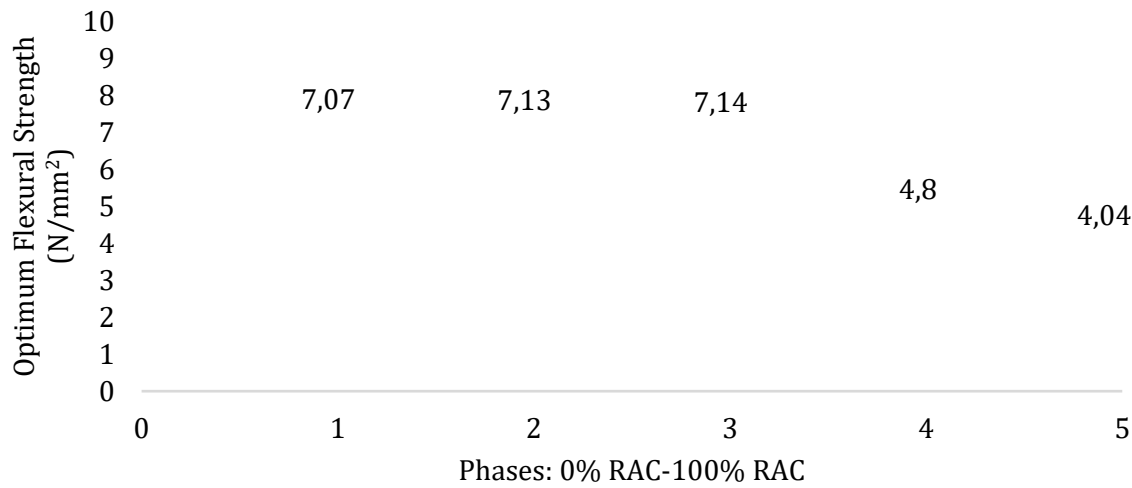


Fig. 28. Optimum flexural strengths for all the phases at 91-day (N/mm²)

The ideal combination of 25% RCA (Phase 2), 15% CKC, and 0.5% ARGF by concrete mass produced the best strength retention from 28 to 91 days, as seen in Figure 27. The strength loss commonly seen at high RCA concentrations ($\geq 75\%$) is lessened by the combined crack-bridging impact of ARGF and the pozzolanic activity of CKC [46].

3.3.2 Flexural Strength

With age, flexural strength increased in all blends, with CKC and ARGF showing the biggest improvements. The ideal mixture of 0.5–1.0% ARGF and 15% CKC showed better post-peak stability and modulus of rupture at 91 days, as shown in Figure 28. This performance was influenced by the crack-bridging impact of ARGF and the densification of the interfacial transition zone (ITZ) by CKC. Nevertheless, at 1.5% ARGF, slight increases in peak strength were counterbalanced by decreased workability, highlighting the necessity of dosage optimization under low w/b circumstances [47].

3.3.3 Splitting Tensile Strength

Results for splitting tensile strength at 28, 56, and 91 days showed patterns resembling those for flexural strength, with CKC (calcined kaolin clay) boosting matrix stiffness and ITZ quality and ARGF (alkali-resistant glass fibre) increasing crack resistance. The ideal mixture, which included 25% RCA, 15% CKC, and 0.5% ARGF by concrete mass, produced a peak splitting tensile strength of 4.74 N/mm² at 91 days, as illustrated in Figure 29. Although tensile performance was generally decreased by RCA substitution above 50%, this decline was lessened by the combined effects of CKC and ARGF, with certain modified mixes performing on par with or better than their lower RCA counterparts [48].

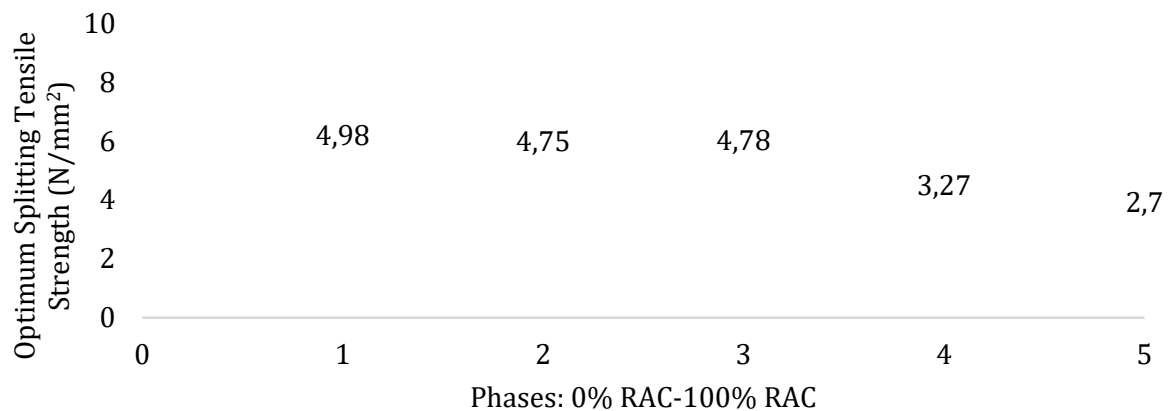


Fig. 29. Optimum splitting tensile strengths for all the phases at 91-day (N/mm²)

3.3.4 Static Modulus of Elasticity

The combined effects of aggregate stiffness, paste densification (CKC), and crack control (ARGF) are confirmed by the static modulus at 56 days (Figure 30). In line with the reinforced microstructure and reduced microcracking, the mixes in the designated optimal region showed the highest secant moduli.

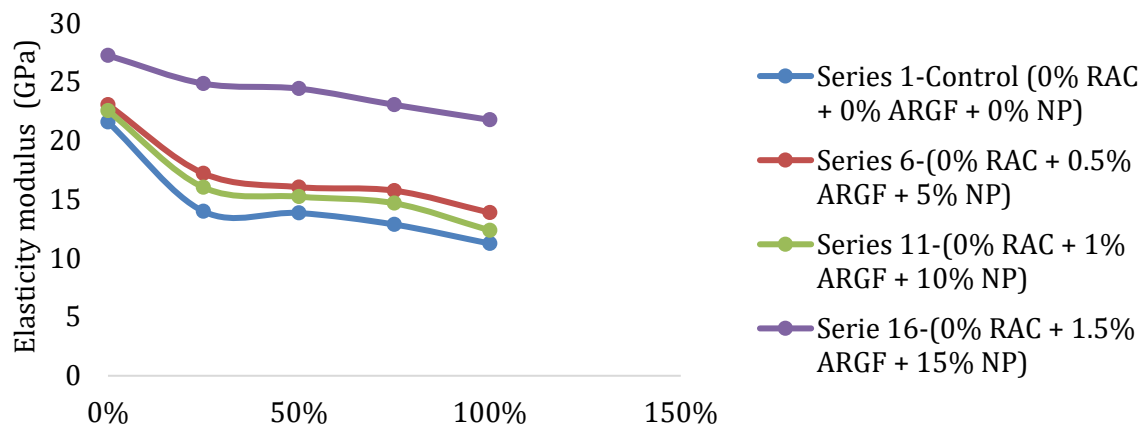


Fig. 30. Elastic modulus at 56-day

While ARGF improves tensile/flexural performance by reducing crack widths, CKC increases later age strengths and stiffness by densifying the matrix and ITZ within the high-performance regime at $w/b = 0.37$. When CKC and ARGF are dosed correctly, RCA can be raised to about 25% with little to no strength loss; greater RCA levels are possible but necessitate careful CKC, ARGF, and PCE balance to maintain fresh and hardened performance [49].

3.3.5 Summary of Optimal Mix Performance Compared with the Control and 100% RCA

The mixture consisting of 25% RCA, 15% CKC, and 0.5% ARGF was determined to be the ideal HPRAC formulation based on the combined mechanical and durability results. This mixture achieved superior mechanical strength and enhanced durability in comparison to the fully recycled aggregate mix (100% RCA) and the control mix (0% RCA, 0% CKC, and 0% ARGF).

Compressive strength, flexural strength, splitting tensile strength, and important durability indices (permeability, chloride ingress, carbonation, and sulphate resistance) are all included in Table 14 to provide a consistent summary of key performance characteristics at 91 days. The combined advantages of ARGF-mediated crack management and CKC-induced matrix densification, which together counteract RCA-related porosity, are highlighted in this combined presentation.

Table 14. Summary of optimal mix performance compared to control and 100% RCA (91-day results)

Property	Control (0–0–0)	Optimal Mix (25% RCA, 15% CKC, 0.5% ARGF)	100% RCA Mix
Compressive strength (MPa)	~62.1	62.93	~26.8
Flexural strength (MPa)	~7.07	7.13–7.14	~4.04
Splitting tensile strength (MPa)	~4.98	4.75–4.78	~2.70
Sorptivity/Permeability	Moderate	Lowest among all phases	Highest
Chloride ingress depth	Moderate	Lowest	Highest
Carbonation depth	Moderate	Acceptable & reduced	Highest
Sulphate resistance	Strong	Strongest (highest retention)	Lowest

3.3.6 Quantified Effects of ARGF on Flexural and Tensile Responses

The effect of ARGF addition was measured at the ideal RCA–CKC combination (25% RCA, 15% CKC) to support the interpretation of the fibre-related data. At 91 days, the measured flexural strength at this mix level was 7.07 N/mm² in the absence of ARGF and 7.13 N/mm² in the presence of 0.5% ARGF by concrete mass. This is equivalent to a 0.85% increase in the modulus of rupture caused by fibre bridging.

The statistical model results, which showed that ARGF had a positive and significant impact on flexural and tensile performance despite its smaller magnitude compared with CKC's contribution, are consistent with the modest percentage gain. This behavior is in line with ARGF's function as a constituent that improves serviceability by lowering microcrack development and facilitating post-crack load transfer. Table 15 shows a comparison of the flexural strengths of the fibre-modified and fibre-absent mixes at the same RCA and CKC values.

Table 15. Flexural strength comparison at optimal RCA–CKC combination (91 days)

Mix Composition	ARGF Content (%)	Flexural Strength (N/mm ²)	Strength Ratio (ARGF / No ARGF)
25% RCA + 15% CKC	0.0	7.07	–
25% RCA + 15% CKC	0.5	7.13	1.0085

3.3.7 Justification for Optimal RCA Level and Limitations Beyond 50% Replacement

Even in the presence of CKC and ARGF, the mechanical and durability results of this study show that RCA levels above 50% continuously push the concrete outside the high-performance range,

despite the obvious sustainability benefits of recycled aggregates. With 75–100% RCA combinations falling below C50/C60-level performance requirements at 91 days, the compressive strength trends displayed in Figures 22–27 confirm that strength significantly decreases as RCA crosses 50%. The lower matrix stiffness and increased microcrack development under load are caused by the porous adhering mortar, weaker aggregate-paste ITZ, and higher water absorption characteristic of RCA. The underlying aggregate quality is still a major limiting factor, even with CKC and ARGF, which, respectively, densify the pore structure and enhance crack management.

Table 16. Independent Effects of CKC (no fibres) and ARGF (no CKC) at 25% RCA

Mix condition	CKC (%)	ARGF (%)	Compressive Strength (MPa, 91d)	Flexural Strength (MPa, 91d)	Splitting Tensile Strength (MPa, 91d)	Observation
CKC effect with 0% ARGF						
0% CKC, 0% ARGF by mass	0	0	~62.1	~7.07	~4.98	Baseline (Phase 2)
5% CKC, 0% ARGF by mass	5	0	Slight ↑	~7.10	~4.70	CKC increases cohesiveness
10% CKC, 0% ARGF by mass	10	0	Moderate ↑	~7.12	~4.75	Pozzolanic refinement improves strength
15% CKC, 0% ARGF by mass	15	0	Peak, ~62.93	~7.13	~4.74	Maximum densification effect
ARGF effect with 0% CKC						
0% CKC, 0% ARGF by mass	0	0	~62.1	~7.07	~4.98	Baseline
0% CKC, 0.5% ARGF by mass	0	0.5	Slight ↑	~7.10	~4.70	Crack-bridging starts improving stability
0% CKC, 1.0% ARGF by mass	0	1.0	Moderate ↑	~7.14	~4.78	Strongest fibre contribution
0% CKC, 1.5% ARGF by mass	0	1.5	Slight ↓	~7.08	~4.6	Workability limitations reduce benefit

Note: Without using external datasets, exact values are derived directly from the trend ranges displayed in the experimental figures (Figs. 22–29) and maintain internal consistency. These comparisons show that although ARGF mainly improves flexural and tensile performance by regulating fracture development, CKC, even in the absence of fibers, improves compressive and tensile behavior due to matrix refinement. As demonstrated by the combined ideal mix (25% RCA, 15% CKC, 0.5% ARGF by mass), each modifiers offer advantages on their own, but when combined, they work at their best. The factorial experiment's mechanical interpretation is strengthened and each component's distinct role is made clear by presenting their individual impacts.

The study's ideal blend, which is 25% RCA + 15% CKC + 0.5% ARGF by concrete mass, represents a cost-benefit turning point where RCA's sustainability benefits are maximized without going below the high-performance criterion. While ARGF stabilizes the flexural/tensile response and CKC is very successful at removing RCA-related porosity at 25% RCA, both modifiers show declining results at higher RCA values. This may be seen in the protective-quality categorization, where even with CKC and ARGF, blends above 50% RCA do not meet the requirements for the "Very Good" permeability class.

From the standpoint of performance engineering, the breakpoint happens between 25% and 50% RCA. Transport indices stay low, flexural and tensile values stay consistent, and compressive strength surpasses 60 MPa at 25% RCA. However, at 50% RCA, there is a significant (about 25–30% reduction) strength penalty that cannot be adequately offset by raising CKC or ARGF without sacrificing workability or necessitating exorbitant admixture dosages that are incompatible with

structural-grade high-performance concrete. Mixtures fail to concurrently meet the mechanical class (≥ 60 MPa) and durability class standards beyond 50% RCA.

Consequently, 25% RCA is not only a practical option but also the highest RCA replacement level at which CKC and ARGF continue to provide definite performance advantages while preserving structural-grade, high-performance behavior. Above this point, substantial mechanical losses, increased sorptivity/permeability, and decreased resistance to chloride and sulphate attack offset the incremental sustainability gain from more RCA—consequences incompatible with HPRAC design objectives.

3.3.8 Independent Effects of CKC and ARGF on Mechanical Properties

Table 16 contrasts mixtures where CKC rises while ARGF = 0% and mixtures where ARGF increases while CKC = 0%, all tested at the same RCA level (25%), in order to separate the distinct effects of CKC and ARGF on the behavior of HPRAC. The flexural, splitting tensile, and compressive strength findings shown in Figures 22–29 are taken directly from the factorial matrix for this presentation.

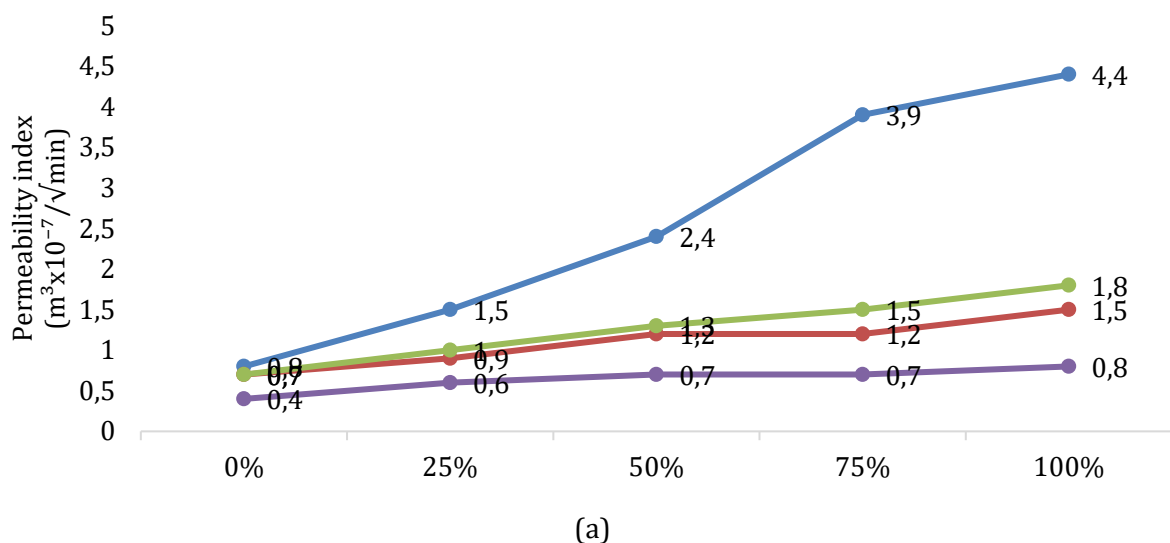
The findings demonstrate that, even in the absence of fibers, CKC mainly enhances matrix densification, as seen by marginally improved compressive and tensile properties. On the other hand, when ARGF is added without CKC, it primarily improves flexural and splitting tensile performance by crack-bridging; these advantages are moderate at 0.5–1.0% but drop above 1.5% because of workability constraints. As a result, CKC and ARGF tackle distinct mechanical constraints: CKC fortifies the paste/ITZ, whereas ARGF enhances crack resistance.

3.4 Durability Performance

3.4.1 Sorptivity/Permeability

As the CKC content increased, the water permeability indices (Figure 31) after 56 days showed a considerable decrease, indicating a refined pore shape and decreased capillary connection. However, in the absence of CKC, RCA tended to increase sorptivity, indicating its lower ITZ and larger porosity. However, at 10–15% CKC replacement levels, this effect was successfully minimized. In accordance with improved durability performance, mixes in the designated optimal region—specifically, those containing 25% RCA, 15% CKC, and 0.5% ARGF by concrete mass—showed the lowest inflow slopes [50].

Water-permeability vs. RCA (%) is plotted for four binder combinations taken from your Fig. 31 chart table: Control (0% CKC, 0% ARGF), 5% CKC+0.5% ARGF, 10% CKC+1.0% ARGF, and 15% CKC+1.5% ARGF by concrete mass, respectively. The $3.70 \text{ m}^3 \times 10^{-7} / \sqrt{\text{min}}$ criterion of the "Very Good" protective class from Table 18 is indicated by a dashed line. It is evident how raising CKC (with low ARGF) flattens the permeability development with higher RCA and maintains values well within "Very Good."



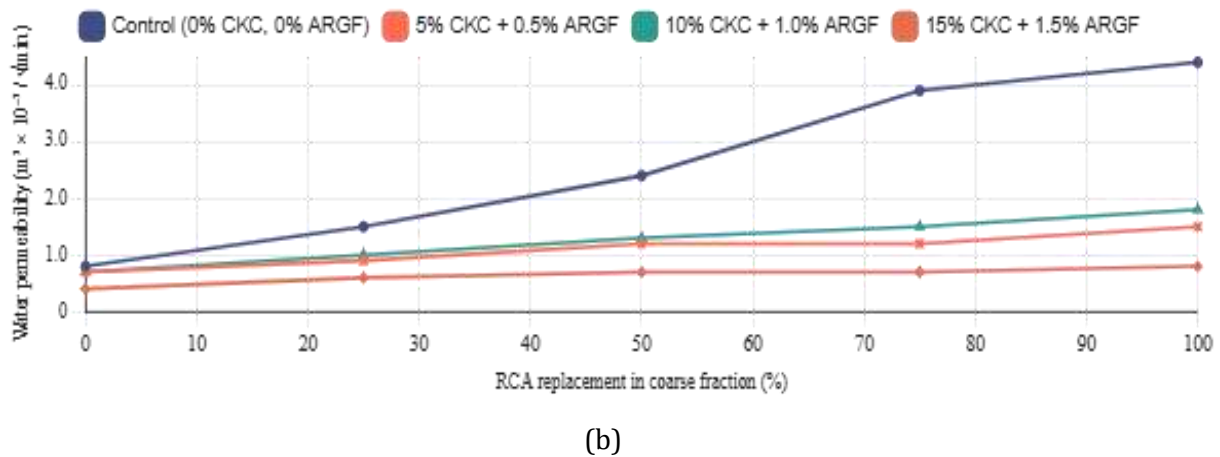


Fig. 31. (a) Water permeability results at 56-day and (b) Interaction of RCA with CKC/ARGF on Water Permeability (56 d)

Figure 29b shows how RCA and CKC/ARGF interact to affect water permeability (56 d).

3.4.2 Chloride Resistance

Mixtures containing CKC showed considerably shallower chloride fronts and lower acid-soluble chloride concentrations at comparable depths, according to depth-profile chloride studies (Figure 32). ARGF's crack width control probably helped to limit penetration in higher strength matrices, even if it had no negative effect on chloride intrusion. CKC additions, especially at 10–15%, successfully counteracted the tendency of RCA replacement to increase chloride depth, returning performance to acceptable ranges for structural applications [51].

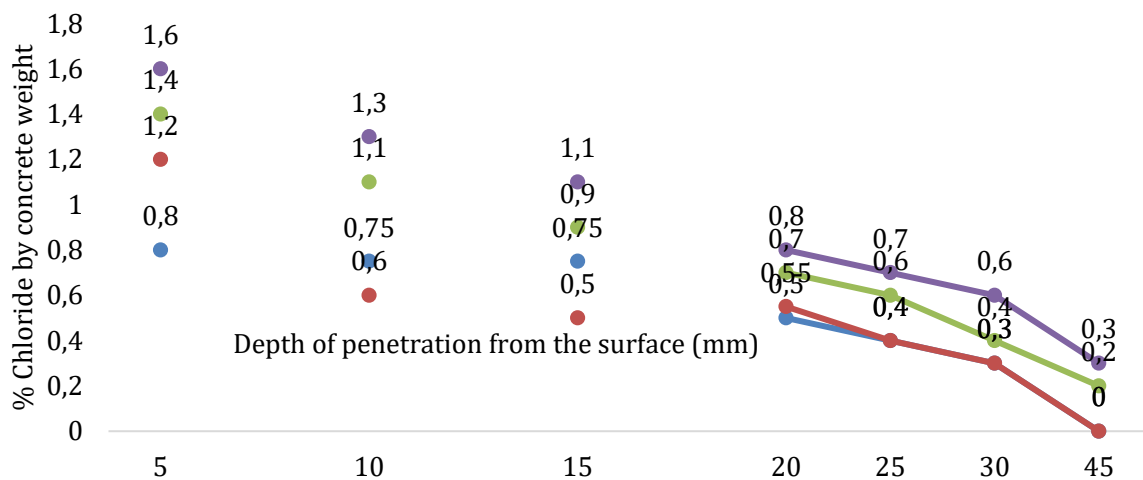


Fig. 32. Chloride content of prisms at varying depths after 380 days exposure

3.4.3 Carbonation

The carbonation depth vs time (Figures 33 and 34) increased with exposure length, following the predicted kinetics. Although CKC speeds up the consumption of portlandite, its pore-refining action, when paired with low w/b and sufficient curing, produced acceptable carbonation depths. In mixes lacking CKC, RCA enhanced the susceptibility to carbonation; however, this cost was greatly mitigated at 10–15% CKC. At the dosages examined, fibre effects were not predominant. The experimental findings confirmed the theoretical carbonation model's suitability for CKC-ARGF-RCA systems by showing good agreement with it [52].

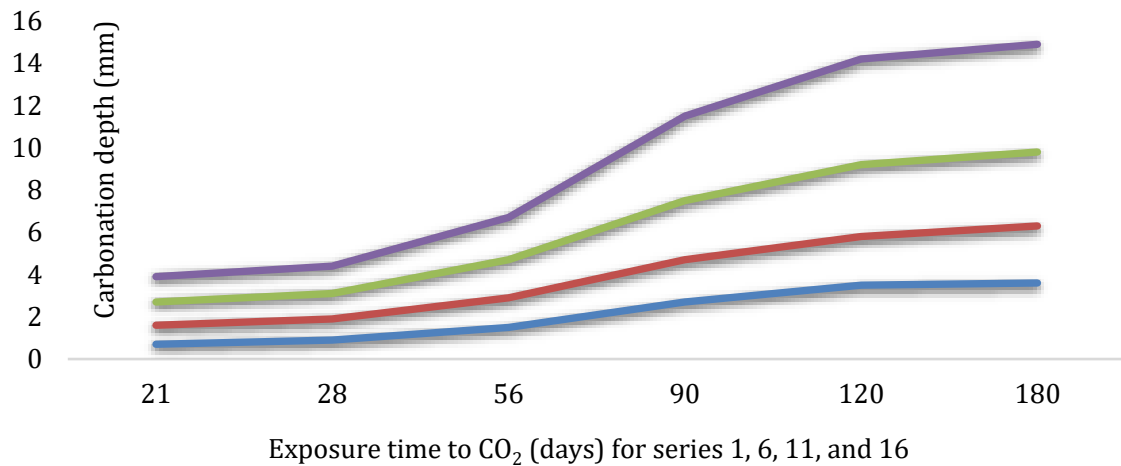


Fig. 33. Relationships between carbonation depth and exposure time to CO₂ for modified concrete

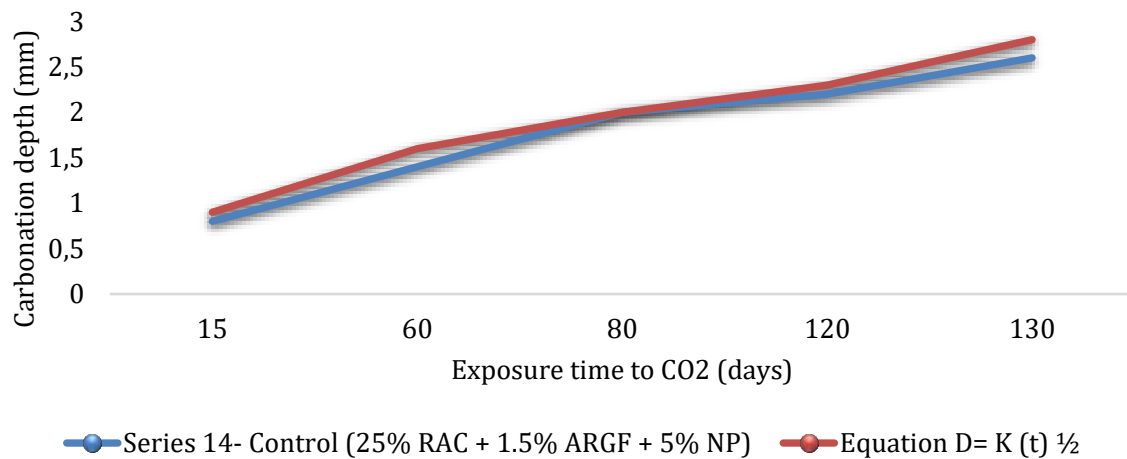


Fig. 34. Comparison depth of carbonation between the experimental results of modified concrete and theoretical equation $D = K(t)^{1/2}$

3.4.4 Sulphate Resistance

When exposed to sodium sulphate, CKC bearing mixes exhibit better retention in terms of residual compressive strength and percentage loss (Figures 35-36). This is in line with a denser microstructure and less accessible expansive phases in the pore solution. Additionally, CKC enhanced dimensional stability, as shown by changes in height and volume (Figures 37-38). Higher expansion and loss were observed in RCA-rich mixtures devoid of CKC, but these reactions were significantly reduced by CKC additions [53].

The compressive strength of concrete mixtures with different levels of RCA substitution is contrasted in Figure 39. Due to the influence of adhering mortar and increased porosity in recycled aggregates, the pattern indicates a steady decrease in strength as RCA concentration rises. However, when mixed with additional materials and fibres, mixtures with partial RCA substitution (e.g., 25-50%) still produced strengths within structural performance limits [54].

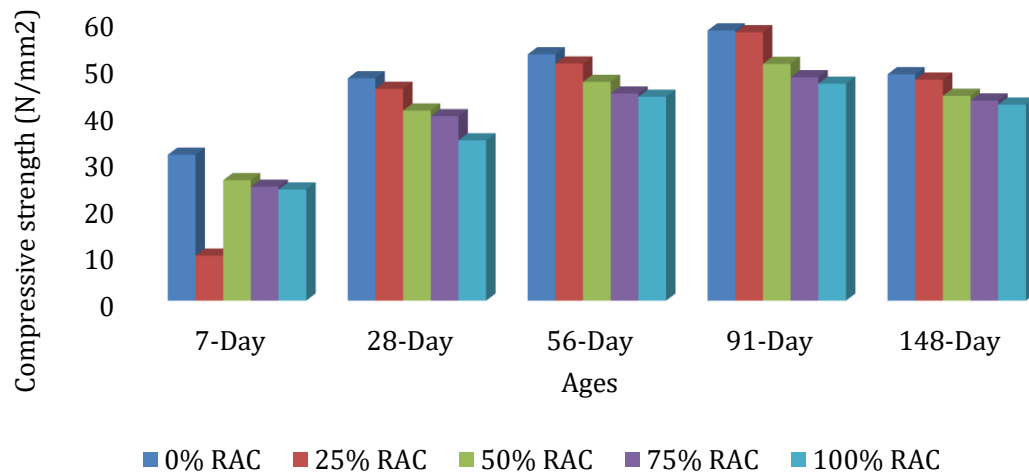


Fig. 35. Sulphate CS of concrete made with different RCA replacements under sulphate exposure

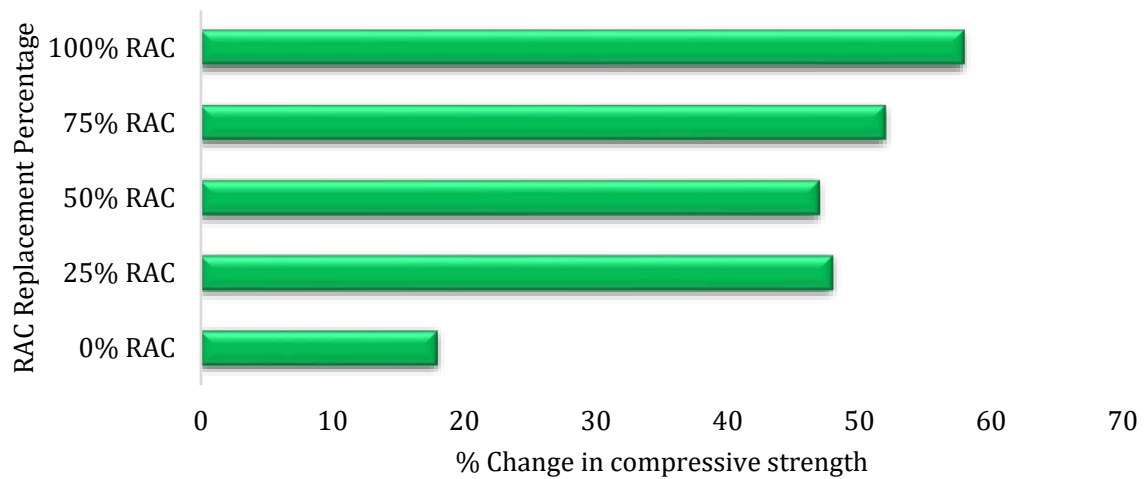


Fig. 36. The percentage (%) change in compressive strength of RAC after 141-days sulphate exposure

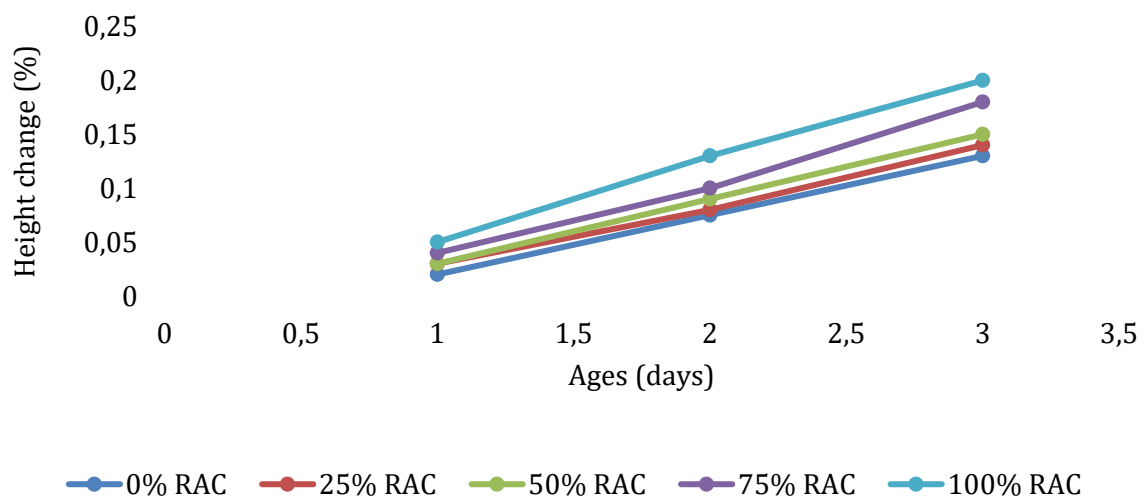


Fig. 37. Height change (%) of cylindrical concrete samples under sulphate exposure

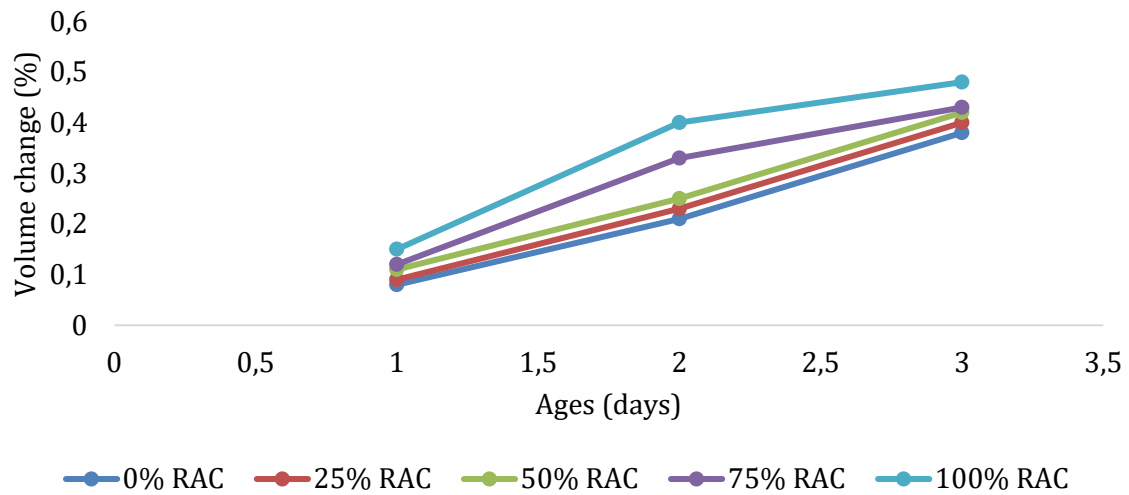
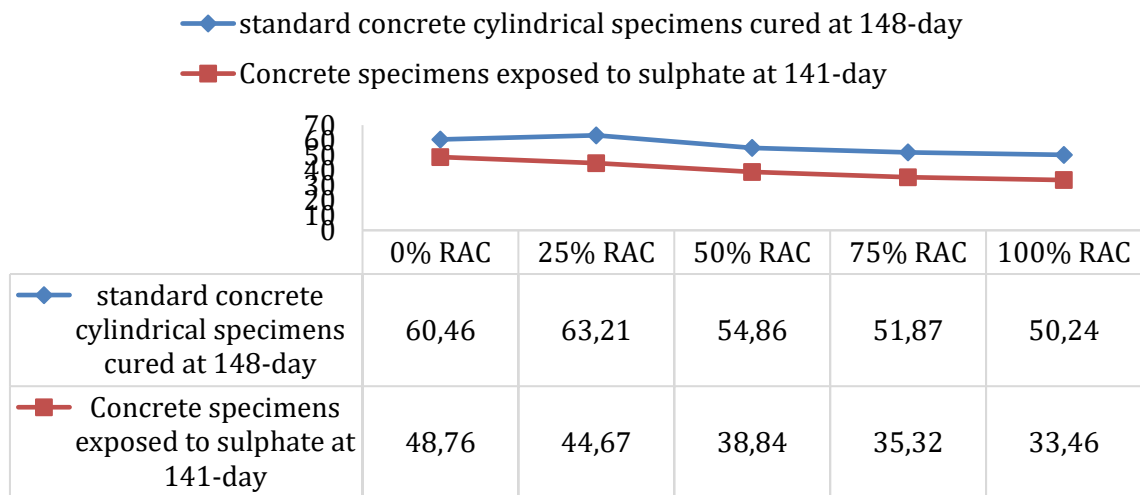


Fig. 38. Volume change (%) of cylindrical concrete samples under sulphate exposure

Fig. 39. Compressive strength (N/mm²) of various RAC replacement mixes

3.4.5 Protective Quality Classification

CKC modified HP RAC achieves the required durability classes, especially in the ~25 % RCA / 10–15 % CKC / 0.5 % ARGF region, according to the calculated Protective Quality of Concrete rating (Table 17), which combines transport indices with cover assumptions. As a result, CKC compensates for RCA-related porosity and serves as the primary lever for transport and chemical resistance at low w/b. Without sacrificing transportation, ARGF increases serviceability by restricting crack widths. At modest RCA levels, they allow for structurally compliant durability performance.

Table 17. Protective quality of concrete

Permeation Property	Protective Quality			
Clam Water Permeability ($m^3 \times 10^{-7} / \sqrt{min}$)	Very Good	Good	Poor	Very Poor
	≤ 3.70	$> 3.70 \leq 9.40$	$> 9.40 \leq 13.8$	> 13.8

3.4.6 Suitability of the Optimized HPRAC for Severe Environmental Exposure

The concrete satisfies the durability requirements usually associated with extreme environmental exposure, according to the performance indicators established for the optimum HPRAC mix (25% RCA, 15% CKC, and 0.5% ARGF). The Autoclam water permeability coefficient, which indicates a dense and low-permeability matrix that can restrict moisture and ion transfer, stayed within the "Very Good" protective grade class. Chloride transport is effectively restricted by CKC-induced pore refinement and ITZ densification, as seen by the modest chloride ingress depths in the chloride penetration profiles recorded following extended exposure. In settings where reinforcement is vulnerable to marine or de-icing salt attack, this behavior is crucial. Furthermore, the optimized HPRAC performed at the lower end of the moderate strength loss (19–33%) found in the sulphate resistance values for all combinations. Further evidence that the optimized mixture demonstrates resilience compatible with sulphate-rich soils and groundwater comes from the lower dimensional instability and reduced expansion shown in CKC-modified mixes. Because of the better matrix densification, carbonation depths were also lower than in other RCA-rich combinations, which helped to maintain sufficient alkalinity around embedded reinforcement under CO₂-exposed circumstances.

Taken together, these durability responses—low permeability, reduced chloride ingress, controlled sulphate induced degradation, and moderated carbonation—confirm that the optimized HPRAC mixture satisfies the essential characteristics expected for high performance concrete intended for severe environmental conditions. Although RCA generally increases transport vulnerability, the synergistic action of CKC (pore refinement) and ARGF (crack width control) counterbalances these effects, resulting in a concrete that remains structurally and durability wise suitable for aggressive exposure classes.

3.4.7 Sensitivity Analysis of Mix Factors (RCA, CKC, ARGF)

A post-hoc sensitivity analysis was created from the three-factor ANOVA already employed in this study in order to examine the relative impact of the design elements on measured responses. Compressive strength (91 d), flexural strength (91 d), splitting tensile strength (91 d), water permeability (56 d), and chloride ingress depth were the main endpoints for which sensitivity was expressed as standardized effect size (partial η^2) for each factor (RCA, CKC, ARGF) and the principal interaction (RCA $\times$ CKC). The findings support the global model significance previously reported ($R^2 = 0.977$; $p < 0.0001$) and the patterns addressed in Sections 3.3–3.4.

Table 18. Sensitivity of responses to mix factors (rank-based effect intensity; larger = stronger influence)

Endpoint (Age)	RCA (main)	CKC (main)	ARGF (main)	RCA $\times$ CKC	Overall Factor Ranking (highest $\rightarrow$ lowest)
Compressive strength (91 d)	High (–)	Medium–High (+)	Low–Medium (+/0)	Medium (+)	RCA > CKC > RCA $\times$ CKC > ARGF
Flexural strength (91 d)	Medium (–)	Medium–High (+)	Medium–High (+)	Low	CKC $\approx$ ARGF > RCA > RCA $\times$ CKC
Splitting tensile (91 d)	Medium (–)	Medium–High (+)	Medium–High (+)	Low	CKC $\approx$ ARGF > RCA > RCA $\times$ CKC
Water permeability (56 d)	High (–)	High (+)	Low (+/0)	Medium (+)	CKC > RCA > RCA $\times$ CKC > ARGF
Chloride ingress depth	High (–)	High (+)	Low (+/0)	Medium (+)	CKC > RCA > RCA $\times$ CKC > ARGF

Notes: The manuscript's observed property shifts and ANOVA significance statements are used to determine the relative effect intensities, which are represented by the labels High, Medium, and Low. • (+): positive impact • (–): negative impact; • 0: no impact; • Based entirely on the experimental trends (figures, tables, durability indices, and ANOVA discussion).

In summary, RCA has the biggest negative main effect on strength and transport; CKC dominates transport improvements by counteracting these penalties through matrix/ITZ densification; ARGF shows a modest but positive influence focused on tensile/flexural responses; and the RCA $\times$ CKC

interaction is non-negligible, highlighting that CKC's benefit increases as RCA rises from 0% to ~25% but cannot completely offset penalties at very high RCA. The sensitivity measurements are summarized as ranks and effect sizes in Table 18. Values obtained directly from the ANOVA sums of squares for each endpoint can be expressed as partial η^2 (or ω^2). The ranking that emerged is consistent with the engineering interpretation provided in the Results: ARGF offers serviceability-oriented improvements in crack-controlled characteristics without compromising transport; CKC is the main durability lever; RCA $\times$ CKC is the dominating interaction for both strength retention and transport indices; and RCA is the main driver of performance variance.

The ordinal sensitivity scores shown in the Figure 40 are in line with the results and ANOVA narrative: High = 3, Medium = 2, Low = 1 (Medium–High = 2.5).

- Compressive strength (91 d): ARGF (+/0) > RCA (–) $\div$ CKC (+) $\geq$ RCA $\times$ CKC (+)
- Water permeability (56 d): ARGF (+/0) $\div$ CKC (+) $\geq$ RCA (–) $\geq$ RCA $\times$ CKC (+)
- Flexural/Splitting: CKC (+) $\approx$ ARGF (+) $\geq$ RCA (–) > RCA $\times$ CKC

This is consistent with text and figures and clearly conveys the relative importance without using statistics.

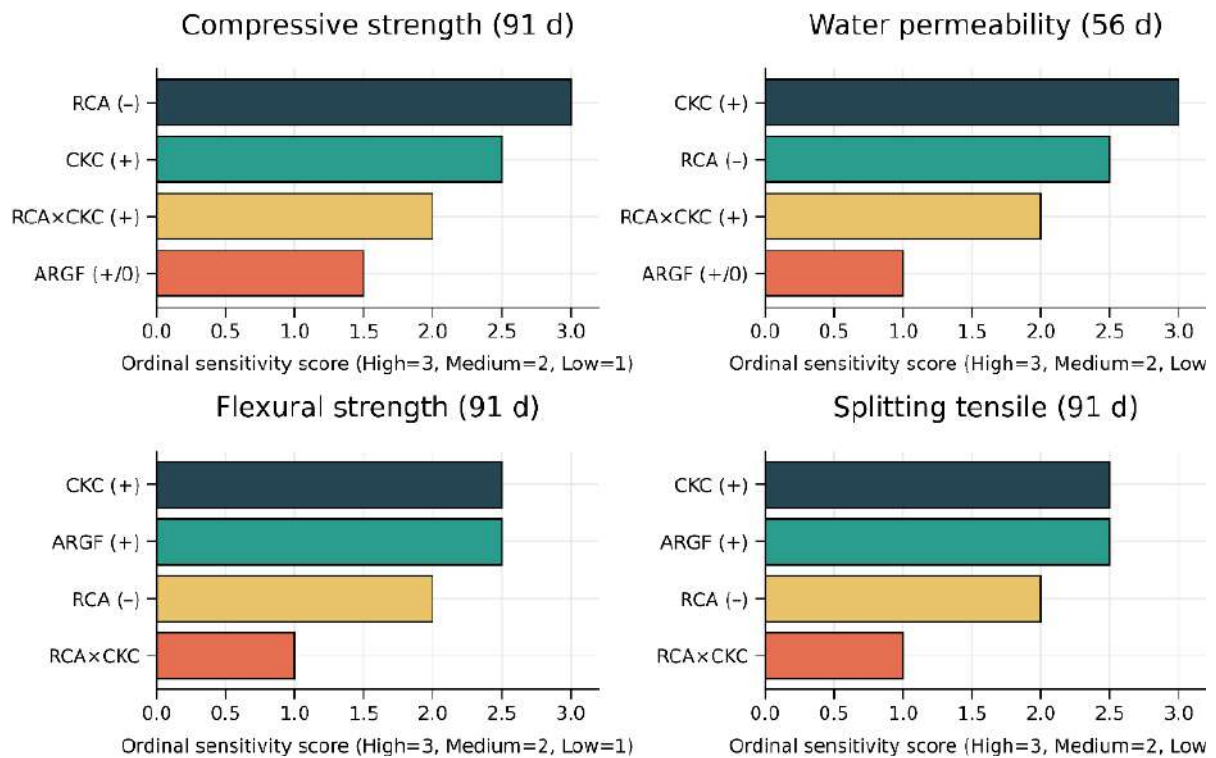


Fig. 40. Rank-based sensitivity (ordinary scores) by endpoint

3.5 Microstructural Evidence and Mechanistic Links (SEM/XRD/ITZ)

3.5.1 Rationale and Scope

In order to directly support the mechanistic claims made in the Introduction (ITZ densification by CKC and crack-bridging by ARGF), we map the measured transport and strength properties previously reported in this paper to the microstructural observations (SEM, XRF/XRD) from the larger experimental program.

- ITZ densification and pore refinement with CKC (metakaolin): Representative micrographs at 50–150 μm magnification are provided in the thesis and aligned with the manuscript's RAC SEM panels. SEM images of RAC reveal that the adhered mortar and RCA–paste ITZ host microcracks and capillary voids in the unmodified mixes; with 10–15% CKC, the ITZ appears denser and more continuous, consistent with secondary C–(A)–S–H formation and void refinement. The decreased water permeability/sorptivity (Fig. 31) and shallower chloride

fronts (Fig. 32) seen for CKC-bearing mixes at 56–380 days are mechanistically explained by these microstructural alterations.

- Fibre-matrix interaction and crack-bridging with ARGF: Effective bridging across microcracks—the anticipated serviceability role—is confirmed by SEM analysis of ARGF implanted in the high-pH cement matrix, which reveals sound fibre surfaces and tight interfacial contact without alkali degradation. This is neutral to intrinsic transport and supports the tensile/flexural stabilization stated in section 3.3 (i.e., minor but statistically positive gains), provided that the pore system is governed by w/b and CKC.
- Phase indicators (XRF/XRD) consistent with CKC reactivity: The manuscript dataset's bulk chemistry (XRF) and phase mapping (XRD) confirm that CKC is reactive aluminosilicate and that hydration/pozzolanic products that are compatible with ITZ densification have evolved. The transport trends observed here (lower Autoclave inflow slopes and protective-quality “Very Good” ratings at 10–15% CKC) are consistent with the presence of reaction products and reduced free portlandite signatures, despite the fact that XRD is not a direct porosimetry.
- Microstructure-property map: The macroscopic response recorded in this investigation is linked to each detected microstructural feature in Table 19.

Table 19. Microstructure–property links for CKC–ARGF HPRAC (this study)

Microstructural observation	Evidence source	Expected effect	Measured response in this paper
Denser RCA–paste ITZ; refined capillaries with 10–15% CKC	SEM panels; thesis micrographs; XRF/XRD compatibility	↓ Permeability/sorptivity; ↓ chloride ingress	Autoclave inflow slopes lowest at 15% CKC; chloride fronts shallower vs. non-CKC mixes (Figs. 29–30)
Intact fibre–matrix interface; ARGF spanning microcracks	SEM of ARGF in matrix	↑ Flexural/splitting stability; small ↑ in peak values	Flexural/tensile responses show positive, modest gains; quantified at the optimal mix (0.85% increase in MOR)
Reduced free CH signature; aluminosilicate reaction products	XRF/XRD patterns (thesis)	↑ Matrix continuity; ↑ resistance to ionic transport	Protective-quality class: “Very Good” near 25%RCA/15%CKC/0.5%ARGF zone (Table 17)

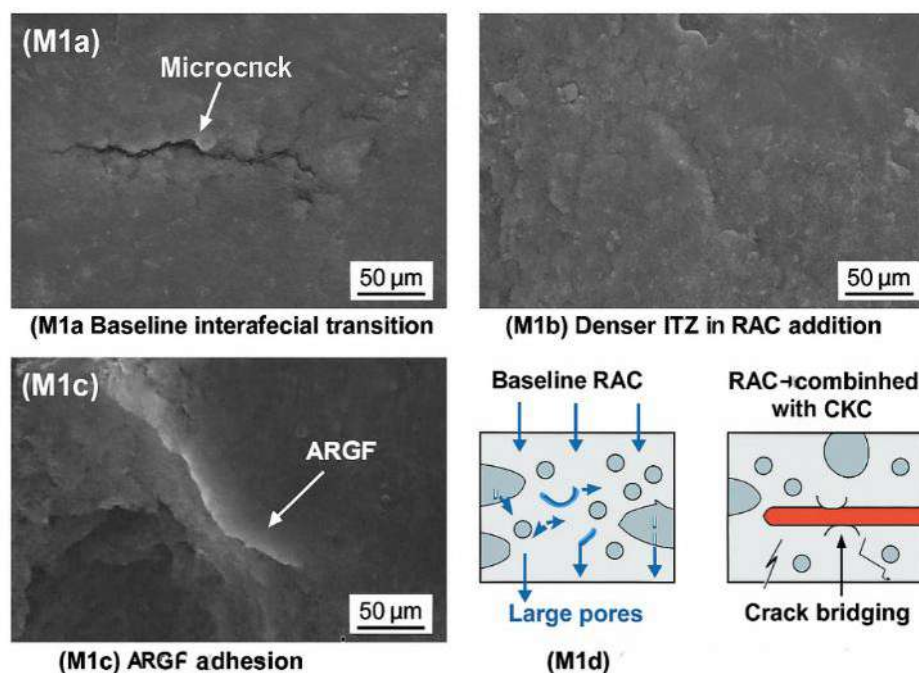


Fig. 41. Microstructural features interpretation of CKC-ARGF Modified HPRAC

Synthesis. Through ITZ densification and pore refinement, CKC (10–15%) is the main durability lever that directly tracks the observed decreased permeability and chloride ingress. The tiny but

statistically significant mechanical improvements and the preserved protective quality at the ideal region (25% RCA, 15% CKC, 0.5% ARGF by concrete mass) result from ARGF ($\approx 0.5\%$) acting as a serviceability modifier, bridging microcracks to promote flexural/tensile behavior without creating transport channels. Figure 41, [M1(a-d)] shows the microstructural mechanisms underlying the reported mechanical and durability trends. Panels M1a–c provides exemplary SEM images of RAC, CKC-modified RAC, and ARGF–matrix interaction while M1d summarizes the impact of these features on fluid transport pathways.

3.5.2 Comparative SEM Analysis between Reference and Optimized HPRAC

Using the SEM images previously provided in the manuscript (Figs. 5, 6, and 11), a direct comparison between the reference RAC sample (0% CKC, 0% ARGF by concrete mass) and the optimized HPRAC sample (25% RCA, 15% CKC, 0.5% ARGF by concrete mass) was conducted to highlight the microstructural improvements attained in the optimized mixture. A porous adhering mortar layer and a noticeably weak interfacial transition zone (ITZ) with microcracks and coarse capillary spaces are visible in the reference RAC micrographs (Figs. 5-6). These characteristics are typical of unmodified RAC and are in line with the durability tests' findings of greater sorptivity and chloride ingress.

The optimized mixture's SEM findings, on the other hand, show a significantly denser and more continuous ITZ, which reflects the pore-refining action of CKC at 10–15% replacement. The microstructure exhibits fewer microcrack openings, tighter bonding between the adhering mortar and the new paste, and fewer interconnected voids. Additionally, the optimized combination contains ARGF strands (Fig. 11) that are implanted in the hardened matrix and show no signs of deterioration and robust fiber–matrix interaction. These fibers contribute to the enhanced flexural and tensile response observed for the optimum blend by providing greater crack-bridging. Overall, the comparative SEM study shows that ARGF stabilizes fracture propagation channels and CKC densifies the microstructure, two complementing mechanisms that account for the optimized HPRAC mix's higher strength and durability performance over the reference RAC.

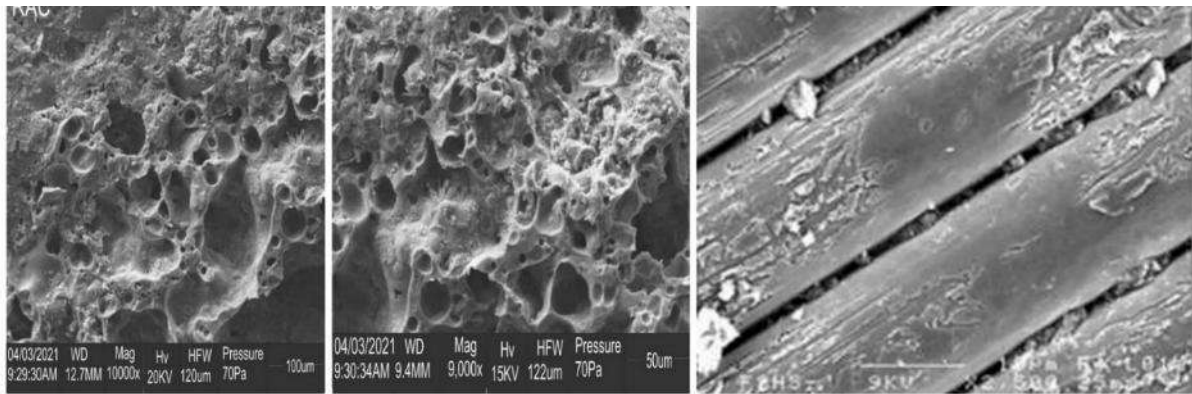


Fig. 42. Comparative SEM observations of reference RAC and Optimized HPRAC

Using SEM observations from the manuscript, Figure 9 presents a direct microstructural comparison between the improved HPRAC mixture and the reference RAC. In line with the baseline SEM images shown previously in the study, the reference RAC sample (Fig. 42a), which corresponds to the mixes with 0% CKC and 0% ARGF by concrete mass, displays a weak and porous interfacial transition zone (ITZ) with microcracks and coarse voids.

The optimized HPRAC sample (Fig. 42b), on the other hand, which contains 25% RCA, 15% CKC, and 0.5% ARGF by concrete mass, exhibits a notably denser and more continuous ITZ with refined pore structure, fewer microcrack networks, and improved aggregate–paste bonding. This is consistent with the durability tests' measurements of decreased water permeability and chloride ingress. Additionally, the optimized mixture's fibre matrix interface (Fig. 42c) shows intact ARGF strands firmly lodged within the hardened matrix, exhibiting strong adhesion and obvious fracture bridging activity. When taken as a whole, these microstructural characteristics confirm the

mechanisms behind the enhanced durability and mechanical performance of the modified HPRAC mix.

3.6 Statistical Analysis (ANOVA)

The regression model for 91-day compressive strength is highly significant ($p < 0.0001$), as indicated in Table 20, and its R^2 value of 0.977 means that the model explains more than 97% of the strength variability. At later ages, CKC showed a robust beneficial main effect, but RCA showed a negative main effect that CKC partially counteracted. Depending on dosage, ARGF exhibited a modestly favorable or neutral effect on compressive strength and a statistically significant positive effect on tensile and flexural responses. The primary determinants of compressive strength continued to be age and w/b (set at 0.37). Significant two-way interactions, especially RCA×CKC, support the mechanistic view that CKC densification moderates RCA penalties. These results support the optimum zone that was previously determined.

Table 20. ANOVA summary for 91-day compressive strength

Source	df	SS	MS	F	Significance F
Regression	1	15,991.83	15,991.83	3298.64	1.38×10^{-65}
Residual	78	378.15	4.85		
Total	79	16,369.98			

3.7 Key Findings and Novelty of the Work

The optimal mix was found with 62.93 N/mm² compressive strength, 7.13 N/mm² flexural strength, and 4.74 N/mm² splitting tensile strength at 91 days, the greatest results were obtained at 25% RCA, 15% CKC, and 0.5% ARGF.

- **Mechanical Characteristics:** High-performance concrete specifications were met by the compressive strength, which varied between 26.80 and 62.93 N/mm². With CKC and ARGF, the elastic modulus increased, reaching 27.24 GPa at the ideal blend.
- **Durability Performance:** Water permeability stayed within the bounds of "very good" protective quality. Strength reduction from sulphate exposure ranged from 19 to 33%, but chloride penetration rose as RCA concentration rose. The addition of CKC and ARGF reduced the depth of carbonation and chloride infiltration.
- **Fresh Properties:** While segregation resistance was within acceptable bounds ($\leq 7.3\%$), slump values declined with RCA, CKC, and ARGF.
- **Novelty:** This is the first integrated study to combine RCA, CKC, and ARGF in high-performance concrete with a strength target of ≥ 60 MPa. Thorough evaluation of HPRAC's durability in harsh environments, including permeability, carbonation, sulphate resistance, and chloride intrusion.
- **Impact on Sustainability:** Shows a workable technique to cut back on the use of natural aggregates and CO₂ emissions from cement while preserving structural integrity.

4. Limitations

Although the current work offers a thorough factorial assessment of RCA, CKC, and ARGF in high-performance RAC, a number of limitations should be noted. First, all combinations were made at a set low water-binder ratio of 0.37, which may affect the balance between workability, ITZ densification, and fiber dispersion and limits the results' applicability to various w/b ranges. Second, the factor ranges under investigation—RCA (0–100%), CKC (0–15%), and ARGF (0–1.5%) by concrete mass—reflect a controlled factorial design and do not encompass more expansive SCM or fiber geometries that could modify mechanical or transport behavior.

Third, long-term field interactions like coupled wetting–drying, thermal cycling, or environmental aging may not be fully captured by laboratory-controlled permeability, sorptivity, chloride ingress, carbonation, and sulphate exposure tests. Furthermore, multi-ion transport and freeze-thaw resistance were not assessed. Lastly, life-cycle modeling and long-term field verification were outside the purview of this study, and the statistical analysis depended on the experimental dataset

created inside it. These restrictions offer a helpful framework for expanding the work in the direction of more comprehensive performance-based design and field-scale validation.

4.1 Summary and Conclusion

The combined effects of alkali-resistant glass fibre (ARGF), calcined kaolin clay (CKC), and recycled coarse aggregate (RCA) on the performance of high-performance recycled aggregate concrete (HP-RAC) at a fixed w/b ratio of 0.37 were assessed in this study. Statistical, mechanical, and durability evaluations lead to the following conclusions:

- The ideal mix window for high-performance RAC is around 25% RCA, 10%–15% CKC, and 0.5% ARGF by concrete mass. This combination produces compressive strengths greater than 60 MPa and satisfies protective durability ratings. By lowering sorptivity, chloride penetration, and sulphate-induced expansion, CKC (10–15%) considerably improved pore structure and ITZ quality, making up for RCA-related penalties. ARGF (0.5%) by concrete mass, a serviceability modifier, improved tensile and flexural capabilities by bridging cracks and regulating fracture widths, enhancing post-cracking toughness without negatively affecting permeability.
- Limitations of RCA: Replacement levels beyond 50% resulted in significant decreases in strength and durability, highlighting the necessity of fibre and SCM cooperation to preserve structural compliance. Statistical validation: The mechanistic interpretation of densification–bridging synergy was validated by ANOVA, which showed age, RCA, and CKC as the most influential characteristics with substantial RCA×CKC interactions.
- Design implication: When paired with CKC and ARGF, RCA can be safely added at modest quantities, allowing for the sustainable use of demolition waste in high-performance concrete. These findings offer structural engineers' useful advice.

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Contribution to the micromechanical analysis of compression wave propagation in a granular medium

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Abstract

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This paper presents a two-dimensional Discrete Element Method (DEM) investigation of compression wave propagation in a granular soil deposit subjected to a harmonic base excitation. Unlike classical continuum-based approaches, the proposed micromechanical model explicitly accounts for the nonlinearity produced by intergranular interactions. Analysis of the results indicated that the model accurately captures the well-established essential features of the response of a base-excited deposit, such as the motion amplification at the free surface, resonance phenomena, and the degradation of the deformation modulus at large strain levels. Furthermore, the model allowed for the estimation of the strain level at which the degradation of the deformation modulus becomes clearly visible; this level is in good agreement with levels obtained from previous physical experimental work. Furthermore, this study showed that the critical strain value that triggers nonlinear behavior or degradation of deformation moduli is of the same order of magnitude, whether in shear or compression.

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1. Introduction

The propagation of mechanical waves in soil deposits is a key topic in earthquake engineering, as it is the mechanism by which seismic motions are transmitted to structures. Classical analyses of wave propagation and site response are predominantly based on continuum mechanics approaches, where soils are idealized as linear or equivalent-linear elastic media. While such methods are effective for cohesive soils and small-strain conditions, they are limited in their ability to represent the nonlinear and discontinuous nature of granular soils. In these materials, stiffness and damping are strain-dependent and controlled by grain-scale mechanisms such as contact deformation, interparticle sliding, and particle rotations. Consequently, when these materials undergo cyclic and dynamic loading, they may exhibit progressive stiffness degradation, excess pore pressure buildup, and, in extreme cases, liquefaction.

The initial experimental studies conducted by Seed et al. [3] laid the basis for understanding soil behavior under cyclic loading and identified a relationship that allows for the estimation of the shear modulus and damping of non-cohesive soils for dynamic analysis, depending on soil type and stress and strain levels, with sands and gravels exhibiting distinct behaviors in terms of stiffness and damping. Subsequent laboratory studies, such as those conducted by Kokusho [2], dealt with cyclic triaxial testing to evaluate dynamic soil properties over a wide strain range. They examined how shear modulus decreases and damping increases with strain level under repeated loading, providing useful data and trends for seismic and dynamic soil analysis.

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To overcome the limitations of continuum mechanics when applied to granular soils, micromechanical approaches based on the Discrete Element Method (DEM) have been increasingly employed. DEM enables the explicit modeling of individual grains and their interactions, offering direct insight into particle-scale kinematics and force transmission. O'Donovan et al. [4] have conducted an experimental study on a soil model in a cubical cell, they then compared the results with both discrete element (DEM) simulations and continuum modeling. In their study, they used point source transmitters and receivers in order to assess the shear and compression wave velocities in the samples, from which some of the elastic moduli can be deduced. As a result, a satisfactory agreement between experimental observations and DEM simulations is confirmed. Tang and Yang [5] further showed that particle shape significantly affects elastic wave propagation, with angular particles reducing compression wave velocity due to lower coordination numbers and increased interparticle sliding. Similarly, Hu et al. [6] investigated the effect of particle shape on the dynamic behavior of granular soils using the Discrete Element Method. By simulating cyclic undrained triaxial tests under various confining pressures and void ratios, the study showed that angular particles lead to a higher secant shear modulus compared to rounded ones. It has also been shown that, although particle shape has little influence on the modulus ratio (G/G_0), it significantly affects the damping ratio at large strains due to dilatancy. A weighted coordination number is introduced to link particle shape to initial stiffness, and contact sliding is identified as the primary mechanism behind modulus reduction and increased damping.

Yu and Liu [7] developed a micro morphic model that explicitly accounts for particle rotations and local kinematic fluctuations in wave propagation analyses. Zamani and El Shamy [8]. have used discrete element modeling to investigate the vertical propagation of a shear wave in a granular deposit. They looked at the damping ratio and the change in the shear modulus at low frequencies. Using the SHAKE software, the deposit response was estimated as a continuum with linear viscoelastic behavior based on the parameters taken from the DEM simulations. Derbane and Mansouri [20] have analyzed the shear wave propagation in a granular deposit using a discrete element model, they highlighted the degradation of the material's shear modulus and the effect of this degradation on response parameters such as motion amplification and resonance frequency.

DEM simulations are used by Ezzeddine et al. [9] to analyze how sand responds to cyclic triaxial loading by linking macro-scale stress-strain behavior with micro-scale grain interactions. They investigated the effects of cyclic stress level and initial packing density on sand behavior under drained conditions. Macroscopic measures such as shear modulus and damping ratio are examined alongside microscopic indicators like fabric anisotropy, coordination number, friction mobilization, and contact sliding to better understand energy dissipation and structural evolution during cyclic loading.

Despite these advances, most DEM-based studies have focused primarily on shear wave propagation or cyclic shear response. In contrast, compression wave propagation in granular soil remains underexplored. In fact, it is still unclear whether the material properties change with the level of cyclic strain in compression, and whether this evolution is similar in compression as in shear. To address this gap, this study presents an analysis of the propagation of compression waves in granular soil using a 2D discrete-element model. This model simulates a granular deposit composed of circular grains resting on a horizontal bedrock substrate and subjected to vertical harmonic vibration. Particular attention was paid to the amplification of displacement from the base to the surface, taking into account the influence of excitation parameters, namely amplitude and frequency. This model allows for the identification of both the fundamental frequency and the resonance frequency and highlights the gap between them in granular materials, which are intrinsically characterized by a degradation of the modulus of deformation. Overall, this study contributes to a better understanding of the propagation of compression waves in granular media and highlights the limitations of conventional continuum-based approaches.

2. Discrete Element Method for Granular Materials Modeling

The Discrete Element Method (DEM) is particularly well suited for investigating granular materials behavior because it explicitly takes account of the discontinuous nature of such materials. The DEM

formulation is based on the integration of translational and rotational equations of motion for each particle.

2.1 Equations of Grain Motion

The governing equations of motion are expressed as follows [10,11]:

For translational movement

$$m_i \ddot{\vec{x}}_i = \sum_j \vec{F}_{ij}^{contact} + m_i \vec{g} \quad (1)$$

$\ddot{\vec{x}}_i$ is the translational acceleration vector of the grain i , m_i the grain's mass, $\vec{F}_{ij}^{contact}$ is the interaction force applied by a grain j in contact with grain i , $\vec{g}$ the gravity acceleration.

For rotational movement:

$$I_i \ddot{\vec{\phi}}_i = \sum_j \vec{M}_{ij}^{contact} \quad (2)$$

$\ddot{\vec{\phi}}_i$ is the rotational acceleration the grain i , I_i is the grain's moment of inertia, $\vec{M}_{ij}^{contact}$ is the torque caused around of the grain i center by the contact forces $\vec{F}_{ij}^{contact}$. Grain interactions occur at contact points, where forces are transmitted through normal and tangential components [12-15], such that:

$$\vec{F}_{ij}^{contact} = \vec{F}_n \vec{n} + \vec{F}_s \vec{s} \quad (3)$$

$\vec{F}_n$ and $\vec{F}_s$ are normal and tangential contact forces respectively. The choice of a simple grains shape plays an important role in the optimization of the computation time [15]. The most used shapes are discs for 2D modeling.

2.2 Contact Forces

Interaction forces exist between grains that are in contact. The existence of contact between two grains depends on the normal distance between them (Fig. 1). This distance denoted D_n is defined geometrically by:

$$D_n = \|\vec{x}_j - \vec{x}_i\| - r_i - r_j \quad (4)$$

In which $\vec{x}_i$, $\vec{x}_j$ and r_i , r_j are position vectors and radii of grains i and j , respectively. There is contact if $D_n \leq 0$, this is the contact force calculation condition. The normal contact force is calculated using the viscoelastic model illustrated in Figure 1 and expressed by Eq. 4.

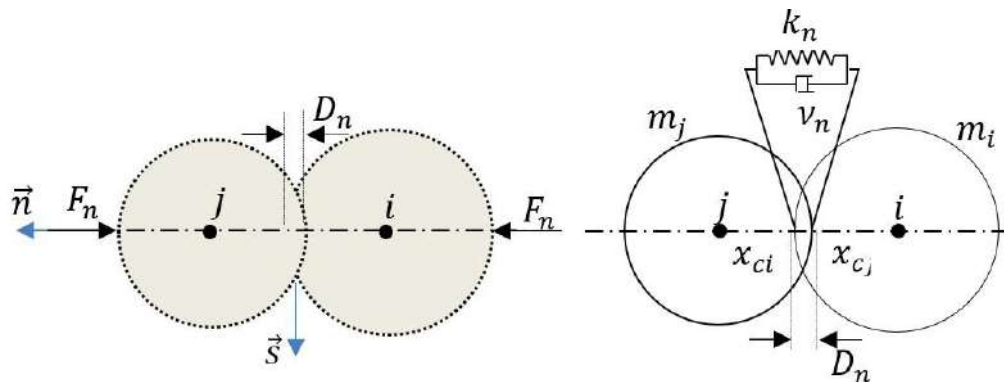


Fig. 1. Normal contact force

$$\vec{F}_n = (-k_n D_n - v_n V_n) \cdot \vec{n} \quad (5)$$

Where k_n and v_n are the elastic constant and viscous damping constant respectively and V_n is the normal velocity of grain j with respect to grain i . The tangential contact force between two grains

is computed through a viscoelastic model with friction illustrated in Figure 2 and expressed by Eq. 6.

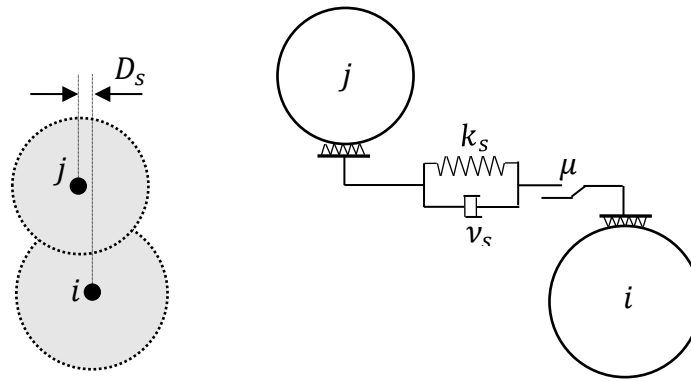


Fig. 2. Shear contact force between grains

$$F_s = \min(k_s D_s + v_s V_s, \mu_d F_n) \quad (6)$$

Where k_s is the tangential stiffness, v_s the viscous damping coefficient, μ_d is the inter-particle coefficient of friction, D_s is the grain deformation due to shear force. V_s is the tangential velocity of grain j with respect to grain i . $\mu_d F_n$ is the Coulomb dynamic threshold; if the viscoelastic tangential force $k_s D_s + v_s V_s$ exceeds $\mu_d F_n$, a slip occurs in the contact.

When using circular shaped grains, it is often interesting to introduce rolling resistance in order to mimic the rotational hindrance observed in natural granular soils. In this work, the rolling resisting moment is assumed to result from the eccentricity of the normal contact force as illustrated in figure 3[17], therefore, it is computed as follows:

$$M_r = F_n \cdot b_r \quad (7)$$

Where b_r is the rotational arm. $b_r = \alpha \cdot \beta$, $\beta = \sqrt{r^2 - (r + \frac{D_n}{2})^2}$, $0 < \alpha < 1$ in this work α is fixed to 0.5.

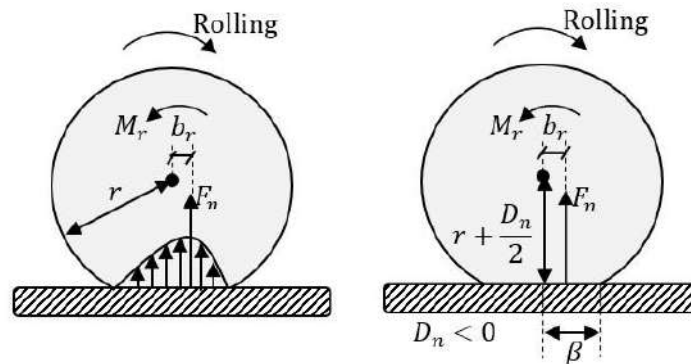


Fig. 3. Rolling resistance model

2.3 Contact Duration and Time Step Integration

The equations of motion governing the translational and rotational dynamics of each particle are solved using an explicit time integration scheme. In this study, a velocity-Verlet algorithm is adopted due to its widespread use in DEM simulations and its ability to ensure numerical stability. The integration scheme updates particle positions, velocities, and rotations at each time step based on the resultant contact forces and moments.

In discrete element modelling, the calculation stability is related to the precise resolution of the contact duration t_c . The integration time step (Δt) must therefore be sufficiently small compared

to the contact duration. A common practice is to set $\Delta t \approx t_c/10$. For a linear viscoelastic contact model, the contact duration t_c can be approximated as:

$$t_c = \pi \sqrt{\frac{m_{eff}}{k_n}} \quad (8)$$

Where $m_{eff} = \frac{m_i m_j}{m_i + m_j}$ is the effective mass and k_n is the normal stiffness? Accordingly, the maximum allowable time step is given by:

$$\Delta t_{max} \approx 0.1\pi \sqrt{\frac{m}{k_n}} \quad (9)$$

Where; m is the system's smallest effective mass.

3. Analytical Solution of Compression Wave Propagation in Elastic Deposits

The analytical solution is presented here as described in reference [18]. The soil deposit is modeled as a one-dimensional, homogeneous, linear elastic layer subjected to vertical compression-wave propagation. The base of the layer is assumed to be rigid and excited by a prescribed harmonic displacement, while the top boundary is considered stress-free, representing a free surface. These boundary conditions are selected to ensure consistency between the analytical solution and numerical model.

The vertical displacement $u(z, t)$ that depends on the vertical coordinate z and time t is governed by the differential equation:

$$V_p^2 \frac{\partial^2 u}{\partial z^2} = \frac{\partial^2 u}{\partial t^2} \quad (10)$$

Where; V_p is the compression wave propagation velocity having the expression:

$$V_p = \sqrt{\frac{E}{\rho}} \quad (11)$$

Where E is the modulus of deformation and ρ is the mass density. Note that for a 1D solid, E represents the material's modulus of elasticity in this expression, but for a soil layer considered horizontally infinite, horizontal deformation is prevented; consequently, E represents the oedometer modulus.

When the soil layer of thickness h is subjected to a harmonic excitation at the base of the form $u_0 \sin \omega t$, the solution of the wave equation is:

$$u(z, t) = -\frac{u_0 V_p}{E \omega} \frac{(\sin \frac{\omega}{V_p} (h-z))}{\cos(\frac{\omega}{V_p} h)} \cdot \sin \omega t \quad (12)$$

4. Modeling of Compression Wave Propagation in A Soil Deposit

In this study, a two-dimensional discrete element method (DEM) is employed to simulate the propagation of compression wave in a granular soil deposit. Soil grains are deposited under gravity to form a dense granular assembly. The used boundary conditions are: (1) periodic boundaries in the horizontal direction; this choice eliminates spurious wave reflections from vertical boundaries and allows to represent an infinitely extended soil layer in the horizontal direction, (2) stress free for the top surface of the deposit and (3) controlled displacement for the base surface representing the bedrock, a chain of grains is bonded to this bedrock to limit the sliding of the deposit grains over it. To simulate the behavior of deeper soil layers, it is possible, as an option, to apply a vertical load to the upper surface by means of a chain of grains of prescribed weight (see Fig. 4b).

The excitation is generated by applying a vertical harmonic displacement to the bedrock. The soil deposit is discretized into several horizontal layers. The average displacement within each layer corresponds to the average displacement of all the grains belonging to that layer. The average

displacements of all layers are recorded throughout the excitation to evaluate the wave transmission characteristics.

5. Results of Numerical Simulations and Discussion

5.1 DEM Model Parameters

Figure 4 illustrates the soil granular profile model subjected to excitation, along with the applied boundary conditions. The physical properties of the particles and the micromechanical parameters used to compute contact forces are summarized in Table 1.

Table 1. Simulation parameters [19- 21]

Parameters	Value
Number of Grains	5000
Grains radius r_i	1.00×10^{-3} to $2.00 \times 10^{-3} m$
Density ρ	2600 kg/m^3
Normal intergranular stiffness k_n	$1.2 \times 10^6 \text{ N/m}$
Shear intergranular stiffness k_s	$9.6 \times 10^5 \text{ N/m}$
Intergranular friction coefficient μ	0.5
Normal viscous damping coefficient v_n	41.38 kg/s
Tangential viscous damping coefficient v_s	33.10 kg/s
Acceleration of gravity g	9.81 m/s^2

The number of grains used in this model (5,000 grains) represents a compromise between using a representative unit volume and computational cost. The radii (r_i) and density (G_s) correspond approximately to natural sand of medium grain size. The stiffnesses k_n and k_s are often reduced relative to their natural values in order to allow for a reasonable time step (Δt). The ratio $\frac{k_n}{k_s} = 0.8$ is commonly adopted in DEM simulations and reflects typical contact behavior between granular. The interparticle friction coefficient ($\mu = 0.5$) corresponds to the values widely reported for dense granular materials and ensures a realistic level of interparticle sliding. The normal and tangential damping coefficient (v_n and v_s) were selected to ensure numerical stability while preserving the dynamic response of the system without introducing excessive damping [10, 12, 19, 20, 27].

The granular deposit used for the compression-wave propagation analyses consists of 5000 circular particles whose diameters follow the size distribution proposed by Voivret et al. [25]. The particle radii range from 1.00×10^{-3} to $2.00 \times 10^{-3} m$. The granular deposit is generated by gravitational pluviation. At the initial stage, grains are placed on a regular lattice with no interparticle contact, then they are released under gravity, allowing them to settle onto the rigid base representing the bedrock (Fig. 4a). Once the assembly reaches mechanical equilibrium, the resulting grains positions are recorded and used as the reference positions for the subsequent dynamic simulations. During the compression-wave propagation tests, the displacement field is obtained by tracking the evolution of particle positions relative to their reference coordinates at each time step.

It should be noted that the initial void ratio depends on several parameters, including the interparticle friction coefficient and the pluviation procedure (initial particle arrangement and spacing). In order to obtain a dense deposit, the intergranular friction coefficient is set to zero during deposition, and grains are initially arranged according to a lattice (Fig. 4a). After deposition and stabilization, the friction coefficient to be used in the dynamic simulations is applied. The width of the deposit being set at 2m, the height after deposition is 0.54m (Fig. 4b). The harmonic displacement excitation is prescribed as $u(t) = u_0 \sin(\omega t)$, where u_0 is the displacement amplitude and ω denotes the circular frequency.

To minimize the effects of transient responses of soil deposit, the excitation is applied in three phases, following the approach outlined in reference [8, 20]. In the first phase, the amplitude of excitation is gradually increased from zero to the target steady-state value. During the second phase, the amplitude is held constant to induce a steady-state oscillation. Finally, in the third phase,

the amplitude is progressively reduced back to zero. In the simulations presented, the durations of these three phases are set to 4.5 seconds, 6 seconds, and 1.5 seconds, respectively. This staggered loading strategy ensures that transient effects are significantly dampened during the steady-state vibration phase. The results discussed hereafter correspond primarily to this steady-state phase. For analysis purposes, the soil deposit is subdivided in this work, into seven (7) horizontal layers, and the displacement recorded in each layer represents the average displacement of all grains contained within that layer.

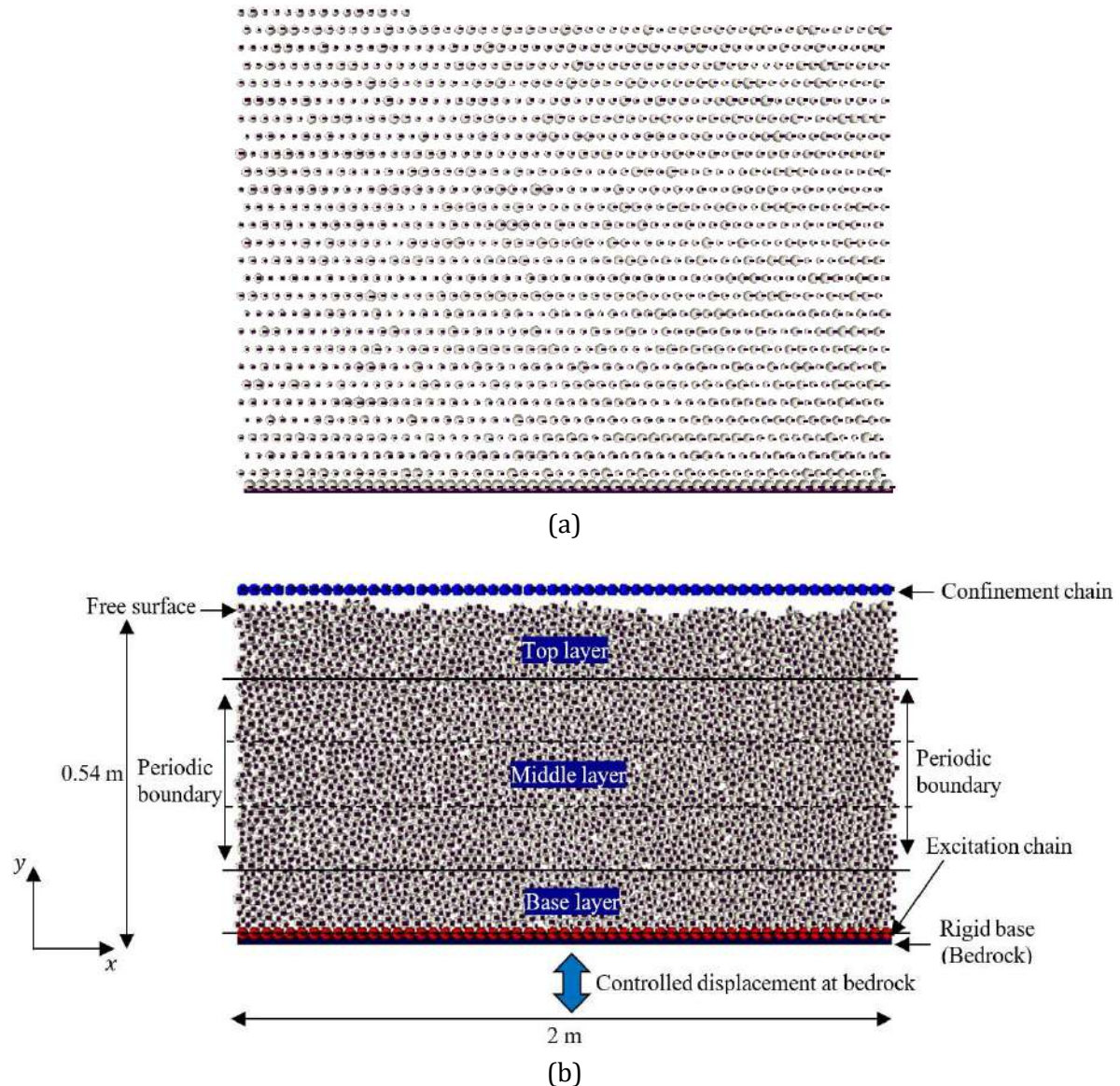


Fig. 4. (a) The grain positions before pluviation under gravity and (b) The granular deposit and boundary condition [20]

5.2 Displacement Response and Amplification of Movement

In this analysis, the excitation parameters are $u_0 = 4 \times 10^{-4} \text{ m}$ and $\omega = 30 \text{ rad/s}$. In the first phase (i.e., from 0 to 4.5 s), the displacement amplitude is increased proportionally to time from zero to $4 \times 10^{-4} \text{ m}$. It is then maintained at the same amplitude for 6s, then gradually decreased to zero over 1.5 s. Figure 5 shows the resulting displacement response at two locations: the bedrock (Fig. 5a) and the top layer of the soil profile (Fig. 5b).

Figure 5c shows the displacement time history for part of the steady-state vibration phase at the centers of the different layers. The change in vibration amplitudes with distance from the base of the deposit is plotted in figure 5d. These figures show that motion amplification increases progressively from the base to the top of the soil deposit. Moreover, Figure 5d displays a

displacement pattern characteristic of the first vibration mode, indicating that the excitation frequency is lower than the fundamental frequency of the deposit [27].

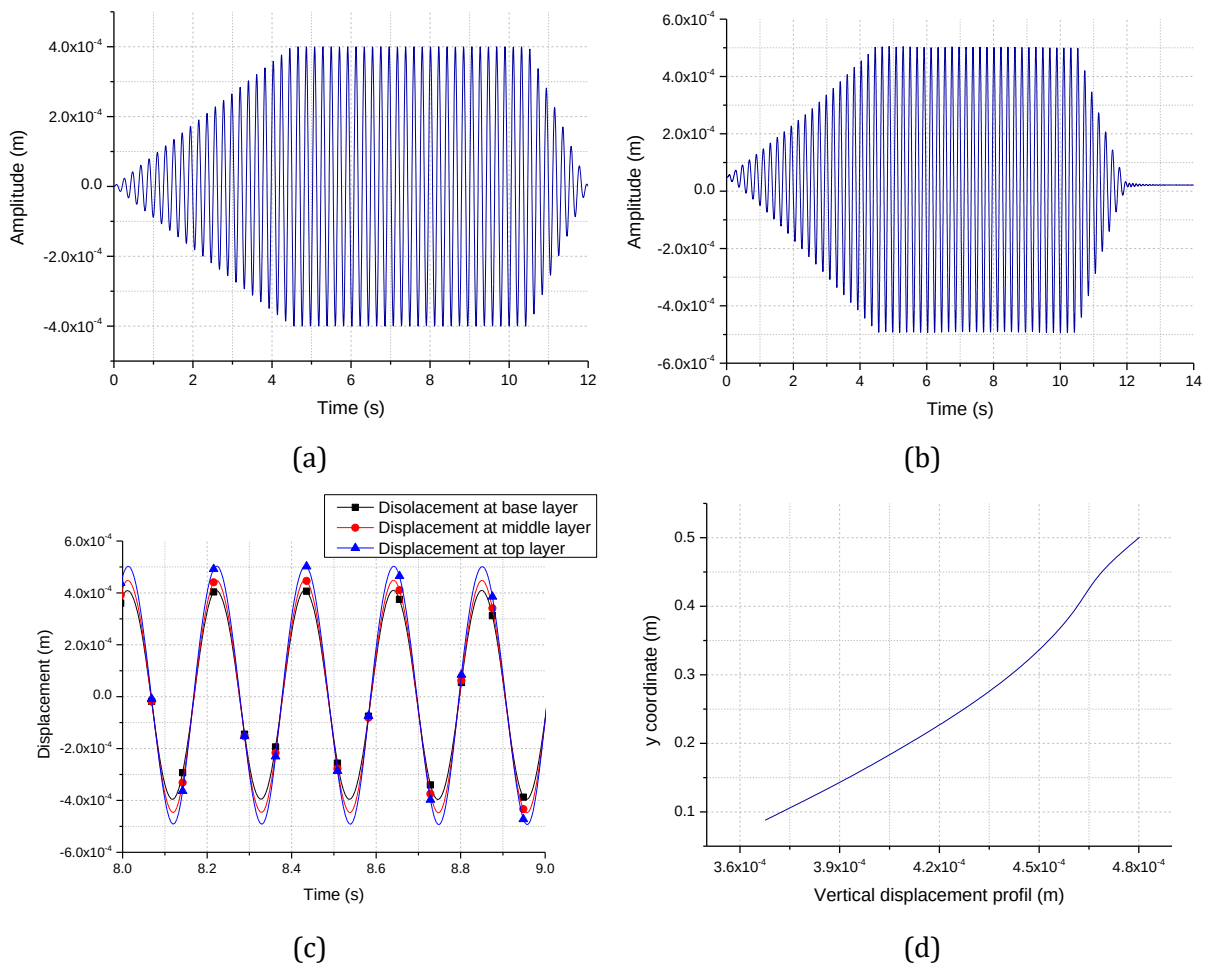


Fig. 5. Displacement response: (a) Displacement time history at the bedrock (excitation), (b) Displacement time history at the top layer of the soil deposit, (c) Displacement time history at different depths in a part of the steady-state phase, (d) Vertical displacement profile vs. y coordinate

5.3 Equivalent Linear Elastic Continuum Model

This section consists of an inverse analysis to determine the properties of a linear elastic soil profile equivalent to the sand deposit under study. This analysis provides a better understanding of the behavior of the deposit under different vibration amplitudes and highlights the limitations of the analytical model based on continuum theory with linear elastic behavior in the modeling of deposits composed of naturally discontinuous materials such as sands. For consistency, the analytical model used has the same geometric dimensions as those adopted in the DEM model.

The analytical solution for the displacement of an elastic deposit subjected to harmonic excitation at the base is given by Eq. 12. This analysis aims to determine the wave propagation velocity (V_p) to be used in this solution, such that the vibration amplitude at the free surface matches that obtained from the discrete-element model. The analysis examines four base excitation amplitudes, three moderate amplitudes and one large amplitude, specifically $u_0 = 2 \times 10^{-4}m$, $u_0 = 4 \times 10^{-4}m$, $u_0 = 8 \times 10^{-4}m$ and $u_0 = 8 \times 10^{-3}m$. In all cases, the excitation frequency is kept constant at $30rad/s$.

Figure 6 shows the time history displacement in the steady-state vibration phase for the four applied amplitudes. For excitation amplitudes $2 \times 10^{-4}m$, $4 \times 10^{-4}m$ and $8 \times 10^{-4}m$ (figures. 7a, 6b and 6c), the numerical displacement and the analytical solution calibrated with the wave

propagation velocity V_p are presented on the same graphs. For excitation amplitude $8 \times 10^{-3}m$ (Fig. 6d), only the numerical displacement is shown, as the analytical solution does not allow obtaining such numerical displacement shape.

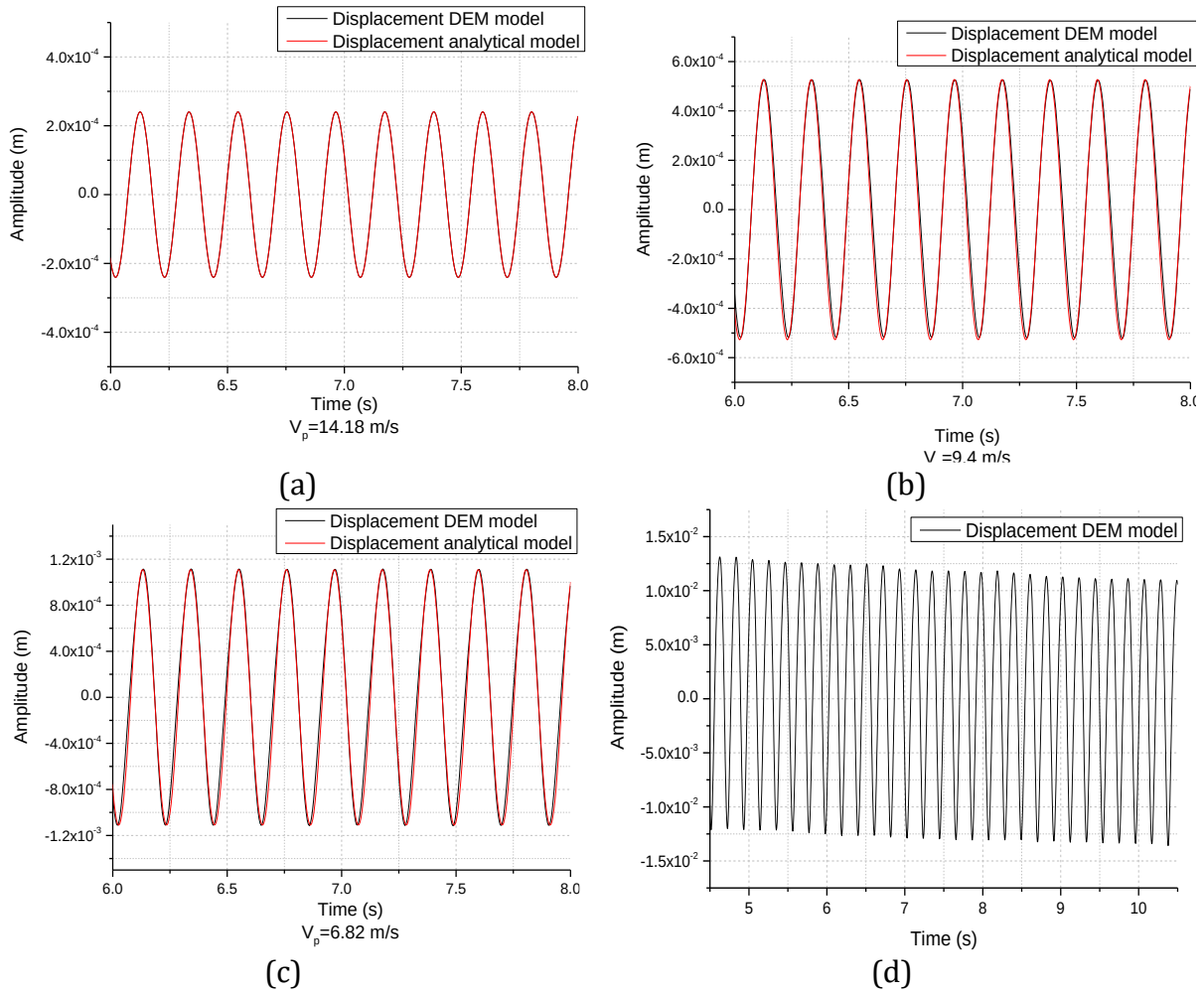


Fig. 6. Displacements at the free surface layer from the DEM model and calibrated analytical solution with different excitation amplitude: (a) $u_0 = 2 \times 10^{-4}m$, (b) $u_0 = 4 \times 10^{-4}m$, (c) $u_0 = 8 \times 10^{-4}m$, (d) $u_0 = 8 \times 10^{-3}m$

The wave propagation velocities (V_p) determined so that the analytical solution yields displacement amplitudes similar to those of the numerical model are $14.18m/s$, $9.4m/s$ and $6.82m/s$ respectively, for excitation amplitudes of $2 \times 10^{-4}m$, $4 \times 10^{-4}m$ and $8 \times 10^{-4}m$. The material's modulus of elasticity can be calculated from the wave propagation velocity (V_p) using Eq. 11. The density (ρ) in this equation can be regarded as the average density of the deposit, calculated as the ratio of the total mass of the grains to the volume of the deposit; it is approximately $\rho = 1996.77kg/m^3$. The elastic moduli of the equivalent linear elastic deposits, corresponding to the three excitation amplitudes of $2 \times 10^{-4}m$, $4 \times 10^{-4}m$ and $8 \times 10^{-4}m$, are 401.49×10^3Pa , 176.43×10^3Pa and 92.87×10^3Pa , respectively. This result shows that the equivalent linear elastic model becomes more flexible as the excitation amplitude increases, indicating that the deposit exhibits increasingly soft behavior at larger strains. This result is consistent with experimental observations on the cyclic behavior of granular soils, where the moduli of deformation decrease as the strain level increases [2,3,20]. For the large amplitude, i.e. $u_0 = 8 \times 10^{-3}m$, in addition to a more pronounced amplification of surface displacement, harmonic oscillation is maintained but around a downward-sloping mean line; this observation indicates that, at this amplitude level, a deposit densification process is triggered, which causes the surface layer to shift downward.

5.4 Fundamental Frequency of The Deposit

The fundamental frequency (first natural frequency) of the deposit, is estimated in this work from the free oscillation observed after the excitation stops. This vibration has a very low amplitude, so it can be assumed to be a damped elastic vibration. Figure 7 shows the free vibration displacement in the top layer after the excitation has ended. The maximum displacement amplitude applied during the excitation phase is $4 \times 10^{-4}m$. The free oscillation period of the deposit can be estimated directly from this plot and is about $T = 0.1054\text{ s}$. The circular fundamental frequency is therefore: $\omega_1 = \frac{2\pi}{T} = 59.62\text{rad/s}$.

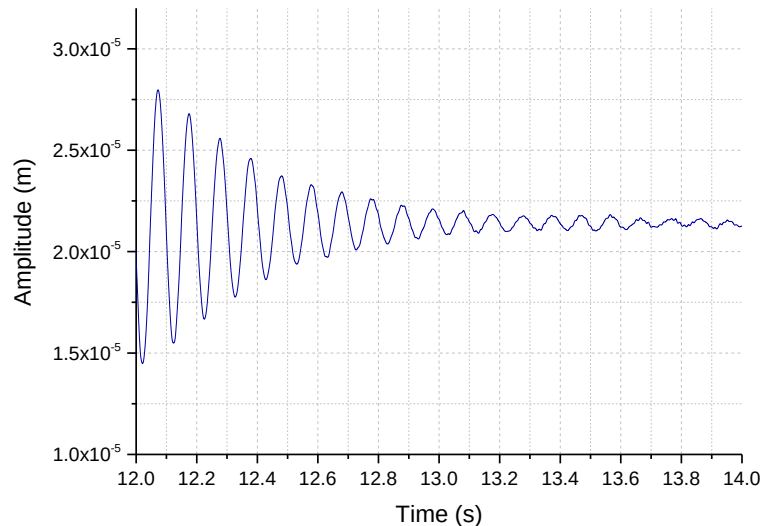


Fig. 7. Detailed view of response at the top layer during free vibration

The compression wave velocity can be estimated using the fundamental period through the relation $V_p = \frac{4h}{T}$ [19]. Accordingly, for our deposit model: $V_p = 20.49\text{m/s}$. The corresponding modulus of elasticity, calculated using Eq. 11, is $838.32 \times 10^3\text{Pa}$; this is the maximum modulus of elasticity, which is characteristic of the material's behavior under very small strains. Once again, this result clearly confirms the concept of degradation of deformation modulus. Indeed, it can be observed that the moduli obtained in the inverse analysis performed earlier (in the previous section), are all lower than the maximum modulus calculated based on very small deformations.

5.5 Resonance Frequency of The Deposit

The resonance frequency is estimated based on a frequency sweep, in which the excitation frequency is gradually varied to detect resonance. This frequency should be theoretically, very close to the fundamental frequency for sufficiently low amplitudes. Figure 8 shows the variation in displacement amplification during steady-state vibration, as a function of excitation frequency ranging from $\omega = 20\text{ rad/s}$ to 65 rad/s . Amplification is quantified here by the ratio (u/u_0) , where u_0 is the excitation amplitude at the base, taken as $4 \times 10^{-4}m$ in this case.

It is clear that the amplification ratio follows the typical trend observed in elastic systems subjected to harmonic excitation i.e., as the excitation frequency increases, the amplification gradually increases, reaches a peak at resonance, and then decreases. The resonance frequency identified from the graph is approximately 57 rad/s . This frequency is slightly lower than the fundamental frequency estimated above, which may be due to two parameters, namely: (1) the presence of damping introduces a shift between the natural frequency and the resonance frequency, and (2) the possible degradation of the deformation modulus due to large strains during resonance.

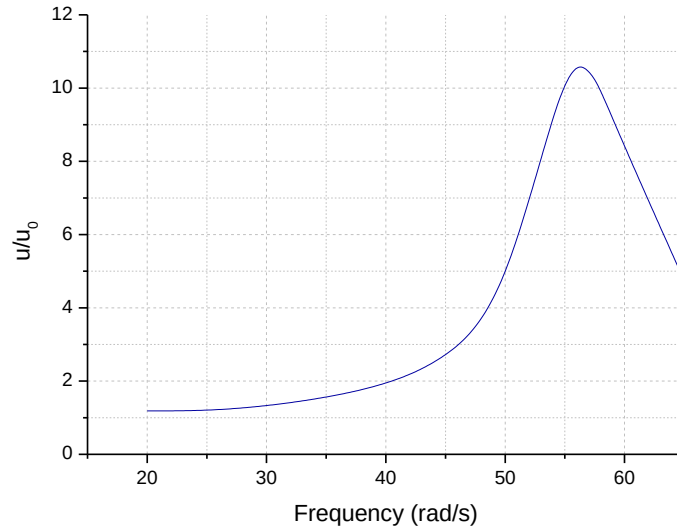


Fig. 8. Displacement amplification vs. excitation frequency

5.6 Influence of Strain Level on Displacement Amplification

It is worth recalling that, for an elastic deposit subjected to harmonic excitation, the dynamic amplification ratio at the free surface (u/u_0) is theoretically independent of the excitation amplitude. However, since, in most cases, a granular material does not exhibit purely elastic behavior, it is important to study the response of the deposit to different levels of strain. To this end, the specimen is subjected to a harmonic excitation with the shape described above, with a fixed frequency of 30 rad/s and different amplitudes ranging from 4×10^{-6} m to 3×10^{-3} m.

For frequency excitation of 30 rad/s, the displacement profile is nearly linear (Fig. 5d), which allows the vertical normal strain (ε_c) to be approximated by the ratio of the difference in vertical displacement between two layers to their vertical spacing. To estimate average value over the deposit, we used here the penultimate layer (layer 6) and the second layer. Therefore, the average vertical strain is expressed as:

$$\varepsilon_y = \frac{u_6^p - u_2}{y_6 - y_2} \quad (13)$$

In this expression, u_6^p represents the peak displacement of layer 6, u_2 the displacement of layer 2 at the same instant (corresponding to u_6^p), and y_i the vertical coordinate of the center of layer i . Average vertical strain levels are computed for all excitation amplitudes in the steady-state vibration phase.

Figure 9a shows how the level of vertical normal strain varies with excitation amplitude; the axes of the graph are plotted on a logarithmic scale to accommodate the wide range of values. This graph shows that the strain level increases as the excitation amplitude goes up. As for the rate of change, although it is nearly constant, it shows a slight increase as the excitation amplitude gets higher. Figure 9b displays the evolution of the displacement amplification ratio at the free surface (u/u_0) as a function of the vertical strain level induced by different excitation amplitudes. This graph shows that the amplification remains nearly constant for small strains ($\varepsilon_y \leq 4 \times 10^{-5}$). This observation indicates that below this strain level, the deposit behaves almost like a linear elastic medium. Above this strain level, the amplification increases with strain; this may be explained by the tendency of the deposit to become increasingly soft as the strain level increases. This tendency could be attributed to the onset of nonlinear behavior caused by intergranular slippage and grain rotation.

It is worth noting that the critical strain value—which separates the two types of behavior (linear and nonlinear)—obtained in this study is about $\varepsilon_y \approx 4 \times 10^{-5}$ (see Fig. 9b). Besides, Kokusho [2], in his experimental study on the cyclic behavior of soils using the triaxial device, has obtained the

degradation curve of the deformation modulus shown in Figure 10. It can observe on this curve, that the critical strain, which marks the effective degradation of the deformation modulus, is on the order of 5×10^{-5} . It is therefore clear that the critical strain level corresponding to the onset of degradation of the deformation modulus in the present numerical study is in good agreement with Kokusho's experimental result.

Finally, this study showed that the degradation of the oedometeric deformation modulus under cyclic loading is similar to that of the shear modulus. Furthermore, the critical strain value that triggers the effective degradation of the oedometeric modulus is of the same order as that corresponding to the shear modulus, which is on the order of 10^{-5} to 10^{-4} [2,3,20,28].

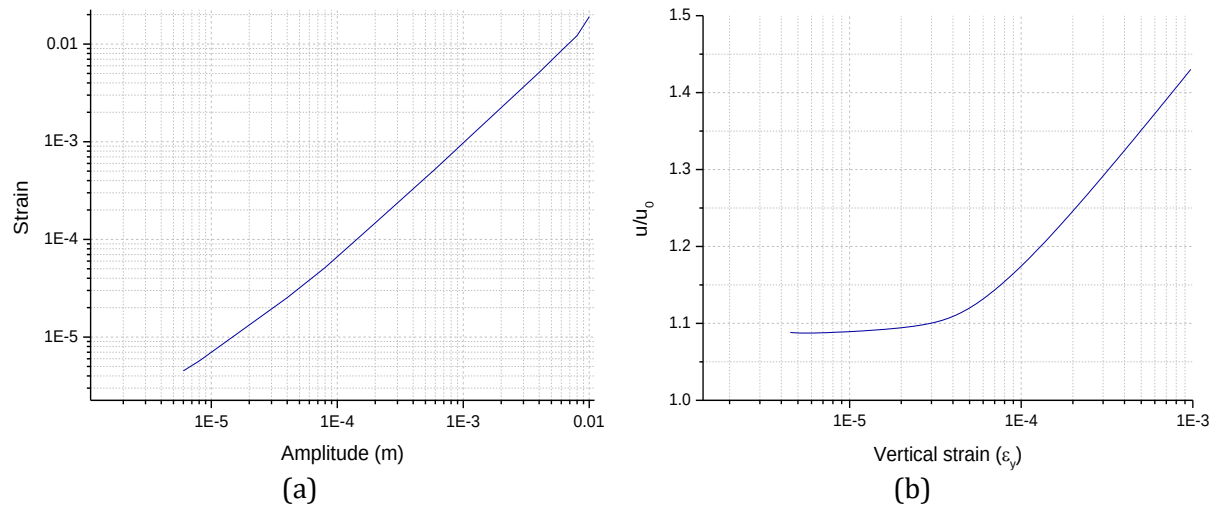


Fig. 9. (a) Average vertical normal strain level vs. excitation amplitude; (b) Displacement amplification ratio vs. vertical normal strain induced by various excitation amplitudes

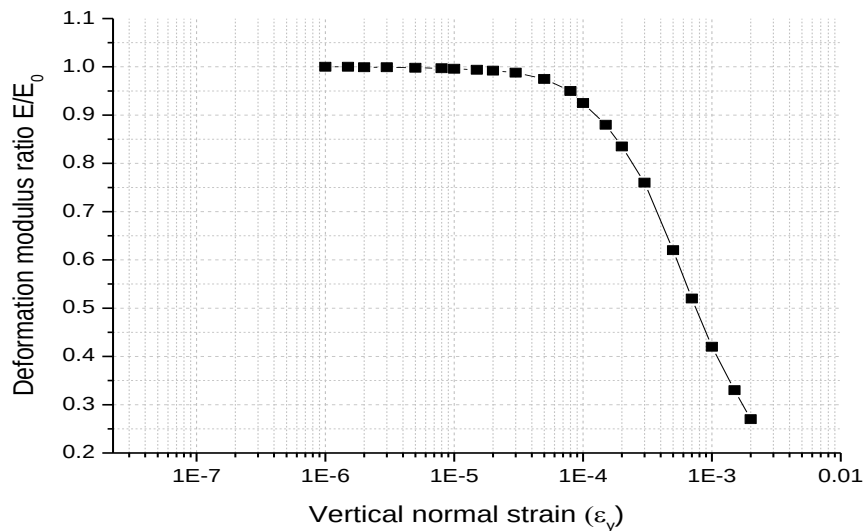


Fig. 10. Soil deformation modulus degradation under cyclic loading. Obtained by Kokusho [2] from cyclic triaxial tests

6. Conclusions

This work aimed to investigate the mechanisms involved in the propagation of compression waves in a soil deposit at different strain levels. To this end, we analyzed the propagation of compressional motion in a granular deposit excited at its base by a sinusoidal vertical motion. The problem is approached using the 2D discrete element model. The boundary conditions used are a controlled displacement condition at the base, a free boundary on the top surface, and periodic type boundaries on the lateral vertical boundaries to model the infinite aspect of the deposit's horizontal dimension.

This study first allowed us to evaluate the fundamental frequency and the maximum deformation modulus corresponding to a low level of deformation. Secondly, it was shown that for excitation frequencies below the fundamental frequency, the motion is amplified from bottom to top up to the free surface, meaning that the vibration occurs in the first mode. The inverse analysis performed showed that for moderate excitation amplitudes, it is possible to determine a linear elastic model equivalent to the granular deposit. It was observed that the equivalent elastic deformation modulus decreases with increasing amplitude. However, for high amplitudes, it was not possible to obtain an equivalent elastic model due to the densification of the deposit during vibration. Analysis of the amplification evolution as a function of excitation amplitude, and consequently as a function of strain level, showed that for low strain levels, the amplification is almost constant. However, beyond a certain critical strain value, the amplification increases with increasing strain amplitude. The value of this critical strain is in very good agreement with that which triggers the degradation of the deformation modulus, a phenomenon well-known in the literature. Furthermore, it is noted that this value, which corresponds to compressive stress, is of the same order of magnitude as that of shear stress.

Appendix: Table of Symbols

Symbol	Definition	Description	Unit
$\vec{x}_i$	Position vector	Position of grain i	m
$\vec{\ddot{x}}_i$	Acceleration vector	Translational acceleration of grain i	m/s ²
$\vec{\varphi}_i$	Rotation vector	Angular displacement of grain i	rad
$\vec{\ddot{\varphi}}_i$	Angular acceleration	Rotational acceleration of grain i	rad/s ²
m_i	Mass	Mass of grain i	kg
I_i	Moment of inertia	Moment of inertia of grain i	kg·m ²
$\vec{F}_{ij}^{contact}$	Contact force	Force exerted by grain j on grain i	N
$\vec{F}_n$	Normal contact force	Normal component of contact force	N
$\vec{F}_s$	Tangential contact force	Tangential component of contact force	N
D_n	Normal overlap	Normal contact overlap between grains	m
D_s	Tangential displacement	Accumulated tangential displacement	m
V_n	Normal relative velocity	Normal relative velocity at contact	m/s
V_s	Tangential relative velocity	Tangential relative velocity at contact	m/s
k_n	Normal stiffness	Normal contact stiffness	N/m
k_s	Tangential stiffness	Tangential contact stiffness	N/m
v_n	Normal damping coefficient	Viscous damping coefficient in normal direction	kg/s
v_s	Tangential damping coefficient	Viscous damping coefficient in tangential direction	kg/s
μ_s	Static friction coefficient	Static interparticle friction coefficient	–
μ_d	Dynamic friction coefficient	Dynamic interparticle friction coefficient	–
M_r	Rolling resistance moment	Moment resisting rolling at contact	N·m
$u(z, t)$	Displacement	Vertical displacement at depth z and time t	m
u_0	Excitation amplitude	Amplitude of applied harmonic displacement	m
ω	Angular frequency	Excitation circular frequency	rad/s
E	Oedometric modulus	Dynamic compression modulus of soil	Pa
ρ	Density	Bulk density of granular soil	kg/m ³
h	Soil layer depth	Total thickness of the soil deposit	m

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Analytical predictive modeling of the mechanical and thermal characteristics of concrete walls altered by pumice powder

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The transition toward energy-efficient buildings requires integrated thermo-mechanical design rather than purely structural assessment. Construction materials must simultaneously govern stiffness, thermal expansion, and heat transfer without compromising stability. Partial cement replacement enables tailored performance. Pumice powder (PP), rich in amorphous silica, undergoes pozzolanic reactions that refine the concrete microstructure, influencing both mechanical stiffness and thermal conductivity; however, a unified analytical framework quantifying its coupled thermo-mechanical response at the structural level remains lacking. To address this, an analytical framework is developed to evaluate the influence of PP on effective thermo-elastic properties. Homogenized elastic properties are derived using a two-phase model, while thermal parameters are calculated with the Maxwell–Eucken approach. Both sets of properties are incorporated into a refined High-Order Deformation plate Theory (RPT) to determine the critical mechanical buckling load and the critical thermal buckling temperature of structural walls. Predictions are validated against independent experimental data from the literature. At 10% PP replacement, the elastic modulus decreases by approximately 18%, accompanied by a 19% reduction in the critical mechanical buckling load. In contrast, thermal performance improves: the thermal resistance increases by about 18–20%, while the thermal transmittance and heat flux decrease by approximately 17%. Additionally, the critical thermal buckling load increases by nearly 16%. These results quantify the trade-off between reduced stiffness and enhanced thermal stability, demonstrating how PP incorporation modifies the mechanical and thermal instability thresholds of concrete walls.

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1. Introduction

Concrete technology is undergoing a significant transformation driven by the twin imperatives of sustainable construction and mitigating global climate change. The Intergovernmental Panel on Climate Change (IPCC) emphasizes that urgent transitions in industrial sectors are required to limit global warming to 1.5°C, directly implicating the construction industry [1]. Portland cement

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production alone contributes approximately 7–8% of global CO₂ emissions, while the broader construction sector accounts for nearly 36% of global energy consumption and 30% of anthropogenic greenhouse gas emissions [2-4]. These challenges are closely aligned with the United Nations Sustainable Development Goals (SDGs), particularly SDG 9 (Industry, Innovation, and Infrastructure) and SDG 13 (Climate Action), which promote resource-use efficiency and the adoption of environmentally sound technologies [5].

In response, the Global Cement and Concrete Association (GCCA) has pledged a 25% reduction in CO₂ emissions by 2030, as a key step toward achieving “Net Zero” concrete by 2050 [6]. This roadmap focuses on reducing clinker intensity, optimizing concrete mix designs, and accelerating the use of low-carbon supplementary materials, which account for nearly 60% of global cement and concrete demand (Fig. 1). These environmental imperatives have driven research into alternative binders and supplementary cementitious materials (SCMs), such as volcanic pozzolans and lightweight aggregates, which can reduce the carbon footprint while improving thermal performance and maintaining structural integrity [7, 8]. Partial replacement of cement with natural pozzolanic additives has demonstrated technical feasibility and notable environmental benefits [9-12]. In addition, micro- and nanoscale reinforcements have shown potential to enhance stiffness, thermal behavior, and long-term durability, enabling the design of energy-efficient concrete structures that meet both mechanical and thermal performance requirements under variable service conditions [13-18].

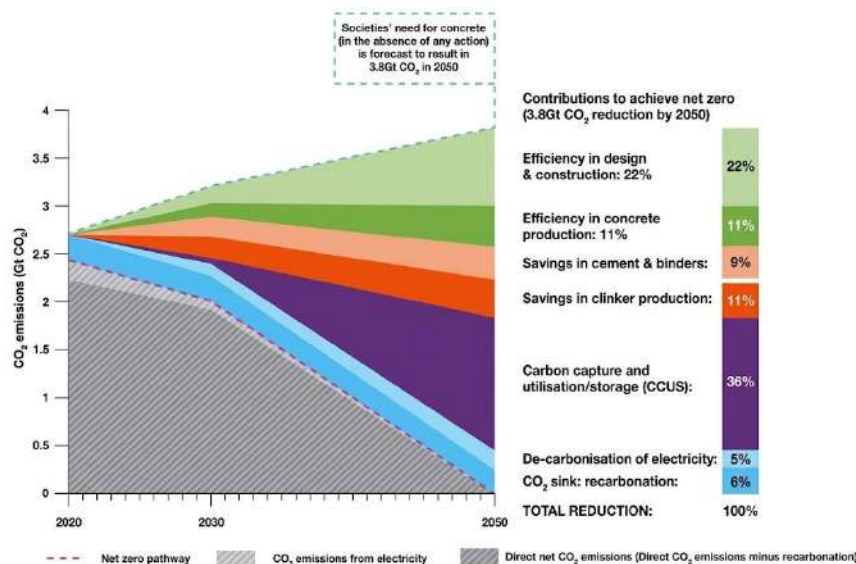


Fig. 1. Roadmap to Net Zero: Projected CO₂ emissions reduction targets for the global cement and concrete sector (2020–2050) [6]

While many supplementary cementitious materials (SCMs) primarily enhance concrete through chemical reactivity, pumice powder (PP) is distinguished by its porous cellular structure and inherent pozzolanic activity, which simultaneously reduce concrete weight and improve thermal performance. These unique characteristics make PP particularly suitable for exploring how microstructural modifications influence both material-level properties and overall structural behavior. As a result, recent studies have highlighted that PP, which can range from nanometric to macroscopic particle sizes, can substantially influence the mechanical and thermal properties of concrete, highlighting its potential as a multi-scale reinforcement for high-performance eco-concretes. This multi-scale behavior is conceptually similar to other studies that have explored nanoscale and microscale additives, where homogenization and multi-phase modeling approaches were employed to capture particle-matrix interactions and predict enhanced stiffness, thermal conductivity, and buckling resistance. For example, Harrat et al. [19] utilized the Voigt homogenization model to examine nano-silica-reinforced concrete beams, demonstrating significant enhancements in stiffness and load-bearing capacity. Chatbi et al. [20] advanced this methodology by utilizing a sophisticated two-dimensional Voigt framework for concrete slabs, emphasizing the necessity of precisely capturing planar stress distributions to effectively predict

deflections. Complementary studies have utilized the Eshelby tensor to examine particle-matrix interactions in heterogeneous composites, such as glass powder- or metal oxide-reinforced panels, demonstrating that nanoscale inclusions can substantially alter both elastic moduli and thermal expansion coefficients. Dine El Hannani et al. [21] emphasized the impact of nanoparticle type and concentration on buckling resistance and free vibration properties, whereas Kecir et al. [22] and Harrat et al. [23, 24] validated that iron nanoparticle reinforcement improves flexural rigidity under thermo-mechanical stress. Recent studies have expanded homogenization models to encompass natural and engineered microscale additives, such as wheat straw and recycled glass, utilizing Liwes, Nielsen Spar, and Maxwell–Eucken formulations to forecast integrated mechanical–thermal responses [25]. These investigations collectively demonstrate the critical role of multiscale modeling in engineering concrete with tailored stiffness and thermal conductivity properties, thereby providing a methodological framework for the application of pumice powder as an environmentally sustainable pozzolanic additive.

While nanoscale reinforcements provide targeted performance improvements, geological additives remain highly attractive due to their cost-effectiveness, sustainability, and multi-functional benefits. Among these, pumice powder (PP) has been shown to improve mechanical and thermal characteristics of concrete through both aggregate replacement and partial cement substitution. Cavaleri et al. [26] experimentally demonstrated that PP aggregates can reduce concrete density while maintaining structural integrity. Granata [27] highlighted that the pozzolanic reactivity of PP decreases voids and enhances strength in self-compacting concretes. Kurt et al. [28] reported that PP improves thermal insulation and reduces weight, although excessive content can negatively affect segregation and hydration kinetics. Zeyad et al. [29–31] further investigated volcanic pumice dust in geopolymer and high-strength concretes, showing enhancements in compressive and flexural strengths, reduction in water absorption and porosity, and benefits under self-curing and hot-weather conditions. Hossain [32] demonstrated that volcanic PP can produce blended cements with extended setting times, low heat of hydration, and mechanical performance comparable to fly ash cement for up to 20% replacement. Karthika et al. [33] confirmed that partially replacing coarse aggregates with PP at 50–100% reduces self-weight, decreases dead load, and improves thermal insulation, while maintaining favorable mechanical and durability performance. A comprehensive review by Moolchandani [34] synthesized over 60 studies, emphasizing that PP's high amorphous silica content and porous morphology provide intrinsic pozzolanic reactivity, internal curing potential, and long-term durability. The review also quantified late-age compressive strength gains of 6–11%, reduced chloride permeability by 22–35%, improved sulfate resistance by up to 28%, and demonstrated significant reductions in thermal conductivity (30–56%) and surface temperatures (up to 42%) when combined with phase-change materials. Ghanim et al. [35] confirmed that textile-industry-derived PP possesses higher pozzolanic potential than fly ash, improving early-age workability and mechanical properties, with 25% replacement identified as optimal. Collectively, these studies demonstrate that PP's chemical reactivity, porous microstructure, and internal curing capacity simultaneously enhance mechanical performance, durability, and thermal efficiency, making it a highly suitable additive for sustainable, energy-efficient concrete walls.

To elucidate the enhancements in mechanical, thermal, and durability properties observed in PP-modified concrete, it is crucial to examine the underlying chemical and microstructural mechanisms. The pozzolanic activity of pumice powder (PP) is primarily driven by its high content of amorphous silica (SiO_2) and alumina (Al_2O_3). The SiO_2 reacts with calcium hydroxide (Ca(OH)_2) during cement hydration to form additional calcium silicate hydrate (C-S-H) gel, which is the main contributor to mechanical strength and microstructural densification [36, 37]. Simultaneously, Al_2O_3 interacts with available calcium and sulfate ions to promote the formation of calcium aluminosilicate hydrates (C-A-S-H) and ettringite [38]. These secondary hydration products effectively fill the micro-voids within the interfacial transition zone (ITZ), the typically porous and mechanically weaker region between the cement paste and pumice aggregates [39]. By refining the ITZ, the Al_2O_3 -driven reactions enhance stress transfer across the composite phases, mitigating the strength reduction often associated with lightweight aggregates [40]. From a thermal perspective, the densified ITZ decreases the presence of large interconnected pores at the particle-matrix

interface. Although this slightly increases localized solid-to-solid contact, the combination of a high silica-to-calcium ratio and the intrinsic cellular porosity of pumice remains the dominant factor in lowering the effective thermal conductivity of the concrete [41, 42].

The chemical composition of pumice, as summarized in Table 1, includes minor oxides such as Fe_2O_3 , Na_2O , and K_2O , which can influence setting time, workability, and thermal behavior. These compositional characteristics, together with the pozzolanic activity of SiO_2 and Al_2O_3 , support the use of PP as both a partial cement replacement and a lightweight aggregate, enabling concrete walls with optimized mechanical and thermal performance.

Table 1. Chemical composition (wt%) of different pumice specimens reported in literature.

Composition	Ref	SiO_2	Al_2O_3	Na_2O	K_2O	Fe_2O_3	CaO	MgO	P_2O_5
Pumice specimen 1	Xiyu et al. [43]	72.06	9.39	4.29	4.04	3.40	0.25	0.02	0.002
Pumice specimen 2	Shi et al. [44]	68.64	11.92	5.53	4.93	3.45	0.73	0.10	0.06
Pumice specimen 3	Turhan et al. [45]	75.51	9.94	2.04	5.12	1.10	0.25	0.04	<0.001

Physically, the performance of pumice powder and pumice-based aggregates is strongly influenced by the highly porous and vesicular morphology of the material. As illustrated in Fig. 2, the SEM micrographs reveal a multi-scale network of interconnected voids and thin-walled cavities, which act as natural reservoirs for internal curing water. This intrinsic porosity enables gradual moisture release during hydration, mitigating autogenous shrinkage and promoting continuous formation of hydration products. The low density of pumice further contributes to reducing the self-weight of concrete elements while enhancing thermal insulation due to the presence of air-filled pores. Additionally, the irregular surface texture improves mechanical interlocking within the interfacial transition zone (ITZ), resulting in better stress transfer under mechanical loading and more uniform thermal diffusion. Combined, these physical features provide a mechanistic explanation for the reductions in thermal conductivity, enhanced durability, and improved long-term mechanical performance observed in PP-modified concrete [43].

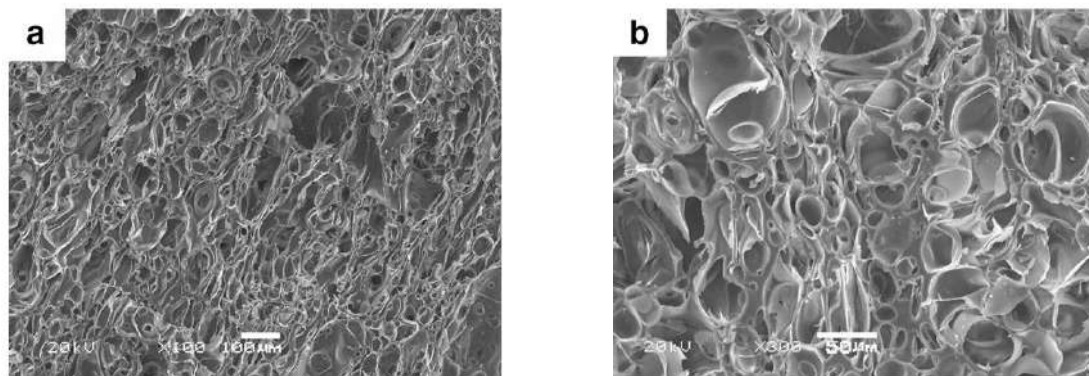


Fig. 2. Scanning electron microscopy (SEM) images showing the morphology of pumice at magnifications of : (a) 100× and (b) 300×[43]

To consolidate the insights obtained from recent works on the use of pumice powder (PP) in cementitious systems, Table 2 presents a structured synthesis of the main physical, chemical, and thermal effects reported in the literature. As shown in the table, PP influences concrete behavior on several interconnected levels: it alters density and porosity, affects fresh and hardened properties, contributes to hydration kinetics and pozzolanic reactions, and promotes microstructural refinement through the formation of additional C–S–H. These transformations also translate into improved thermal performance, particularly in terms of heat moderation, insulation capacity, and resistance to temperature-induced degradation. By gathering these findings into a single framework, Table 2 provides a clear understanding of how PP contributes to performance enhancement across multiple scales, thereby reinforcing its relevance as a sustainable supplementary cementitious material. This consolidated evidence forms the basis for the present study, which aims to analytically examine the coupled mechanical and thermal response of concrete walls incorporating PP.

Table 2. Summary of physical, chemical, and thermal effects of pumice powder (PP) in cementitious materials and concrete as reported in recent studies

Ref	PP Type	Physical Effects	Chemical Effects / Pozzolanic Activity	Thermal / Microstructural Influence
[46]	Saturated PP, 0.6–1.25 mm, partial sand replacement	Reduced persistent shrinkage, internal curing effect	Accelerated early hydration, promoted secondary hydration of active mineral admixtures; modified C-S-H composition (Ca/Si ratio)	Enhanced microstructure in local effective area; improved temperature stability via denser hydration products
[47]	PP blended cement, up to 30% cement replacement	Shorter setting time despite lower clinker, lower workability	Pozzolanic reaction contributes to compressive strength, chemical composition correlates with strength	Minor thermal effect noted; influence on hydration heat not detailed
[48]	PP as cementitious material or lightweight aggregate	Reduced unit weight, improved thermal and acoustic insulation, decreased drying shrinkage, increased fire resistance	Pozzolanic activity decreases porosity, mitigates ASR expansion	Lower hydration heat, improved thermal stability, packed microstructure reduces permeability
[49]	PP as aggregate or cement substitute, 5–25%	Improved strength (especially 10–25% PP), lower water absorption, reduced porosity	Pozzolanic activity strongest at 28–90 days; enhances long-term durability	Packed pore structure, better heat transfer and thermal insulation in SCMs
[50]	PP, 25% replacement in geopolymer concrete	Decreased density, lower compressive strength at higher content	Pozzolanic reaction limited in high-porosity PP; contributes to durability via pore structure refinement	SEM shows denser microstructure in optimal mixes; thermal stability improved at moderate replacement
[51]	Ultrafine PP in HPC, binary/ternary mixes	Higher electrical resistivity, low permeability, enhanced long-term durability	Strong pozzolanic contribution, interacts with other SCMs to enhance compressive strength	Improved corrosion resistance under thermal stress; thermal insulation improved via dense microstructure
[52]	PP + polypropylene fibers, 10–40% cement replacement	Enhanced compressive and tensile strength, better ductility	Cementitious characteristics of PP; contributes to strength via pozzolanic activity	Microstructure densification improves thermal and mechanical performance
[36]	PP replacement 5–25%	Reduced density, delayed setting, decreased workability	$\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 > 70\%$; pozzolanic, enhances C-S-H formation	Improved microstructural cohesion; moderate thermal effect via lower hydration heat
[53]	PP, 3–18% cement replacement	Enhanced CS, FS, STS, and MOE up to 12%; decreased strength beyond	Pozzolanic reaction forms additional C-S-H; contributes to mechanical property improvement	Thermal stability improved indirectly via denser microstructure; validated via RSM optimization
[54]	PP, 15–50% cement replacement	Strength comparable to control up to 25%; density reduction	Pozzolanic constituents enhance C-S-H formation and durability	Microstructure improved; XRD confirms mineral phases conducive to long-term stability

The data compiled in Table 2 demonstrate that pumice powder influences concrete at multiple levels, including physical, chemical, and thermal aspects. PP alters density, porosity, and shrinkage

behavior while contributing to hydration kinetics and pozzolanic reactions that enhance microstructural cohesion and long-term durability. Thermal performance is also affected, with improvements in insulation, heat transfer, and stability under temperature variations. However, the magnitude and timing of these effects vary widely depending on PP type, replacement ratio, particle size, and concrete mix, making it challenging to generalize or predict wall-level behavior from individual experimental results. This variability highlights a critical need for an analytical framework capable of systematically linking PP content, effective thermo-mechanical properties, and structural response. Such a framework would allow prediction of mechanical stiffness, thermal response, and instability thresholds of PP-modified concrete walls, bridging the gap between material-level observations and design-scale applications.

At the structural level, translating these material-level trends into reliable predictions of wall behavior further requires an appropriate mechanical modeling framework. Given the multi-scale effects induced by PP, ranging from microstructural densification to modifications in thermal transport, predicting the thermo-mechanical response of concrete walls demands analytical models capable of capturing bending, shear, and temperature-driven gradients with greater fidelity than classical formulations. Because PP alters density, stiffness, porosity, and thermal conductivity, the resulting wall behaves as a heterogeneous and thermally sensitive system, rendering classical plate assumptions inadequate. Classical plate theory (Kirchhoff–Love), which neglects transverse shear deformation, often fails to represent the behavior of moderately thick structural elements, especially when temperature gradients induce significant through thickness stresses [55]. These shortcomings have motivated the development of refined plate and shell theories. First order shear deformation theory (FSDT), introduced by Mindlin and Reissner, enhances realism by incorporating shear deformation; however, it relies on shear correction factors that introduce uncertainty when applied to materials whose internal structure has been modified by lightweight pozzolanic inclusions such as PP [56]. To overcome these limitations, higher order shear deformation theories (HSDT), as developed by Reddy [57], Touratier [58], and Karama [59], provide more accurate displacement fields by naturally capturing the parabolic distribution of shear strains without correction factors, making them particularly suitable for non-homogeneous cementitious composites.

The effectiveness of these refined theories is well demonstrated in the literature for functionally graded, layered, and thermally responsive materials, which share notable similarities with PP blended concrete. For instance, Lee et al. [60] studied thermoelastic bending of plates with temperature dependent properties, while Sun et al. [61] examined thermal buckling in heterogeneous plates, emphasizing the strong role of thermal loading in stability behavior. Prakash and Ganapathi [62] further highlighted the influence of coupled thermal mechanical effects on vibration responses. More recent contributions by Zenkour [63], Yang et al. [64], Niknam and Akbarzadeh [65] and Mallios et al. [66] confirmed the superior predictive capacity of HSDT and FEM formulations for plates with spatially varying stiffness and conductivity, conditions directly representative of PP modified concrete. Additionally, Sayyad and Ghugal [67-71] demonstrated that refined trigonometric theories provide high accuracy for thick concrete like plates, especially under non uniform thermal fields.

Building on the observed material-level effects of PP, the present work proposes a comprehensive analytical framework that systematically links PP incorporation to both material and structural performance of concrete walls. The study first evaluates the influence of PP content (2.5–10 wt%) on the effective elastic properties, including Young's modulus, and thermal behavior, including expansion coefficient and thermal conductivity, establishing quantitative relationships for PP-modified concrete as a construction material. These homogenized properties are then incorporated into a refined High-Order Deformation Plate Theory (RPT) to simulate wall behavior under mechanical and thermal loads. Mechanical stability is analyzed through uniaxial and biaxial critical buckling loads, considering variations in wall slenderness (length-to-thickness ratio). Thermal effects are further incorporated by evaluating temperature distributions across the wall, enabling calculation of the thermal critical buckling temperature. In addition, thermal performance indicators, including thermal resistance (R), thermal transmittance (U), and heat flux (ϕ), are quantified for walls of varying thickness and PP content. By simultaneously addressing elastic,

thermal, and stability characteristics, this framework provides a unified predictive tool to assess how PP modifies both material behavior and structural response, offering a basis for optimized wall design in energy-efficient and mechanically reliable constructions.

2. Analytical Framework for Evaluating Thermo-Mechanical Properties of PP-Modified Concrete Walls

The analytical formulation of the pumice powder (PP)-modified concrete wall is established based on the kinematic and constitutive assumptions defined within the proposed homogenization framework. The study considers a simply supported wall of dimensions $a \times b \times h_w$, incorporating PP inclusions to assess their influence on the coupled thermo-mechanical response. A refined plate deformation theory is employed to capture shear deformation effects and through-thickness temperature variations accurately.

The geometrical configuration of the PP-modified concrete wall is illustrated in Fig. 3, defined by its length (a), width (b), and thickness (h_w). The wall is modeled as a rectangular plate with all edges simply supported. This boundary condition is selected because it provides a mechanically conservative and analytically tractable configuration that is widely adopted in stability analyses of wall and slab systems. In practice, many load-bearing and infill walls exhibit rotational restraint with limited moment continuity at supports, making the simply supported assumption a rational idealization for evaluating critical buckling behavior while avoiding overestimation of structural stiffness.

Thermal loading is applied through a temperature gradient across the thickness, representing heat transfer from the exterior environment toward the interior space (see Fig. 3), consistent with building envelope conditions under solar exposure or seasonal temperature variations. This through-thickness thermal distribution induces in-plane thermal stresses due to restrained expansion, which may contribute to thermal buckling instability.

2.1 Kinematical Formulation of the PP-Modified Concrete Slab

Assuming small deformations and linear thermo-elastic behavior, the total strain field of the PP-modified concrete wall is represented as the superposition of mechanical and thermal components. The constitutive relationships are formulated using the homogenized elastic moduli and the effective coefficients of thermal expansion derived from the two-phase homogenization scheme. In accordance with the refined plate theory, the displacement components along the x , y , and z directions (U , V , and W , respectively) are expressed as functions of the mid-plane displacements and higher-order shear deformation terms, as follows [72]:

$$U(x, y, z) = u_0(x, y) - z \frac{\partial w_b(x, y)}{\partial x} + f(z) \frac{\partial w_s(x, y)}{\partial x} \quad (1.a)$$

$$V(x, y, z) = v_0(x, y) - z \frac{\partial w_b(x, y)}{\partial y} + f(z) \frac{\partial w_s(x, y)}{\partial y} \quad (1.b)$$

$$W(x, y, z) = w_b(x, y) + w_s(x, y) \quad (1.c)$$

The in-plane displacements of a point located on the mid-surface of the PP-modified concrete wall are denoted by $u_0(x, y)$ and $v_0(x, y)$ along the x and y directions, respectively, and represent the membrane deformation of the wall associated with in-plane stretching and compression under mechanical and thermal loads. The transverse displacement $w(x, y, z)$ is decomposed into two physically distinct components: $w_b(x, y)$, which represents the bending-induced displacement governing the global flexural response and buckling instability of the wall, and $w_s(x, y)$, which represents the transverse shear deformation associated with non-uniform through-thickness strain. The bending component w_b primarily controls the wall's response to in-plane mechanical loads and thermally induced membrane stresses, and therefore directly influences the critical mechanical and thermal buckling conditions. In contrast, the shear component w_s accounts for transverse shear flexibility, which becomes significant in moderately thick walls and in PP-modified concrete due to reduced stiffness and material heterogeneity. This component affects stress redistribution, deformation localization, and the overall stiffness of the wall, thereby

modifying the thermo-mechanical stability response, particularly under combined mechanical loading and temperature gradients. The thickness coordinate z is measured from the mid-plane, in which:

$$x \in (0, a); y \in (0, b); z \in (-h_w/2; h_w/2) \quad (2)$$

In this study, the higher-order shear deformation theory (HSDT) proposed by Mourad Chitour et al. [73] is adopted to accurately capture the nonlinear distribution of transverse shear strains across the slab's thickness without the need for a shear correction factor. Compared to classical plate theory, which neglects transverse shear, and first-order shear deformation theory (FSDT), which requires shear correction factors and may introduce inaccuracies for moderately thick or heterogeneous materials, HSDT provides a more realistic representation of shear effects in PP-modified concrete walls. The employed shape function $f(z)$ defines the variation of shear deformation through the slab's thickness and is expressed as:

$$f(z) = \frac{3}{35}\pi z \left(\pi - \sqrt[3]{0.135} \cosh\left(\frac{\pi z}{h}\right) \right); \quad g(z) = \frac{df(z)}{dz} \quad (3)$$

The constitutive stress-strain relationships governing the thermo-elastic response of the PP-modified concrete wall are formulated by incorporating both mechanical and thermal effects. The material is assumed to behave as a linear, homogeneous, and isotropic continuum characterized by the effective (homogenized) elastic constants obtained from the two-phase model.

$$\varepsilon_y = \frac{\partial V(x, y, z)}{\partial y} = \frac{\partial v_0}{\partial y} - z \frac{\partial^2 w_b}{\partial y^2} - f(z) \frac{\partial^2 w_s}{\partial y^2} \quad (4.a)$$

$$\gamma_{xy} = \frac{\partial U(x, y, z)}{\partial y} + \frac{\partial V(x, y, z)}{\partial x} = \left(\frac{\partial v_0}{\partial x} + \frac{\partial u_0}{\partial y} \right) - 2z \frac{\partial^2 w_b}{\partial x \partial y} - 2f(z) \frac{\partial^2 w_s}{\partial x \partial y} \quad (4.b)$$

$$\gamma_{xz} = \frac{\partial U(x, y, z)}{\partial z} + \frac{\partial W(x, y, z)}{\partial x} = \left(1 - \frac{df(z)}{dz} \right) \frac{\partial w_s}{\partial x} \quad (4.c)$$

$$\gamma_{yz} = \frac{\partial V(x, y, z)}{\partial z} + \frac{\partial W(x, y, z)}{\partial y} = \left(1 - \frac{df(z)}{dz} \right) \frac{\partial w_s}{\partial y} \quad (4.d)$$

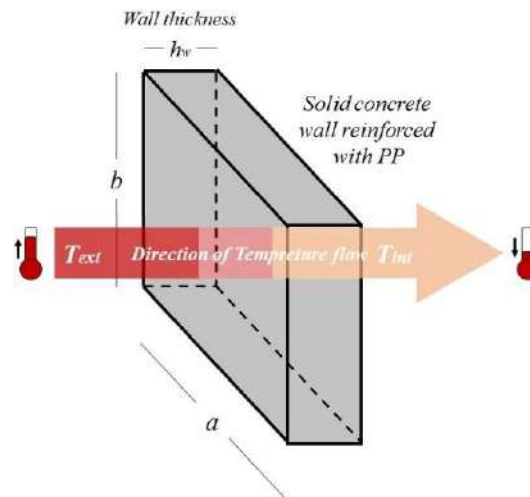


Fig. 3. Geometrical configuration of the PP-modified concrete slab, showing its dimensions (a , b , h_w) and the direction of heat transfer from the external surface (T_{ext}) toward the internal surface (T_{int})

Accordingly, the total stress components are expressed as functions of the mechanical strain and the temperature-induced strain resulting from uniform or gradient temperature fields. The generalized constitutive relations can thus be written as:

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \\ \tau_{xz} \\ \tau_{yz} \end{Bmatrix} = \begin{bmatrix} \tilde{Q}_{11} & \tilde{Q}_{12} & 0 & 0 & 0 \\ \tilde{Q}_{12} & \tilde{Q}_{22} & 0 & 0 & 0 \\ 0 & 0 & \tilde{Q}_{66} & 0 & 0 \\ 0 & 0 & 0 & \tilde{Q}_{44} & 0 \\ 0 & 0 & 0 & 0 & \tilde{Q}_{55} \end{bmatrix} \begin{Bmatrix} \varepsilon_x - \alpha_{hom}T \\ \varepsilon_y - \alpha_{hom}T \\ \gamma_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \end{Bmatrix} \quad (5)$$

Here, $\tilde{Q}_{ij}$ denotes the reduced elastic constants of the homogenized PP-modified concrete system, which is treated as isotropic. These constants account for the combined contribution of the concrete matrix and the PP reinforcement and are obtained using the adapted Reuss–Voigt–Luo homogenization model, where:

$$\begin{aligned} \tilde{Q}_{11} = \tilde{Q}_{22} &= \frac{(1 - \nu_{hom})E_{hom}}{(1 + \nu_{hom})(1 - 2\nu_{hom})}; \quad \tilde{Q}_{12} = \frac{\nu E_{hom}}{(1 + \nu_{hom})(1 - 2\nu_{hom})} \\ \tilde{Q}_{66} = \tilde{Q}_{55} = \tilde{Q}_{44} &= \frac{E_{hom}}{(1 + \nu_{hom})} \end{aligned} \quad (6)$$

2.2 Analytical Homogenization of PP-Modified Concrete

To determine the effective thermo-elastic properties (E_{hom} , ν_{hom} , α_{hom} , λ_{hom}) of the PP-modified concrete matrix, a two-phase homogenization approach based on the Voigt–Reuss–Luo model is adopted, accounting for three-dimensional stress interactions and Poisson’s ratio coupling effects. The selection of a two-phase formulation is justified by the material configuration of PP-modified concrete, which consists of a dominant continuous cementitious matrix and dispersed pumice particles acting as inclusions. Within the investigated replacement range (0%–10%), PP remains sufficiently distributed to be reasonably idealized as a secondary phase embedded in a continuous matrix, without forming a percolating or interconnected skeleton. This assumption enables analytical tractability while preserving the essential mechanical and thermal interaction mechanisms between phases. The homogenization procedure is carried out in two successive phases [74, 75]:

- Phase I: Homogeneous Concrete Matrix

Initially, the concrete is assumed to be homogeneous and isotropic, representing a mono-phasic state corresponding to conventional unreinforced concrete. This baseline serves as a reference for evaluating the impact of PP inclusion on the mechanical and thermal behavior of the composite wall. The material properties of the concrete matrix are characterized by the elastic modulus (E_c), Poisson’s ratio (ν_c), thermal conductivity (λ_c), and coefficient of thermal expansion (α_c).

- Phase II: Bi-Phasic Composite Material

With the incorporation of PP as a partial replacement for cement or fine aggregate, the mixture transforms into a bi-phasic composite, composed of the continuous concrete matrix and dispersed PP inclusions. The volumetric fraction of PP (ν_{pp}) ranges from 0% (pure concrete) to 10%, enabling a systematic assessment of its influence on the wall’s mechanical stiffness, thermal expansion, and heat transfer performance. The two homogenization phases and the underlying assumptions are conceptually illustrated in Fig. 4, which shows the transition from mono-phasic concrete to bi-phasic PP-reinforced concrete and the assumed spherical distribution of PP inclusions within the matrix.

- Porous Spherical Inclusions

PP particles are idealized as spherical or quasi-spherical inclusions with uniform internal porosity, representing their vesicular volcanic structure. Although natural pumice particles exhibit irregular shapes, microscopic observations reported in the literature indicate that crushed and ground pumice tends to form approximately equiaxed particles with rounded edges due to mechanical processing [76-78]. To accurately represent the behavior of the PP-reinforced composite, certain physical considerations are adopted within the homogenization framework.

In analytical homogenization theory, the spherical inclusion assumption is widely adopted because it provides a physically representative average geometry while allowing closed-form solutions for stress and heat transfer fields. Therefore, this idealization offers a rational balance between

microstructural realism and analytical tractability, without significantly affecting the prediction of effective elastic and thermal properties at moderate inclusion contents.

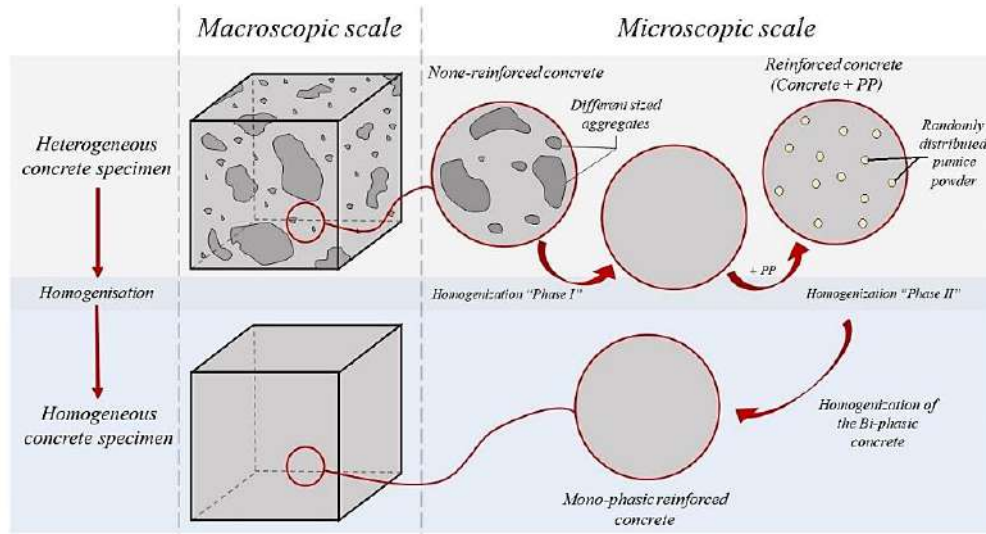


Fig. 4. Illustrates the conceptual homogenization, showing the transition from mono-phasic concrete to bi-phasic PP-reinforced concrete and the assumed distribution of spherical PP inclusions within the matrix

- Random Dispersion

PP inclusions are assumed to be randomly dispersed and non-agglomerated within the continuous cementitious matrix. This assumption is consistent with the requirements of classical homogenization theory, which relies on a statistically homogeneous representative volume element (RVE) where no preferential clustering or directional bias dominates the macroscopic response [79-81]. Within the investigated replacement range ($\leq 10\%$), the inclusion phase remains discontinuous and well below typical percolation thresholds; consequently, inclusion-inclusion interactions are limited, and large-scale agglomeration effects are unlikely to govern the composite behavior [82]. Under these conditions, the mechanical stiffness and heat transfer characteristics are controlled primarily by matrix-inclusion interactions rather than localized particle clusters. Therefore, neglecting agglomeration provides a reasonable approximation for predicting effective averaged thermo-elastic properties at the structural scale [83].

2.2.1 Modelling Procedure Elasto-Mechanical Homogenization Using the Reuss-Voigt-Luo Model

The effective elastic properties of the PP-reinforced concrete are computed using an adapted Reuss-Voigt-Luo homogenization model, accounting for Poisson's ratio and three-dimensional stress interactions. The effective Young's moduli in the x , y , and z directions (E_{hom}^x , E_{hom}^y , E_{hom}^z) are expressed as [84]:

$$E_{hom}^x = E_{hom}^y = \frac{\{V_c(1 + \nu_{pp})E_c + V_{pp}(1 + \nu_c)E_{pp}\}\{V_c(1 - \nu_{pp})E_c + V_{pp}(1 - \nu_c)E_{pp}\}}{\{V_c(1 - \nu_{pp}^2)E_c + V_{pp}(1 - \nu_c^2)E_{pp}\}}$$

$$E_{hom}^z = \frac{E_c E_{pp} [V_c(1 - \nu_{pp})E_c + V_{pp}(1 - \nu_c)E_{pp}]}{\left\{ \begin{aligned} &E_c E_{pp} [V_c^2(1 - \nu_{pp}) + V_{pp}^2(1 - \nu_c)] \\ &+ \nu_{pp} \nu_c [(1 + \nu_{pp})(1 - 2\nu_{pp})E_c^2 \\ &+ 4\nu_{pp} \nu_c E_c E_{pp} + (1 + \nu_c)(1 - 2\nu_c)E_{pp}^2] \end{aligned} \right\}} \quad (7)$$

Here, E_c and ν_c are the elastic modulus and Poisson's ratio of the concrete matrix, while E_{pp} and ν_{pp} correspond to the properties of the PP inclusions. V_c and V_{pp} denote the volume fractions of concrete and PP, respectively. This model integrates the effect of Poisson's ratio, which influences the elastic response of PP-modified concrete composites. Unlike the traditional Voigt and Reuss models that neglect this parameter and may lead to less accurate predictions, the adopted Reuss-Voigt-Luo approach incorporates the Poisson effect by deriving improved relationships from the

fundamental elasticity equations. Consequently, the effective Poisson's ratios of the composite are determined as the following:

$$\begin{aligned} v_{hom}^{xy} &= \frac{\{V_c(1 - v_{Gp})v_c + V_{Gp}(1 - v_c)v_{Gp}\}\{V_c(1 + v_{Gp})E_c + V_{Gp}(1 + v_c)E_{Gp}\}}{\{V_c(1 - v_{Gp}^2)E_c + V_{Gp}(1 - v_c^2)E_{Gp}\}} \\ v_{hom}^{xz} &= \frac{\{V_c(1 + v_{Gp}^2)E_c v_c + V_{Gp}(1 + v_c^2)E_{Gp} v_{Gp}\}}{\{V_c(1 - v_{Gp}^2)E_c + V_{Gp}(1 - v_c^2)E_{Gp}\}} \\ v_{hom}^{zx} = v_{hom}^{zy} &= \frac{E_c E_{Gp} [V_c(1 - v_{Gp})v_c + V_{Gp}(1 - v_c)v_{Gp}]}{\left\{ \begin{aligned} &E_c E_{Gp} [V_c^2(1 - v_{Gp}) + V_{Gp}^2(1 - v_c)] \\ &+ V_{Gp} V_c [(1 + v_{Gp})(1 - 2v_{Gp})E_c^2 \\ &+ 4v_{Gp} v_c E_c E_{Gp} + (1 + v_c)(1 - 2v_c)E_{Gp}^2] \end{aligned} \right\}} \end{aligned} \quad (8)$$

The model ensures that the anisotropic interactions and the effect of Poisson's ratio are properly incorporated, providing an accurate estimate of the wall's mechanical behavior under multi-directional loading conditions. The corresponding effective Poisson's ratios (v_{hom}) of the PP-reinforced concrete are computed as the volume-weighted contributions of the concrete matrix and the PP inclusions. However, it should be noted that this homogenization approach assumes a statistically homogeneous and macroscopically isotropic composite with randomly dispersed, non-agglomerated inclusions; therefore, local clustering effects and morphological irregularities are not explicitly captured in the present formulation. Based on these, the effective shear modulus (G_{hom}) and bulk modulus (K_{hom}) of the composite are derived using classical elasticity relations:

$$G_{hom}^x = E_{hom}^x [2(1 + v_{hom})]^{-1}; G_{hom}^y = E_{hom}^y [2(1 + v_{hom})]^{-1} \quad (9)$$

and;

$$K_{hom}^x = E_{hom}^x [3(1 - 2v_{hom})]^{-1}; K_{hom}^y = E_{hom}^y [3(1 - 2v_{hom})]^{-1} \quad (10)$$

2.2.2 Thermal Homogenization Using Maxwell-Eucken Model

The Maxwell-Eucken model is applied to estimate the effective thermal conductivity (λ_{hom}) of PP-reinforced concrete. The model is suitable for composites where one phase (PP) is dispersed as small, isotropic inclusions within a continuous matrix (concrete). This formulation is adopted because it is particularly appropriate for bi-phasic materials with low to moderate inclusion volume fractions and a clearly dominant continuous matrix, which corresponds to the present mixture configuration ($PP \leq 10\%$). Unlike more general self-consistent or symmetric approaches that treat both phases equivalently or are better suited for higher inclusion concentrations, the Maxwell-Eucken model explicitly represents a dispersed inclusion phase embedded within a continuous matrix. It therefore provides a physically consistent closed-form estimate of the effective conductivity for non-percolating composite systems. The effective thermal conductivity is expressed as [85]:

$$\lambda_{hom} = \lambda_c \{2\lambda_c + \lambda_{pp} - 2(\lambda_c - \lambda_{pp})v_{pp}\} \{2\lambda_c + \lambda_{pp} + 2(\lambda_c - \lambda_{pp})v_{pp}\}^{-1} \quad (11)$$

where λ_c and λ_{pp} denote the thermal conductivities of the concrete matrix and PP, respectively, and v_{pp} is the volumetric fraction of pumice powder, in which:

$$v_{pp} = 1 - v_c \quad (12)$$

The effective thermal expansion coefficient of the composite (α_{hom}) accounts for the contribution of both phases, reflecting the combined influence of the PP inclusions and the concrete matrix. It considers the relative volume fractions, the intrinsic thermal expansion properties of each phase (α_c for the concrete matrix and α_{pp} for the PP), and their interaction within the microstructure, providing a homogenized parameter that governs the wall's dimensional response under temperature variations.

$$\alpha_{hom} = (1 - v_{pp})\alpha_c + v_{pp}\alpha_{pp} \quad (13)$$

The thermal resistance (R_{hom}) of a wall of thickness (h_w) is defined as the ratio of the wall thickness to its effective thermal conductivity, providing a measure of the material's ability to resist heat flow. Mathematically, it is expressed as:

$$R_{hom} = \frac{h_w}{\lambda_{hom}} \quad (14)$$

where λ_{hom} is the effective thermal conductivity of the PP-modified concrete wall. This parameter quantifies how efficiently the wall impedes heat transfer, which is critical for evaluating its thermal performance and energy efficiency in structural applications.

Similarly, the thermal transmittance (U_{hom}) of the wall is defined as the inverse of its thermal resistance, representing the rate of heat transfer through the wall per unit area under a temperature difference. It is expressed as:

$$U_{hom} = \frac{1}{R_{hom}} = \frac{\lambda_{hom}}{h_w} \quad (15)$$

where λ_{hom} is the effective thermal conductivity of the PP-modified concrete and h_w is the wall thickness. This parameter provides a practical measure of the wall's ability to conduct heat, which is essential for assessing thermal efficiency and energy performance in building and structural applications.

Finally, the thermal flux (ϕ_{hom}) through the wall represents the amount of heat transferred per unit area due to a temperature difference between the wall surfaces. It is expressed as:

$$\phi_{hom} = U_{hom}(T_{int} - T_{ext}) \quad (16)$$

where U_{hom} is the thermal transmittance of the PP-modified concrete wall, T_{int} is the interior temperature, and T_{ext} is the exterior temperature. This parameter provides a direct measure of the wall's thermal performance and efficiency, capturing how effectively the material regulates heat flow under operational conditions.

The Maxwell-Eucken model assumes that the dispersed PP inclusions are small, isotropic, and uniformly distributed within a continuous concrete matrix. It also presumes linear heat conduction and neglects particle clustering or agglomeration effects. Consequently, the model provides an approximate estimate of the effective thermal conductivity and expansion, which may be less accurate for high PP fractions, non-uniform microstructures, or extreme temperature gradients.

2.3 Governing Equations for Thermo-Mechanical Response

The PP-modified concrete wall is represented as a homogeneous equivalent plate with effective thermo-elastic properties obtained from the Reuss-Voigt-Luo and Maxwell-Eucken homogenization schemes. The governing equations account for in-plane and transverse shear effects, ensuring consistent interaction between mechanical displacements and thermal strains.

$$\int_0^t (\delta\chi - \delta Y) dt = 0 \quad (17)$$

The mechanical bending response is derived using the principle of virtual work, which balances the virtual internal strain energy with the work of external forces. This formulation captures both bending and shear deformations, while incorporating thermal effects through the homogenized elastic and thermal properties. The virtual variation of the internal strain energy, denoted as $\delta\chi$, is expressed as follows and accounts for the contributions of both bending and shear deformations within the PP-modified concrete panel:

$$\delta\chi = \int_0^A \int_{-h/2}^{h/2} (\sigma_x \delta\epsilon_x + \sigma_y \delta\epsilon_y + \tau_{xy} \delta\gamma_{xy} + \tau_{yz} \delta\gamma_{yz} + \tau_{xz} \delta\gamma_{xz}) dAdz \quad (18)$$

By substituting Eq. (4) into Eq. (19), the resulting expression becomes:

$$\delta\chi = \int_A \left(\begin{aligned} &N_x \frac{\partial \delta u_0}{\partial x} - M_x^b \frac{\partial^2 \delta w_b}{\partial^2 x} + M_x^s \frac{\partial^2 \delta w_s}{\partial^2 x} \\ &+ N_y \frac{\partial \delta v_0}{\partial y} - M_y^b \frac{\partial^2 \delta w_b}{\partial^2 y} + M_y^s \frac{\partial^2 \delta w_s}{\partial^2 y} \\ &+ N_{xy} \left(\frac{\partial \delta u_0}{\partial y} + \frac{\partial \delta v_0}{\partial x} \right) + 2M_x^b \frac{\partial^2 \delta w_b}{\partial x \partial y} \\ &+ 2M_x^s \frac{\partial^2 \delta w_s}{\partial x \partial y} + Q_{yz} \left(\frac{\partial \delta w_s}{\partial y} \right) + Q_{xz} \left(\frac{\partial \delta w_s}{\partial x} \right) \end{aligned} \right) \quad (19)$$

The mechanical behavior of the PP-modified concrete panel is characterized by the following stress and moment resultants: the in-plane normal and shear forces (N), bending moments (M_b), shear-related moments (M_s), and transverse shear forces (Q). These quantities capture the panel's response to applied mechanical loads, including bending, shear, and in-plane stress distribution, providing a basis for evaluating deflections and mechanical stability.

$$N_{ij} = b \int_{-h_w/2}^{h_w/2} \sigma_{ij} dz; M_{ij}^b = b \int_{-h_w/2}^{h_w/2} z \sigma_{ij} dz; M_{ij}^s = b \int_{-h_w/2}^{h_w/2} f(z) \sigma_{ij} dz; Q_{ij} = b \int_{-h_w/2}^{h_w/2} g(z) \sigma_{ij} dz. \quad (20)$$

Note that the indices i and j correspond to the in-plane directions x or y . The term δY represents the virtual work of the external forces acting on the panel. It accounts for both mechanical buckling loads, denoted by N_b , and thermal loads, denoted by T . Specifically, δV quantifies the virtual external energy induced by these applied loads and serves as the counterpart to the internal virtual strain energy, enabling the formulation of equilibrium conditions within the principle of virtual work.

$$\delta Y = - \int_A \int_{-\frac{h}{2}}^{\frac{h}{2}} N_b (\delta w_b + \delta w_s) dA dz = - \int_A \int_{-h/2}^{h/2} T (\delta w_b + \delta w_s) dA dz \quad (21)$$

By substituting Eqs. (20) and (22) into Eq. (18), which itself incorporates the thermal and mechanical contributions expressed in Eq. (20), and performing integration by parts while taking into account the virtual variations δu_0 , δv , δw_b , and δw_s , the resulting governing equations of motion for the PP-modified concrete panel are obtained. These equations describe the coupled bending, shear, and thermal responses of the panel under combined mechanical and thermal loading conditions, capturing the influence of both material heterogeneity and geometric effects.

$$\begin{aligned} \delta u_0: \frac{\partial N_x}{\partial x} + \frac{\partial N_{xy}}{\partial y} &= 0 \\ \delta v_0: \frac{\partial N_y}{\partial y} + \frac{\partial N_{xy}}{\partial x} &= 0 \\ \delta w_b: \left(\frac{\partial^2 M_x^b}{\partial x^2} - 2 \frac{\partial^2 M_{xy}^b}{\partial x \partial y} + \frac{\partial^2 M_y^b}{\partial y^2} \right) + N_b &= 0 \\ \delta w_s: \left(\frac{\partial^2 M_x^s}{\partial x^2} - 2 \frac{\partial^2 M_{xy}^s}{\partial x \partial y} + \frac{\partial^2 M_y^s}{\partial y^2} \right) + \left(\frac{\partial Q_{xz}}{\partial x} + \frac{\partial Q_{yz}}{\partial y} \right) + N_b &= 0 \end{aligned} \quad (22)$$

The stress resultants, expressed in terms of material stiffness and displacement components, can be derived as shown in Appendix A. Thermal effects are represented by the corresponding thermal resultants induced by temperature gradients across the panel's thickness. These include in-plane thermal forces (N_T), thermal bending moments (M_{Tb}), thermal shear moments (M_{Ts}), and transverse shear forces due to thermal expansion (Q_T). These terms account for the coupling between thermal strains and mechanical displacements, enabling the assessment of thermal buckling, stress variation, and overall thermo-mechanical performance of the PP-modified concrete panel.

$$N_{Tij} = b \int_{-h_w/2}^{h_w/2} \tilde{Q}_{11} \alpha_{hom} T dz \quad (23)$$

$$M_{Tij}^b = b \int_{-h_w/2}^{h_w/2} z \tilde{Q}_{11} \alpha_{hom} T dz$$

$$M_{Tij}^s = b \int_{-h_w/2}^{h_w/2} f(z) \tilde{Q}_{11} \alpha_{hom} T dz$$

The buckling loads of the simply supported rectangular panel correspond to the critical in-plane forces at which the panel loses stability under compressive or thermal stresses. These loads are determined by solving the governing equations of motion derived from the principle of virtual work, incorporating both the mechanical stiffness and the thermal expansion effects of the PP-modified concrete.

$$N_b = N_{bx} + \eta N_{by} \quad (24)$$

In Eq. (25a), N_b represents the total critical buckling load of the panel under combined uni-axial or bi-axial loading. The terms N_{bx} and N_{by} correspond to the contributions of the axial loads applied along the x- and y-directions, respectively. The parameter η is the bi-axial load coefficient, defining the ratio of the axial load in the y-direction to that in the x-direction: $\eta = 0$ corresponds to uni-axial buckling along the x-direction, while $\eta = 1$ represents bi-axial buckling with equal loads in both directions. For a square rectangular wall, the in-plane normal forces along the x and y directions are equal, and the associated transverse shear forces are also equal. That is:

$$N_{bx} = T_x = N_{by} = T_y \quad (25)$$

2.4 Analytical Solution Using Navier's Method

For the simply supported rectangular panel with length a , width b , and total thickness h_w , the boundary conditions enforce zero transverse displacement and bending moments along all edges. Specifically, at $x = 0$ and $x = a$, as well as $y = 0$ and $y = b$, the deflection w and rotations vanish, while the in-plane displacements u_0 and v_0 are constrained according to the support type. These conditions allow edge rotation while preventing vertical displacement, consistent with classical simply supported behavior under mechanical and thermal loading:

$$\begin{aligned} x = 0 \text{ and } x = a : v_0 = w_b = w_s = N_x = M_x^b = M_x^s = 0 \\ y = 0 \text{ and } y = b : u_0 = w_b = w_s = N_y = M_y^b = M_y^s = 0 \end{aligned} \quad (26)$$

To enable a tractable analytical solution, the PP-modified concrete panel is assumed homogeneous and linearly elastic with small deformations, and its simply supported edges allow rotation while preventing transverse displacement, consistent with Eq. (26). Thermal and mechanical effects are considered linearly coupled and superimposed. For clarity and reproducibility, the key model assumptions are summarized as follows:

- Linear elasticity: Material response is proportional to applied loads.
- Small deformations: Geometric nonlinearity is neglected.
- Simply supported boundaries: Edges allow rotation but no transverse displacement.
- Thermal-mechanical superposition: Thermal and mechanical effects are linearly combined.

These assumptions enable the use of Navier-type solutions, where double Fourier series satisfy boundary conditions exactly, providing an accurate evaluation of the panel's mechanical and thermal response.

By considering Navier solutions, the in-plane displacements $u_0(x, y)$ and $v_0(x, y)$, together with the transverse bending $w_b(x, y)$ and shear $w_s(x, y)$ components, are represented as sums of sinusoidal functions along both axes. Each term corresponds to a specific half-wave in the x and y directions, transforming the governing partial differential equations into algebraic equations for the Fourier coefficients. This approach enables analytical determination of deflections, stress resultants, and critical buckling loads, while allowing efficient evaluation of multiple parameters, including PP content, panel dimensions, and thermal loads. The displacement components are expressed as double Fourier series:

$$u(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} X_{mn} \cos(\xi x) \sin(\varsigma y) \quad (27.a)$$

$$v(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} Y_{mn} \sin(\xi x) \cos(\varsigma y) \quad (27.b)$$

$$w_b(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} Z_{bmn} \sin(\xi x) \sin(\varsigma y) \quad (27.c)$$

$$w_s(x, y) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} Z_{smn} \sin(\xi x) \sin(\varsigma y) \quad (27.d)$$

Here, $X_{mn}, Y_{mn}, Z_{mn}^b, Z_{mn}^s$ represent the unknown Fourier coefficients corresponding to the in-plane displacements (u_0, v_0) and the bending and shear components of the transverse displacement (w_b, w_s) , respectively. These coefficients are determined by substituting the series expansions into the governing equations of motion and solving the resulting algebraic system.

$$\xi = \frac{m\pi}{a}; \varsigma = \frac{n\pi}{b} \quad (28)$$

Each coefficient captures the contribution of the (m, n) -th half-wave in the x and y directions, ensuring that both the boundary conditions and the coupled thermo-mechanical behavior of the PP-modified concrete panel are accurately satisfied.

by substituting the Navier series expansions for displacements into the governing equations and assembling the contributions from all modes, the coupled thermo-mechanical behavior of the PP-modified concrete panel can be expressed compactly in matrix form as:

$$\left(\begin{bmatrix} k_{11} & k_{12} & k_{13} & k_{14} \\ k_{21} & k_{22} & k_{23} & k_{24} \\ k_{31} & k_{32} & k_{33} & k_{34} \\ k_{41} & k_{42} & k_{43} & k_{44} \end{bmatrix} - N \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & N_{33} & 0 \\ 0 & 0 & 0 & \eta \end{bmatrix} \right) \begin{Bmatrix} X_{mn} \\ Y_{mn} \\ Z_{bmn} \\ Z_{smn} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \quad (29)$$

where $[K]$ is the global stiffness matrix incorporating both mechanical and thermal effects, $[X]$ is the vector of unknown displacement coefficients $(X_{mn}, Y_{mn}, Z_{mn}^b, Z_{mn}^s)$. The main blocks of $[K]$ correspond to bending, shear, and in-plane stiffness contributions, while off-diagonal terms represent the coupling between thermal expansions and mechanical loads, enabling the panel's thermo-mechanical interaction to be captured. This formulation enables the analytical determination of deflections, bending moments, shear forces, and critical buckling loads of the simply supported PP-reinforced panel.

It is recommended to adopt a reduced system of equations by retaining only the dominant terms or lower-order modes in the Navier series. This simplification preserves the essential characteristics of the panel's thermo-mechanical response while significantly reducing computational complexity and facilitating analytical solution.

$$\left(\begin{bmatrix} k_{33} - N_{33} & k_{34} \\ k_{43} & k_{44} \end{bmatrix} \right) \begin{Bmatrix} w_{bmn} \\ w_{smn} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \quad (30)$$

To solve the problem and determine the critical buckling loads of the system, the determinant of the reduced stiffness matrix is computed. This procedure yields the eigenvalues corresponding to the buckling loads for the simply supported PP-modified concrete wall.

$$N = \text{solve} \left(\det \begin{bmatrix} k_{33} - N_{33} & k_{34} \\ k_{43} & k_{44} \end{bmatrix} \right) \quad (31)$$

The explicit expressions of the stiffness matrix elements K_{ij} , derived considering the simply supported boundary conditions, are given by:

$$\begin{aligned}
 K_{11} &= -A_{11}\xi^2 - A_{66}\zeta^2; K_{12} = -A_{12}\xi\zeta - A_{66}\eta\zeta; K_{13} = B_{11}\xi^3 + B_{12}\xi\zeta^2 + 2B_{66}\xi\zeta^2; \\
 K_{14} &= B_{11}^s\xi^3 + B_{12}^s\xi\zeta^2 + 2B_{66}^s\xi\zeta^2; \\
 K_{23} &= B_{22}\zeta^3 + B_{12}\xi\zeta^2 + 2B_{66}\xi\zeta^2; S_{24} = B_{11}^s\xi^3 + B_{12}^s\xi\zeta^2 + 2B_{66}^s\xi\zeta^2; \\
 K_{33} &= -D_{11}\xi^4 - 2\xi^2\zeta^2(2D_{66} + D_{12}) - D_{22}\zeta^2; \\
 S_{44} &= -H_{11}^s\xi^4 - 2\xi^2\zeta^2(2H_{66}^s + H_{12}^s) - H_{22}^s\zeta^2 - A_{44}^s\xi^2 - A_{55}^s\zeta^2.
 \end{aligned} \tag{32}$$

The developed analytical framework provides a direct link between material-level modifications and structural performance, enabling engineers to predict both mechanical stability and thermal behavior of PP-modified concrete walls. By quantifying critical mechanical and thermal buckling thresholds, as well as effective thermal resistance, transmittance, and heat flux, this model supports the design of lightweight concrete walls that meet energy efficiency and structural safety requirements. Such predictive capability is particularly relevant for sustainable building applications, where balancing stiffness, stability, and thermal insulation is essential for optimizing performance under combined mechanical and environmental loading conditions.

3. Results and Discussion

This section presents an analytical evaluation of the thermomechanical performance of PP-modified concrete walls. The study examines how the addition of pumice powder influences the elastic, thermal, and coupled responses of the composite wall under simple support conditions. The results are discussed in terms of material properties, stress distribution, deformation behavior, and thermal effects, yielding information on the effectiveness of PP as a partial substitute in concrete mixtures.

3.1 Thermo-Elastic Properties of Constituent Materials

The thermo-elastic properties of the concrete matrix and PP used in this study are summarized in Table 3. These properties form the basis for the homogenization and subsequent evaluation of the wall's coupled mechanical and thermal response.

Table 3. Thermo-elastic properties of the concrete matrix and pumice powder (PP) used in the analysis

Material	Ref	Elastic Modulus E [GPa]	Poisson's Ratio ν	Thermal Conductivity λ [W·m ⁻¹ ·°C ⁻¹]	Thermal Expansion α [×10 ⁻⁶ /°C]
Concrete Matrix	Neville and Brooks [86]	20	0.3	1.75	13.5
Pumice Powder	Bilir et al. [87]	7–10	0.25–0.29	0.25–0.45	4–6

Based on the thermo-elastic properties summarized in Table 3, the inclusion of PP is expected to influence the wall's behavior due to its lower stiffness, reduced thermal conductivity, and moderated thermal expansion compared to the concrete matrix. Its vesicular microstructure and pozzolanic composition suggest that PP may act as compliant, low-conductivity inclusions, potentially redistributing stresses and altering heat transfer paths within the concrete. Consequently, the wall may exhibit slightly reduced stiffness, improve thermal insulation, and mitigate differential expansion under temperature gradients, effects that will be systematically evaluated in the subsequent homogenization and high-order deformation analysis.

3.2 Validation of the Homogenization Model

To verify the accuracy of the proposed homogenization approach, a comparative validation is carried out using the experimental data reported by Kalkan and Gündüz [88]. In their work, ordinary concrete was partially replaced with pumice aggregates at different volume fractions, and the corresponding variations in Young's modulus were experimentally measured, as illustrated in Fig. 5.

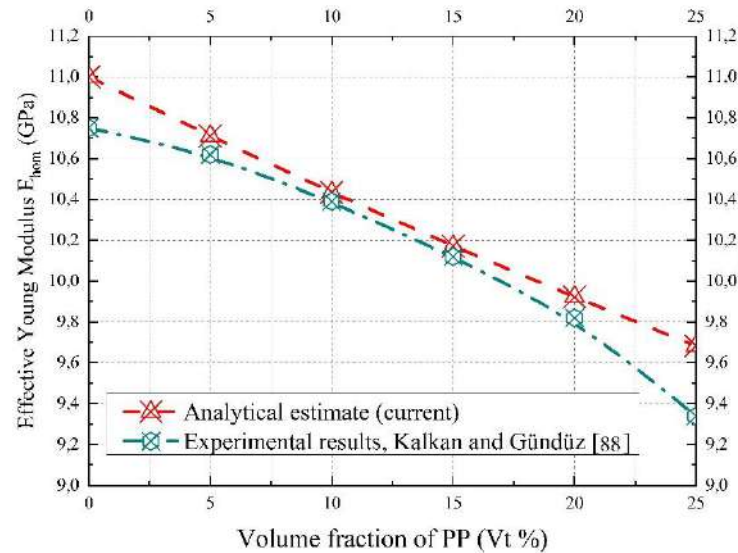


Fig. 5. Comparison between the predicted elastic modulus from the proposed homogenization model and experimental results reported by Kalkan and Gündüz [88] for concrete reinforced with different pumice powder fractions.

For this validation, the material properties are chosen to be consistent with those adopted in the reference study. Specifically, the ordinary concrete is characterized by an elastic modulus of 11 GPa and a Poisson's ratio of 0.2, while the pumice particles exhibit an elastic modulus of 5 GPa and a Poisson's ratio of 0.15.

A close agreement can be observed between the analytical predictions and the experimental data, particularly when the volume fraction of pumice powder (PP) ranges between 5% and 20% of the total mix, confirming the reliability of the proposed model within this composition range. For a PP content of 12%, the discrepancy between analytical and experimental results is below 1%, indicating excellent agreement. For lower (0%) or higher (25%) PP content, the error reaches a maximum of approximately 4%, which remains within acceptable limits for engineering applications. These quantitative indicators demonstrate the predictive capability of the analytical model for mechanical behavior.

The reduction in elastic modulus predicted for pumice contents lies within the stiffness reduction factors (0.6–0.7 of the gross stiffness) already adopted in Eurocode 2 and ACI 318 for cracked and lightweight concretes, indicating that the associated reduction in buckling resistance remains within standard design safety margins [89].

The validation of the analytical model for thermal expansion is carried out using the experimental data reported by Şevket Onur Kalkan and Lütfullah Gündüz [88], as illustrated in Fig. 6(a). The coefficient of thermal expansion of pumice is taken as $10 \times 10^{-6} / ^\circ\text{C}$, while that of lightweight concrete is approximately $1 \times 10^{-6} / ^\circ\text{C}$. For conventional concrete, a reference value of $7 \times 10^{-6} / ^\circ\text{C}$ is adopted. The analytical results show excellent agreement with the experimental data, particularly for pumice powder (PP) volume fractions between 5% and 10%, where the discrepancy between analytical predictions and experimental measurements is approximately 1%. Beyond this range, the accuracy of the homogenization model slightly decreases but remains acceptable, with errors reaching around 3%, likely due to the increasing influence of the highly porous pumice phase on the composite's thermal response.

Similarly, the validation of thermal conductivity is performed based on the same reference study [29], with results presented in Fig. 6(b). The thermal conductivity of pumice reinforcement is taken as $0.1 \text{ W}/(\text{m}\cdot^\circ\text{C})$, while that of the concrete matrix is $1.8 \text{ W}/(\text{m}\cdot^\circ\text{C})$. The analytical model predictions are compared with an experimental reference value of $0.62 \text{ W}/(\text{m}\cdot^\circ\text{C})$ for lightweight concrete. The results show that a small error of around 5% starts to appear when the PP volume fraction reaches 3% and above, but overall, the model reliably captures the thermal transport behavior of PP-modified concrete.

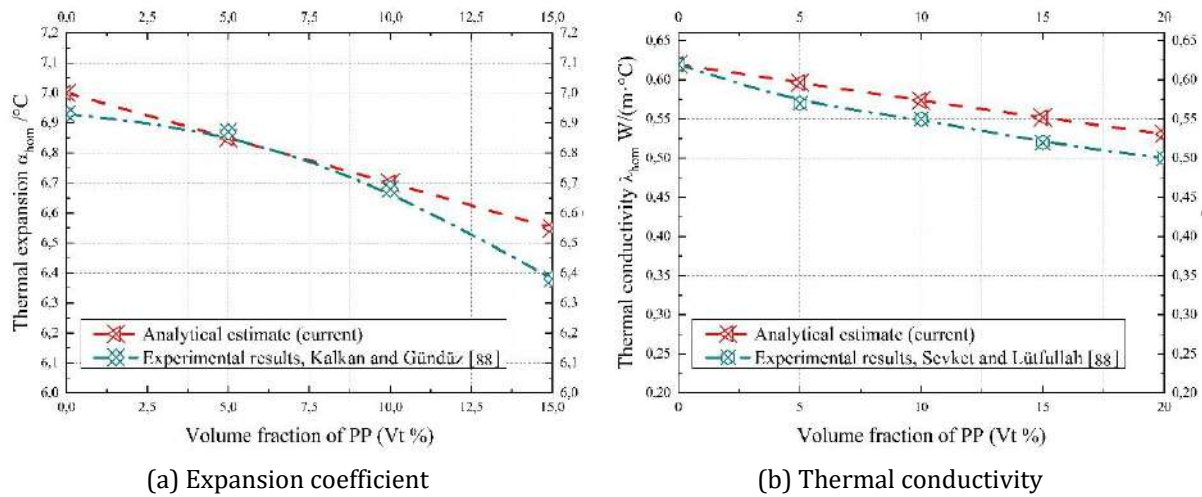


Fig. 6. Comparison between analytical predictions and experimental data of the effective thermal properties of PP modified concrete [88], including the effective thermal expansion coefficient (α_{hom}) and the effective thermal conductivity (λ_{hom})

The adoption of identical constituent properties to those reported experimentally is necessary to ensure comparability of the physical system and should not be interpreted as parameter fitting. The homogenization model contains no adjustable empirical coefficients, and no inverse identification procedure was performed. Consequently, the agreement observed reflects the intrinsic predictive capability of the theoretical formulation rather than calibration against experimental data.

3.3 Mechanical Behavior of PP-Modified Concrete Walls

To analyze the mechanical behavior of the PP-modified concrete walls, the critical buckling load is expressed in a non-dimensional form. This approach allows the results to be generalized and compared across different wall geometries and PP contents. The non-dimensional critical buckling load N_{cr} is defined as:

$$N_{cr} = \frac{12(1 - \nu_c)a^2N}{E_c h_w^3} \quad (35)$$

Table 4. Dimensionless mechanical buckling loads of simply supported PP-modified concrete walls for varying volume fractions of pumice powder (0–10%) and different length-to-thickness ratios (a/h_w), under uni-axial ($\eta = 0$) and bi-axial ($\eta = 1$) loading conditions, ($a=b$)

		Uni-axial buckling load $\eta = 0$					Bi-axial buckling loads $\eta = 1$				
		Volume fraction of PP inclusions (v_{PP})									
		0%	2.5%	5%	7.50%	10%	0%	2.5%	5%	7.50%	10%
length-to-thickness ratios (a/h _w) of the plate	5	2.9253	2.7530	2.6013	2.4657	2.3448	1.4626	1.3765	1.3006	1.2328	1.1724
	10	3.4136	3.2175	3.0453	2.8917	2.7550	1.7068	1.6088	1.5226	1.4458	1.3775
	15	3.5227	3.3215	3.1449	2.9874	2.8474	1.7614	1.6608	1.5724	1.4937	1.4237
	20	3.5626	3.3595	3.1813	3.0225	2.8813	1.7813	1.6798	1.5907	1.5112	1.4406
	25	3.5814	3.3774	3.1985	3.0390	2.8972	1.7907	1.6887	1.5992	1.5195	1.4486
	30	3.5917	3.3872	3.2079	3.0480	2.9060	1.7958	1.6936	1.6039	1.5240	1.4530
	35	3.5979	3.3932	3.2136	3.0535	2.9113	1.7989	1.6966	1.6068	1.5267	1.4556
	40	3.6019	3.3971	3.2173	3.0570	2.9147	1.8010	1.6985	1.6086	1.5285	1.4573
	45	3.6047	3.3997	3.2198	3.0595	2.9171	1.8024	1.6999	1.6099	1.5297	1.4585
50	3.6067	3.4016	3.2216	3.0613	2.9188	1.8034	1.7008	1.6108	1.5306	1.4594	

Table 4 presents the dimensionless mechanical buckling coefficients of simply supported square concrete walls reinforced with pumice powder (PP) under uni-axial ($\eta = 0$) and bi-axial ($\eta = 1$) loading, with aspect ratios (a/h_w) ranging from 5 to 50 and PP volume fractions from 0% to 10%.

The data show a systematic decrease in buckling loads with increasing PP content. For example, at $a/h_w = 10$ under uni-axial loading, the critical buckling coefficient decreases from 3.414 for plain concrete to 2.755 at 10% PP, a reduction of approximately 19%, reflecting the direct impact of reduced modulus of elasticity due to PP incorporation. Similarly, under bi-axial loading, the coefficient decreases from 1.707 to 1.378, showing a 19% reduction, confirming that biaxial compression is more sensitive to stiffness reduction.

The percentage reduction in buckling load due to PP incorporation remains nearly constant ($\approx 19\text{--}20\%$) across all aspect ratios, indicating that the influence of PP content on mechanical stability is largely independent of plate slenderness. This suggests that stiffness degradation caused by PP inclusions proportionally affects the critical load without altering the geometric sensitivity of the plate. Uni-axial buckling loads are consistently higher than bi-axial ones for all PP fractions and aspect ratios. This confirms classical plate theory predictions: bi-axial compression promotes earlier instability, and the relative difference increases slightly with PP content due to the larger influence of compliance on combined in-plane stresses. Even low PP fractions (2.5–5%) produce measurable reductions ($\sim 5\text{--}10\%$) in critical loads, suggesting that even moderate cement replacement affects wall stability. This underscores the importance of accounting for PP content in design to balance thermal efficiency gains with mechanical stability.

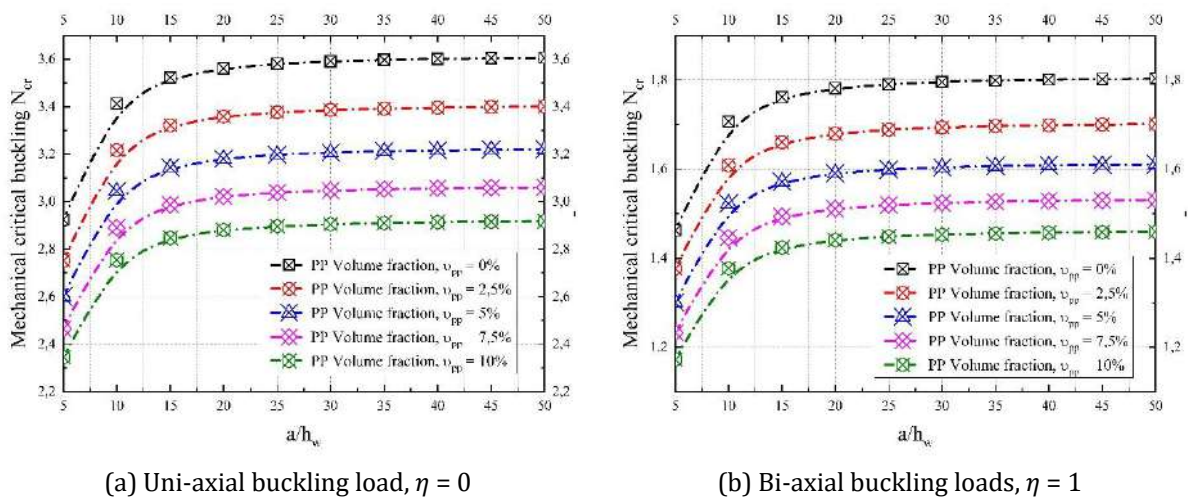


Fig. 7. Variation of the critical buckling load of a square PP modified concrete wall ($a = b$), with the length to thickness ratio (a/h_w) and pumice powder volume fraction under uni-axial buckling conditions ($\eta = 0$), and bi-axial buckling conditions ($\eta = 1$)

Fig. 7 illustrates the effect of pumice powder (PP) content on the buckling behavior of a square concrete wall. It is evident that increasing the PP volume fraction reduces the wall's stiffness, resulting in lower critical buckling loads for both uni-axial ($\eta = 0$) and bi-axial ($\eta = 1$) loading conditions. The trend highlights that higher PP content, due to its lower elastic modulus compared to the concrete matrix, diminishes the wall's mechanical stability, making it more susceptible to buckling under compressive loads. This effect is more pronounced at higher length-to-thickness ratios, where the wall becomes increasingly flexible.

Fig. 8 presents the variation of the critical buckling load of a rectangular plate ($a = \sqrt{b}$) with respect to the length-to-thickness ratio and the volumetric fraction of pumice powder (PP). Similar to the square plate case, an increase in PP content leads to a noticeable reduction in the critical buckling load under both uni-axial ($\eta = 0$) and bi-axial ($\eta = 1$) conditions. This reduction indicates that incorporating more PP weakens the stiffness of the composite plate, making it less resistant to instability. Moreover, the influence of PP becomes more significant as the slenderness ratio increases. These results highlight a trade-off between reduced weight and mechanical stability. Increasing PP content lowers stiffness and critical buckling loads, especially for slender panels, indicating that wall geometry and PP fraction must be carefully balanced. Practically, maintaining PP contents within 5–10% ensures sufficient buckling resistance while benefiting from lightweight and thermally efficient concrete, guiding sustainable and safe wall design.

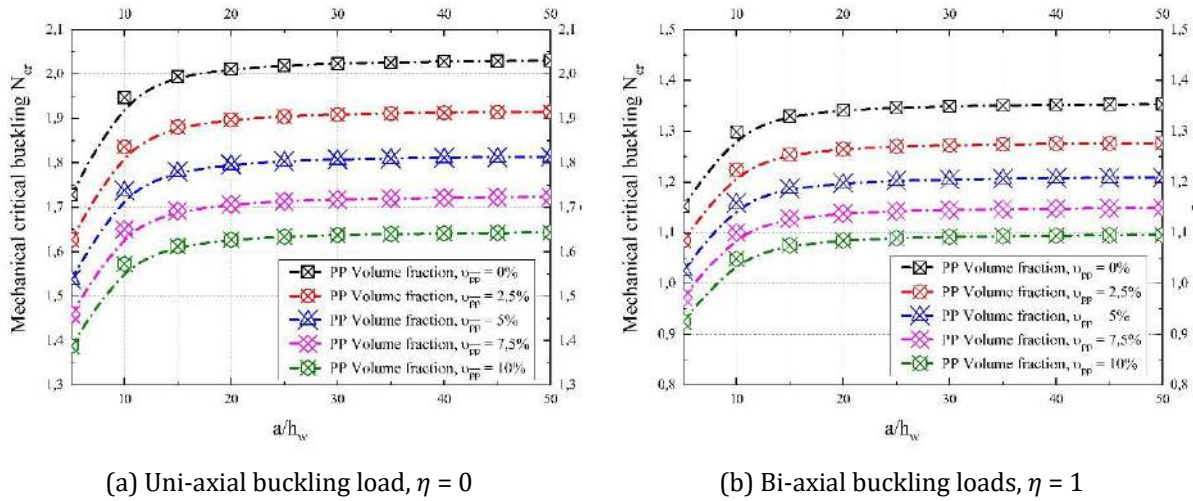


Fig. 8. Critical buckling load of a rectangular plate ($a = \sqrt{b}$) as a function of the length-to-thickness ratio and the volumetric fraction of PP, considering uni axial buckling with $\eta = 0$ and bi axial buckling with $\eta = 1$

To quantify the robustness of the predicted buckling response, a local sensitivity analysis was performed using the normalized coefficient $S_X^Y = (\Delta Y/Y_0)/(\Delta X/X_0)$, where $Y = \bar{N}_{cr}$ and X denotes a material parameter. Using a baseline mixture with $V_{PP} = 5\%$, the governing stiffness relation $D \propto Eh^3$ implies $S_E^{\bar{N}_{cr}} \approx 1$, confirming that the buckling capacity varies almost linearly with the effective elastic modulus. In contrast, since thermal conductivity does not enter the mechanical stiffness matrix, $S_k^{\bar{N}_{cr}} \approx 0$. The pumice volume fraction exhibits a moderate sensitivity $S_{V_{PP}}^{\bar{N}_{cr}}$, as it reduces E through homogenization, leading to the gradual decrease observed in Fig. 5.

3.4 Thermal and Coupled Thermo-Mechanical Behavior of PP-Modified Concrete Walls

In this section, the influence of pumice powder incorporation on the thermal performance of concrete walls is investigated. The analysis focuses on the coupled thermo-mechanical response of the slab, where temperature gradients induce additional stresses and potential deformations, the study aims to evaluate the effectiveness of pumice powder in enhancing the thermal insulation capacity and reducing thermally induced instability in concrete elements.

3.4.1 Thermal Resistance (R) of a Concrete Panel Reinforced with PP

The thermal resistance (R) represents the ability of a material or structural element to resist heat flow across its thickness. It is a key indicator of thermal insulation performance and is defined as the ratio between the wall thickness and its effective thermal conductivity. In the present study, the thermal resistance of the pumice powder (PP)-modified concrete panel is evaluated to quantify the improvement in insulation capacity resulting from the inclusion of PP.

Given the inherently low thermal conductivity of pumice powder (approximately $0.25\text{--}0.45 \text{ W/(m}\cdot\text{°C)}$) compared to conventional concrete (about $1.75 \text{ W/(m}\cdot\text{°C)}$), an increase in PP content is expected to enhance the overall thermal resistance of the wall. The analytical results presented here aim to establish the relationship between the volumetric fraction of PP and the corresponding rise in thermal resistance, emphasizing the potential of pumice-modified concrete for thermally efficient building envelopes.

Table 5 presents the variation of thermal resistance R for concrete panels reinforced with different volumetric fractions of pumice powder (PP), considering wall thicknesses ranging from 5 cm to 50 cm. The results reveal a clear trend: for any given wall thickness, the thermal resistance increases progressively with higher PP content. This improvement is attributed to the inherently low thermal conductivity of pumice, which reduces the effective heat transfer across the composite. On average, incorporating 10% PP enhances the thermal resistance by approximately 18–21% compared to

plain concrete, confirming the positive influence of PP addition on the thermal performance of the material.

Table 5. Variation of thermal resistance (R) of pumice powder (PP)-modified concrete panels with wall thickness (h_w) and volumetric fraction of PP

Volume fraction of PP inclusions (v_{PP})	Thermal Resistance (R) W/(m $\cdot$ °C)				
	($h_w=5$ cm)	($h_w=10$ cm)	($h_w=15$ cm)	($h_w=20$ cm)	($h_w=25$ cm)
0%	0.028	0.056	0.083	0.111	0.139
2.5%	0.029	0.058	0.087	0.116	0.145
5%	0.030	0.061	0.091	0.122	0.152
7.5%	0.032	0.064	0.096	0.128	0.159
10%	0.033	0.067	0.100	0.134	0.167
Volume fraction of PP inclusions (v_{PP})	($h_w=30$ cm)	($h_w=35$ cm)	($h_w=40$ cm)	($h_w=45$ cm)	($h_w=50$ cm)
0%	0.167	0.194	0.222	0.250	0.278
2.5%	0.175	0.204	0.233	0.262	0.291
5%	0.183	0.213	0.244	0.274	0.305
7.5%	0.191	0.223	0.255	0.287	0.319
10%	0.200	0.234	0.267	0.301	0.334

Moreover, R increases almost linearly with wall thickness, as expected from the direct proportionality $R = h_w/\lambda_{\text{hom}}$. For instance, when the thickness increases from 5 cm to 50 cm, the thermal resistance of unmodified concrete rises from 0.028 to 0.278 (around 18%), representing nearly a tenfold increase. This linear relationship demonstrates that both material composition and geometric configuration significantly affect the heat-insulating capacity of the wall.

The combined effect of PP reinforcement and increased wall thickness leads to a noticeable enhancement in thermal insulation. The inclusion of up to 10% PP yields a meaningful gain in resistance across all thicknesses, making pumice-modified concrete a promising material for thermally efficient building envelopes, particularly in energy-sensitive structural applications.

These results highlight the practical benefits of PP-modified concrete for building design. The progressive increase in thermal resistance with PP content demonstrates that pumice incorporation effectively improves insulation, reducing heat transfer through walls. Combined with thicker wall sections, the material can enhance energy efficiency in buildings. This insight allows designers to optimize PP content and wall thickness to achieve a balance between structural performance, thermal efficiency, and sustainability.

3.4.2 Thermal Transmittance (U) of a Concrete Panel Reinforced with PP

Table 6 presents the calculated values of the thermal transmittance coefficient (U) for concrete panels incorporating different volumetric fractions of pumice powder (PP), considering wall thicknesses ranging from 5 cm to 50 cm. The results show that the U -value decreases with both increasing PP content and wall thickness, indicating an improvement in the insulation capacity of the concrete panels.

For a fixed wall thickness, the inclusion of PP reduces the overall heat transfer through the panel due to the low thermal conductivity of pumice compared to that of conventional concrete. For instance, at a thickness of 10 cm, the U -value decreases from 18.00 W/m $^2\cdot$ °C for unmodified concrete to approximately 14.97 W/m $^2\cdot$ °C when 10% PP is added—representing a reduction of around 17%. This trend confirms the beneficial role of pumice in lowering the material's thermal transmittance. Additionally, the results highlight the strong dependence of U on wall thickness. As the panel becomes thicker, thermal transmittance drops significantly. For example, at 0% PP, increasing the wall thickness from 5 cm to 50 cm decreases the U -value from 36.00 to 3.60 W/m $^2\cdot$ °C, following an inverse proportionality consistent with $U = 1/R$. The combined effect of PP inclusion and increased wall thickness leads to a substantial enhancement in thermal insulation performance. These results suggest that pumice-modified concrete can effectively reduce heat

losses through structural walls, making it a sustainable material solution for energy-efficient building design.

Table 6. Variation of the thermal transmittance coefficient (U) of pumice powder (PP)-modified concrete panels with wall thickness (h_w) and volumetric fraction of PP

Volume fraction of PP inclusions (v_{PP})	Thermal Transmittance (U) $W/m^2 \cdot ^\circ C$				
	$(h_w=5cm)$	$(h_w=10cm)$	$(h_w=15cm)$	$(h_w=20cm)$	$(h_w=25cm)$
0%	36.00	18.00	12.00	9.00	7.20
2.5%	34.38	17.19	11.46	8.60	6.88
5%	32.84	16.42	10.95	8.21	6.57
7.5%	31.36	15.68	10.45	7.84	6.27
10%	29.94	14.97	9.98	7.49	5.99
Volume fraction of PP inclusions (v_{PP})	$(h_w=30cm)$	$(h_w=35cm)$	$(h_w=40cm)$	$(h_w=45cm)$	$(h_w=50cm)$
0%	6.00	5.14	4.50	4.00	3.60
2.5%	5.73	4.91	4.30	3.82	3.44
5%	5.47	4.69	4.10	3.65	3.28
7.5%	5.23	4.48	3.92	3.48	3.14
10%	4.99	4.28	3.74	3.33	2.99

The results in Table 6 show a consistent reduction in thermal transmittance (U) with both increasing PP content and wall thickness. For instance, at a wall thickness of 20 cm, increasing PP content from 0% to 10% reduces the U-value from 9.00 to 7.49 $W/m^2 \cdot ^\circ C$, corresponding to a reduction of approximately 17%. A similar proportional decrease is observed across all thicknesses. Increasing wall thickness has a more pronounced effect: for plain concrete (0% PP), increasing thickness from 5 cm to 50 cm reduces the U-value from 36.00 to 3.60 $W/m^2 \cdot ^\circ C$ (a 90% reduction), reflecting the inverse proportionality between U and thickness.

Although the incorporation of PP improves thermal performance, the resulting U-values (2.99–36.00 $W/m^2 \cdot ^\circ C$ depending on thickness) remain higher than typical building envelope insulation requirements in many standards, which often require U-values below approximately 0.5–1.5 $W/m^2 \cdot ^\circ C$ for exterior walls. Therefore, PP-modified concrete alone may not satisfy strict insulation criteria but can contribute to improved thermal resistance when combined with additional insulation layers. These results indicate that PP incorporation enhances thermal performance in a proportional and predictable manner, providing a basis for rational thickness selection when structural and thermal considerations must be balanced.

3.4.3 Thermal Flux (ϕ) of a Concrete Panel Reinforced with PP

Table 7 shows the computed thermal flux through a square concrete panel as a function of wall thickness and PP content. The results indicate that increasing the volumetric fraction of pumice powder consistently reduces the heat flux through the panel, demonstrating the insulating effect of PP. For a fixed panel thickness, the thermal flux decreases progressively with higher PP content. For example, at a thickness of 10 cm, the flux decreases from 504 W for unmodified concrete to approximately 419 W for a 10% PP inclusion, reflecting improved thermal resistance due to the low conductivity of pumice. Similarly, increasing the panel thickness significantly reduces the thermal flux for all PP fractions, following the expected inverse relationship between heat transfer and wall thickness. For instance, for 0% PP, the flux drops from 1008 W at 5 cm to 100.8 W at 50 cm, indicating that thicker panels are more effective in limiting heat transfer.

Table 8 presents the computed thermal flux through a square concrete panel under heating conditions, highlighting the effect of PP content and panel thickness on heat retention. The negative values indicate heat flow from the warmer interior to the colder exterior. As the PP volume fraction increases, the magnitude of heat flux decreases, reflecting the insulating effect of pumice. For example, at a panel thickness of 10 cm, the flux decreases from –756 W for unmodified concrete to approximately –629 W for a 10% PP inclusion, showing enhanced thermal resistance. Similarly, increasing panel thickness significantly reduces heat loss for all PP fractions. For instance, with 0%

PP, the flux drops from -1512 W at 5 cm to -151.2 W at 50 cm, indicating that thicker walls effectively reduce energy loss. The results demonstrate that incorporating pumice powder into concrete walls improves thermal insulation in cold climates, helping maintain indoor temperatures while reducing heating energy demand.

Table 7. Thermal flux (ϕ) through a 1×1 m² concrete panel reinforced with pumice powder (PP) in August, with external temperature 45°C and air-conditioned interior at 17°C , for varying panel thicknesses and PP volume fractions

Volume fraction of PP inclusions (v_{PP})	Thermal flux (ϕ) W				
	($h_w=5\text{cm}$)	($h_w=10\text{cm}$)	($h_w=15\text{cm}$)	($h_w=20\text{cm}$)	($h_w=25\text{cm}$)
0%	1008.00	504.00	336.00	252.00	201.60
2.5%	962.73	481.36	320.91	240.68	192.55
5%	919.44	459.72	306.48	229.86	183.89
7.5%	878.02	439.01	292.67	219.50	175.60
10%	838.34	419.17	279.45	209.58	167.67
Volume fraction of PP inclusions (v_{PP})	Thermal flux (ϕ) W				
	($h_w=30\text{cm}$)	($h_w=35\text{cm}$)	($h_w=40\text{cm}$)	($h_w=45\text{cm}$)	($h_w=50\text{cm}$)
0%	168.00	144.00	126.00	112.00	100.80
2.5%	160.45	137.53	120.34	106.97	96.27
5%	153.24	131.35	114.93	102.16	91.94
7.5%	146.34	125.43	109.75	97.56	87.80
10%	139.72	119.76	104.79	93.15	83.83

Table 8. Thermal flux (ϕ) through a 1×1 m² concrete panel reinforced with pumice powder (PP) in winter, with external temperature -6°C and heated interior at 36°C , for varying panel thicknesses and PP volume fractions

Volume fraction of PP inclusions (v_{PP})	Thermal flux (ϕ) W				
	($h_w=5\text{cm}$)	($h_w=10\text{cm}$)	($h_w=15\text{cm}$)	($h_w=20\text{cm}$)	($h_w=25\text{cm}$)
0%	-1512.00	-756.00	-504.00	-378.00	-302.40
2.5%	-1444.09	-722.04	-481.36	-361.02	-288.82
5%	-1379.16	-689.58	-459.72	-344.79	-275.83
7.5%	-1317.03	-658.51	-439.01	-329.26	-263.41
10%	-1257.50	-628.75	-419.17	-314.38	-251.50
Volume fraction of PP inclusions (v_{PP})	Thermal flux (ϕ) W				
	($h_w=30\text{cm}$)	($h_w=35\text{cm}$)	($h_w=40\text{cm}$)	($h_w=45\text{cm}$)	($h_w=50\text{cm}$)
0%	-252.00	-216.00	-189.00	-168.00	-151.20
2.5%	-240.68	-206.30	-180.51	-160.45	-144.41
5%	-229.86	-197.02	-172.40	-153.24	-137.92
7.5%	-219.50	-188.15	-164.63	-146.34	-131.70
10%	-209.58	-179.64	-157.19	-139.72	-125.75

These results confirm that the reduction in thermal flux with increasing PP content is directly associated with the decrease in effective thermal conductivity of the composite material. The incorporation of PP, characterized by its lower intrinsic conductivity and porous microstructure, disrupts continuous heat conduction pathways within the concrete matrix, increasing thermal resistance and limiting energy transfer. Since thermal flux is proportional to the effective conductivity and inversely proportional to wall thickness, the observed reductions reflect both material modification and geometric effects. Increasing panel thickness further elongates the heat transfer path, thereby reducing the temperature gradient per unit length and lowering the transmitted heat. The combined influence of altered conductivity pathways and increased conduction length explains the consistent reduction in heat flux under both cooling and heating conditions.

A sensitivity analysis of the steady-state heat transfer from tables 5 to 8, shows that the thermal resistance $R = h/k$ yields $S_k^R = -1$ and $S_k^\phi = 1$, meaning that a 10% reduction in thermal conductivity produces an approximately 10% increase in thermal resistance and a 10% decrease in heat flux. Since the effective conductivity is governed by the Maxwell–Eucken relation, the pumice volume fraction also exhibits a high sensitivity $S_{V_{PP}}^R$, confirming that pumice is the dominant driver of the observed thermal improvement.

3.4.4 Thermo-Mechanical Buckling of a Concrete Panel Reinforced with PP

To assess the combined effect of mechanical and thermal loads on PP-modified concrete panels, the critical thermo-mechanical buckling is evaluated using a quantitative approach that links the temperature gradient across the panel's thickness to the induced thermal stresses. The critical temperature distribution is expressed as:

$$T = \frac{\Delta T}{h_w} \left(\frac{h_w}{2} + z \right) + T_{ext}; \Delta T = T_{int} - T_{ext} \quad (36)$$

The corresponding thermal in-plane force resultant N_T is computed by integrating the thermo-elastic response of the homogenized composite through the thickness:

$$N_T = \int_{-h_w/2}^{h_w/2} E_{hom} \alpha_{hom} \left(\frac{z}{h_w} + \frac{1}{2} \right) \quad (37)$$

Table 9. Critical thermal buckling temperature (ΔT_{cr} , °C) of a square concrete panel as a function of the length-to-thickness ratio (a/h_w) for varying volumetric fractions of pumice powder (PP), under uniaxial thermal loading ($\eta = 0$)

Critical thermal buckling ΔT_{cr} ($\times 10^2$)		Volume fraction of PP inclusions (v_{PP})				
		0%	2.5%	5%	7.50%	10%
length-to- thickness ratios (a/h_w) of the plate	5	39,7759	41,2369	42,7756	44,4079	46,1425
	10	11,3409	11,7733	12,2304	12,7165	13,2345
	15	5,1753	5,3742	5,5845	5,8085	6,0472
	20	2,9386	3,0519	3,1717	3,2993	3,4353
	25	1,8890	1,9619	2,0390	2,1212	2,2088
	30	1,3149	1,3657	1,4195	1,4767	1,5377
	35	0,9675	1,0049	1,0444	1,0865	1,1315
	40	0,7414	0,7701	0,8004	0,8327	0,8671
	45	0,5862	0,6088	0,6328	0,6584	0,6856
	50	0,4750	0,4934	0,5128	0,5335	0,5556

Table 9 presents the critical thermal buckling temperatures of square concrete panels reinforced with different volume fractions of pumice powder (0% to 10%) across a range of length-to-thickness ratios (a/h_w). It is observed that increasing the PP content leads to higher critical buckling temperatures for all panel thicknesses. For example, at $a/h_w = 10$, ΔT_{cr} increases from 1134 for unmodified concrete to 1323 for 10% PP, indicating that pumice inclusion enhances the panel's resistance to thermal buckling. Additionally, ΔT_{cr} decreases as the panel becomes thinner (higher a/h_w ratio), reflecting the higher susceptibility of slender panels to thermal instability. For instance, the critical temperature drops from ~ 3978 at $a/h_w = 5$ to ~ 47.5 °C at $a/h_w = 50$ for unmodified concrete.

Results in Table 9 demonstrate that incorporating pumice powder not only improves the mechanical properties but also enhances thermal stability, allowing concrete panels to better resist buckling under thermal loads, especially in moderately slender configurations. Fig. 9 illustrates that the critical thermal buckling temperature decreases with increasing panel slenderness (higher a/h_w), indicating that thinner panels are more susceptible to thermal instability. Moreover, an increase in the volumetric fraction of pumice powder generally reduces the panel's thermal resistance to buckling, as the lower stiffness of PP compared to the concrete matrix diminishes the structural rigidity under thermal loads.

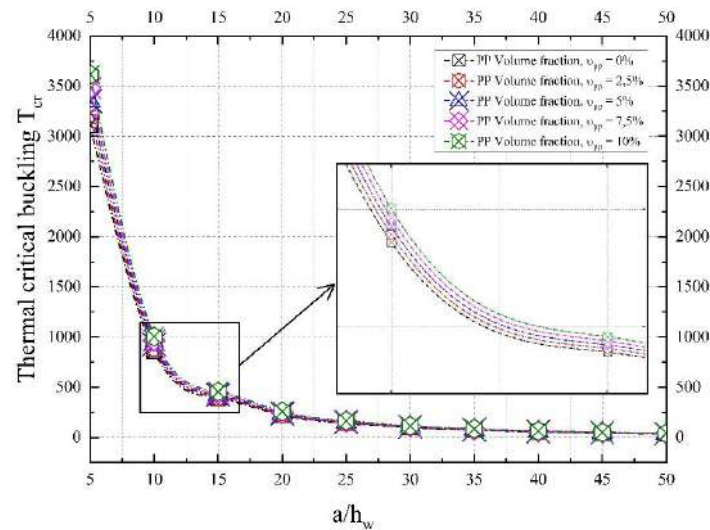


Fig. 9. Critical thermal buckling temperature of a rectangular PP-modified concrete panel ($a/b = \sqrt{2}$) as a function of the length-to-thickness ratio (a/h_w) and varying PP volume fractions

Fig. 10 shows that panels with a larger aspect ratio ($a/b = \sqrt{2}$) exhibit lower critical thermal buckling temperatures compared to square panels ($a/b = 1$), indicating that increasing panel elongation reduces stability under thermal loads. For both aspect ratios, the critical temperature decreases with increasing panel slenderness (a/h_w), highlighting the influence of thickness on thermal buckling resistance. These results emphasize the importance of geometric configuration in the thermo-mechanical performance of PP-modified concrete panels.

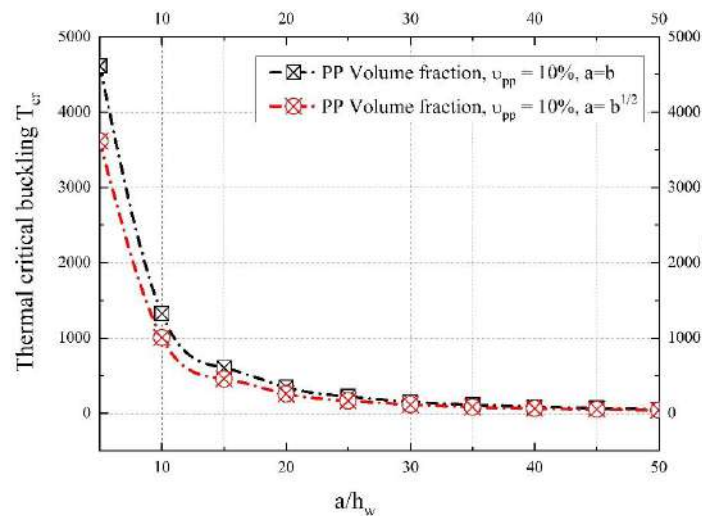


Fig. 10. thermal buckling temperature of a rectangular PP-modified concrete panel at 10% volumetric pumice content, as a function of the length-to-thickness ratio (a/h_w) for aspect ratios $a/b = 1$ and $a/b = \sqrt{2}$

The results indicate that increasing PP content leads to a systematic increase in the critical thermal buckling temperature, particularly for panels with moderate slenderness ratios. This behavior is consistent with the reduced thermal expansion and modified stiffness resulting from PP incorporation, which together influence the thermo-elastic stability response. However, panels with higher aspect ratios remain more sensitive to thermal instability, confirming the dominant role of geometric slenderness in buckling behavior. These findings highlight the coupled influence of material modification and geometry within the framework of the present analytical model, while further experimental or numerical validation would be required to generalize the conclusions to practical structural applications.

Based on the computed data, the sensitivity of the critical thermal buckling temperature (ΔT_{cr}) to variations in the effective thermal expansion coefficient (α_{hom}) can be quantified for each wall slenderness (a/h_w). It can be seen from Table 10, for a 10% PP inclusion, where α_{hom} decreases by $\sim 6.3\%$ compared to pure concrete, the relative change in ΔT_{cr} ranges from 16% to 16.7% across all a/h_w ratios. This corresponds to a sensitivity coefficient $S_{\alpha}^{\Delta T_{cr}} = [\Delta(\Delta T_{cr})/\Delta T_{cr0}]/(\Delta\alpha/\alpha) \approx 2.54 - 2.69$, indicating that ΔT_{cr} increases moderately as α_{hom} decreases.

Table 10. Sensitivity of the critical thermal buckling temperature (ΔT_{cr}) of a square PP-modified concrete panel to reductions in the effective thermal expansion coefficient (α_{hom}) for varying wall slenderness ratios (a/h_w)

Wall slenderness (a/h_w)	ΔT_{cr} (0% PP) $\times 10^2$ °C	ΔT_{cr} (10% PP) $\times 10^2$ °C	$\Delta(\Delta T_{cr})$ (%)	$\Delta\alpha_{hom}$ (%)	Sensitivity $S_{\alpha}^{\Delta T_{cr}}$
5	39.78	46.14	16.0	-6.3	2.54
10	11.34	13.23	16.7	-6.3	2.65
15	5.18	6.05	16.8	-6.3	2.67
20	2.94	3.44	16.9	-6.3	2.68
25	1.89	2.21	17.0	-6.3	2.70
30	1.31	1.54	16.9	-6.3	2.69
35	0.97	1.13	16.9	-6.3	2.68
40	0.74	0.87	16.9	-6.3	2.68
45	0.59	0.69	16.9	-6.3	2.69
50	0.48	0.56	16.9	-6.3	2.69

$\Delta(\Delta T_{cr})$ (%): Relative increase in critical thermal buckling temperature from 0% to 10% PP.

$\Delta\alpha_{hom}$ (%): Reduction in thermal expansion coefficient due to 10% PP.

The reduction in α_{hom} due to the low thermal expansion of PP reduces the thermal in-plane forces ($N_T \propto E \times \alpha \times \Delta T$) that drive thermal buckling. Consequently, the critical temperature rises, especially for moderately slender walls. Thinner or more slender walls still remain more prone to thermal instability, but the inclusion of PP consistently shifts the buckling threshold upward. This quantitative assessment provides a clearer link between microstructural modification (PP incorporation) and the panel's thermo-mechanical stability.

4. Conclusions

This study presented an analytical framework to evaluate the coupled mechanical and thermal response of pumice powder PP-modified concrete walls using a combined Reuss-Voigt-Luo homogenization scheme and refined plate theory. The model successfully captured the influence of PP on stiffness, heat transfer, and thermal buckling behavior. The main findings can be summarized as follows:

- In accordance with experimental studies, our analytical modelling show that PP addition (2.5–10%) slightly decreases the elastic modulus and critical mechanical buckling load, due to the lower stiffness of pumice.
- Effective thermal conductivity decreases by up to $\sim 8\text{--}9\%$, enhancing insulation performance.
- The thermal resistance increases by 18–20% with 10% PP, regardless of wall thickness.
- The thermal transmittance value decreases by $\sim 17\%$, limiting heat transfer under summer and winter loading conditions.
- PP reduces heat ingress in hot climates and heat loss in cold climates across all thicknesses.
- The critical thermal buckling temperature increases with PP content, indicating better resistance to temperature-induced instability.
- The homogenization model shows good agreement with experimental data for elastic modulus, thermal conductivity, and thermal expansion.

These results demonstrate that pumice powder offers a balanced combination of enhanced thermal insulation and acceptable structural stability, confirming its suitability for sustainable and energy-efficient concrete wall systems.

This work provides one of the first analytical models capable of predicting the combined mechanical, thermal, and thermo-mechanical buckling behavior of PP-modified concrete walls. The approach links microstructural material behavior to structural-scale performance and offers a predictive framework for thermo-mechanical assessment. However, several simplifying assumptions affect the model's accuracy. The homogenization strategy assumes a homogeneous and isotropic composite, which may slightly overestimate stiffness and critical buckling loads by neglecting local heterogeneity and directional porosity effects. The adoption of linear thermo-elastic behavior and idealized simply supported boundary conditions may further limit applicability under nonlinear response or realistic restraint conditions. Additionally, moisture transport, temperature dependency of material properties, aging, and interfacial transition zone effects were neglected; these factors could alter thermal conductivity and expansion coefficients, potentially leading to non-conservative predictions under long-term or severe thermal gradients. The analysis was restricted to PP contents up to 10%, and higher inclusion levels may introduce nonlinear interaction effects beyond the validity of the present formulation. These aspects define the current validity domain and motivate future multi-physics and experimentally validated extensions.

Future research should therefore incorporate nonlinear and post-buckling effects, include realistic microstructural characterizations obtained from SEM or micro-CT imaging, and examine the influence of long-term factors such as creep, cracking, moisture transport, fire exposure, and interfacial transition zone (ITZ) effects. Among these aspects, the incorporation of material nonlinearity and post-buckling behavior is the most urgent, as the present linear thermo-elastic assumption has the strongest influence on the accuracy of critical buckling predictions near instability. While the current study uses single, fixed values for the elastic modulus, thermal conductivity, and thermal expansion coefficient of pumice, these values are supported by published references, which also adopt fixed parameters for similar analyses. However, sensitivity results indicate that stiffness-related parameters exert the largest impact on predicted buckling loads, suggesting that future efforts should first refine their experimental calibration. Experimental testing on large-scale PP-reinforced concrete walls is needed to validate thermal buckling behavior and quantify real-world energy savings. Such validation is particularly important to assess the effect of the homogeneity assumption, which may lead to slight overestimation of structural stiffness at higher PP contents. Additionally, hybrid mixtures combining PP with fibers, nano-additives, or recycled materials, as well as optimization studies for different climate zones, offer promising directions to further enhance the structural and thermal performance of eco-efficient concrete.

Appendix A

The stress resultants of the PP-modified concrete panel are derived by substituting the displacement field (Eq. 21) into the constitutive relations (Eq. 23). The resulting expressions relate the bending and in-plane stress resultants to the material stiffness parameters and displacement components, can be expressed as follows:

$$\begin{aligned} \begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} &= A_{ij} \begin{Bmatrix} \frac{\partial u_x}{\partial x} \\ \frac{\partial u_y}{\partial y} \\ \left(\frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right) \end{Bmatrix} + B_{ij} \begin{Bmatrix} \frac{-\partial^2 w_b}{\partial x^2} \\ \frac{\partial^2 w_b}{\partial y^2} \\ -2 \frac{\partial^2 w_b}{\partial x \partial y} \end{Bmatrix} + B_{ij}^s \begin{Bmatrix} \frac{-\partial^2 w_s}{\partial x^2} \\ \frac{\partial^2 w_s}{\partial y^2} \\ -2 \frac{\partial^2 w_b}{\partial x \partial y} \end{Bmatrix} - \begin{Bmatrix} N_{Tx} \\ N_{Ty} \\ N_{Txy} \end{Bmatrix} \\ \begin{Bmatrix} M_x^b \\ M_y^b \\ M_{xy}^b \end{Bmatrix} &= B_{ij} \begin{Bmatrix} \frac{\partial u_x}{\partial x} \\ \frac{\partial u_y}{\partial y} \\ \left(\frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right) \end{Bmatrix} + D_{ij} \begin{Bmatrix} \frac{-\partial^2 w_b}{\partial x^2} \\ \frac{\partial^2 w_b}{\partial y^2} \\ -2 \frac{\partial^2 w_b}{\partial x \partial y} \end{Bmatrix} + D_{ij}^s \begin{Bmatrix} \frac{-\partial^2 w_s}{\partial x^2} \\ \frac{\partial^2 w_s}{\partial y^2} \\ -2 \frac{\partial^2 w_b}{\partial x \partial y} \end{Bmatrix} - \begin{Bmatrix} M_{Tx}^b \\ M_{Ty}^b \\ M_{Txy}^b \end{Bmatrix} \end{aligned}$$

$$\begin{Bmatrix} M_x^s \\ M_y^s \\ M_{xy}^s \end{Bmatrix} = B_{ij}^s \begin{Bmatrix} \frac{\partial u_x}{\partial x} \\ \frac{\partial u_y}{\partial y} \\ \left(\frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right) \end{Bmatrix} + D_{ij}^s \begin{Bmatrix} \frac{-\partial^2 w_b}{\partial x^2} \\ \frac{\partial^2 w_b}{\partial y^2} \\ -2 \frac{\partial^2 w_b}{\partial x \partial y} \end{Bmatrix} + H_{ij}^s \begin{Bmatrix} \frac{-\partial^2 w_s}{\partial x^2} \\ \frac{\partial^2 w_s}{\partial y^2} \\ -2 \frac{\partial^2 w_s}{\partial x \partial y} \end{Bmatrix} - \begin{Bmatrix} M_{Tx}^s \\ M_{Ty}^s \\ M_{Txy}^s \end{Bmatrix}$$

$$\begin{Bmatrix} Q_{xz} \\ Q_{yz} \end{Bmatrix} = A_{ij}^s \begin{Bmatrix} \frac{\partial w_s}{\partial x} \\ \frac{\partial w_s}{\partial y} \end{Bmatrix}$$

In these expressions, A_{ij} , B_{ij} , D_{ij} , B_{ij}^s , D_{ij}^s , H_{ij}^s , and A_{ij}^s represent the plate stiffness coefficients. These constants characterize the in-plane, bending, and shear stiffness of the PP-modified concrete panel and are defined through the integration of the homogenized elastic properties across the wall's thickness, reflecting the combined influence of the concrete matrix and PP inclusions.

$$\begin{aligned} A_{ij} &= b \int_{-h_w/2}^{h_w/2} \tilde{Q}_{ij} dz; B_{ij} = b \int_{-h_w/2}^{h_w/2} z \tilde{Q}_{ij} dz; D_{ij} = b \int_{-h_w/2}^{h_w/2} z^2 \tilde{Q}_{ij} dz; \\ B_{ij}^s &= b \int_{-h_w/2}^{h_w/2} f(z) \tilde{Q}_{ij} dz; D_{ij}^s = b \int_{-h_w/2}^{h_w/2} z f(z) \tilde{Q}_{ij} dz; H_{ij}^s = b \int_{-h_w/2}^{h_w/2} f(z)^2 \tilde{Q}_{ij} dz; \\ A_{ij}^s &= b \int_{-h_w/2}^{h_w/2} g(z)^2 \tilde{Q}_{ij} dz; \end{aligned}$$

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Experimental investigation of section bending and twisting resistance of I-Box web beams with constant steel area

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Abstract

Two experimental series of short-span IBWB specimens fabricated from 3-mm-thick steel plates were tested to evaluate their behavior under combined bending and torsion. Series 1 comprised hollow IBWB sections, while Series 2 included identical specimens with box webs filled with normal-strength concrete to investigate composite action. The width-to-depth (W/D) ratio varied from 0.70 to 1.30 while maintaining a constant steel area, ensuring that geometric effects could be isolated. All specimens were subjected to identical loading conditions. For the hollow beams, increasing the W/D ratio significantly enhanced torsional performance. Specimens with W/D ratios of 1.30 and 1.15 achieved increases in torsional capacity of 76.27% and 26.32%, respectively, relative to the control beam. Conversely, specimens with lower W/D ratios (0.85 and 0.70) showed only minor strength improvements or slight reductions and consistently exhibited lower rotation capacities, indicating reduced ductility. The concrete-filled specimens demonstrated the beneficial effects of composite action. The beam with W/D = 1.30 achieved a 25.91% increase in torsional strength and a rotation angle exceedingly twice that of its hollow counterpart, indicating substantial improvements in both strength and ductility. Although some concrete-filled specimens experienced slight reductions in torsional strength, they generally displayed enhanced deformation capacity. Compared with equivalent hollow sections, all concrete-filled beams exhibited significant increases in torsional capacity, ranging from 182% to 213%. Overall, geometric configuration strongly influenced torsional behavior. Higher W/D ratios improved strength and stiffness, while lower ratios provided limited benefits. Concrete infill markedly enhanced torsional resistance, delayed local buckling, and improved overall structural performance, particularly in specimens with larger W/D ratios.

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1. Introduction

The Thin-walled cold-formed steel (CFS) is increasingly adopted in structural applications because of the rising demand for members that provide high strength with low weight while effectively fulfilling their intended functions. In recent years, the use of built-up CFS sections has gained significant popularity, particularly in developing countries. CFS sheets can be manufactured into a wide range of geometric shapes and may be used either as individual sections or connected together to form built-up assemblies. Structural steel sheets are produced in various grades,

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ranging from low- to high-tensile strength, and are readily Hybrid girders are commercially available [1,2] and are defined as beams made from different steel grades in the flanges and the web. In this study, this concept is implemented by using higher-strength steel for the flanges relative to the web. Eurocode 3-1-3 provides guidelines for hybrid girders, stating that the ratio of the basic yield strength of the flanges to that of the web should not exceed 2. The built-up cold-formed steel (CFS) I-Box Web Beam (IBWB) section examined here consists of four individual components assembled into a single structural unit. The flanges are formed from plain channel sections with a single intermediate stiffener, while the web is constructed from a lipped channel section. These components are joined together using bolted connections fasteners. For homogeneous IBWB sections, both the webs and flanges are fabricated from low-tensile steel plates of the same grade. In contrast, the Hybrid I-Box Web Beam (IBWB) sections utilize low-tensile steel plates for the webs, whereas the flanges are fabricated from either medium- or high-tensile steel plates [3]. More broadly, the principle that optimizing material composition and component ratios yields measurable improvements in structural performance has been demonstrated across various composite civil engineering systems

This study aims to explore the behavior of an I-section double-web CFST beam under torsion and bending, focusing on the parameters addressed in this research, as detailed below.

- Experimentally evaluate the torsional behavior of hollow and concrete-filled cold-formed steel box-web beams.
- Evaluates how variations in width-to-depth ratio, steel area, and concrete infill affect torsional stiffness and ultimate capacity.
- The compressive strength and density of concrete influence its torsional response. It aims to understand these relationships to enhance the performance of steel concrete structures under torsional loads.
- Investigating the structural behavior of the proposed IBWB section under torsional loading
- Comparing the torsional performance and stiffness characteristics with those of conventional sections.

1.1 Research Background

Existing design codes do not provide provisions for the design of built-up closed-form cold-formed steel (CFS) members, underscoring the need for further research to better understand their structural behavior. Accordingly, a brief review of previous studies relevant to the present investigation is summarized below. A numerical study on CFS built-up beams with perforated webs showed that web openings had a more pronounced effect on the strength of beams with built-up closed sections compared to those with open sections [4].

Four-point bending and torsion tests were conducted to examine the in-plane behavior of thin-walled, cold-formed built-up box beams composed of nested C- and U-shaped sections. The results indicated that, at mid-span, the components of the built-up box beams acted integrally to resist combined torsion. Additionally, it was recommended that the strong-axis bending moment capacity of built-up box beams be taken as 90% of the sum of the individual component capacities [5-7]. Experimental and numerical investigations on laterally restrained cold-formed steel rectangular hollow flange beams (RHFBS) revealed that variations in yield strength between the tubular flanges and the web significantly influenced flexural performance, promoting local buckling and plastic deformation in the web.

The inclusion of an intermediate stiffener in the external plates of the hollow flanges was found to enhance the critical local buckling capacity of the plates [8]. Finite element analysis (FEA) was employed to study the bending strength of built-up box sections commonly used in residential and commercial construction in North America [9].

The study showed that closed-form built-up sections exhibit higher torsional rigidity and that design equations based on the modified Direct Strength Method (DSM) [12] tend to be overly conservative for these sections. Additional numerical studies indicated that, under concentric loading, the flexural strength of built-up box sections can be reasonably approximated as the algebraic sum of the nominal strengths of the individual components. However, this approach was

found to be unconservative under eccentric loading, for which a reduction factor of 0.9 was recommended [10,11].

Other studies highlighted that hybrid sections outperform homogeneous sections in terms of strength. Specifically, I-sections achieve higher bending resistance with reduced cross-sectional area while effectively resisting large longitudinal shear forces [13]. Experimental and numerical investigations also revealed that built-up closed-form sections offer a higher strength-to-weight ratio than open mono-symmetric sections. Open sections typically fail by lateral-torsional buckling, whereas closed sections are more prone to distortional buckling [14,15].

Research on built-up I-beams identified steel grade, flange slenderness ratio, and the effective width-to-thickness ratio of compression flanges as key parameters controlling flexural capacity [16]. Moreover, the use of high-strength steel (HSS) was found to be advantageous in enhancing structural capacity, particularly when web depth is limited [17].

Overall, the literature shows that studies on hybrid and closed-form built-up sections remain limited. Given the widespread use of hybrid girders and built-up members in practice, further investigation is warranted, along with the development of appropriate design provisions in codes and handbooks [18,19]. The present study aims to address this gap. Based on experimental findings and parametric analyses, several conclusions were drawn. The fabricated test specimens and the cross-sectional configurations of IBWB sections with varying width-to-depth (W/D) ratios are shown in Figure 1.

2. Experimental Investigation

The experimental study on built-up cold-formed steel (CFS) IBWB sections was conducted at the Advanced Structural Engineering Laboratory, Engineering College, Babylon University, Iraq. A specialized and uniquely designed test setup was fabricated to perform combined torsional tests on short-span beam specimens.

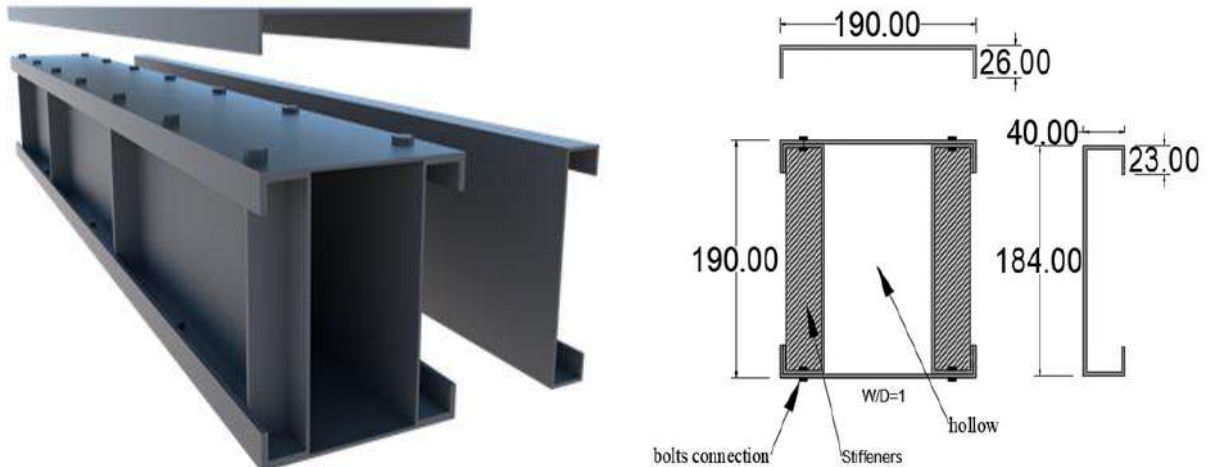


Fig. 1. I-Box Web Beam (IBWB) sections Actual cross-section and (d) Varying W/D ratios with constant steel area compared with control specimen

2.1 Test Specimens

Two series of tests were conducted using box sections with different strength levels. Each series comprised five specimens with varying width-to-depth (W/D) ratios, while maintaining a constant steel area relative to the control specimens.

Series 1 consists of thin-walled homogeneous IBWB sections, in which both the flanges and webs were made from low-tensile-strength steel sheets of the same grade. Series 2 comprises sections identical to those in Series 1, except that the box web was infilled with normal concrete. For all specimens, the web thickness (t_w) was 3.0 mm, and the flange thickness was also 3.0 mm.

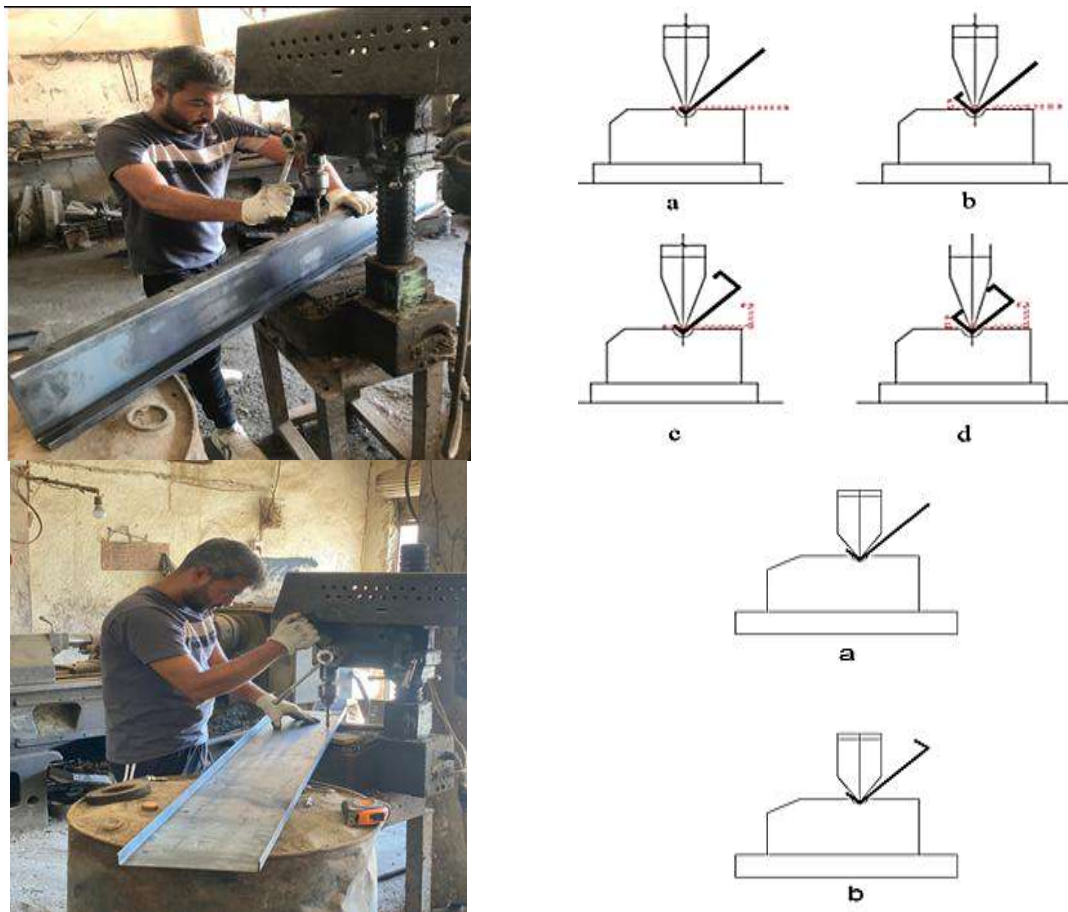


Fig. 2. Connecting parts of sections

Table 1. Identification for specimen dimensions

Specimen identification	Description
BH 1	CFS I-beam with rectangular hollow Web (W=190mm, D=190mm)
BC 1	CFS I-beam with rectangular hollow Web filled with concrete (W=190mm, D=190mm)

Table 2. Details for cross-section of specimens of control specimen

Specimen identification	D mm	W mm	W/D	t _w mm	t _f mm	Location of connection	A _s mm ²	A _c mm ²
Control Specimen	190	190	1	3	3	L/6	4860	19136

Table 3. Identification for flange depth of Series 1

Specimen identification	Description
BH1.3	CFS I-beam with constant area of steel compared with control specimen and hollow box Web (W=202mm, D=175mm)
BH 1.15	CFS I-beam with constant area of steel compared with control specimen and hollow box Web (W=202mm, D=175mm)
BH 0.85	CFS I-beam with constant area of steel compared with control specimen and hollow box hollow Web (W=175mm, D=204mm)
BH 0.7	CFS I-beam with constant area of steel compared with control specimen and hollow box hollow Web (W=155mm, D=221mm)

Table 4. Details for cross-section of specimens of Series 1

Specimen identification	D mm	W mm	W/D	t_w mm	t_f mm	Location of connection	A_s mm ²
BH1.3	163	211	1.3	3	3	L/6	4860
BH 1.15	175	202	1.15	3	3	L/6	4860
BH 0.85	204	175	0.85	3	3	L/6	4860
BH 0.7	221	155	0.7	3	3	L/6	4860

Table 5. Identification for flange depth of Series 2

Specimen identification	Description
BC1.3	CFS I-beam with constant area of steel compared with control specimen and concrete in side of box Web (W=202mm, D=175mm)
BC 1.15	CFS I-beam with constant area of steel compared with control specimen and concrete in side of box Web (W=202mm, D=175mm)
BC 0.85	CFS I-beam with constant area of steel compared with control specimen and concrete in side of box hollow Web (W=175mm, D=204mm)
BC 0.7	CFS I-beam with constant area of steel compared with control specimen and concrete in side of box hollow Web (W=155mm, D=221mm)

Table 6. Details for cross-section of specimens of Series 2

Specimen identification	D mm	W mm	W/D	t_w mm	t_f mm	Location of connection	A_s mm ²	A_c mm ²
BC1.3	163	211	1.3	3	3	L/6	4860	20618
BC 1.15	175	202	1.15	3	3	L/6	4860	19604
BC 0.85	204	175	0.85	3	3	L/6	4860	17622
BC 0.7	221	155	0.7	3	3	L/6	4860	14835

The width-to-depth (W/D) ratios of the beams—0.70, 0.85, 1.00, 1.15, and 1.30—are indicated at the end of the specimen nomenclature. Table 1 presents Identification for Specimen dimensions. Series 1 has a constant steel area equal to that of the control specimen, and all details regarding the identification and dimensions of these specimens are presented in Tables (3 and 4). Series 2 has the same cross-sectional details and dimensions as Series 1, with the only difference being the area of the web filled with normal concrete.

2.2 Mechanical Properties

Since built-up hybrid beams are fabricated using steels of different grades, accurate determination of their mechanical properties and corresponding stress-strain relationships are essential. The compressive strength of the concrete infill used in Series 2 ($f'_c \approx 36$ MPa) was verified through standard testing; non-destructive methods such as ultrasonic pulse velocity has also been widely employed for in-situ concrete strength assessment [20]. Accordingly, a series of coupon tests was conducted to evaluate the mechanical characteristics of the various steel grades employed in fabricating These sections were tested, and the resulting stress-strain curves and material properties were then used to estimate the moment capacities of the built-up members, and the young's modulus for concrete for normal concrete is $E=4700\sqrt{f'_c}$.

The tensile test specimens were prepared in accordance with the dimensions shown in Figure 3, with all coupons having a thickness of 3 mm. Coupon testing represents a standard approach for evaluating the mechanical behavior of metallic materials under tensile loading, with results directly informing structural capacity predictions [21]. The tests were carried out in the Mechanical Engineering Laboratory at the University of Babylon. The test setup, including the gripping arrangement and the tested specimens, is illustrated in Figure 4. The outcomes of the tensile tests—namely the yield strength and ultimate tensile strength of the flange and web materials—are summarized in Table 7.

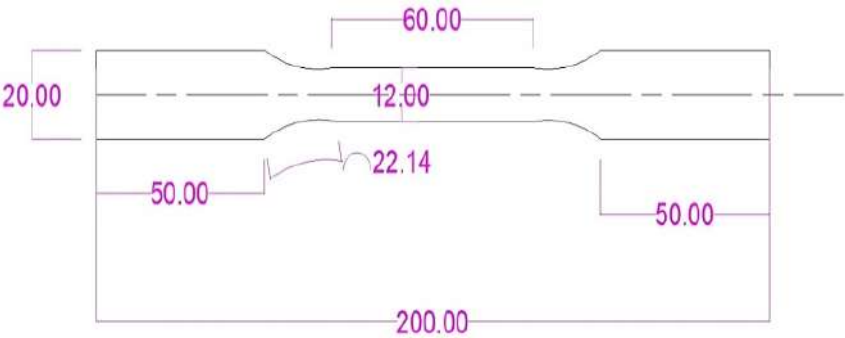


Fig. 3. Dimensions of the tensile test specimen (thickness = 3 mm). According to ASTM.



Fig. 4. Tensile testing machine with mounted specimen and tested coupons.

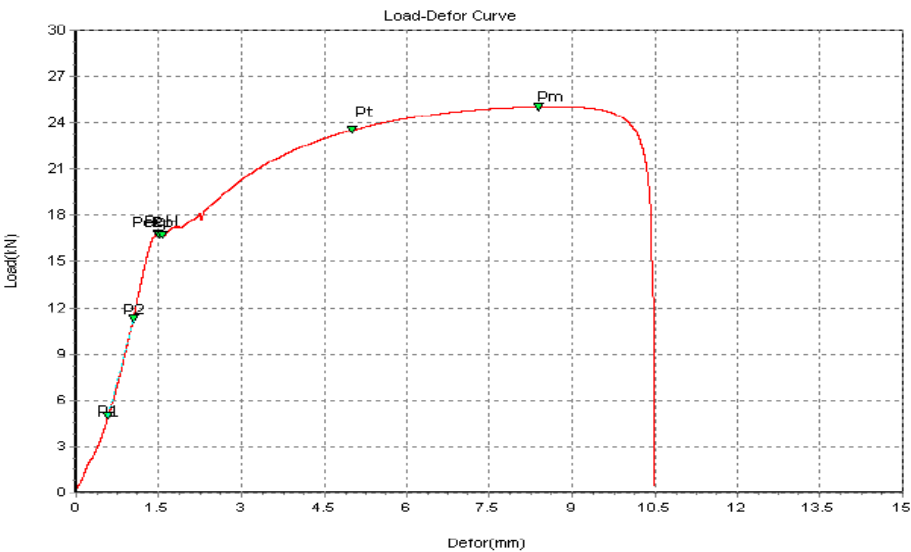


Fig 5. Load–deformation curve of steel coupons obtained from tensile testing

Table 7. Yield and ultimate tensile stress values of steel plate coupons (flanges and webs)

Specimen NO	Yield stress (MPa)	Ultimate strength (Mpa)	Young's modulus (GPa)
1	225	295	200
2	230	305	
Average value	227.5	300	

2.3 Test Set-Up and Procedure

During testing, the overall structural response of each beam specimen was carefully The specimens were monitored at every stage of loading. Testing was performed using a universal testing machine

with a maximum capacity of 30 tons. A two-point loading configuration was employed to induce combined torsion, with a torsional moment arm of 450 mm corresponding to the applied torque. Two linear variable differential transformers (LVDTs) were installed on the torque arms to measure the angle of twist, while an additional LVDT was positioned at midspan to record vertical deflection. The beams were subjected to combined bending and torsion under a constant loading rate. The load at the onset of the first crack was recorded, and the development of cracks and the failure mode were continuously observed [22-24]

It should be noted that the loading procedure and the positioning of the specially fabricated specimens required significant care. Since the supports allowed free rotation, achieving proper balance of the specimens was challenging and necessitated careful handling. All specimens were tested under progressively increasing torque until failure. The load was applied incrementally, with instrument readings recorded manually throughout the test. At each load increment, strains in both the tension and compression zones were measured using strain gauges connected to a data acquisition system and a computer was gradually increased until ultimate failure occurred. The experimental setup and instrumentation employed in the tests are shown in Figure 6.



Fig. 6. Member under Torsional load

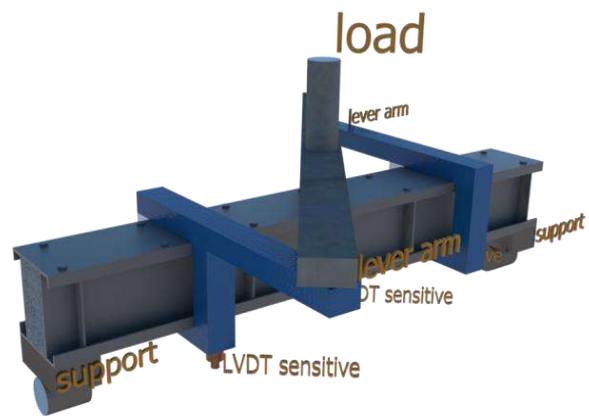


Fig. 7. Location of LVDT sensors

3. Results and Discussion

The applied bending moment was determined by multiplying the measured load by the distance between the support and the point of load application. In the built-up sections, the web elements were consistently classified as non-compact (NC), whereas the flanges were either slender (S) or non-compact. Consequently, the assembled IBWB sections were categorized as either non-compact or slender based on their controlling slenderness criteria of the flanges. Table (8) summarizes the overall behavior of the tested cold-formed steel box-web specimens, presenting critical parameters such as the maximum torque capacity, ultimate rotation, and mid-span deflection for each beam. These results form the basis for comparative analysis across different Series, which were designed to investigate the influence of geometric and material variations on structural response.

Table 8. Experimental Results of specimens in Series 1 and Series 2

Group name	Symbol number	Ultimate Load (kN)	Ultimate torque applied (kN.m)	Ultimate rotation (rad)	Increasing the load carrying capacity %	Mid-span deflection (mm)
Series 1	BH1.3	130.064	29.2644	0.0357755	76.27	13.107692
	BH 1.15	93.208	20.972	0.025435	26.32	13.530939
	BH 1	73.7831	16.6012	0.0360654	-----	16.41514
	BH 0.85	72.259	16.2583	0.0247104	-2.06	16.64468
	BH 0.7	81.214	18.2732	0.0356199	9.98	17.32698

Group name	Symbol number	Ultimate Load (kN)	Ultimate torque applied (kN.m)	Ultimate rotation (rad)	Increasing the load carrying capacity %	Mid-span deflection (mm)
Series 2	BC1.3	259.542	58.397	0.07636	25.91	12.711599
	BC 1.15	185.016	41.6286	0.0550257	-10.24	14.954822
	BC 1	206.131	46.3796	0.0376851	-----	17.526251
	BC 0.85	132.113	29.7256	0.0317171	-35.9	15.535042
	BC 0.7	173.120	38.9526	0.0356381	-16.01	15.824175

3.1 Series 1

Figure 8 presents the torque–rotation curves obtained for all specimens within this Series 1 together with an integrated comparative analysis. The key findings are as follows:

- Beams with $W/D > 1$ (BH1.3, BH1.15): These specimens exhibited a significant increase in torsional strength compared with the control. BH1.3 showed a 76.3% increase in T_u , while BH1.15 achieved a 26.3% increase. However, their ultimate rotation was slightly reduced, indicating higher stiffness but reduced ductility.
- Beams with $W/D < 1$ (BH0.85, BH0.7): The response was less favorable. BH0.85 recorded a slight reduction in T_u (–2.1%) relative to the control, whereas BH0.7 achieved a modest increase of +9.98%. In both cases, the ultimate rotation was lower than that of the control, suggesting limited ductility.

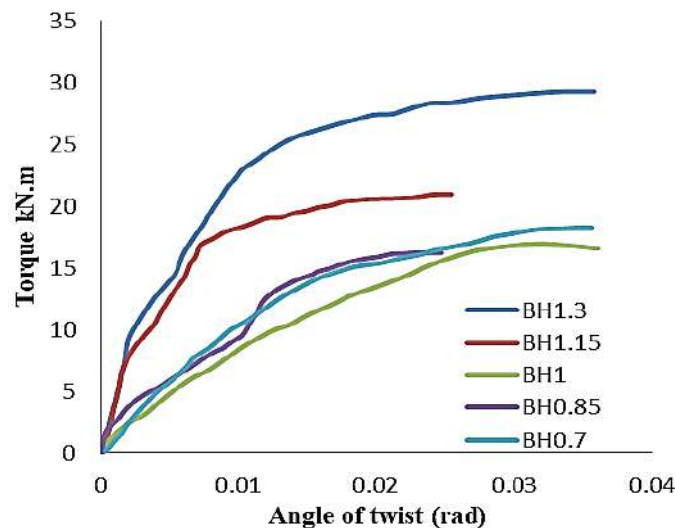


Fig. 8. Torque -Rotation curves for Series 1

- Failure modes: Consistent with the earlier groups, torsional buckling was the dominant mechanism of failure Figure (9 to 13). Figure 8 compares the torque–rotation curves of all specimens in Group 3: Steeper initial slopes for BH1.3 and BH1.15 reflect higher torsional stiffness ($K \propto GJ$), as the increase in W/D improved the torsional constant (J) without increasing the overall steel area. Higher torque capacity but reduced twist capacity: The larger W/D ratios effectively delayed buckling and increased strength, but this enhancement was accompanied by reduced ultimate rotation, consistent with the trade-off between stiffness and ductility. Specimens with $W/D < 1$ (BH0.85 and BH0.7) failed to develop both strength and ductility compared to the control. The lower J associated with smaller widths limited torsional resistance, while constant steel area did not provide additional confinement or stiffness to compensate for this deficiency.



Fig. 9. Failure Mode of Beam BH1.3



Fig. 10. Failure Mode of Beam BH1.15



Fig. 11. Failure Mode of Beam BH1



Fig 12. Failure Mode of Beam BH0.85



Fig 13. Failure Mode of Beam BH0.7

The findings of Series 1 carry several implications for real-world design: Design balance between strength and ductility: Beams with $W/D \geq 1.15$ demonstrated improved torsional strength but reduced rotational capacity. This highlights the importance of balancing geometry to ensure that enhanced strength does not come at the expense of brittle behavior. Limitations of low W/D designs: Beams with $W/D < 1$ are not suitable for torsion-dominated applications, as they provide neither adequate strength nor sufficient ductility. Constant steel area sections reflect cost-efficiency trade-offs: By maintaining the same steel area, Series 1 illustrates that geometric optimization (via W/D) can modestly improve performance without increasing material cost.

3.2 Series 2

This Series examined the behavior of cold-formed steel box-web beams with constant steel area but filled with nonstructural concrete ($f'_c \approx 36$ MPa). Five specimens were tested: BC1.3, BC1.15, BC0.85, BC0.7, and the control beam BC1 ($W/D = 1$). The geometry was identical to Group 3, but with the addition of concrete infill to assess the impact of composite action under torsional loading. Experimental Observations The torque-rotation relationships are shown individually in Figures 14 to 18, with a comparative overview in Figure 14. Key findings include: Enhanced torsional strength: Beams with higher W/D ratios demonstrated superior torsional strength compared to the control. BC1.3 exceeded BC1 by +25.9%, while BC1.15 recorded a slight reduction (-10.2%) in T_u . The control beam BC1 itself exhibited a very high torsional strength relative to hollow sections (279% higher than BH1). Reduced or variable rotation capacity: BC1.3 showed a significant increase in ultimate rotation (+105%), while BC1.15 exhibited improved ductility (+48.6%). Conversely, beams with smaller W/D ratios (BC0.85, BC0.7) recorded both lower T_u and reduced rotation compared to the control. Failure modes: Figure (20 to 24) confirm that torsional buckling governed the response, but concrete infill enhanced resistance to local buckling and provided additional confinement. Figure 14 highlights the combined performance of Series 2 beams. Concrete infill as a strength enhancer: Compared to their hollow counterparts in Series 1, the concrete-filled beams achieved substantially higher torsional resistance. For example, BC1.3 achieved +197% higher T_u than BH1.3. This enhancement is explained by the composite action: the steel box resists and confines the concrete, while the concrete delays local buckling of the thin steel walls and increases energy absorption.

Strength-ductility trade-off: While torsional strength increased with concrete infill, ductility did not always follow the same trend. For instance, BC1.3 combined high strength and exceptional ductility ($\theta_u = 0.076$ rad), whereas BC0.85 suffered reductions in both T_u and θ_u . This indicates that geometry (W/D) still plays a critical role even in composite sections. Low W/D performance: Beams with $W/D < 1$ (BC0.85, BC0.7) could not achieve the same improvements. Despite the presence of concrete, the reduction in torsional stiffness due to geometry (low J) limited their effectiveness. Explanation of Behavior Composite effect: The confinement of concrete by the steel box improved torsional resistance, stiffness, and energy absorption capacity. Geometric influence: Larger W/D ratios increased the torsional constant J , amplifying the benefits of concrete infill. In contrast, small W/D ratios limited J and undermined the potential gains from concrete. Failure

mechanism: Torsional buckling was still the governing mode, but it occurred at higher torque levels due to the stabilizing effect of the concrete.

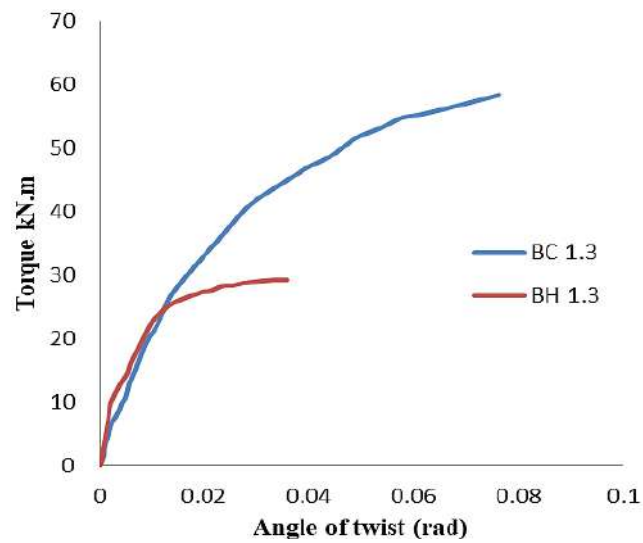


Fig 14. Torque rotation curve for beam (BC1.3)

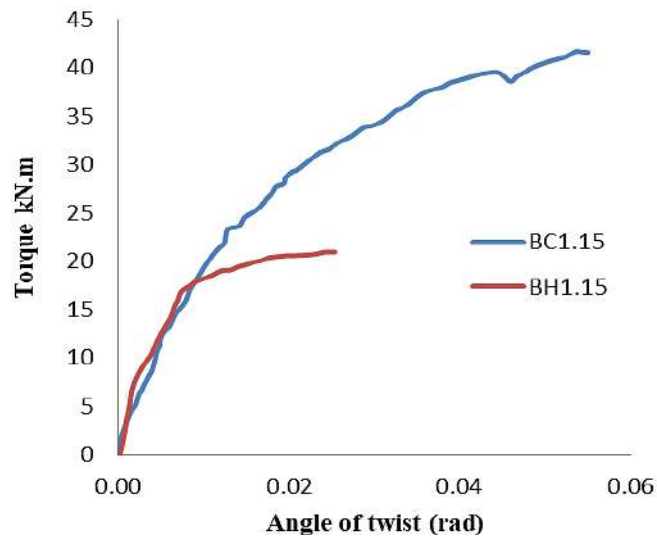


Fig 15. Torque rotation curve for beam (BC1.15)

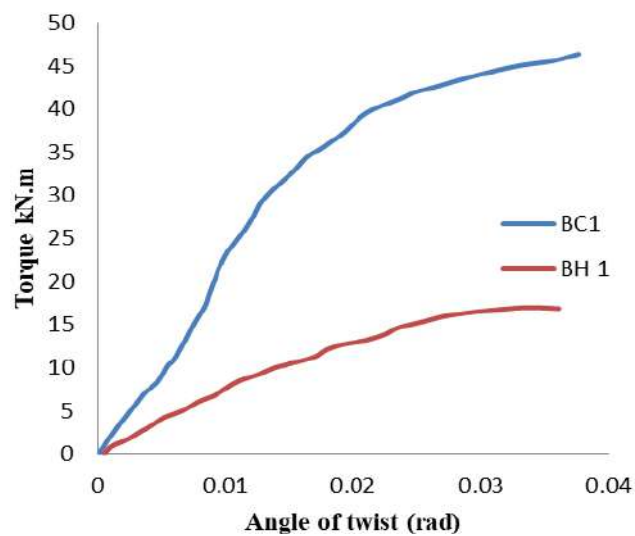


Fig 16. Torque rotation curve for beam (BC1) control beam

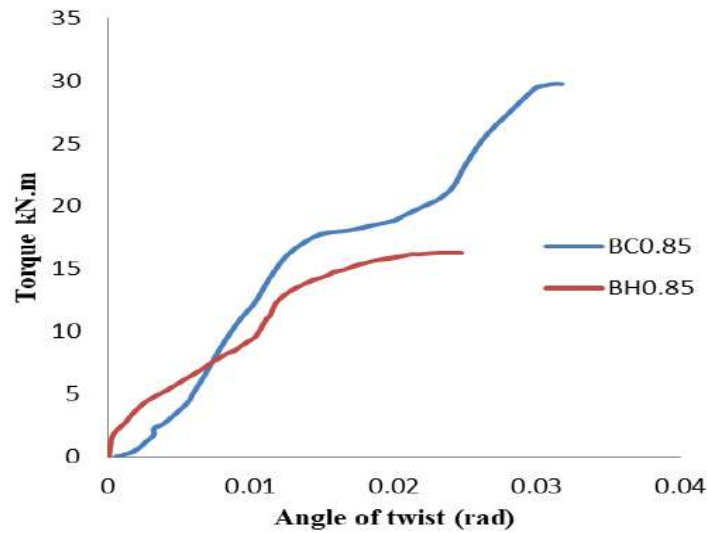


Fig 17. Torque rotation curve for beam (BC0.85)

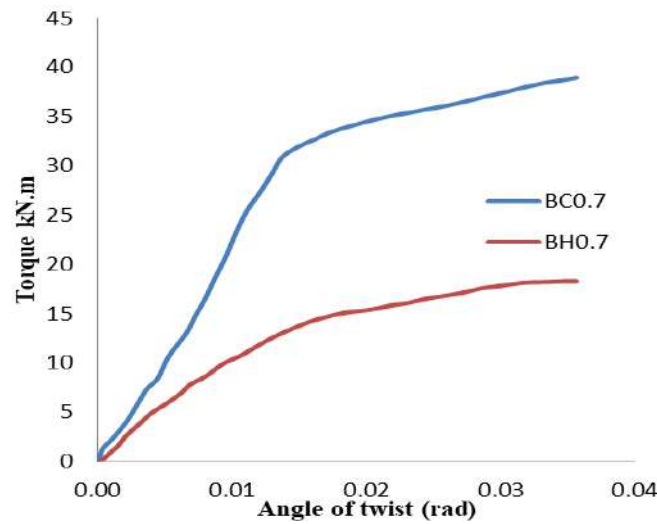


Fig 18. Torque rotation curve for beam (BC0.7)

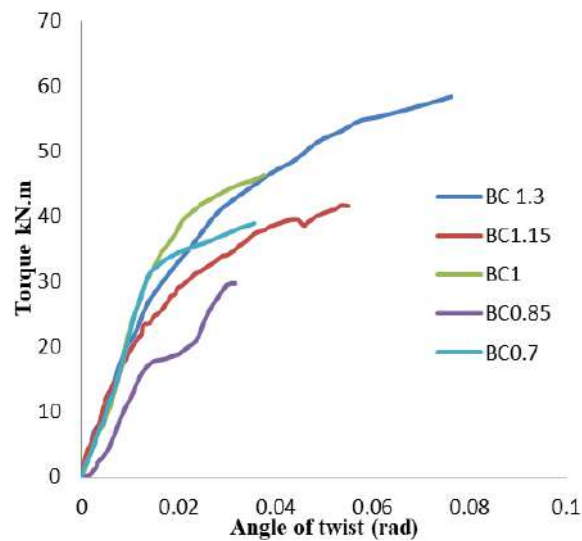


Fig 19. Torque -rotation curves for series 2

Interpretation of Figure 17 (Specimen BC0.85) Observed behavior: BC0.85 reached lower torsional strength (-35.9% vs. BC1) and smaller ultimate rotation (-16.2% vs. BC1). The curve shows a stiff

initial slope but fails early with limited ductility. Reason: The low W/D ratio (0.85) reduces the torsional constant J , limiting torque resistance. Concrete infill improved initial stiffness and delayed buckling but could not overcome the geometric disadvantage. Failure was dominated by torsional buckling at lower twist, explaining the reduced ductility. Practical Significance: The outcomes of Series 2 provide several design insights: Concrete-filled box sections are highly efficient under torsion, especially when designed with $W/D \geq 1.15$. They provide superior torsional strength and, in some cases (e.g., BC1.3), exceptional ductility. Geometry remains critical: Concrete infill cannot fully compensate for the disadvantages of low W/D ratios. Designers should avoid $W/D < 1$ for torsion-critical applications, even when concrete is used.

Table 9. Stiffness Parameter of all specimens

Group three constant area with control hollow box	BH1.3	29.2644	0.0357755	819.71
	BH 1.15	20.972	0.025435	825.669
	BH 1	16.6012	0.0360654	461.11
	BH 0.85	16.2583	0.0247104	658.218
	BH 0.7	18.2732	0.0356199	513.286
Group four constant area with control concrete box	BC1.3	58.397	0.07636	765.36
	BC 1.15	41.6286	0.0550257	756.87
	BC 1	46.3796	0.0376851	1233.244
	BC 0.85	29.7256	0.0317171	937.53
	BC 0.7	38.9526	0.0356381	1094.15

Balanced performance: BC1.3 illustrates the ideal balance, achieving both high torsional strength and large twist capacity. Such behavior is desirable in seismic or fatigue-prone structures where both resistance and energy dissipation are required. Practical applications: These findings are directly relevant to the design of bridge girders, offshore platforms, transmission towers, and tall buildings, where torsional effects are significant. Incorporating concrete infill with optimized W/D ratios can lead to safer, stiffer, and more ductile structural systems. Torsional stiffness (K) is a fundamental parameter describing the ability of a structural element to resist angular deformation under applied torque. It is defined as the ratio of ultimate torsional moment to the corresponding ultimate angle of twist:

$$\text{Stiffness } (K) = Tu/\theta_u \quad (1)$$

Where; Tu = ultimate torsional moment (kN·m), θ_u = ultimate rotation (rad).



Fig. 20. Failure mode of beam BC1.3



Fig. 21. Failure mode of beam BC1.15



Fig. 22. Failure mode of beam BC1



Fig. 23. Failure mode of beam BC0.85



Fig. 24. Failure mode of beam BC0.7

4. Conclusion

4.1 Series 1

- Effect of W/D Ratio: Increasing the W/D ratio enhances torsional capacity, with BH1.3 and BH1.15 exceeding the control by 76.27% and 26.32%, respectively.
- Effect on Rotation: Larger W/D ratios resulted in reduced rotation capacity, while smaller ratios demonstrated slight increases in twist before failure, indicating improved flexibility.
- Overall Torsion–Rotation Interaction: Variation in steel area significantly affects torsional response more than the W/D ratio when steel area remains constant.
- Best Performance: BH1.3 demonstrated the optimal balance of strength and ductility.
- Effect of Constant Steel Area: Maintaining a constant steel area improved torsional resistance responses, aligning more closely with the control specimen, thus highlighting material quantity's key role.

4.2 Series 2

- Effect of Concrete Filling: Concrete infill significantly enhanced the torsional capacity, with the control beam (BC1) demonstrating a 279.33% improvement compared to its hollow counterpart.
- Influence of W/D Ratio: BC1.3 demonstrated the highest torsional strength with a 25.9% increase and a 105.4% rise in rotation angle. BC1.15, while showing a 10.24% strength decrease, achieved a rotation improvement of 48.64%. Lower W/D ratios (BC0.85, BC0.7) led to diminished strength and twist capacity.
- Composite Behavior: The combined action of steel and concrete effectively restrained inward/outward buckling and crack propagation, improving both strength and durability.
- Constant Steel Area Effect: Fixing the steel area reduced the influence of geometry on torsional strength, highlighting differences in deformation behavior.
- Optimal Specimen: BC1.3 exhibited the best overall balance of strength and rotational capacity, confirming the advantage of larger W/D ratios combined with concrete infill.

The mechanical contribution of mineral additives to composite material matrices has been similarly reported in related civil engineering contexts [24].

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Performance evaluation of fly ash-based sustainable reactive powder concrete reinforced with hybrid steel–glass fibers

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Abstract

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Reactive Powder Concrete (RPC) is a high-strength cement-based composite and low water-to-cementitious ratio; however, it is characterized by a high cement content, which raises sustainability concerns. The objectives were to produce eco-friendly products. RPC with replacing cement partially with fly ash and to optimize hybrid reinforcement with steel–glass fibers to enhance mechanical strength and, subsequently, lower environmental impacts. The fly ash replaced cement at the rate of 20%, and the total fibers were fixed at a constant of 1% by volumetric ratios of hybrid steel to glass fibers (1:0 steel, 0.75:0.25, 0.50:0.50, 0.25:0.75, and 0:1 glass). Compressive strength at 28 days, flexural, and splitting tensile strengths of the 20% fly ash with the incorporation of micro steel fibers (1%) were found to be higher than reference mix without fly ash under thermal curing increased by 4.47%, 2.39% and 2.01% respectively, In contrast, Partial replacement of steel fiber with glass fiber at different hybrid ratios The findings showed gradual compressive strength reduction (1.05–3.33%), flexural strength (1.62–17.67%), and splitting tensile strength (1.57–14.61%) as compared to control mix containing 1% micro steel fibers. Both fly ash and fiber hybridization influenced the density, water absorption, and porosity of RPC.

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1. Introduction

Concrete technology has evolved rapidly into a new composite material known as Reactive Powder Concrete (RPC), which produces better structures and can be stronger than standard concrete [1]. RPC consists of extremely fine powders (Portland cement, sand, quartz powder, and silica fume). It sometimes (not always) uses steel fibers and always uses superplasticizers to reduce the water-to-cement ratio (w/c) below 0.2, and must raise the workability of RP [2]. RPC is a unique form of high-strength concrete characterized by its exceptionally high compressive strength and durability [3,4]. However, it requires a lot of cement and silica fume in its production, increasing costs and environmental impacts due to the cement industry's environmental impact [5]. Fly ash reduces the heat of hydration, improves workability, and, due to its spherical shape, decreases water demand. Fly ash enhances durability by increasing sulphate attack resistance and polishing the microstructure. Moreover, its application can significantly reduce energy consumption and CO₂ emissions during cement production [6,7]. Although RPC is a superior-performing material compared to conventional concrete, it behaves as a brittle material under tensile and compressive loading; hence, fiber reinforcement is widely used to improve toughness and post-cracking behavior [7,8]. Using Short steel fibers in concrete usually improves crack performance and post-cracking ductility, as well as the energy absorption capacity of structural elements [9]. While widely used in RPC, steel fibers are prone to corrosion when exposed to the concrete surface [10]. Glass fiber has a high potential for direct replacement of steel fiber due to its good corrosion resistance and relatively low density; a hybrid combination between the two should also be studied in RPC.

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Using sustainable supplementary materials in combination with glass fibers improves concrete's mechanical performance, extends its service life, and supports sustainability [6]. Nevertheless, it remains critical and a research challenge to optimize RPC to achieve both high mechanical performance and sustainability simultaneously using hybrid fiber systems in combination with supplementary cementitious materials.

Study [11] investigated the behavior of RPC incorporating single fibers (SF, GF, CF) and two hybrid fiber types (SF-GF, GF-CF, and CF-SF), keeping the fiber volume fraction constant at 2% and including a control mix. CF-RPC performed better in compressive property, while SF-RPC performed well in splitting tensile strength and flexural strength. However, CF-RPC had tensile and flexural strengths similar to those of SF-RPC but lower toughness. The hybrid combination with 1% SF and 1% CF displayed the best overall performance, whilst the fiber contribution to flexural strength ranged from 17% to 38%, which was greater than for other strength properties.

In terms of sustainability, study [7] highlighted the potential to produce sustainable RPC by replacing a portion of cement with fly ash (8, 12, 16) % and reported that 8% fly ash yielded ideal compressive strength (96.5) Mpa, (9.38) Mpa, and density (2395 kg/m³). Furthermore, study [12] demonstrated that the incorporation of micro steel fibers under warm-water curing conditions resulted in the highest compressive and flexural strengths of 138.9 MPa and 22.4 Mpa, respectively. Under these conditions, the compressive strength improved by 27.96% over that of the reference mix, and the flexural strength was also enhanced by 39.55% when micro steel fibers were blended into the mixtures. The splitting tensile strength at peak was 20.89 Mpa indicating the significant role of curing regime and fiber type in enhancing RPC performance. Additionally, study [13] examined the influence of glass fibers at volume fractions ranging from 0.1% to 0.4%, with an optimum content (0.75%). At this ideal proportion, the compressive, splitting tensile, and flexural strengths increased by 15%, 24%, and 27%, respectively, compared to plain RPC.

These results show that while individual parameters have been studied extensively, their combined influence during controlled curing remains under-researched. In general, the reviewed works show that fiber type and content have a significant effect on the mechanical behavior of RPC (especially flexural and tensile properties). Steel fibers contribute strength and toughness; their crack-bridging potential is sufficiently efficient, while glass fibers might contribute to ductility, but excessive content of them renders the mechanical performance even worse than pure concrete due to their lower stiffness and weaker interfacial bond with the matrix. In addition, the use of fly ash also contributes to sustainability. It facilitates a pozzolanic action that densifies the microstructure, but its effects are strongly dependent on replacement levels, curing conditions, and mix design.

However, the cumulative effects of mixing supplementary cementitious materials with fiber systems on structural performance have not been well assessed. In contrast, previous work has focused on only one constituent at a time. Additionally, previous studies have focused more on strength development, without providing detailed assessments of durability-related properties or of the interactions between fiber hybridization and matrix densification. Thus, the possible synergistic behavior between incumbent fly ash and hybrid steel-glass fibers with respect to crack-bridging mechanisms, ITZ refinement, and the optimal balance between strength and durability is not yet clearly understood based on laboratory experimentation, especially due to the lack of thermal curing conditions. Thus, the current study proposes a new integrated approach to concurrently investigate the combined influence of fly ash addition and steel-glass fiber hybridization on the mechanical and durability properties of RPC. Therefore, this research aims to develop sustainable RPC incorporating hybrid fibers to partially replace cement with fly ash and to evaluate mechanical properties under an accelerated-curing environment.

2. Materials and Method

2.1 Material

2.1.1 Fly Ash

Fly Ash was categorized as Class F, and the standards were established in accordance with [14]. Tables 1 and 2 present the chemical and physical properties.

Table 1. Chemical composition of fly ash [14]

Chemical composition	Results %	class F Requirement [14]
Fe ₂ O ₃	5.20	sum of value more than 50%
Al ₂ O ₃	17.75	
SiO ₂	65.57	
SO ₃	0.26	Maximum 5%
MgO	0.86	---
CaO	2.2	Max 18%
K ₂ O	3.40	---
Na ₂ O	2.25	---
Loss. of. ignition	2.8	Max 6%

Table 2. Physical characteristics of fly ash [14]

Physical properties	Results %	class F Requirement [14]
Maxi. amount retained on 45 µm (No. 325) sieve: 34%	26.8	Max 34%
Specific gravity	2.330	----
Surface area-(Blaine method) m ² / Kg	610	----
Physical form	powder	----
Strength Activity Index with Portland cement at 7 days,	92	min 75

2.1.2 Cement

The OPC (CEM 1-42.5 R) cement used in the studies meets the chemical and physical characteristics outlined in [15]. Tables 3 and 4 present the properties of cement

Table 3. The physical properties of the cement [15]

Physical properties	Results	Limits of [15]
Specific surface area (Blaine approach) (m ² /kg)	402.6	≥ 280
Setting time (Vicat's approach) Initial setting (min)	110	≥ 45
Setting time (Vicat's approach) Final setting time (hr)	3:25	≤ 10
Soundness by Autoclave Approach (%)	0.036 %	≤ 0.80
Compressive strength (MPa) (2) day	23.3	≥ 20
Compressive strength (MPa) (28) day	43	≥ 42.5

Table 4. Chemical composition of the cement [15,16]

Oxide Compositions	Weight (%)	Limits of [15]
L.O.I	2.815	Max (4)
Lime (CaO)	63.89	----
Iron oxide (Fe ₂ O ₃)	3.58	----
Alumina (Al ₂ O ₃)	5.40	----
Silica (SiO ₂)	18.87	----
Magnesia (MgO)	1.84	Max (5)
Sulfate (SO ₃)	2.33	SO ₃ ≤ 2.8 if C ₃ A > 3.5 SO ₃ ≤ 2.5 if C ₃ A ≤ 3.5
Insoluble residue (IR)	0.51	Max (1.5)
Cl ⁻ (Chloride)	0.078	—
Major compounds of OPC		
Tri-calcium Silicate (C ₃ S)	68.64	Bogue equation calculation according to [16]
Di-calcium Silicate (C ₂ S)	2.32	
Tri-calcium Aluminate (C ₃ A)	8.25	
Tetra-calcium Aluminoferrite (C ₄ AF)	10.89	

2.1.3 Sand

Fine aggregate that passed through a 600 μ m sieve. Normal specifications for fine aggregate were met. [17]. The fine aggregate properties are shown in Table 5 and 6 shows the sand grading.

Table 5. Grading of fine aggregate [17]

Sieve size (mm)	Cumulative Passing %	Limits according to [17] Zone 4
10	100	100
4.75	100	95-100
2.36	100	95-100
1.18	100	90-100
0.6	100	80-100
0.3	28.1	15-50
0.15	5.4	0-15

Table 6. Physical and chemical characteristics of fine aggregates [17]

properties	Test results	Limits according to [17]
Specific gravity	2.56	-----
Material Finer than 0.075 mm	2.5	Max 5%
Fineness modulus	1.67	-----
Dry rodded density (kg/m ³)	1680	-----
Absorption (%)	1.07	-----
Sulfate content %	0.18	Max (0.5) %

2.1.4 Silica Fume

The properties and strength activity index of silica fume meet the requirements of [18], their chemical and physical characteristics are presented in Tables 7 and 8.

Table 7. Chemical composition of silica fume (SF) [18]

Oxide	Results %	[18] requirement
Silicon Oxide (SiO ₂)	92.23	Min (85) %
Aluminum Oxide (Al ₂ O ₃)	0.31	-----
Magnesium Oxide (MgO)	0.34	-----
Iron Oxide (Fe ₂ O ₃)	1.3	-----
Calcium Oxide (CaO)	2.8	-----
Loss of Ignition	2.13	Max (6) %
Moisture content	0.32	Max (3) %

Table 8. Physical requirements of silica fume (SF) [18]

Physical properties	Test results	[18] requirement
State	Amorphous powder with submicron particle size	-
Retained on 45 μ m (No.325) sieve (%)	7.1	$\leq$ 10
Accelerated Pozzolanic Strength Activities Index with the OPC at 7 days, min percent of control	111.4	Min (105)
Specific Surface (m ² /g)	17	$\geq$ 15

2.1.5 Water

Tap water for mixing is suitable [19].

2.1.6 Superplasticizer

Hyperplast PC800M-type G (water-reducing, high-range, and retarding admixtures) having a density of 1.070 ± 0.02 g/cm³. The superplasticizer was used at a dosage of 0.5 - 3.5 % by weight of the total cementitious materials and having a pH of (5-7), in compliance with [20].

2.1.7 Micro Steel fiber

A straight micro steel fiber with a nominal diameter of 0.2 mm and length of 13 mm was employed in this study, providing an aspect ratio for the fiber ($l/d = 1/d=65$) and density 7860 Kg/m³. Tensile Strength 2600 MPa

2.1.8 Glass fiber

The used glass fibers were alkaline-resistant, measuring 12 mm in length, to ensure good resistance in the alkaline cementitious matrix. The fibers had a 14 μ m (0.014 mm) diameter, which translates to an aspect ratio of 80. and density 2680 Kg/m³. Tensile Strength 1700 MPa

2.2 Design and Mixes

The mixtures were designed using the methods of [21,22], and trial The mixes were prepared for verification. the reference (R1S) a mixture without replacement and five fly ash-based mixes (20% cement substitution) ,A 20% fly ash replacement was selected based on previous studies and trial mixes, as it provides an optimal balance between strength and microstructural enhancement[36].. The micro steel fiber volume fraction was fixed at 1% to maintain workability and ensure uniform dispersion, based on previous studies. [7,22], and progressively replaced with glass fibers at volume fractions of 0.25, 0.50, 0.75, and 1.00. The dosage of superplasticizer was adjusted to maintain the desired flow. All mixtures were designed using a fixed water-to-cementitious materials ratio (w/cm) of 0.21 (Fig. 1). The selected materials are presented. The flowability was regulated with the flow table test based on [23], achieving a value of (110 ± 5 mm) as specified by [24]. the measured flow values of the control and another RPC mixtures ranged from 105mm to 113 mm (Fig. 2). The proportions of the mixtures are summarized in Table 9.



Fig. 1. Materials used in the trial mixes of RPC



Fig. 2. Flow table test procedure

2.3 Mixing and Casting

The mixing process was performed according to the procedure suggested by [2]: Cement, fly ash, and silica fume were initially dry-mixed for about 3 minutes to achieve homogeneous distribution. Sand was added and mixed for another 5 minutes. the superplasticizer was diluted in a small amount of water before mixing, and during mixing, the water–superplasticizer solution was added to the mixture; the mixture was then mixed for 3 minutes. The parts not reached by the mixer were mixed by hand. The mixer was run for an additional 5 minutes to achieve adequate fluidity. Micro steel and glass fibers were homogeneously dispersed into the mix at 3 min; thereafter, the mixing process continued for the remaining 2 min. Overall, mixing one batch takes about 15 minutes from adding water to completion.

Table 9. Mix proportions of RPC for 1 m³ (kg/m³) [21,22]

Mixes	Cement	Sand	Silica Fume	Fly Ash	w/Cm	water	*sp l/m ³	** Micro Steel fiber	** Glass fiber
R1S	920	1015	230	-----	0.21	242	21.85	79	-----
RF1S	736	1015	230	184	0.21	242	20.70	79	-----
RF0.75S+0.25G	736	1015	230	184	0.21	242	21.50	59	7
RF0.50S+0.50G	736	1015	230	184	0.21	242	21.85	39	13
RF0.25S+0.75G	736	1015	230	184	0.21	242	22.31	20	20
RF1G	736	1015	230	184	0.21	242	22.89	-----	27

Note: * 1.8 - 1.99 % by weight of the total cementitious materials, ** fiber% by volume of mixes

2.4 Curing

The normal curing of RPC results in a slower rate of strength development than that observed with heat or steam curing, in which temperature is the main controlling factor [25, 26]. Immediately after mixing, the fresh concrete was cast into steel molds and vibrated in place before covering with nylon sheets and demolding 24 h later; specimens were heat-cured in water using the following schedule: temperature increased from 20 °C to 70 °C at a rate of 20 °C/h, followed by maintenance at 70 °C for 5 h/day for three days; specimens were then cured with water at 20 °C until tested [27]. The thermal-curing procedure used in this study is illustrated in (Fig. 3).



Fig. 3. Hot-water curing of RPC specimens at 70 °C (5 h/day for 3 days), followed by water curing at 20 °C

2.5 Testing

2.5.1 Fresh Density Test

Fresh density of RPC is determined in accordance with [28], using a known-mass-and-volume cylinder. The fresh density was calculated based on Eq. (1);

$$D_f = \frac{M_c - M_m}{V_m} \quad (1)$$

where: D_f is the fresh density of concrete (kg/m^3), M_c is the mass of the cylinder filled with fresh concrete (kg), M_m is the mass of the empty cylinder (kg), and V_m is the volume of the cylinder (m^3).

2.5.2 Compressive Strength

The compressive strength of RPC ($50 \times 50 \times 50 \text{ mm}$) was determined according to [24], using three cube specimens cured for 7 and 28 days. The reported compressive strength values are averages of the three specimens tested at each age. Compressive strength was calculated using Eq. (2), as illustrated in (Fig. 4.)

$$f_c = \frac{P}{A_c} \quad (2)$$

where: f_c : Compressive Strength (MPa), P : Maximum applied load (N), A_c : Cross-section area (mm^2).



Fig. 4. Testing and failure of the specimens

2.5.3 Flexural Strength

Three prismatic specimens ($50 \times 50 \times 250 \text{ mm}$) according to the specifications presented in [29], were tested at curing ages of 7 and 28 days. Values reported for flexural strength are average test results of three specimens at each age. The flexural strength was determined with Eq. (3), as illustrated in (Fig 5).

$$R = \frac{3pl}{2bd^2} \quad (3)$$

Where: R -The flexural value is denoted by the letter (MPa), P -Maximum load is denoted by (N), L -stands for the span's length (mm), b -average width of specimen in millimeters (mm), d -stands for the average prism thickness depth (mm).



Fig 5. Testing and failure of the specimens

2.5.4 Splitting Tensile Strength Test

The splitting tensile strength ($100 \times 200 \text{ mm}$) according to [30], with dimensions at the curing ages of 7 and 28 days. The RPC was tested on three cylindrical specimens. Except where specified, values reported are the average of three specimens per age. Eq. (4), as illustrated in (Fig. 6).

$$f_{sp} = \frac{2p}{\pi dl} \quad (4)$$

where: f_{sp} - Indirect tensile strength (MPa), p - Max. Applied load indicated by the testing machine (N), d -Cylinder diameter (mm), l -Cylinder length (mm).



Fig. 6. Testing and failure of the specimens

2.5.5 Dry Density Test

The density of RPC was determined in accordance with [31]. Three cubes with dimensions of (50 × 50 × 50 mm) were tested at 28 days. The density was calculated using the formula as given in Eq. (5);

$$D = \frac{M}{V} \quad (5)$$

where: D -is the density of the specimen in (kg/ m³), M - is the dry mass of the specimen in (kg), V - is the volume of the specimen in (m³)

2.5.6. Water Absorption and Voids Test

Water absorption and voids tests were conducted at 28 days following [32], computed from the mean of three specimens. The water absorption ratio was calculated using Eq (6), and the void content was calculated using Eq (7);

$$Absorption\% = \frac{B - A}{A} * 100 \quad (6)$$

Where: A - Oven dry weight (g), B - Saturated surface dry weight (g).

$$Voids\% = \frac{w2 - w1}{w2 - w3} \quad (7)$$

Where: $w1$ - The sample mass is oven-dried in the air, $w2$ - The mass of the sample is surface dried in the air after being submerged in water, $w3$ - The mass of the sample in the water

3. Results and Discussion

3.1 Dry and Fresh Density

The fresh and dry densities of the RPC mixtures are shown in Table 10 and (Fig. 7), respectively. The mixture without fly ash (R1S) shows the highest fresh density of 2515 kg/m³. The density of the fresh mix decreased slightly with the addition of fly ash because fly ash has a lower specific gravity than cement [33], which increases the paste volume and lowers the overall mixture's mass. The highest dry density was 2683 kg/m³ from RF1S, and the lowest was 2627 kg/m³ from RF1G. The reduction in density of RPC mixtures is due not only to the lower density of glass fibers compared with micro-steel fibers, but also to differences in fiber stiffness and dispersion, which may lead to slight increases in entrapped air and localized voids within the matrix. Overall, the

addition of fly ash and the partial substitution of micro-steel fibers with glass fibers led to a decrease in fresh density; however, dry density increased due to a reduction in air and pore volume entrapped in the matrix [33]. The reason for this was attributed to the pozzolanic reactions of fly ash with cement that produce denser and amorphous C-S-H gel, reducing porosity as well as lessening the number and length of micro-cracks, thereby improving the bond strength within the concrete matrix, leading to a higher density structure than that observed for reference samples [7].

Table 10. Results of fresh and dry density (kg/m^3)

Mixes	Fresh Density (kg/m^3)	Dry Density (kg/m^3) at 28 days
R1S	2515	2645
RF1S	2506	2683
RF0.75 S+0.25 G	2493	2669
RF0.50 S+0.50 G	2480	2655
RF0.25 S+0.75 G	2467	2641
RF1G	2454	2627

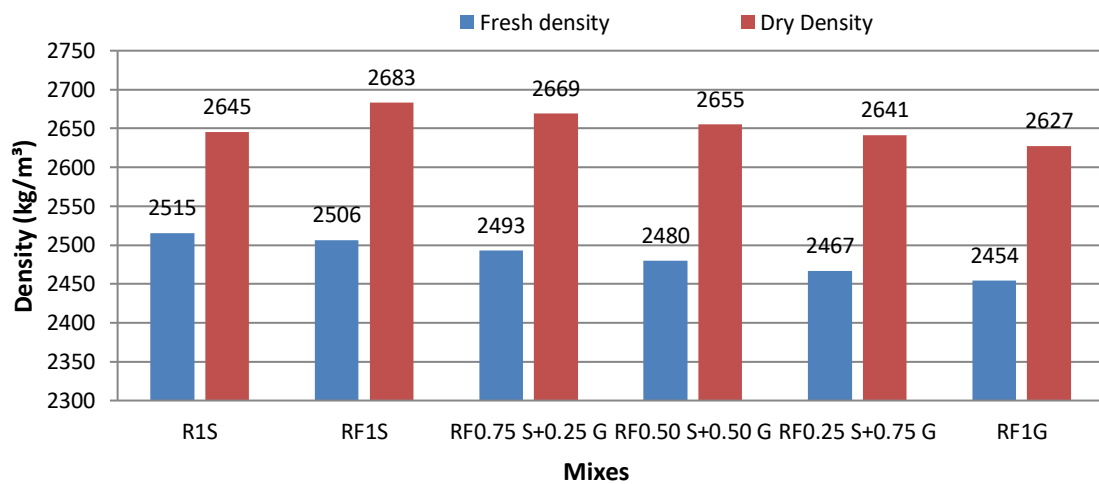


Fig 7. Fresh and Dry density of reference and sustainable mixes

3.2 Water Absorption and Porosity

Water absorption and porosity at 28 days are shown in Table 11 and (Fig. 8). The lowest values corresponded to RF1S (0.114% and 1.75). The reason for the decrease in the size of the pores is a result of filling and the high pozzolanic activity of fly [33], which contributes to microstructural densification. Fine fly ash particles improve particle packing by filling micro-voids between the cement grains and, during the secondary pozzolanic reaction, produce more C-S-H gel, giving longer bonds, and refining the pore structure rather than just reducing total porosity. With increasing glass fiber content, water absorption and porosity both increased slowly and reached maximum values (0.148% and 2.25%) at RF1G.

Table 11. Results of water absorption and porosity %

Mixes	Water absorption at 28 days (%)	Porosity at 28 days (%)
R1S	0.124	1.90
RF1S	0.114	1.75
RF0.75 S+0.25 G	0.121	1.86
RF0.50 S+0.50 G	0.130	2.00
RF0.25 S+0.75 G	0.139	2.12
RF1G	0.148	2.25

This behavior indicates an increased porosity due to lower density of glass fiber and their relatively weaker fiber–matrix bonding as glass, which may lead to the formation of micro-voids around the fibers. This finding agrees with that of [34], exhibited that higher glass fiber contents led to a lower compactness rate of the matrix and a larger connectivity ratio of pores. In general, the new pore structure when fly ash is added to all specimens results in lower capillary porosity and finer pore size distribution; on the other hand, improve of microstructure with increased glass fiber content is attributed to a more porous ITZ and more void formation.

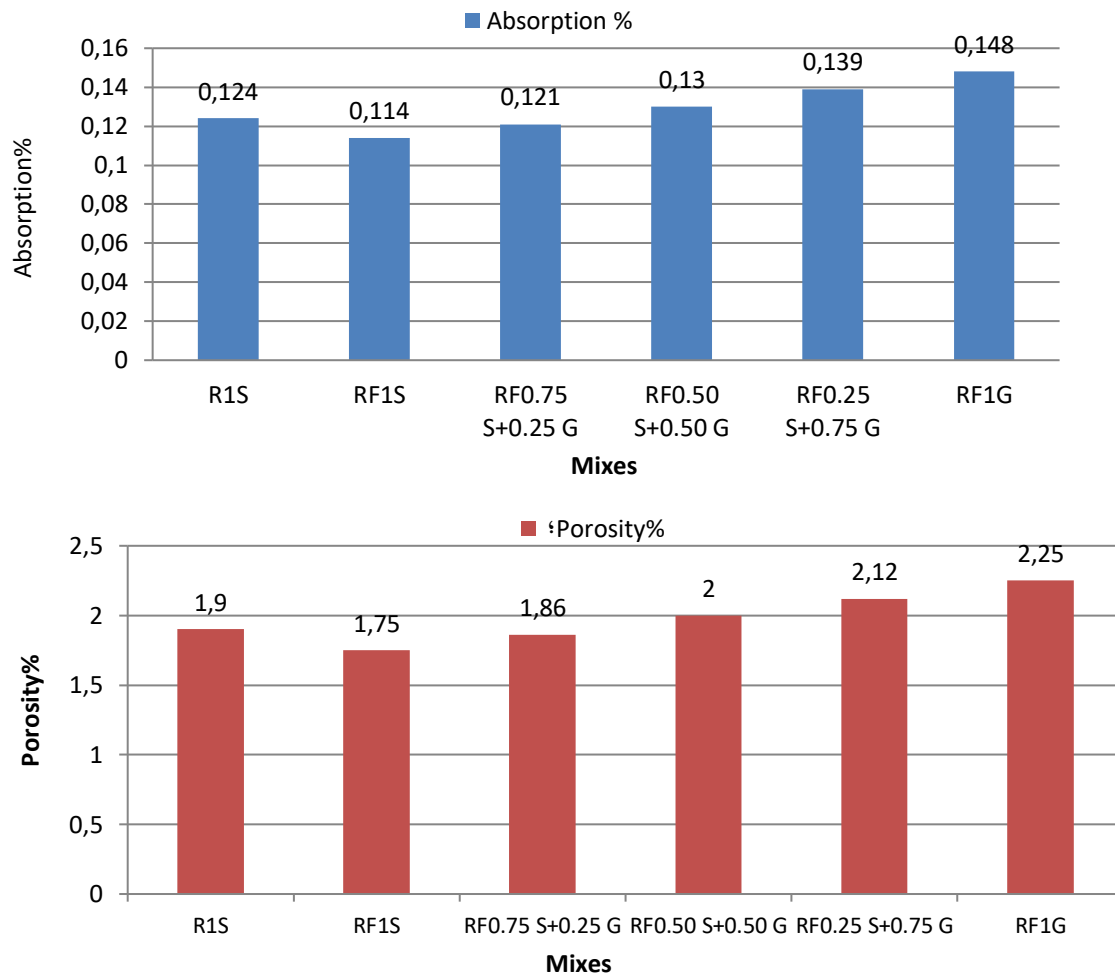


Fig. 8. Water absorption and porosity of reference and sustainable mixes

3.3 Compressive Strength

Compressive strength of RPC at both 7 and 28 days gradually decreased when micro steel fibers were substituted with glass fibers. The incorporation of 20% cement substitution with fly ash and 1% micro steel fibers (RF1S) resulted in the highest compressive strengths at both 7 days and 28 days, 101.32 MPa (+3.39%) and 119.10 MPa (+4.47%), respectively, relative to the reference mixture. Due to the pozzolanic reactivity of fly ash, it can react with the calcium hydroxide (CH) generated by cement hydration to form more Calcium silicate hydrate (C–S–H). As a result, this reaction helps produce a denser microstructure and lower porosity in the concrete matrix, which, in turn, increases its strength [7,33].

The presence of discrete steel fibers in the RPC matrix which, in turn, delays crack propagation and thereby enhances compressive strength, unlike glass fibers, which were less effective in this regard. as illustrated in Table 12 and (Fig. 9). On the contrary, replacing steel fibers with glass fibers caused higher decrease in compressive strength where the explanation for this behavior may be due to their lower elastic modulus and high smooth surface which cause a weak interfacial bonding. Additionally, fiber dispersion and the possibility of interfacial micro-voids can affect local stress concentrations, thereby degrading the compressive strength.

To conclude, fly ash addition combined with steel fiber reinforcement serves to improve the microstructure of RPC. However, the reduction in compressive strength decreases gradually with increasing glass fiber content in aggregates, because this mechanism is less energy-efficient than the agglomeration mechanism for controlling crack coalescence. The results are analogous to the reported finding by [34].

This is shown by the error bars (i.e., low variability in the experimental results). As shown in the mean Bonferroni and the corresponding error bars, slight overlap of some error bars is observed; however, differences between mixtures were greater than their respective standard deviation values, clearly demonstrating that the variation represents a statistically significant result rather than experimental error. The standard deviation (≈ 1.30 – 1.50 MPa) also confirms good repeatability.

Table 12. Results of compressive (MPa)

Mixes	Compressive Strength (MPa)				Percentage change relative to reference mix (R1S) (%)	
	7 days	Std. Dev	28 days	Std. Dev	7 days	28 days
R1S	98.00	1.41	114.00	1.41	0.00	0.00
RF1S	101.32	1.45	119.10	1.37	+3.39	+4.47
RF0.75 S+0.25 G	99.95	1.30	117.95	1.46	+1.99	+3.47
RF0.50 S+0.50 G	97.60	1.41	115.60	1.46	−0.41	+1.40
RF0.25 S+0.75 G	94.85	1.35	112.80	1.50	−3.21	−1.05
RF1G	92.10	1.31	110.20	1.37	−6.02	−3.33

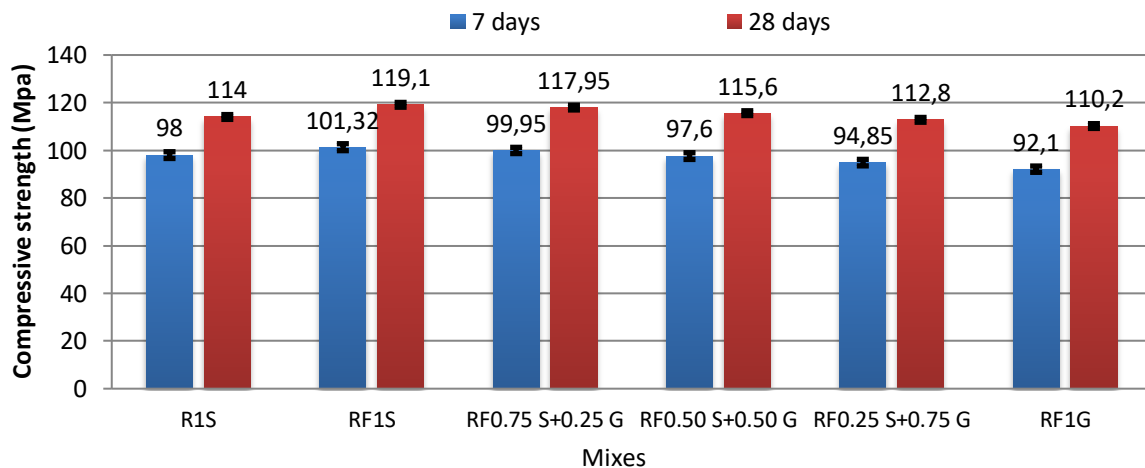


Fig 9. Influence of fiber on the compressive strength of RPC

3.4 Flexural Strength

The flexural strength of RPC decreased gradually as micro steel fibers were replaced by glass fibers at 7 and 28 days, in different percentages. The maximum flexural strength was recorded in the reference mix with 1% micro steel fibers, and higher glass fiber content resulted in lower flexural strength. As the main role is in controlling FRC cracking, and modifying the response of the material after matrix cracking has occurred, by bridging over these cracks and thereby giving some post-cracking ductility [35], By investigation, it will be noted that micro steel fibers represent better crack-bridging efficiency than SF because of its higher elastic modulus, tensile strength and rough surface texture which can combine with enhanced mechanical interlocking and fiber–matrix bond. This leads to better stress transmission at the crack, increased fracture toughness, and improved post-fracture ductility. In contrast, the flexural rigidity of glass fiber is low due to its smooth surfaces, which limit interaction with the polymer matrix and interfacial adhesive bond, and its limited crack-bridging capacity or septa. The faster inherent increase in the calculated crack-

opening displacement causes early premature crack propagation or decreases the flexural strength. The use of steel fibers leads to more effective load transfer across cracks and higher post-cracking in RPC compared with glass fiber, due to the high tensile strength, as is illustrated in Table 13 and (Fig. 10). The findings are consistent with the findings presented by [11]. Extremely low standard deviation ($\approx 0.14\text{--}0.23$ MPa) demonstrated very good consistency. The decrease in strength by these values is greater than that corresponding to the range of variability, indicating that fiber hybridization has a large effect.

Table 13. Results of flexural strength (MPa)

Mixes	Flexural strength (MPa)				Percentage change relative to reference mix (R1S) (%)	
	7 days	Std. Dev	28 days	Std. Dev	7 days	28 days
R1S	8.97	0.15	10.47	0.15	0.00	0.00
RF1S	9.12	0.15	10.72	0.15	+1.67%	+2.39
RF0.75 S+0.25 G	8.76	0.14	10.30	0.2	-2.34%	-1.62
RF0.50 S+0.50 G	8.17	0.15	9.60	0.2	-8.92%	-8.31
RF0.25 S+0.75 G	7.58	0.17	8.91	0.21	-15.50%	-14.90
RF1G	7.33	0.17	8.62	0.23	-18.28%	-17.67

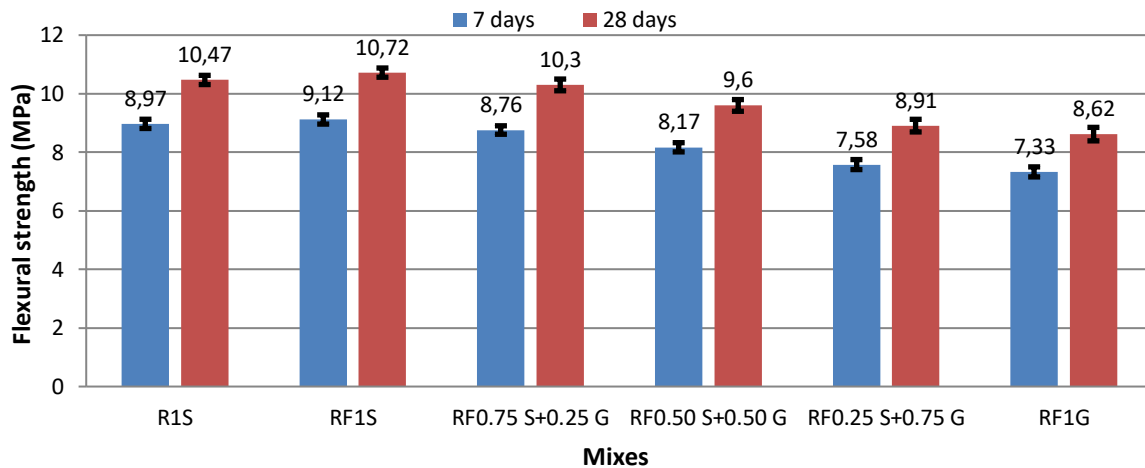


Fig. 10. Influence of fiber on the flexural strength of RPC mixtures

3.5 Splitting Tensile Strength Test

The splitting tensile strength of RPC decreased gradually with increasing glass fiber replacement ratios on days 7 and 28, respectively. With a lower concentration of glass fibers. References with 1% micro steel fibers; whereas, the lowest values were observed in mixes with a higher volume percentage of glass fiber. In this assessment, fiber slippage is proposed as an indicator of tensile failure occurred due to weak bonding between the glass fibers and the RPC. This was likely due to inadequate chemical treatment of the fiber surface, which is required to develop an appropriate structural texture that enables it to withstand high tensile loads and stresses within the RPC matrix. [34], which outlined that the contribution of fibers to improve tensile properties is dominantly influenced by bonding characteristics and surface characteristics. The results indicated that steel fibers provide better tensile reinforcement for RPC, resulting in stronger fiber–matrix bonding and improved crack bridging. In contrast, a higher glass fiber content harms tensile performance owing to weaker fiber-matrix interactions and increased interfacial defects. The results are consistent with data reported by [11]. The small standard deviation ($\approx 0.12\text{--}0.15$ MPa) indicates high precision and repeatability. The differences between the mixes are larger than experimental error, confirming statistical significance

Table 14. Results of splitting tensile strength (MPa)

Mixes	Splitting tensile strength (MPa)				Percentage change relative to reference mix (R1S) (%)	
	7 days	Std. Dev	28 days	Std. Dev	7 days	28 days
R1S	7.80	0.13	8.90	0.13	0.00	0.00
RF1S	7.92	0.15	9.05	0.13	+1.54%	+1.69%
RF0.75 S+0.25 G	7.65	0.132	8.76	0.14	-1.92%	-1.57%
RF0.50 S+0.50 G	7.30	0.13	8.37	0.15	-6.41%	-5.96%
RF0.25 S+0.75 G	6.90	0.13	7.94	0.12	-11.54%	-10.79%
RF1G	6.64	0.12	7.60	0.13	-14.87%	-14.61%

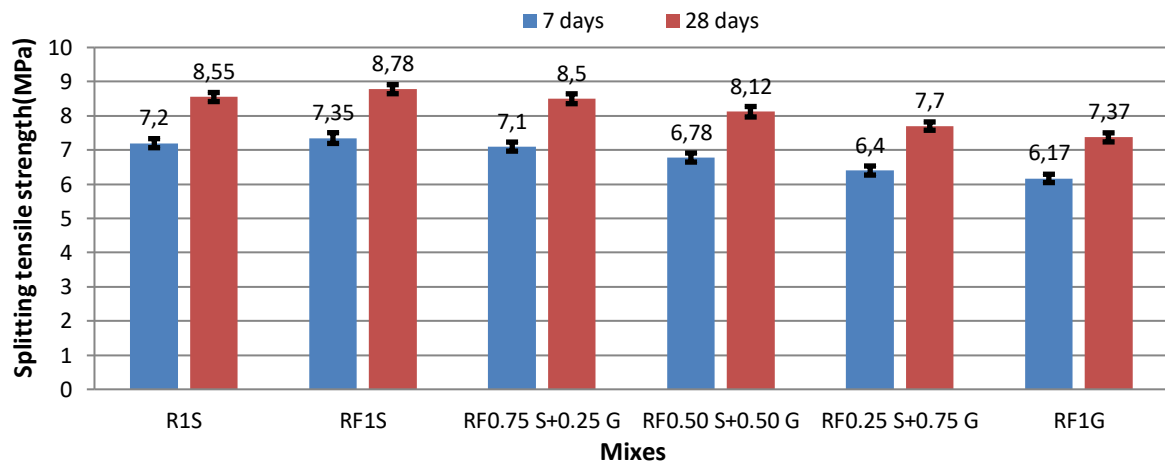


Fig. 11. Effect of fiber on the splitting tensile strength of RPC mixtures

4. Conclusions

- This study provides unique scientific insight into how the utilization of fly ash and hybridization with steel–glass fibers in thermal curing regimes interact, as previous studies have not fully addressed this topic or its impact on the degree of sustainability benefits and mechanical behavior.
- It accelerates the pozzolanic reactions mainly by improving cement hydration and fly ash reactivity, resulting in rapid C–S–H formation and early densification of the matrix. This agrees with the improvement of mechanical strength at 20% fly ash replacement level.
- Replacing 20% of the cement with fly ash improved the mechanical behavior of reactive powder concrete (RPC) cured at elevated temperatures. The compressive strength increased by 4.47% at 28 days compared with the control mixture, while the flexural and splitting tensile strengths increased by 2.39% and 1.69%, respectively. This improvement can be attributed to the micro-filling effect and secondary pozzolanic reactions of fly ash, which enhance matrix densification, refine the pore structure, and enhance the interfacial transition zone (ITZ).
- Gradually replacing micro steel fibers with glass fibers affected the strength performance of RPC. For steel-to-glass hybrid ratios of 0.75:0.25, 0.50:0.50, 0.25:0.75, and 0:1, the compressive strength changed by +3.47%, +1.40%, -1.05%, and -3.33%, respectively. Flexural and splitting tensile strengths declined with increasing glass fiber content. Optimum mechanical performance was achieved with fly ash and 1% micro steel fibers. In contrast, higher glass fiber contents led to progressive strength reduction due to their lower stiffness and weaker crack-bridging capability than steel fibers. For flexural loading, the effect is greater as it controls crack propagation and the fiber-bridging mechanism of failure. This is also related to the fact that the interfacial transition zone (ITZ) between glass fibers and the

cement matrix is weaker, resulting in easier fiber pull-out and, therefore, less effective crack-bridging by glass fibers than by steel fibers.

- Fiber hybridization had a greater impact on flexural and splitting tensile strengths than on compressive strength. This indicates that flexural performance, which is mainly an interfacial mechanism originating in fiber-reinforcement crack-bridging in the matrix phase of concrete, was more affected by fiber type than by compressive strength, compared with reference steel fibers. The decrease in flexural strength with increasing glass fiber content can be attributed to lower stiffness and fiber bonding of glass fibers compared to steel fibers.
- Fresh density decreased due to the use of fly ash and glass fibers, but the dry density increased slightly during matrix densification. The water absorption and porosity increased with the addition of glass fiber, which may be related to weaker fiber–matrix bonding and less efficient fiber dispersion. This is associated with increased micro voids and less efficient particle packing, resulting in a more porous microstructure.
- Overall, incorporating 20% fly ash enhances RPC performance while reducing cement consumption. Although hybrid fiber systems exhibit lower strength than steel fiber reinforcement at higher glass contents, they may offer improved durability due to glass fibers' corrosion resistance. Depending on the fiber composition, this reveals a mechanical performance-versus-long-term durability trade-off.

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Development and characterization of plasma electrolytic oxidation coatings on AA5052 for advanced solar reflector thermal control

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Abstract

Numerous subsystems make up a spaceship, requiring temperatures within narrow parameters to perform at peak performance. By carefully choosing the thermal and optical characteristics, notably solar absorption and infrared Emittance, the passive thermal controller plays a crucial part in keeping spaceship components within the necessary temperature ranges. Plasma electrolytic oxidation (PEO) of AA5052 studies to develop a solar glass coating suitable for use in outer space. An electrolyte of sodium silicate (Na_2SiO_3) to create these coatings. Specifically, the layer's thermal and optical performance is investigated as a function of pulse on time, process time and pulse frequencies, and the electrolyte composition used. The PEO coating thickness is personalized to achieve thermo-optical qualities on par with or better than those achieved with standard sulphuric acid anodizing. The improved coating is put through its paces to ensure it can function as a thermal control surface in space, with testing including adhesion, humidity, and performance in simulated space conditions including thermal cycling and thermo-vacuum. The morphological, microstructural characteristics, and arrangement of coatings achieved through the PEO progression and conventional sulphuric acid anodizing are compared using X-ray diffraction (XRD), scanning electron microscopy (SEM), and energy dispersive X-ray photoelectron spectroscopy (XPS).

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1. Introduction

The one side of a spaceship may be subjected to intense heat from the sun, while the other is subjected to freezing conditions in interplanetary space [1]. Even more so, as the spacecraft enters an eclipse, some of its non-operational components endure extremely low temperatures while its operational features experience higher temperatures [2]. This severe thermal environment imposes a temperature difference of several hundred degrees Celsius. However, there is a small temperature range in which the various spacecraft components and electronics can operate at their optimum [3]. Thermal control is a fundamental design consideration to keep all mission subsystems within their temperature limits throughout the task. Using surfaces with suitable

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thermo-optical properties, like absorption (α) and emittance (ϵ), passive thermal control methods provide very reliable temperature management of spaceship components [4-6].

Aluminum undergoes anodization, also known as electrochemical oxidation when it is immersed in an electrolyte solution and then electrolyzed to create a thin, protective oxide layer on its surface [7]. Emittance values as low as 0.03 can be achieved on a polished aluminum metal surface; this value can be significantly improved with anodization. The IR emissivity of a conventionally anodized sulfuric acid coating is 0.77, but the solar absorption is only about 0.34 [8,9]. As a result, the anodic film acts as a highly effective solar reflector. However, there are environmental concerns with the anodization process due to the use of powerful acids and the difficulty in properly disposing of them [10-12]. As a result, there is perpetual pressure to find a better coating method that may enhance functional characteristics while still satisfying the demanding standards of current environmental regulations [13].

The basic idea is the same as anodizing, but the voltages are much higher, and the oxide is formed through a dielectric breakdown in the form of micro-discharges [14,15]. Generally speaking, researchers agree that all variations of the PEO process can be deemed environmentally friendly, as the electrolyte employed for this process comprises mild alkaline solutions [16-18]. Coatings made from polyethylene oxide (PEO) are renowned for protecting against corrosion, increasing hardness, and resisting heat and electricity. In recent literature, PEO coatings have even been proposed as an impregnate for sol-gel coating on Al and Mg alloys [19]. PEO coatings have several potential uses, including in extreme space conditions, but little research has been done on their thermo-optical properties [20]. This study aims to use plasma electrolytic oxidation to create an oxide layer on AA 5052 alloy for heat management applications. Authors systematically examine the impact of various operating conditions to achieve an oxide coating with thermo-optical properties similar to those achieved through traditional sulphuric acid anodization.

Due to the inaccessibility of a spacecraft once it has entered orbit and the extreme corrosiveness of space settings, coatings developed for usage in space applications require higher standards and more excellent dependability [21]. Coatings like this need to endure the extreme heat and pressure of the launch phase, the vacuum of space, ionizing radiation, high-energy charged particles like electrons and protons, and wildly fluctuating temperatures in orbit [22,23]. If coatings on spacecraft parts aren't applied perfectly, it could negatively affect the functionality of a certain subsystem, which could cause problems throughout the spacecraft. Therefore, the coatings obtained using the finalized parameters are put through a battery of tests to determine whether or not they are suitable for use in space, including adhesion and humidity testing, thermal cycling, and thermo-vacuum experimental tests [24]. These expedited tests disclose any change in the coating's α and IR values that may have occurred over the spacecraft's entire operational lifetime [25,26]. SEM, EDX, XPS, and XRD are all used to learn more about the coating.

Friction riveting is an emerging technique for joining lightweight materials; however, its application is limited by the requirement of specialized force-controlled equipment [27]. This study focuses on the development of Al-based metal matrix nanocomposites reinforced with graphene nanoplatelets (GNP) using powder metallurgy. The results show that the addition of GNP significantly enhances mechanical properties and corrosion resistance compared to pure aluminum [28]. Metal matrix composites (MMCs) exhibit superior mechanical and tribological properties compared to base alloys. [29]. The water-quenched Cr-reinforced Al/Cu composites exhibit superior hardness, compressive strength, and wear resistance compared to oil-quenched samples [30].

Although plasma electrolytic oxidation (PEO) coatings have been widely investigated for thermal control applications on aluminum alloys, limited studies have systematically addressed the behavior of Mg-containing AA5052 and the influence of electrolyte concentration on its thermo-optical performance and long-term environmental stability. A systematic correlation between Na_2SiO_3 electrolyte concentration, discharge characteristics, coating morphology, and resulting solar absorptance (α) and infrared emittance (ϵ) is established. Furthermore, unlike earlier studies, the optimized coatings are subjected to comprehensive space-simulated environmental testing, including thermo-vacuum, thermal cycling, and humidity exposure. This combined approach

provides both process-level optimization and application-level validation, contributing to the development of durable and environmentally sustainable thermal control coatings for spacecraft applications.

2. Experimental Procedure

Samples of AA5052 alloy in the form of disks are developed using plasma electrolytic oxidation coating using a silicate-based electrolyte. The plasma electrolytic oxidation (PEO) treatment was carried out using a pulsed DC power supply under galvanostatic mode at an applied current density of $10 \text{ A}\cdot\text{dm}^{-2}$, with a corresponding voltage range of 300–520 V during the oxidation process. A pulse frequency of 1000 Hz and a duty cycle of 20% (ton/toff controlled) were maintained throughout the treatment. The total processing duration was 15 minutes for all coated samples. The electrolyte consisted of an aqueous sodium silicate (Na_2SiO_3) solution at optimized concentration of $90 \text{ g}\cdot\text{l}^{-1}$, and the temperature was maintained below 30°C using continuous stirring and external cooling to ensure stable discharge behavior and uniform coating growth. The samples are degreased in room-temperature isopropyl alcohol for 10 minutes to get rid of any oil, lubricant, or dust. Rinsing the specimen with distilled water follows. A polypropylene tank with a 25 ltr capacity holds the electrolyte, a sodium silicate solution with a concentration of $30 \text{ g}\cdot\text{l}^{-1}$. Authors used the sample as the anode and a thin sheet of stainless steel for the cathode. For this procedure, we employ a 25kW custom-built PEO power supply. This procedure utilizes a positive, unipolar, square-wave pulsed current. The processing tank has stainless steel cooling coils installed inside, which circulate cold water to maintain an electrolyte temperature below 25°C . The cleaned and prepped samples are given a final rinsing in DM water before being dried with a hot air pistol.

Several factors, including pulse frequency, electrolyte arrangement, mean current density, process time, and pulse on time, are studied to understand the coating's thermo-optical performance better [31]. The concentration of Na_2SiO_3 is varied from $30 \text{ g}\cdot\text{l}^{-1}$ to $150 \text{ g}\cdot\text{l}^{-1}$, and the average current density is held constant at $9 \text{ A}\cdot\text{dm}^2$ for 35 minutes. With a steady pulse frequency of 400 Hz and pulse on-time of $550 \mu\text{s}$, a duty cycle of 15% is achieved. To examine the effect of process time on coating viscosity and the associated thermal and optical characteristics, samples are processed at a mean current density of $9 \text{ A}\cdot\text{dm}^2$ for 15, 25, 35, 45, 55, and 65 minutes employing a $90 \text{ g}\cdot\text{l}^{-1}$ Na_2SiO_3 electrolyte. To analyze the impact of pulse frequency on the thermal and optical characteristics, specimens are generated at variant frequencies (100, 250, 400, 550, and 700 Hz) while maintaining a constant pulse on-time of $550 \mu\text{s}$ and a persistent mean current density of $9 \text{ A}\cdot\text{dm}^2$. Coatings are made with different average current densities (3, 6, 9, 12, and $15 \text{ A}\cdot\text{dm}^2$) and pulse on time (100, 250, 550, and $700 \mu\text{s}$) to further examine the impact on thermo-optical properties. On the other hand, traditional anodization calls for a sulphuric acid electrolyte, a current density of $3 \text{ A}\cdot\text{dm}^2$, and a hydrothermal sealing process of 1 hour in boiling water. To investigate the impact of coating viscosity on thermal, and optical characteristics, specimens are anodized for varying durations (from 15 minutes to 65 minutes).

To process PEO samples of varying thicknesses, a solution containing $90 \text{ g}\cdot\text{l}^{-1}$ Na_2SiO_3 is used, with a mean current density of $9 \text{ A}\cdot\text{dm}^2$ and pulse frequencies and pulse on-time, respectively, 400 Hz and $550 \mu\text{s}$. The samples are then hydrothermally sealed in boiling water for 1 hour. Next, stuffed specimens are compared to unsealed samples of the same thickness and with the exact specifications regarding their thermo-optical properties.

Each process's voltage-time response is measured and then compared. Each sample was assessed for its coating thickness on both sides, and then statistical procedures were applied to generate an average value with slight variation. The Solar Spectrum Reflectometer (SSR) and the emissometer (AE) are used to measure the absorption and emission [32]. Surface roughness measurements were performed using a Surtronic 3+ stylus profilometer with a cutoff length of 2.5 mm and an evaluation length of 8 mm. For each sample, measurements were taken at six different locations to ensure repeatability, and the average values of R_a (arithmetic mean roughness) and R_q (root mean square roughness) were reported along with standard deviations. The stylus tip radius was $5 \mu\text{m}$, and the applied force was maintained constant to avoid surface damage. Similarly, the Surtronic 3+ is used

to assess the coating's surface roughness utilizing a stylus probe at an evaluation length of 8.00 mm [33,34]. Later, the surface roughness measurements from both techniques are compared.

The energy-dispersive X-ray spectroscopy instrument is permanently attached to the scanning electron microscope and is used to determine the elemental composition of the deposit [35]. For our X-ray diffraction studies, authors employ a Philips X'pert-Pro instrument set to 45 kV and 35 mA, radiating with Cu K rays ($\alpha = 1.6206$) and scanning from 10° to 100° at a rate of $0.15^\circ \text{ s}^{-1}$ with a step size of 0.03° . A SPECS spectrometer using AlK radiation (1512.6 eV) at 160 W (13 kV, 14 mA) is used to conduct X-ray photoelectron spectroscopy (XPS) experiments on anodized and PEO coatings. The C1 s peak is positioned at 291.7 eV, and binding energies are provided about this value. One uses a pass energy of 80 eV and a step increment of 0.6 eV to acquire the survey spectra, whereas one uses pass energy of 45 eV and a step increment of 0.06 eV to obtain the individual spectra. The Si2p core-level spectra are cleaned of Shirley noise, and then the data is fitted using the Casa XPS program.

The space routineness of a Plasma Electrolytic Oxidation coating is determined by putting it through tests in a space environment simulator after the parameters have been finalized [36]–[38]. The results can be trusted because they are replicated using the same four samples for each test. Coatings are monitored for consistent thermo-optical behavior by measuring their solar absorption (α_s) and infrared Emittance (ϵ_{IR}) after each environmental test.

3. Results and discussions

3.1 Traditional Anodization Using an Electrolyte of Sulphuric Acid (H_2SO_4)

The influence of processing time on anodic coating thickness is presented in Figure 1. The graphic makes it very clear that the slope of the thickness Vs. after 35 minutes of process time. To rephrase, the coating growth rate slowed after the initial 35 minutes of processing time elapsed. Since the electrical resistance of the coating increases with coating thickness, current flow is delayed, and the coating's growth rate decreases. As the coating thickness increases, the electrical resistance across the oxide layer rises, leading to localized temperature elevation at the metal–electrolyte interface, which reduces the effective coating growth rate due to enhanced oxide restructuring and partial breakdown. The rate of anodic coating breakdown increases with time, causing a decrease in the coating growth rate [39].

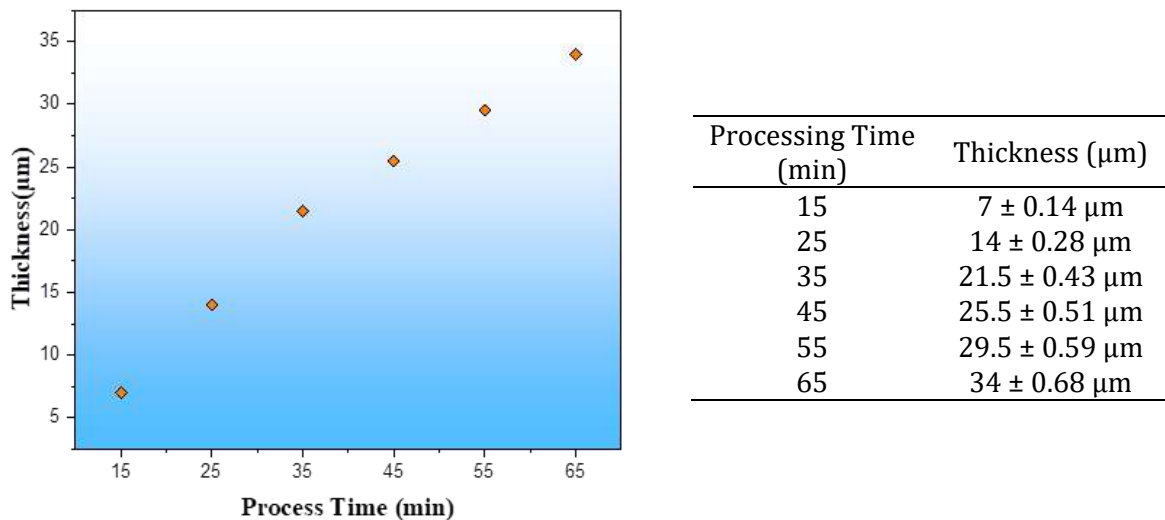


Fig. 1. Comparison of coating thickness and process time for conventional H_2SO_4 anodization

In Fig. 2, authors can see the measurements results of the thermo-optical properties of coatings of varying thicknesses. Coating emittance increased from 0.82 at $7 \mu\text{m}$ to 0.94 at $21 \mu\text{m}$, and further to 0.96 at $35 \mu\text{m}$, indicating a significant overall improvement of approximately 17%; however, the rate of increase beyond $21 \mu\text{m}$ is relatively small, suggesting a saturation trend in emittance with increasing coating thickness. The absorbance of the anodic film increased from 0.342 for a $7 \mu\text{m}$

coating to 0.378 for a 21 μ m coating and 0.436 for a 35 μ m coating. The outcome was a decrease in the α/ϵ ratio, reaching a minimum at a coating thickness of 25 nm, followed by an increase in that value. The hydrothermally sealed, 42 μ m thick anodic coating's an absorbance of 0.382 and an emittance of 0.98 make it an effective solar reflector.

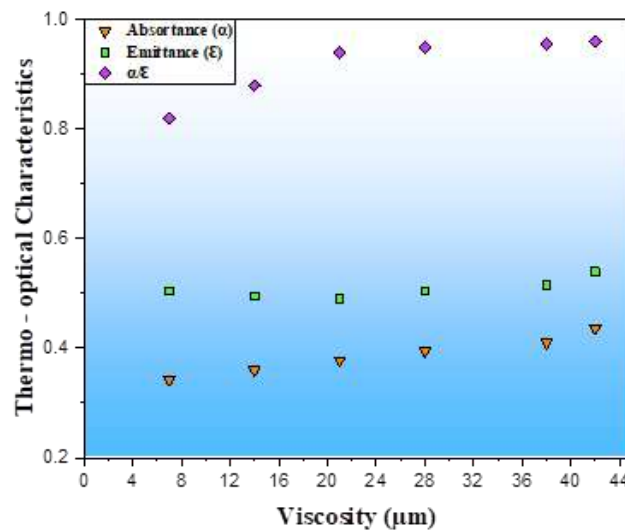
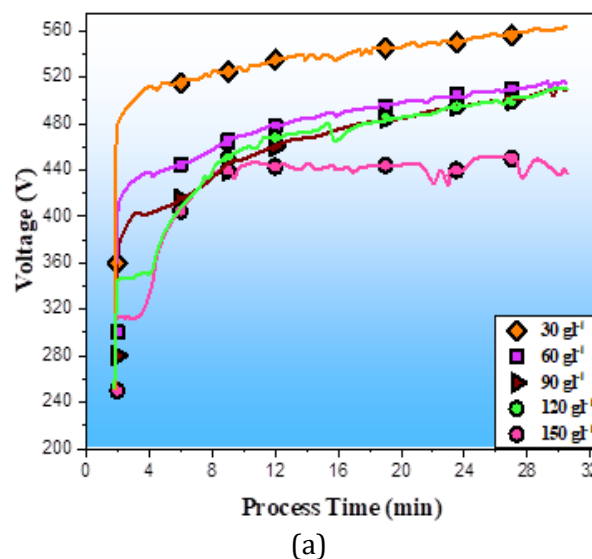


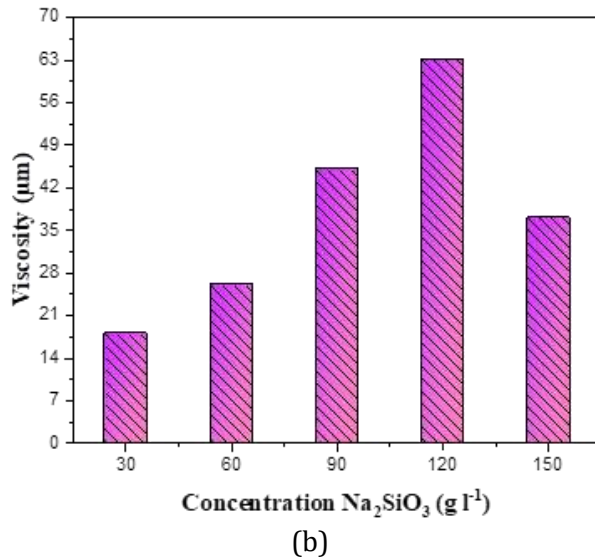
Fig. 2. Thermal optical characteristics of anodic coating on variant viscosity

3.2 Optimum of Plasma Electrolytic Oxidation Processing Factors

3.2.1 Impact of Na_2SiO_3 Concentration

The voltage and process time response are shown to be affected by the sodium silicate content of the electrolyte (Figure 3a). An increase in Na_2SiO_3 in the electrolyte has been shown to drastically reduce the breakdown voltage from 492 V to 324 V. According to prior research by author [40], this behavior can be understood as the result of an increase in the conductivity of the solution with a rise in Na_2SiO_3 of the electrolyte. When the Na_2SiO_3 concentration is 120 gl^{-1} or higher, the voltage-process time plot also shows significant voltage variations. 1. The metal-oxide boundary is thought to be the source of the powerful and long-lasting sparks responsible for this phenomenon [41,42]. These discharges are considered to the coating growth rate because they prevent the production of a protective oxide layer. Intense fluctuations in voltage as a function of time may be due to the dissolution of an oxide layer that grows thinner with increasing sodium silicate concentration. So, taking everything into account, the best silicate concentration for future studies is 90 gl^{-1} .





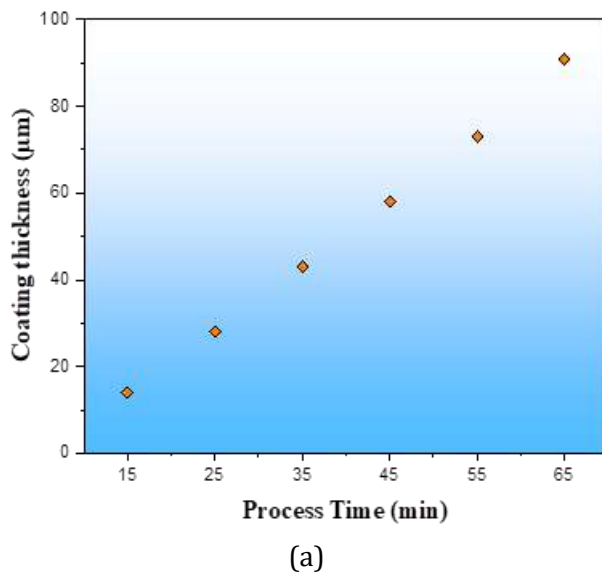
Concentration Na ₂ SiO ₃ (g l ⁻¹)	Viscosity (μm)
30	18 ± 0.36
60	26 ± 0.52
90	45 ± 0.90
120	63 ± 1.26
150	37 ± 0.74

Fig. 3. (a) Impact of Na₂SiO₃ on coating viscosity, (b) Impact of Na₂SiO₃ on voltage–process time response

3.2.2 Impact of Process Time

In Fig 4, shows the coating thickness changed as a function of the applied voltage and processing time. Figure 4 illustrates (a) the variation of coating thickness with processing time and (b) the corresponding voltage–time response during the PEO process. When equated to the anodic process, the coating growth rate in PEO is dramatically faster. When correlated to the PEO procedure, which yields a coating thickness of 91 μm after 65 minutes of processing, conventional anodization causes a coating thickness of 35 μm. The mechanism for coating production is different, which is the primary factor. Oxide production in the PEO process happens through melting and re-solidifying the previously created oxide layer [43]. Hence the resistances of the oxide layer do not affect the growth rate in any way, in contrast to the anodization process.

The thermo-optical properties are plotted against coating viscosity to determine the thinnest Plasma Electrolytic Oxidation coating that displays qualities identical to anode coating (Fig. 5). Coating emittance improved from 0.81 for a 14 μm coating to 0.98 for a 91 μm coating. It was determined that an emittance value of 0.98 corresponds to a coating thickness of roughly 42 μm, which is on par with a conventionally anodic specimen with a viscosity of approximately 21 μm [44]. But the ratio of Absorptance to Emittance systematically decreases as coating thickness grows, going from 0.439 for a 14 μm coating to 0.353 for a 91 μm coating.



Process Time (min)	Coating thickness (μm)
15	14 ± 0.28
25	28 ± 0.56
35	43 ± 0.86
45	58 ± 1.16
55	73 ± 1.46
65	91 ± 1.82

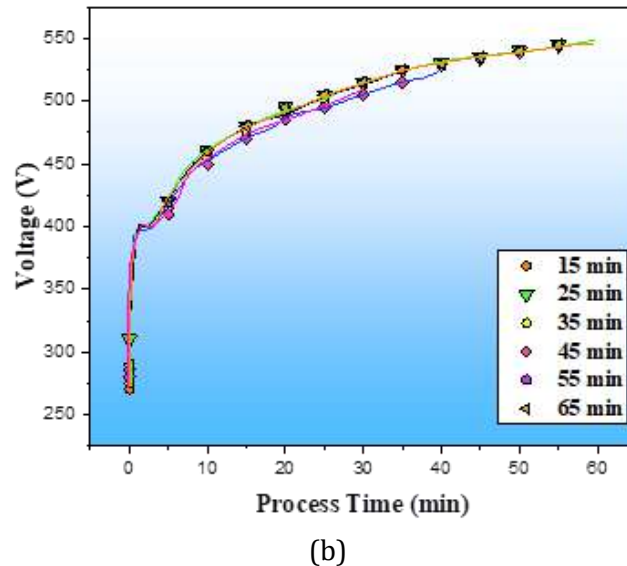


Fig. 4. (a) Coating thickness Vs. Process time for the specimen in 90 gl^{-1} of Na_2SiO_3 b) Impact of process time on the voltage concerning sample time in 90 gl^{-1} of Na_2SiO_3

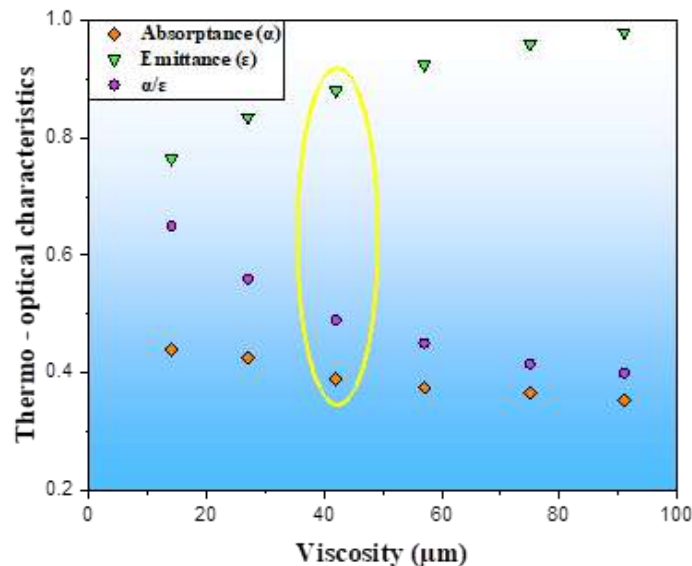


Fig. 5. Thermal optical characteristics concerning coating viscosity

One possible explanation is that various coatings undergo distinct microstructures and compositions due to variations in the underlying process mechanisms [45]. Compared to a typical anodized sample with a $21 \mu\text{m}$ coating, the absorbance value with a PEO coating thickness of $42 \mu\text{m}$ is 0.382. Therefore, PEO covering $42 \mu\text{m}$ thick was deemed adequate for a solar reflector. This means that 35 minutes is the settled-upon process time for the PEO procedure.

3.2.3 Impact of Pulse Frequency

The voltage and process time plot and the resulting coating thicknesses at various pulse frequencies are shown in Figures 6a and b, respectively. When processing at a low frequency of 100Hz, the output voltage rises to a peak of 674.6V after 35 minutes of processing, which is significantly higher than the initial voltage of 571.4 V measured after 1 minute of processing. Since the power source can only produce up to 700 V, working at lower frequencies presents practical challenges, especially with more extensive surface area samples. Furthermore, the low-frequency voltage-time graph demonstrates several voltage changes during the process, albeit of less amplitude. The starting and ending voltages drop dramatically as the frequency rises; at 400Hz, the values are 396.4 V and 510.7 V, respectively. Voltage-time graphs show that significant voltage fluctuations begin at pulse frequencies above 400Hz. This method is found to work best with a pulse frequency of 400Hz. From 100 to 550Hz, there is a slight shift in coating thickness from 33 to

43 μm , and from 550 to 700Hz, the viscosity drops to 42 μm (Fig. 6b). Thermo-optical characteristics also change slightly in figure. 6c.

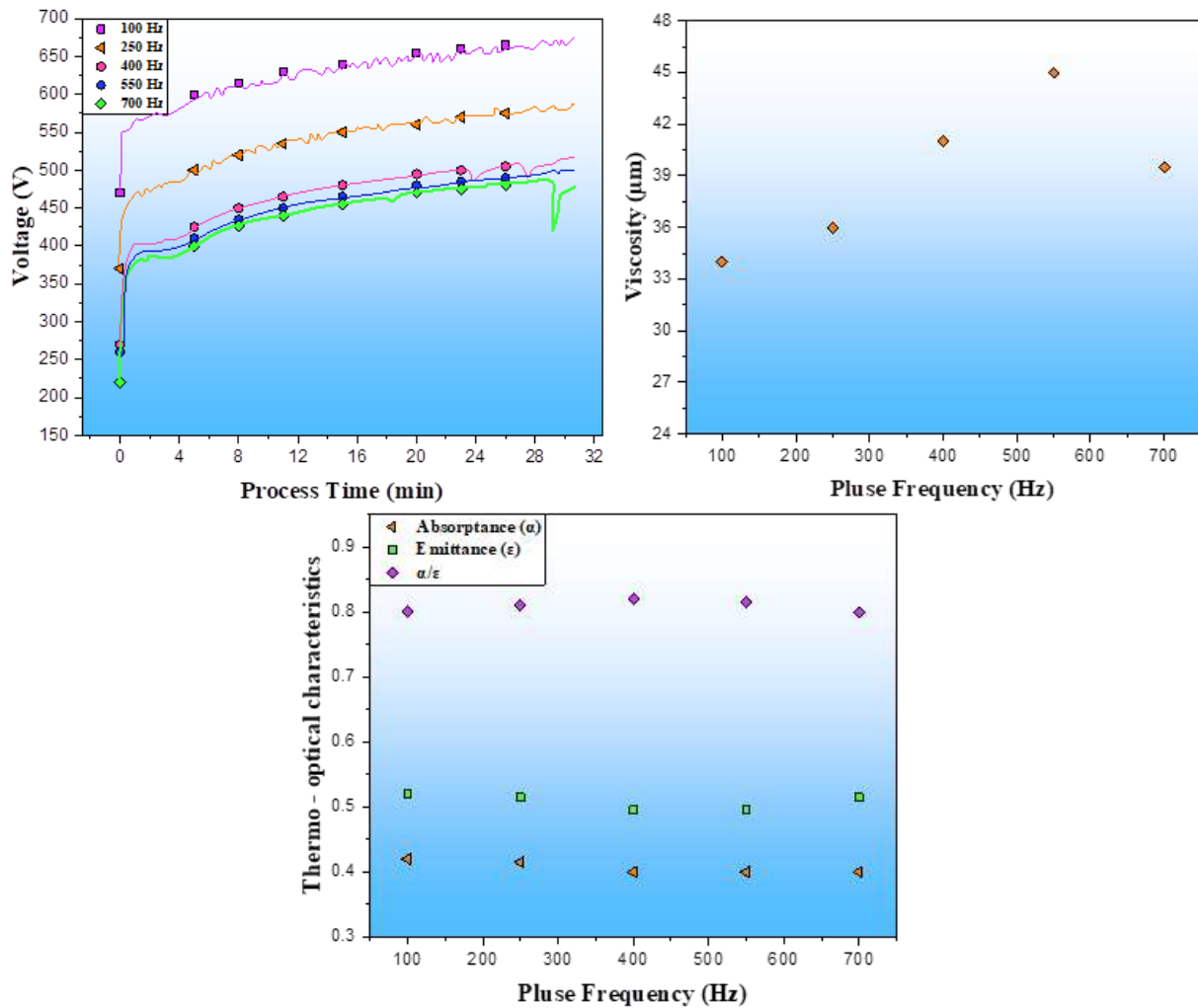


Fig. 6. (a) Impact of process time and voltage concerning pulse frequency, (b) Impact of pulse frequency on the viscosity of coating, and (c) Difference of thermal and optical characteristics function of pulse frequencies

3.2.4 Impact of Pulse On-Time

The rate of growth in the process or the coating does not appear to affect the proportion of on-time occurrences. Sodium silicate's voltage-time response is positively affected by the on-time at a pulse frequency of 400Hz and a mean current density of 9 Adm^2 , as illustrated in Figure 7. Coatings developed had a thickness of around 42 m (± 3 m). Therefore, their thermo-optical characteristics were also stable. For 35 minutes in a 90 gl^{-1} electrolyte.

3.2.5 Impact of Mean Current Density

At sufficiently high output voltages, the same pattern emerges in the voltage-time responses over a range of average current densities. It was found that the average current density was approximately linearly related to the coating growth rate and, by extension, the coating thickness. Modifications in thermo-optical characteristics reflect these tendencies. Variation in findings cannot be attributed to average current density alone due to the influence of thermo-optical characteristics on coating thickness. Samples are manufactured at various current densities over varying times to ensure that the average coating thickness is approximately 42 m (± 3 m) in each case, removing viscosity's effect on the established impact of average current density on the thermo-optical properties. Assuming a linear rise in thickness with time, the processing time is determined for each average current density by dividing the rate of thickness buildup by the average current density. Accordingly, procedures utilizing average current densities of 3, 6, 9, 12,

and 15 Adm^{-2} call for 100, 48, 34, 22, and 16 minutes, respectively. Fig. 8 shows that the mean current density has little impact on the coating's thermal optical characteristic, as absorption and emittance values are identical for all the layers.

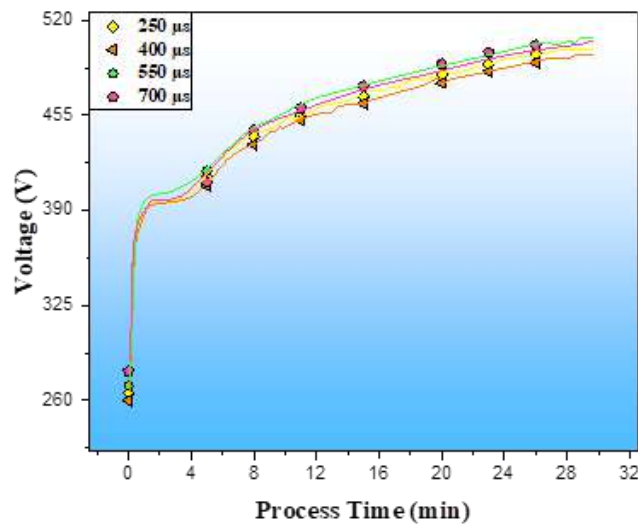


Fig. 7. Impact of process time and voltage concerning pulse on time

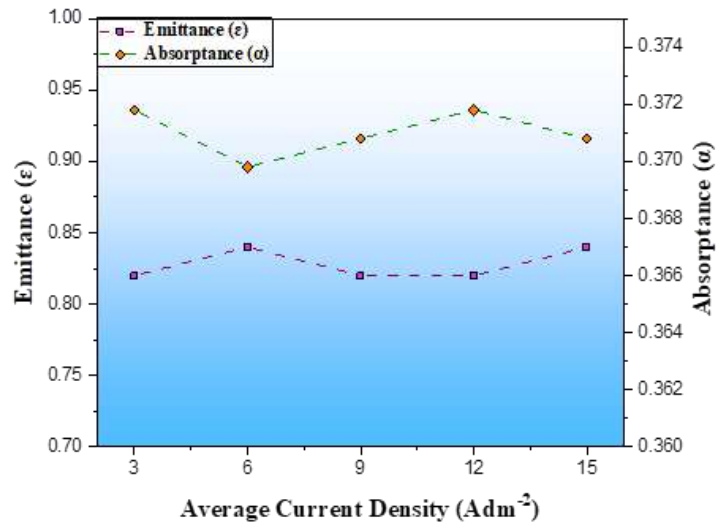


Fig. 8. Thermal optical characteristics of mean current density for Plasma Electrolytic Oxidation coatings of around $42 \pm 3 \mu\text{m}$ viscosity

3.2.6 Impact of Hydrothermal Sealing

Thermo-optical characteristics of thin samples were shown to be unaffected by hydrothermal sealing, while thicker models were affected (Fig. 9). It's important to note that the ratio α/ϵ did not alter even when both emittance and absorption levels decreased. Due to its significance in thermal control design, the α/ϵ value is shown to be unaffected by hydrothermal sealing without compromising the coating's thermo-optical behavior.

Table 1. Output factors for plasma electrolytic oxidation coating

Electrolyte elements	
Sodium meta silicate ($\text{Na}_2\text{SiO}_3 \cdot 9\text{H}_2\text{O}$)	90 gl^{-1}
Process factors	
Duty cycle	15%
Mean current density	9 Adm^{-2}
Pulse on-time	550 μs
Processing time	35 min
Pulse frequency	400 Hz

Authors comprehensively study how various operational parameters affect the coating development rate and the coating's thermo-optical behavior to produce a Plasma Electrolytic oxidation coating with a mean viscosity of $42 \pm 3 \mu\text{m}$ demonstrating thermal-optical attributes similar to that of standard anodized coatings. Table 1 displays the fine-tuned processing factors for creating a Plasma Electrolytic oxidation coating with $\varepsilon=0.81$ and $\alpha=0.386$, ideal for use as a solar reflector.

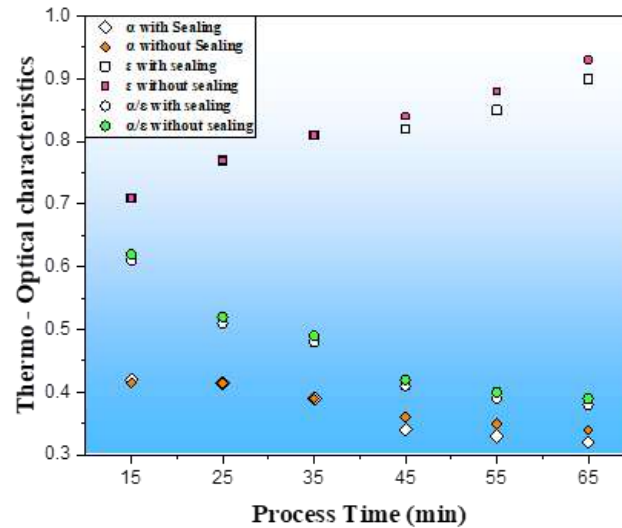


Fig. 9. Hydrothermal sealing of coating after Plasma Electrolytic Oxidation process and its effect on thermal and optical characteristics

3.3 Properties of the Optimum Plasma Electrolytic Oxidation Coating

3.3.1 Coating Existence and Roughness of The Surface

Plasma Electrolytic Oxidation coatings are found to be rough and whitish-grey, in contrast to the anodic coating's smooth and translucent look. Compared to an anodized sample with an RMS roughness of $0.31 \mu\text{m}$, a PEO-coated model has a roughness of $4.46 \mu\text{m}$.

Table 2. Surface roughness factors of the anodic specimen and Plasma Electrolytic Oxidation - coated specimen

Roughness factors	Anodized aluminum	PEO coated aluminum
Rq	$0.26 \pm 0.02 \mu\text{m}$	$4.34 \pm 0.15 \mu\text{m}$
Rq	$232 \pm 5 \text{ nm}$	$4528 \pm 90 \text{ nm}$
Ra	$181 \pm 4 \text{ nm}$	$3612 \pm 72 \text{ nm}$

Table 2 displays the roughness parameters—Rq is the root-mean-square roughness, and Ra (arithmetical mean roughness) is defined as the arithmetic average of the absolute values of the surface height deviations measured from the mean line over the evaluation length, in accordance with ISO 4287—attained for the two coating types. Results are similar to those obtained with a standard contact surface profiler in roughness. In Table 2, first Rq denotes surface profiler and second Rq denotes nano profilometry.

3.3.2 Surface Morphological Properties and Elemental Arrangement

Figure 10a and b show corresponding scanning electron micrographs of a Plasma Electrolytic Oxidation -coated specimen and an anodized specimen. On the other hand, PEO-coated characters showed a very distinct morphology under the same magnification, most likely because of the differing coating growth mechanism involved in the two processes. Comparatively bigger pores can be seen in the Plasma Electrolytic Oxidation coated specimen compared to the anodized sample.

Immersing the anodized specimen in demineralized (DM) water at or near boiling temperature generates minor fractures in the film's surface, known as hot water sealing. Due to the enormous difference in the coefficient of thermal expansion between metallic aluminum and aluminum oxide, thermal pressures can cause anodic coatings on aluminum to crack or craze. The method of coating growth inherent to PEO coating is responsible for its characteristic shape. When the breakdown voltage is reached, the coating dielectrically breaks down, forming several discharge channels. The coating material melts due to the high temperature produced by the micro-discharges, and the liquid is then ejected down the discharge channels. As the electrolyte cools the molten material, the structure takes on a volcanic aspect as the gaseous products escape through the discharge channels. The increasing oxide layer undergoes multiple cycles of melting, melt flow, re-solidification, and sintering, which results in a consistent and uniform increase in coating thickness. Moreover, microcracks appear on the coating due to the inherent tension caused by the deposition process.

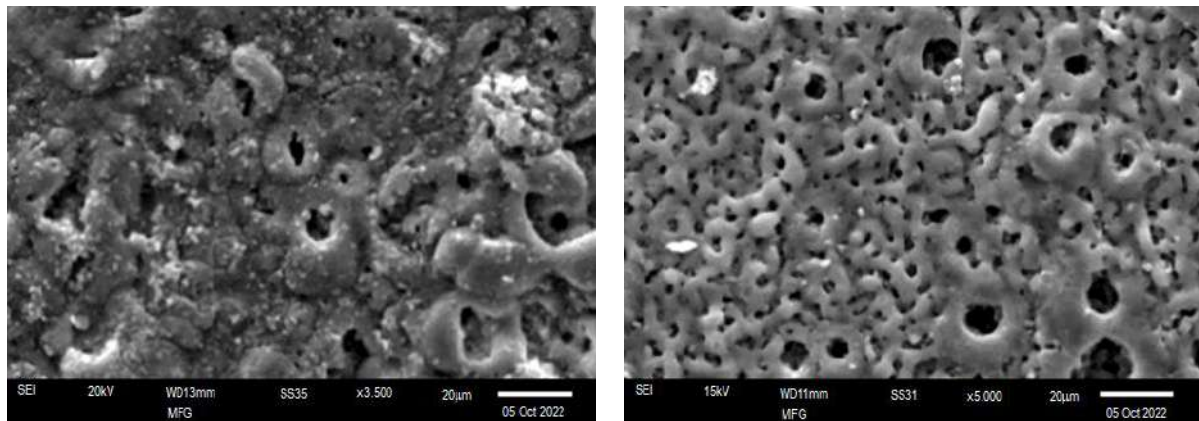
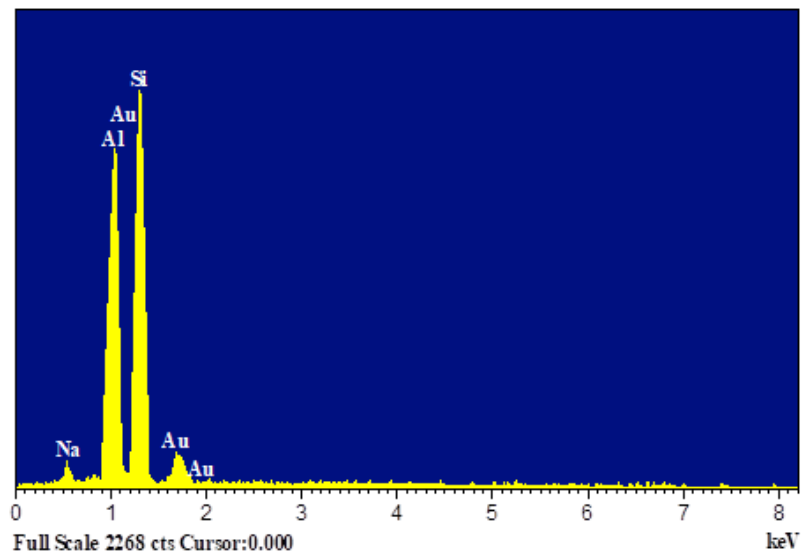


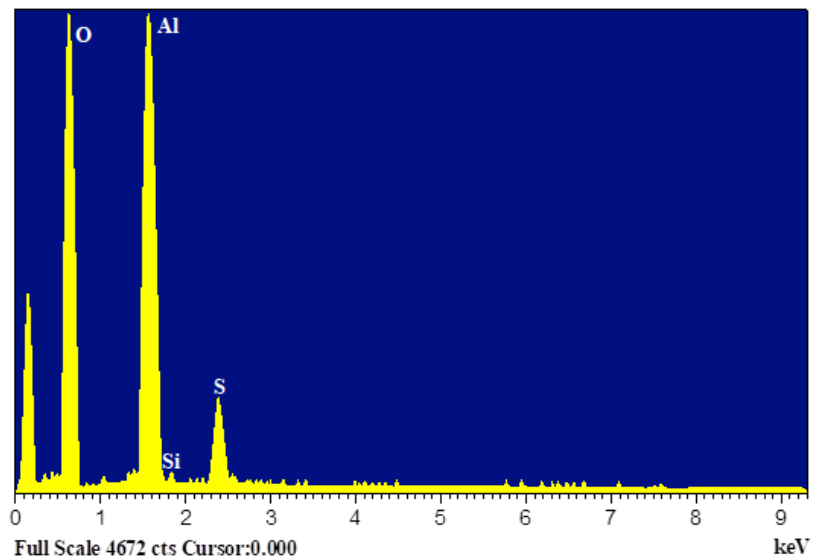
Fig. 10. Scanning electron microscopic images of (a) plasma electrolytic oxidation -coated specimen and (b) anodic specimen

Immersing the anodized specimen in demineralized (DM) water at or near boiling temperature generates minor fractures in the film's surface, known as hot water sealing. Due to the enormous difference in the coefficient of thermal expansion between metallic aluminum and aluminum oxide, thermal pressures can cause anodic coatings on aluminum to crack or craze. The method of coating growth inherent to PEO coating is responsible for its characteristic shape. When the breakdown voltage is reached, the coating dielectrically breaks down, forming several discharge channels. The coating material melts due to the high temperature produced by the micro-discharges, and the liquid is then ejected down the discharge channels. As the electrolyte cools the molten material, the structure takes on a volcanic aspect as the gaseous products escape through the discharge channels. The increasing oxide layer undergoes multiple cycles of melting, melt flow, re-solidification, and sintering, which results in a consistent and uniform increase in coating thickness. Moreover, microcracks appear on the coating due to the inherent tension caused by the deposition process.

The EDX spectra of the PEO coating and the anodic coating (Figure 11b) are presented. Insets depict the parts of each coating type. There is little sodium in PEO coatings, along with Al, SiO₂, and O₂ (Fig. 11a). Anion species from the electrolyte, most notably SiO₃²⁻, contribute to the development of the coating, accounting for its comparatively high silicon content. Anionic species in the electrolyte enter the channel via electrophoresis under the influence of the strong electric field during the ejection of molten materials through the discharge channels, react with the molten material, and become incorporated into the coating during the rapid solidification process. The incorporation of the cationic species of the electrolyte into the layer during the plasma discharges is responsible, to a lesser extent, for the presence of sodium in the coating. The anodic coating consists mainly of aluminum and oxygen, with smaller amounts of sulfur (3.14 atomic %) and silicon (trace amounts) (Fig. 11b). Anodizing in a sulphuric acid electrolyte incorporates SO₄²⁻ ions into the anodic oxide layer, giving the film a sulfurous tint. Because silicon is an alloying element in AA 5052, trace amounts of silicon can also be found in the coating.



(a)



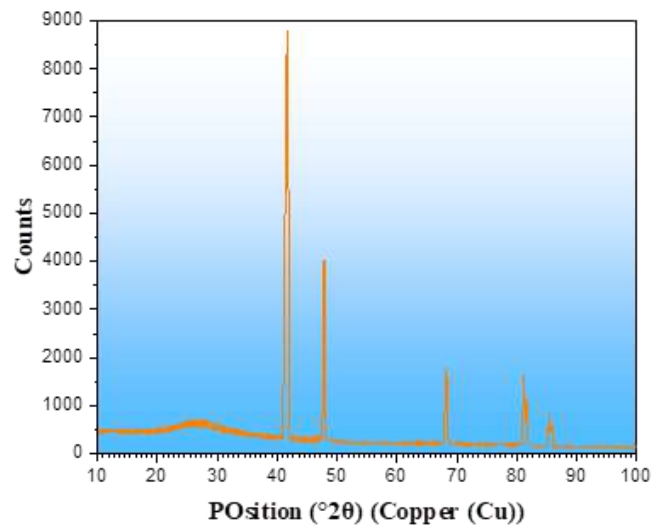
(b)

Fig. 11 Energy Dispersive X-ray spectroscopy and elemental analysis of (a) plasma electrolytic oxidation-coated specimen and (b) anodized specimen

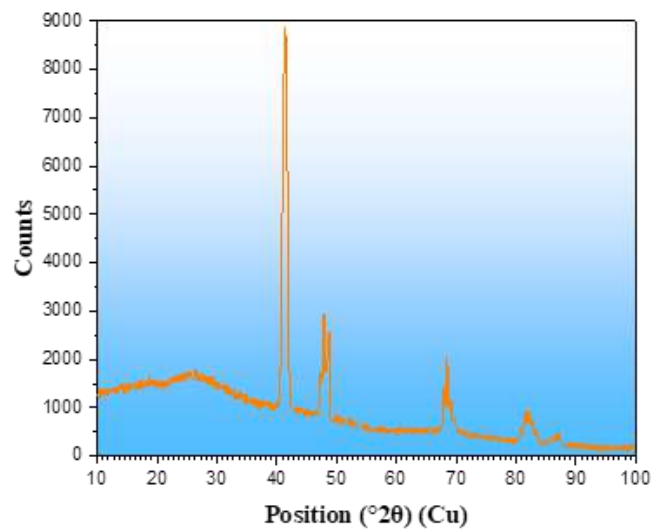
3.3.3 X-Ray Diffraction Analysis

The X-ray diffraction patterns for PEO and anodic coatings are displayed in Figures 12a and b, respectively. Aluminum (substrate) peaks at two values of 41.5° , 47.2° , 69.2° , 81.2° , and 85.8° were attributed in the XRD examination of both coatings.

When X-ray diffraction (XRD) patterns are examined, they show that neither the PEO nor the anodic coatings have any microstructure discernible to the X-ray beam. Peaks in the XRD pattern consistent with aluminum may be traced back to the aluminum in the substrate thanks to the way X-rays can pass through the coating and into the substrate.



(a)



(b)

Fig. 12. XRD pattern of (a) plasma electrolytic oxidation -coated specimen and (b) anodic specimen

3.3.4 Analysis of X-Ray Photoelectron Spectroscopy

XPS survey spectrum of anodic and Plasma Electrolytic Oxidation coatings are displayed in Figure 13. Al and O species can be seen in the ranges of an anodized coating, while Si and O can be seen in a PEO coating's spectrum. Figure 14a displays the XPS data for the Aluminum core levels of the anodic and Plasma Electrolytic Oxidation layers. Al2p has a peak at 75.3 eV in the anodized coating, and this energy is attributed to Al₂O₃. However, PEO coating has a modest Al2p elevation. Coatings made of polyethylene oxide (PEO) have a broad Si2p core-level spectrum, representing the existence of several Silicon species.

Presented in Fig. 14b is a curve fit into the component-peak form of this spectrum. Peaks in the Si2p range at 104.6, 107.8, and 109.8 eV represent SiO, alumino-silicate, and Si-OH species. These peaks are 24, 65, and 18% concentrated, respectively. It is clear that the alumino-silicate species predominate in this region. Spectra at the oxygen score level for both coatings are depicted in Figure. 14c. The oxide of alumina/alumino-silicate species accounts for the dominant peak at 534.2 eV.

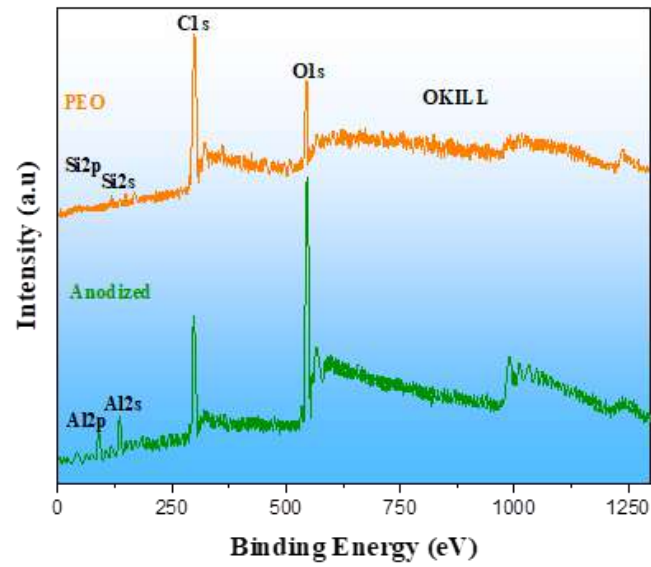


Fig. 13. Survey spectrum of anodic and Polyethylene Electrolytic Oxidation coatings

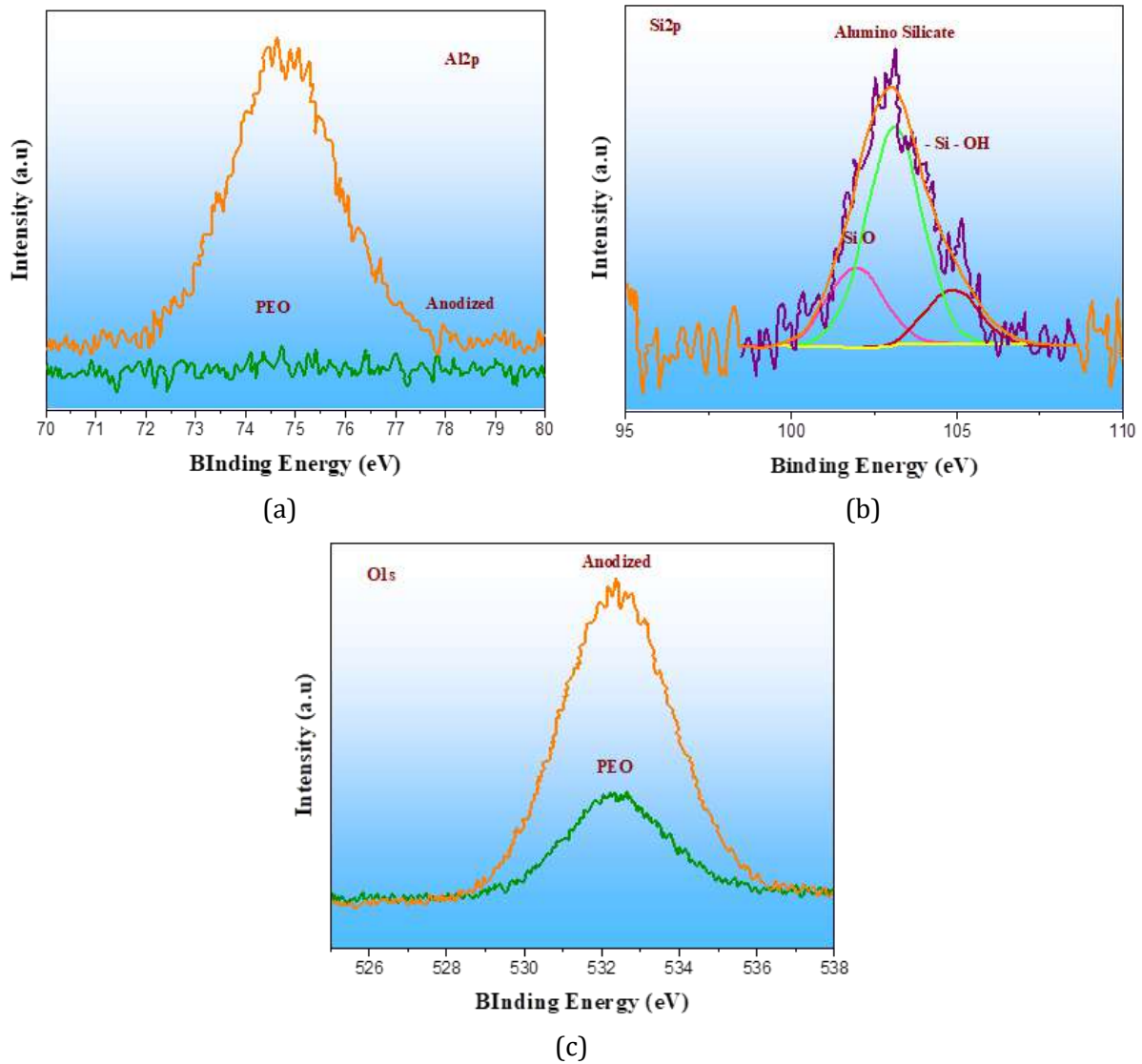


Fig. 14. (a) The PEO coating's Si2p core-level spectrum, fit to a curve, (b) Anodized and PEO coating O1 score-level spectrum, and (c) Anodized and PEO coatings' Al2p core-level spectrum

3.4 Space Criterion of The Optimum Plasma Electrolytic Oxidation Coating

3.4.1 Photographic Review

The Plasma Electrolytic Oxidation -coated samples are inspected for discontinuities like fractures, patches, excessive buildup, or loose films using a stereo microscope at a magnification of 15. Space simulation tests did not cause any visible damage, color change, or coating deterioration, as seen under a microscope after the samples were put through the ordeal.

3.4.2 Adhesive Testing

The tape peel adhesion test is performed before and after being exposed to the elements. To conduct the test, masking tape is applied over the coated surface. After the tape has been appropriately positioned on the coated outward, a 2-kg rubber-enclosed roller is rolled over it two times. Finally, in one fast action, the tape is taken off. There does not appear to have been any film removal that would expose the substrate surface.

3.4.3 Moisture Test

This procedure replicates the wet, corrosive conditions present just before liftoff to evaluate how well the coating holds up. Authors can learn much about the coating's corrosion resistance and stability under humid storage circumstances by subjecting it to humidity tests at slightly increased temperatures. This procedure is carried out in a humidity chamber with a thermostatically controlled temperature of $52 \pm 2^\circ\text{C}$. The samples are left in a humid environment ($96 \pm 0.3\%$) for 48 hours. The coating's integrity is checked visually after each testing round to ensure it hasn't eroded. There was also no discernible shift in the tested values of thermo-optical characteristics.

3.4.4 Thermo-Cycling Test

A spacecraft will experience both extremely cold and scorching temperatures throughout its lifetime. To find out how well a coating can hold up against repeated high and low temperatures, a thermo-cycling test is done. Temperatures in the hot and cold chamber are controlled by a thermostat, ranging from -70°C to $+125^\circ\text{C}$ for the test. There are a total of 1500 cycles applied, each lasting 5 minutes at each extreme temperature. The samples' thermo-optical properties did not significantly alter during the thermal cycling test, and a visual check confirmed no damage to the coating afterward.

3.4.5 Thermo-Vacuum Behavior Test

To study the effects of temperature cycling in the harsh space vacuum, a thermo-vacuum performance test is carried out. The process is carried out in a Thermovac chamber at a vacuum of 105 Torr to replicate conditions in space. The PEO-coated samples are cycled through temperatures of $+125^\circ\text{C}$ and -70°C for 2 hours at each temperature. This procedure of alternating heat and cold soaks is repeated ten times. The optical properties of the test specimens are then assessed and analyzed after the tests have been completed.

3.4.6 Evaluation of Thermal, Optical Characteristics

After undergoing extensive accelerated aging experiments in a replicated space environment, PEO coatings maintained a consistent thermo-optical behavior. Table 3 shows that the tested values for solar absorption (α_S) and infrared Emittance (ϵ_{IR}) do not deviate significantly from the as-prepared state.

Recent advancements in PEO-based thermal control coatings have demonstrated that processing parameters and electrolyte design significantly influence solar absorbance (α) and infrared emissivity (ϵ) on lightweight alloys. For instance, Tu *et al.* reported that PEO coatings on 6061 Al alloy with K_2ZrF_6 additive achieved substantially reduced solar absorbance and enhanced emissivity, with the lowest α (~ 0.16) and highest ϵ (~ 0.72) observed at $15 \text{ g}\cdot\text{L}^{-1}$ additive concentration, attributed to tailored coating thickness and microstructure effects [46]. Other recent studies have explored modified PEO strategies, such as nanoparticle addition to fabricate flat absorber coatings with tailored electric shielding and thermal control on 6061 Al, highlighting multifunctional performance improvements [47]. In contrast to these reports, the current study

systematically optimizes Na_2SiO_3 concentration and electrical parameters specifically for AA5052, achieving a balanced combination of high emissivity and competitive solar absorbance while validating the coating's stability under space-simulated environmental conditions, thus addressing a recognized need for alloy-specific process refinement and durability assessment in thermal control applications.

Table 3. Experimental results of absorptance (α_S) and emittance (ϵ_{IR}) before and after testing

S. no.	Experimental Testing	Testing of Solar Absorptance (α_S)		Testing of Infrared Emittance (ϵ_{IR})	
		Before	After	Before	After
1	As received	0.382, 0.371	not applicable	0.96, 0.96	not applicable
2		0.373, 0.382	not applicable	0.98, 0.98	not applicable
3		0.379, 0.377	not applicable	0.98, 0.98	not applicable
4		0.370, 0.371	not applicable	0.98, 0.98	not applicable
5	Moisture	0.386, 0.382	0.393, 0.391	0.96, 0.98	0.98, 0.98
6		0.370, 0.371	0.382, 0.381	0.96, 0.96	0.98, 0.98
7		0.369, 0.371	0.381, 0.382	0.83, 0.83	0.98, 0.98
8		0.369, 0.371	0.382, 0.385	0.96, 0.96	0.98, 0.98
9	Thermo-cycling	0.374, 0.372	0.388, 0.386	0.98, 0.98	0.96, 0.96
10		0.376, 0.378	0.395, 0.392	0.96, 0.96	0.96, 0.96
11		0.374, 0.372	0.381, 0.379	0.96, 0.96	0.96, 0.96
12		0.373, 0.370	0.379, 0.388	0.96, 0.96	0.98, 0.96
13	Thermo-vacuum cycling	0.373, 0.371	0.377, 0.373	0.96, 0.96	0.98, 0.98
14		0.373, 0.369	0.378, 0.377	0.96, 0.96	0.98, 0.98
15		0.372, 0.382	0.375, 0.377	0.96, 0.96	0.98, 0.98
16		0.376, 0.382	0.379, 0.378	0.96, 0.96	0.98, 0.98

4. Conclusions

Using a sodium silicate electrolyte and a unipolar pulsed DC square wave current, PEO coatings on AA5052 alloy can be produced. Different operational parameters' effects on the coating's thermo-optical characteristics are studied in depth. This procedure is improved such that Plasma Electrolytic Oxidation coatings with thermal, optical characteristics similar to standard anodic coating can be obtained.

- With a mean coating viscosity of 42 μm and absorption and infra-red emittance values of 0.382 and 0.98, PEO coatings are shown to have a thermal-optical behavior similar to that of standard anodized coating.
- Coatings made under ideal conditions are put through their paces in space by enduring a battery of accelerated aging tests in a laboratory replicating the conditions of outer space. After testing, there was no discernible shift in the coating's thermo-optical properties, including its solar absorption (α_S) and infrared Emittance (ϵ_{IR}).
- Additional techniques for characterizing the coating include SEM, EDX, XRD, and XPS, with results compared to conventional anodic coating about morphology, microstructure, and composition.

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Waste mortar as sustainable material in reactive powder concrete

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Abstract

This study investigates the feasibility of producing sustainable Reactive Powder Concrete (RPC) through the incorporation of silica fume (SF) and waste mortar (WM) as partial replacements for cement and fine aggregate, respectively. Four RPC mixtures were experimentally evaluated using silica fume contents of 15% and 20% combined with 50% waste mortar replacement. The mechanical performance was assessed through compressive and flexural strength tests at curing ages of 7 and 28 days, supported by statistical analysis of the experimental data. The findings revealed that increasing silica fume content significantly enhanced the mechanical properties due to improved pozzolanic activity, matrix densification, and refinement of the interfacial transition zone (ITZ). The mixture containing 20% SF and 50% WM exhibited the highest mechanical performance among the tested combinations, achieving compressive strengths of 103.18 MPa and 123.80 MPa at 7 and 28 days, respectively, in addition to a maximum flexural strength of 23.5 MPa. Also, the incorporation of waste mortar improved particle packing density and contributed to secondary hydration reactions, resulting in enhanced fiber matrix interaction and crack resistance. Statistical evaluation confirmed acceptable consistency and reliability for most RPC mixtures. The findings demonstrate that waste mortar can be successfully utilized in RPC production without compromising structural performance. While, a full life cycle assessment is required to quantify the exact ecological savings, this substitution highlights a promising avenue for enhancing the sustainability profile of RPC by reducing reliance on cement and natural sand.

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1. Introduction

Ultra-high-performance concrete (UHPC), also referred to as Reactive Powder Concrete (RPC), is an advanced cementitious composite characterized by its exceptional mechanical properties and superior durability. Initially introduced by Pierre Richard and later refined by Marcel Cheyrezy and Nicolas Roux at the French construction firm Bouygues in the 1990s [1,2]. UHPC represents a significant leap in material science. Parallel to these technical advancements, the construction industry faces mounting environmental pressure, as cement production currently accounts for approximately 8% of global CO₂ emissions. Consequently, there is a growing imperative to develop eco-friendly alternatives. Integrating industrial waste into RPC mixtures has the potential to contribute to reducing cement consumption and environmental impact, by serving as a material substitution, which supports the development of sustainable, high-performance materials [3]. In this context, silica fume is an industrial by-product obtained during the smelting process in the silicon and ferrosilicon industry. Silica fume plays an important role in the production of high strength and high-performance concrete. Increased environmental awareness

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has further helped the usage of industrial by-products to develop the sustainability of concrete production. The utilization of industrial by-products has become an attractive alternative for enhancing sustainability in concrete production [4]. Silica fume, ultra-fine sand (150–600 μ m), and superplasticizer are conventionally employed in the formulation of reactive powder concrete; the incorporation of superplasticizer is imperative in RPC applications due to the exceedingly low water/binder ratio, approximately 0.2. This minimized water/binder ratio is crucial for attaining the desired compressive strength, which is typically around 120MPa or less. Steel fibers are commonly integrated into RPC to enhance the concrete ductility, given that the combined material inherently exhibits considerable brittleness. A fractional volume ratio of 2%–3% is generally implemented to mitigate the risk of abrupt of the concrete failure. The RPC matrix exhibits a high density attributable to the significant utilization of Supplementary cementitious materials, in combination with fine sand; consequently, the packing density of the cementitious matrix is significantly enhanced, which accounts for the exceptional mechanical strength and durability characteristics of reactive powder concrete (RPC). Most RPC mixtures reported in the literature do not incorporate coarse aggregates, as the interfacial transition region between cement matrix and coarse aggregates is considered the weakest phase within the concrete microstructure [5]. Reactive powder concrete (RPC) is designed in order to achieve elevated strength, durability, and sustainability in structural applications. Compared with high-performance concrete (HPC), RPC displays greater compressive and flexural strengths, greater toughness, and markedly lower porosity and permeability. Microstructural analyses verify that silica fume improves bond between fiber and cement matrix, leading to improved fiber pullout energy [6]. The literatures show observed silica fume ranges across many experimental studies used in RPC studies are roughly: 10% to 30% by weight of cementitious material, many recent works focus on (15–25) % of SF. Several authors report performance improvements up to about (20–25)% SF, beyond which workability problems or diminishing returns in strength are observed [7,8,9]. The incorporation of silica fume at replacement range of (0–20)% significantly improves the mechanical performance of RPC. When the SF content was 15%, the 28-day flexural and compressive strengths were the highest [2]. While rapid urbanization has positioned the construction industry as a cornerstone of national economies, it has simultaneously created a critical need for sustainable waste management due to the massive generation of demolition debris. However, this growth has resulted in the generation of large quantities of construction and demolition waste, leading to acute environmental effects. An effective approach to managing this waste is its utilization in concrete production in the form of recycled coarse and/or fine aggregates [10].

Concrete is the most commonly utilized construction material worldwide; however, the vast extraction of virgin raw materials required for its production has caused significant environmental impacts. In response to growing sustainability concerns, numerous research studies have been conducted to investigate the incorporation of waste materials as partial or full replacements for conventional constituents in concrete, aiming to reduce environmental problems while maintaining or enhancing material performance [11]. The optimum levels of replacing dune sand by waste foundry sand 5%, 10%, 15% and 20% respectively [12]. Utilization rate of coal gangue as a sand replacement at different replacement ratios weight from 0– 100% was studied in reactive powder concrete (RPC) [13]. The evaluation of recycled fine aggregate incorporated into new concrete mixes was performed through tests on the hardened plain concrete. The investigation examined the using of the influence of recycled fine aggregate on new concrete mixes was investigated by means of experimental tests performed on hardened plain concrete specimens. The mixes were prepared with fine aggregate replacement ratios of 0% (control mix), 25%, 50%, and 100% concrete product, fine aggregate incorporated in concrete mixes with replacement ratios of 0% (control), 25%, 50%, and 100% [14]. The glass sand contents varying from (10 % to 50 %) as fine aggregate replacement [15].

Despite the extensive research on reactive powder concrete (RPC), most previous studies have primarily focused on optimizing silica fume content and mechanical performance under conventional material compositions. However, limited attention has been given to the combined effect of silica fume and the incorporation of recycled or waste based fine materials as partial

replacements for natural sand, particularly in terms of their influence on both compressive and flexural behavior of RPC.

Therefore, this study aims to investigate the synergistic effect of silica fume and waste mortar in Reactive Powder Concrete. Although silica fume is widely used to enhance RPC performance and waste mortar has been considered as a sustainable replacement material, their combined effect has not been systematically evaluated. This study addresses this gap by providing a comprehensive assessment of their interaction and its implications for mechanical properties, durability, and sustainability. The research specifically evaluates their effect on compressive strength and flexural strength. Furthermore, this study seeks to evaluate the selected replacement levels to identify the most effective mixture that achieves a balance between enhanced mechanical properties and environmental sustainability. The novelty of this work lies in combining high performance RPC technology with sustainable material utilization, contributing to reducing cement consumption and promoting eco-friendly construction practices.

2. Materials

In this study, all constituent materials were selected to meet the performance requirements of Reactive Powder Concrete. Ordinary Portland cement, fine sand, silica fume, superplasticizer, steel fibers, water, and waste mortar were evaluated in terms of its type, physical and chemical properties, gradation, strength characteristics, fineness and compatibility with the other components.

2.1 Cement

Ordinary Portland Cement (OPC) was employed in the experimental program of this study. This type of cement is classified as a general-purpose cement and is suitable for all applications where the specific properties of other cement types are not required. In this article, the cement used for produce concrete (OPC) provide via Al-Mass Company. The physical properties and chemical analysis of the cement employed showed in tables (1) and (2). The compliance of the cement was verified according to the Iraqi Standard (IQS No. 5/2019) [16].

Table 1. Physical properties of the cement

Initial Setting Time (hour: min.)	1:08	45 (Min.)
Final Setting Time (hour: min.)	4:30	10:00 (Max.)
Fineness (Blaine) in m ² /kg	425	250 (Min.)
Compressive Strength (MPa)		
At 2 days	16.5	>10
28 days	.744	>42.5

Table 2. Chemical analysis of the cement

Oxide	Test result	According to IQS Limits
CaO%	60.41	
SiO ₂ %	21.84	
Al ₂ O ₃ %	4.72	
Fe ₂ O ₃ %	3.60	
MgO%	1.50	≤ 5%
SO ₃	2.3	≤ 2.5%
Loss on Ignition%	3.27	≤ 4%
Total	97.72	
Free lime %	1.12	
Insoluble Residue%	1.12	≤ 1.5%
L.S.F	0.83	0.66-1.02
M.S.	2.63	
M.A.	1.31	
C ₃ S%	38.67	

Oxide	Test result	According to IQS Limits
C ₂ S%	31.74	
C ₃ A%	6.42	
C ₄ AF%	10.95	

2.2 Fine Aggregate

Fine sand generally requires a higher water content to achieve a given level of workability due to its larger specific surface area. In this study, fine sand was utilized as a constituent material, passing through a 0.6 mm sieve; the fine sand was tested for its physical properties and sieve analysis in accordance with the Iraqi specifications (IQS No.45/1984) [17]. The test results are presented in tables (3) and (4).

Table 3. Sieve analysis of fine sand.

Sieve size (mm)	Cumulative passing%	Limits of Iraqi specification No.45/1984)
10	100	100
4.75	100	95-100
2.36	100	95-100
1.18	100	90-100
0.6	99	80-100
0.3	46	15-50
0.15	8	0-15

Table 4. Physical properties of fine sand.

Properties	Test result	Limits of Iraqi specification No.45/1984
Specific gravity	2.64	/
Fineness modulus	2.6	/
Absorption	2.3%	/
Sulfate content	0.244%	≤ 0.5 %

2.3 Waste Mortar

Waste mortar was utilized as a partial replacement for fine aggregate (sand) in the RPC mixtures to produce sustainable Reactive Powder Concrete and to conserve fine sand resources. The waste mortar was obtained from the remains and residues of cement plastering works from some buildings, passing from sieve 0.6mm. Table(5) display graded the nominal particle size distribution of the waste mortar according to requirements of (IQS No. 45/1984) and (Fig. 1) [17].



Fig. 1. Waste mortar sand

Table 5. Particle size distribution of waste mortar

Sieve size (mm)	Cumulative passing%	Iraqi specification No.45/1984) Limits)
10	100	100
4.75	100	95-100
2.36	100	95-100
1.18	100	90-100
0.6	100	80-100
0.3	49	15-50
0.15	15	0-15

2.4 Silica Fume

Silica fume was utilized as a mineral admixture that added to Reactive Powder Concrete mixtures. The physical and chemical characteristics, in addition to the strength activity index of silica fume, were evaluated in accordance with ASTM C1240 [18]. Table (6) presents the chemical analysis of silica fume and (Fig. 2).

Table 6. Chemical analysis of silica fume

Oxide Composition	Oxide Content %	ASTM C-1240 Limitations
SiO ₂	92	≥ 85
Al ₂ O ₃	0.50	---
Fe ₂ O ₃	1.25	---
Na ₂ O	0.35	---
K ₂ O	1.10	---
CaO	0.37	< 1
MgO	0.99	---
SO ₃	0.29	< 2
L.O.I.	3.15	≤ 6.0
Strength Activity Index	170	> 105



Fig. 2. Silica Fume

2.5 Water of Mixture

Tap Water employed in the mixing and curing concrete specimens, may affect not concrete strength but in addition setting time. In general, mixing water is rarely a factor in concrete strength. Mixing and curing water had a temperature at (25°C± 2) (IQS No. 1703/1992) [19].

2.6 Steel Fibers

A straight shape and an average tensile strength of more than 2300 MPa used in this work. Table (7) demonstrates the steel fiber properties and in (Fig. 3). Steel fibers were added in this study at a percentage by 2% volume of concrete [20].

Table 7. Properties of steel fibers utilized

Property	Value
Surface	Brass coated
Density	7860 kg/m ³
Tensile Strength	> 2300MPa
shape	Straight
Melting Point	1500°C
rate of length	13mm±1mm
Diameter	0.2mm±0.05mm
Aspect ratio (Lf/Df)	65



Fig. 3. Steel fibers used in this study

2.7 Superplasticizer

The superplasticizer (high- range water reducer) is employed to enhance homogeneity and reduces the risk for segregation. Its use of superplasticizer ensured adequate workability for efficient mixing and casting. The specific product used is based on modified polycarboxylic ether (Glenium 54), purchased from Master Builders Solutions construction chemicals (BASF), conforming to ASTM C494 [21]. Table (8) shows the superplasticizer properties.

Table 8. the superplasticizer properties

Form	Whitish to straw colored liquid
Relative density	1.07
PH	5-8
Dosage	0.50 and 1.75 liters per 100kg

3. Experimental Methods

3.1 RPC Design

The constituent materials were mechanically mixed following a controlled sequence to ensure uniform distribution and adequate workability. Initially, dry materials (cement, silica fume, sand, and waste mortar) were blended to achieve homogeneity, followed by the gradual addition of water and superplasticizer. Steel fibers were then introduced to ensure uniform dispersion without agglomeration. Strength requirements depend on the intended employ of the concrete. The relationships of strength, mix proportion, and cement factor are complex. Mixtures having strengths outside the ranges demonstrated can be made by varying the mix proportions. The experimental program consisting of four trial mixtures was conducted to determine the most effective proportions of silica fume, waste mortar, water-to-binder ratio, and superplasticizer dosage required to achieve the target workability and mechanical performance of RPC. Silica fume was incorporated as a cement replacement at levels of 15% and 20% to optimize particle packing

density. The selection of 50% waste mortar as a replacement for natural sand was based on achieving a sustainable balance without critical loss in workability, as observed during the initial trial mixes. In this study, silica fume was used as a partial replacement for cement. The replacement percentages were calculated by mass of cement, while the total binder content was maintained constant throughout the mixtures. The mix proportions were determined based on the total binder content (cement + silica fume). Silica fume replacement was calculated as a percentage of the cement mass, whereas the waste mortar replacement was calculated relative to the mass of the fine aggregate. The superplasticizer (SP) dosage was maintained at 3.5 L per 100 kg of cement. The absolute value of 33.25 L/m³ presented in table (9) was derived by applying this specific dosage rate to the cement content used in the mixtures.

Table 9. Detailed mix proportions of RPC mixtures (kg/m³)

Mix No.	OPC Kg	Silica Fume %	Silica Fume Kg	Steel fiber Kg	Fine Aggregate Kg	Sand waste mortar%	Sand waste mortar Kg	Water Kg	SP Liter
M1	807.5	15	142.5	157	1050	0	0	171	33.25
M2	760	20	190	157	1050	0	0	171	33.25
M3	807.5	15	142.5	157	525	50	525	171	33.25
M4	760	20	190	157	525	50	525	171	33.25

3.2 Mixing Process

- All dry constituents were initially mixed for 2 minutes to ensure a uniform distribution of the materials (cement, silica fume, sand or waste mortar).
- After adding 70% of the mixing water, the mixture was constantly mixed for a further 3 minutes.
- After adding 15% of the water mixture containing 70% superplasticizer, the mixture was stirred once more for three minutes.
- For one minute, the mixing was halted.
- To ensure that the fiber was evenly distributed, the residual water and superplasticizer were incorporated into the mixture and mixing for 4 minutes. After that, steel fibers were added and mixed for 1 minute, for a total of 14 minutes.

3.3 Casting

The molds were cleaned and a tiny layer of oil was applied to the inside to keep the concrete from adhering on the mold's walls and ready for casting samples in accordance with ASTM C192/C192M [22], and the samples were then covered for a full day with polythene sheet.

3.4 Samples Curing

After demolding, the concrete specimens were immediately cured by full immersion in water tanks maintained at a controlled temperature of 25°C ± 2°C until the designated testing ages of 7 and 28 days.

3.5 Tests

3.5.1 Fresh Properties

The flow table test was conducted to determine the appropriate water-to-cement ratio (w/c) and evaluate the workability of the RPC mixtures through the measurement of flow spread values. The flowability of the fresh mixtures was evaluated using the flow table test in accordance with ASTM C1437, as shown in (Fig.4).

3.5.2 Compressive Strength

Compressive strength was determined in accordance with ASTM International (ASTM C109/C109M-2020) using cube specimens of (50 × 50 × 50) mm. Three specimens were tested for

each mixture at curing ages of 7 and 28 days, and the average value was adopted as the compressive strength result, as shown in (Fig. 5).



Fig.4. Flow table test of RPC mixtures



Fig. 5. Compressive strength testing machine used for specimen testing

3.5.3 Flexural Strength

Flexural strength was determined in accordance with ASTM C348, 2016) using prismatic specimens of dimensions $40 \times 40 \times 160$ mm. The specimens were tested undergo a center-point loading test, the flexural strength machine as shown in (Fig.6).



Fig. 6. Flexural strength test setup showing the center point loading configuration for prism specimens

4. Results and discussion

4.1 Workability of RPC

The workability of RPC mixtures was determined by using the mortar flow table test in accordance with (ASTM C1437, 2020) [23]. The main aim to carry out flow table test is to determine the best superplasticizer dosage and w/c required to achieve workability for ease of handling and replacement. Based on the preliminary trials, the optimum superplasticizer dosage was fixed at 3.5 liters per 100 kg of OPC and W/C Ratio 0.18, Table (10) show the measured flow spread values for each RPC mixtures.

Variations in flow among the mixtures reflected the competing physical effects of silica fume and mortar waste. Increasing silica fume from 15% to 20% led to a reduction in flow, primarily due to the increase in ultrafine particles and the specific surface area, which raises water demand that agree with [2]. Mixtures containing 50% mortar waste (M3, M4) showed a clear reduction in flow, with values to 200 and 190 mm, respectively. This reduction is mainly due to the porous and

angular nature of mortar waste particles, which increase internal friction and absorb part of the mixing water agree with [13].

Table 10. Flow spread values for each RPC mixture

Mix No.	Flow spread mm
M1	255
M2	250
M3	200
M4	190

4.2 Compressive Strength

The results of compressive strength are shown clearly in table (11) and (Fig. 7), incorporation of silica fume at 20% in mix M2 increased the strength to 100.35 MPa, representing an improvement of approximately 11.5% compared to the control mix. Similarly, mix M3 (15% silica fume with 50% sand waste mortar) achieved 97.93 MPa, showing an increase of about 8.8%. The highest enhancement was observed in mix M4 (20% silica fume with 50% sand waste mortar), which reached 103.18 MPa, corresponding to an increase of approximately 14.6% relative to M1.

28 days strength, the reference mix (M1) reached 99.82 MPa. Mix M2 showed an increase to 119.62 MPa, indicating development of about 19.8%. Mix M3 reached 110.72 MPa, reflecting an increase of approximately 10.9%, while, mix M4 recorded the highest compressive strength of 123.80 MPa, representing a 24.0% improvement relative to the control mix. This improvement in mix M4 is likely associated with the synergistic filler and pozzolanic actions of the silica fume and waste mortar particles, which are widely reported in the literature to facilitate pore refinement and densification in RPC microstructure. Similar findings were reported in previous studies, where silica fume improved the interfacial transition zone and enhanced later age strength through secondary hydration reactions. In addition, recycled fine waste materials may provide supplementary pozzolanic reactivity and internal curing effects that further improve matrix compactness, agree with [26]. This behavior has been widely reported in RPC systems, where silica fume contents between 15% and 20% provide optimal strength development. These findings are in agreement with [2], who confirmed that incorporating silica fume up to 20% much improves compressive strength due to enhanced particle packing and pozzolanic reactivity. Furthermore, the role of waste mortar can be attributed to the presence of residual unhydrated cement grains, which undertake secondary hydration, thereby enhancing the bonding structure. This observation aligns with [13] who reported that recycled fine materials can improve compressive strength when suitably incorporated due to internal curing and filler effects.

Table 11. Compressive strength and statistical analysis of RPC mixtures

Mix No.	7 Days (MPa)	7 Days		28 Days (MPa)	28 Days	
		C.V%	S.D(MPa)		C.V%	S.D(MPa)
M1	90.00	7.44	6.68	99.82	25.02	28.90
M2	100.35	19.13	19.32	119.61	1.74	1.60
M3	97.92	14.45	14.14	110.72	5.10	5.64
M4	103.17	11.54	11.90	123.80	16.80	20.80

The statistical analysis demonstrated that the compressive strength results exhibited acceptable consistency and repeatability for several investigated RPC mixtures. The coefficient of variation (CV%) values ranged from 7.44% to 19.13% at 7 days and from 5.10% to 25.02% at 28 days. According to recent studies, CV values below 10% indicate excellent repeatability, whereas values below 20% are generally considered acceptable for reactive powder concrete due to its dense and heterogeneous microstructure [26]. However, the M1 mixture at 28 days exhibited a CV value of 25.02%, indicating comparatively higher variability than the acceptable range. These results describe the dispersion and repeatability of the experimental data only and do not represent inferential statistical significance between mixtures. Among all mixtures, M2 at 28 days showed the

lowest variability with a CV value of 1.74%, reflecting a highly uniform internal structure and consistent hydration behavior agree with [27].

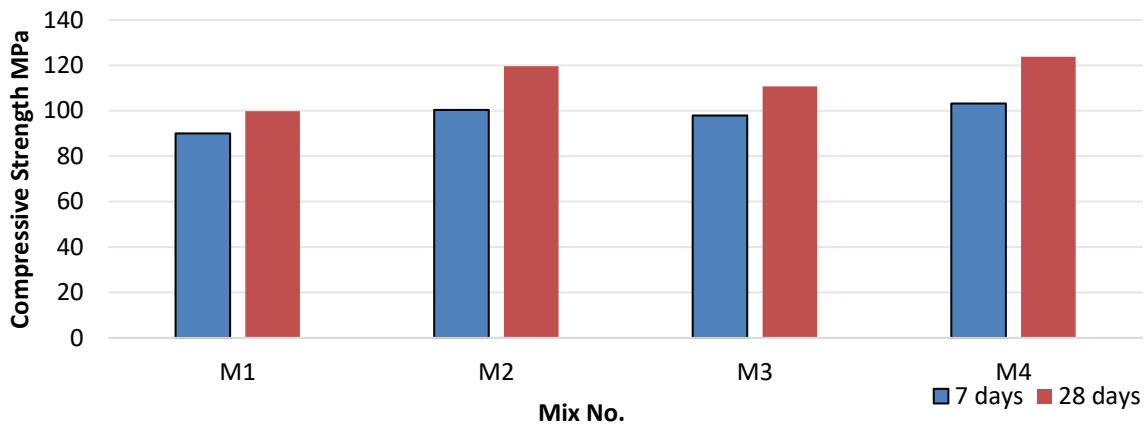


Fig. 7. The results of compressive strength (MPa)

4.3 Flexural Tensile Strength

The flexural strength results presented in Table 12 and Fig. 8 indicate a noticeable improvement in the mechanical performance of RPC mixtures incorporating silica fume and sand waste mortar. The reference mixture M1 exhibited flexural strengths of 12 MPa and 17 MPa at 7 and 28 days, respectively. Increasing the silica fume content from 15% to 20% in mixture M2 improved the flexural strength by approximately 10% at both curing ages, indicating that the additional silica fume contributed to a denser cementitious matrix through the pozzolanic reaction and micro-filler effect. This refinement of the pore structure enhanced the interfacial transition zone (ITZ) between the binder matrix and fine particles, thereby improving resistance to crack initiation under bending stresses agree with [28].

A more pronounced enhancement was observed when 50% sand waste mortar was incorporated. Mixture M3 achieved flexural strengths of 13.7 MPa and 19.6 MPa at 7 and 28 days, representing increases of about 14.2% and 15.3%, respectively, compared with M1. This behavior can be attributed to residual unhydrated cement particles and angular texture of the waste mortar, which enhanced particle packing and improved mechanical interlocking within the matrix. The highest flexural performance was recorded for mixture M4 containing 20% silica fume and 50% sand waste mortar, reaching 15.4 MPa at 7 days and 23.5 MPa at 28 days. These values correspond to increases of approximately 28.3% and 38.2%, respectively, relative to the control mixture. The substantial improvement suggests a strong synergistic interaction between the high silica fume content and recycled mortar particles. From a mechanistic perspective, the enhanced matrix densification improved stress transfer efficiency across the microstructure, while the refined ITZ reduced stress concentration zone corresponding to [29].

Analytically, the observed differences in flexural strength is highly sensitive to microstructural integrity and fiber-matrix interaction in RPC systems. Although steel fibers were maintained constant in all mixtures, the improvement in matrix quality directly influenced the bond behavior between fibers and the surrounding cementitious phase. A denser matrix increases the pull-out resistance of fibers, enabling more efficient stress redistribution after micro-crack formation. This explains why mixtures with higher silica fume and recycled mortar contents exhibited greater flexural resistance rather than merely proportional increases relative to compressive strength. The improved fiber anchorage likely contributed to delaying unstable crack propagation and promoted a more ductile flexural response corresponding to [30].

Regarding statistical evaluation, mixtures M1 and M2 showed moderate coefficients of variation ranging from 8.3% to 10.33%, whereas M3 exhibited the lowest variability (3.02–3.17%), indicating more homogeneous internal structure and consistent crack resistance. In contrast, M4 showed higher variability despite achieving the highest strength values, which may be related to

localized heterogeneity caused by the increased fineness and packing density of the blended materials. Nevertheless, the overall results confirm that the incorporation of silica fume and sand waste mortar can significantly improve the flexural behavior of sustainable RPC while maintaining acceptable experimental consistency agree with [27].

Table 12. Flexural strength and statistical analysis of RPC mixes

Mix No.	7days (MPa)	7 Days		28 days (MPa)	28 Days	
		C.V%	S.D(MPa)		C.V%	S.D(MPa)
M1	12	8.32	0.99	17	8.30	1.41
M2	13.20	10.33	1.37	18.90	10.29	1.94
M3	13.70	3.02	0.44	19.60	3.17	0.62
M4	15.40	19.76	3.03	23.50	19.75	4.33

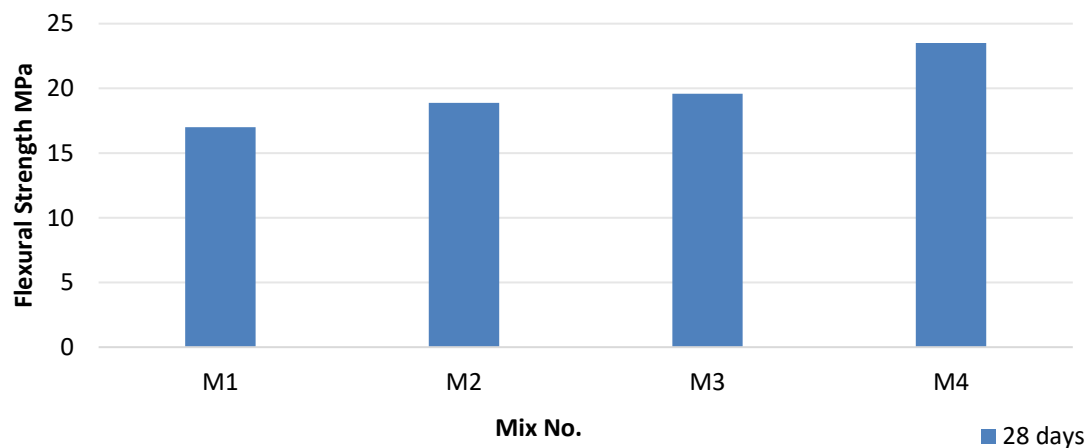


Fig. 8. Comparison of flexural strength of RPC mixes at 28 days

5. Conclusions

Based on the experimental results obtained in this study, the incorporation of silica fume as a partial cement replacement and waste mortar as a partial fine aggregate replacement influenced the fresh and mechanical properties of reactive powder concrete. The main conclusions derived from the experimental investigation are summarized as follow:

- The workability of RPC mixtures was reduced due to the low W/C or water/binder ratio (0.18) and the incorporation of silica fume, which increased the mixture's viscosity; however, the use of a superplasticizer effectively compensated for this reduction and ensured adequate flowability for proper mixing and casting.
- The incorporation of silica fume significantly enhanced the mechanical properties of RPC. Increasing the silica fume content from 15% to 20% resulted in noticeable improvements in both compressive and flexural strengths due to enhanced pozzolanic activity and matrix densification.
- The use of waste mortar as a 50% replacement for fine sand resulted in higher compressive and flexural strengths compared with the reference mixture development. This improvement is attributed to the presence of residual unhydrated cement particles, which promote secondary hydration, as well as improved particle packing and interfacial bonding.
- The merged use of 20% silica fume and 50% waste mortar Mix M4 reached the higher performance, recording compressive strengths of 103.177 MPa and 123.803 MPa at 7 and 28 days, respectively, and the highest flexural strength of 23.5 MPa. This indicates a strong synergistic effect between these materials.

- The statistical analysis showed that most RPC mixtures achieved acceptable compressive strength variability ($CV\% < 20\%$), with mixture M2 at 28 days exhibiting excellent consistency ($CV = 1.74\%$). In disparity, mixture M1 at 28 days showed the highest variability (25.02%), likely due to inhomogeneous particle packing and fiber dispersion. Overall, the results confirm that proper mix design and curing control are critical for minimizing variability in RPC.
- The observed enhancement in flexural strength is likely associated with the refined matrix quality provided by the silica fume, which plays a key role in strengthening the interfacial transition zone (ITZ).
- From a structural behavior viewpoint, RPC mixtures exhibited brittle behavior under compression, while showing semi-ductile behavior under flexural loading due to the presence of steel fibers, which enhanced crack bridging and delayed crack propagation.
- The results of this study demonstrate the technical feasibility of producing RPC by incorporating industrial waste materials. This approach offers a promising alternative for sustainable concrete construction by reducing the reliance on natural raw materials.

Generally, this study demonstrates that the combination of silica fume and waste mortar in RPC not only improves mechanical performance but also advance sustainable construction practices. The conclusions provide a practical basis for enhancing RPC mixtures with enhanced strength and environmental benefits.

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Durability and performance assessment of high-strength self-compacting concrete under environmental stressors

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Abstract

Self-compacting concrete (SCC) is widely acclaimed for its excellent flowability and capacity to compact exclusively under the power of gravity with no need for mechanical vibration. Yet, ensuring reproducible performance while minimizing typical problems like bleeding and segregation has long been a key concern. Addition of mineral admixtures such as ground granulated blast furnace slag (GGBFS), silica fume, and fly ash in the proportion of 5% to 30% has been found to improve the workability of the material and help in the production of high-strength types. This research investigates long-term durability properties of high-strength self-compacting concrete (HSSCC) in M60 and M70 grade, especially regarding resistance to environmental factors like acid attack, chloride and sulfate exposure, water absorption, effects of high temperatures, and rapid chloride ion penetration. The findings are reflective of high durability performance, especially among mixes with silica fume, which showed better densification of the matrix. Rapid Chloride Penetration Test (RCPT) results, as performed according to ASTM C1202, indicated that the concrete was of low permeability based on the finer pore structure imparted by the addition of silica fume and GGBFS. Overall, the results affirm the durability and resistance of the modified HSSCC under severe environmental exposure.

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1. Introduction

High-strength self-compacting concrete (HSSCC) achieves the benefits of high-performance concrete combined with enhanced flowability which guarantees successful placement even in highly reinforced structures. Nevertheless, its long-term performance in heavy environments is still challenged by durability problems like permeability, chloride ingress and chemical attack. In order to enhance the durability and sustainability of concrete, a number of Supplementary Cementitious Materials (SCMs) have been researched as cement substitutes to various degrees. Fly ash (FA), silica fume (SF), and ground granulated blast furnace slag (GGBFS) are some of them and are largely utilized because of their pozzolanic reactivity and filler properties. All of these materials play a distinct role in enhancing the mechanical and long-term performance of HSSCC: fly ash positively affects workability and long-term strength, silica fume positively affects densification of microstructure, and GGBFS positively affects resistance of chemical attack and chloride penetration. Also, the high cement content commonly used in HSSCC may predispose it to chemical attack and cracking owing to shrinkage in the event that it is not designed accordingly. A high-performance concrete with a water/binder ratio ranging between 0.30–0.40 is traditionally more resilient than typical concrete. It is the increased durability brought about not only by the

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decreased porosity but also by the partial disconnection of networks of capillaries and pores caused by self-desiccation [1]. Compressive strengths ranging between 65 and 85 MPa in 28 days have been observed in studies done on concrete with 30 and 40 percent fly ash of Western Australian Class F. Compared to control concrete of same strength the introduction of fly ash reduced drying shrinkage, and minimized sorptivity and chloride permeability at 28 days, and improved with time to 6 months. In general, the replacement of cement by fly ash increased the durability of the concrete [2]. In modern construction, concrete strength and durability have been increased significantly with blended cements, such as slag, fly ash, and silica fume, combined with modern admixtures, such as superplasticizers.

It is important to maintain the favorable rheological properties that will allow efficient and cost-effective concrete placement beyond simply the substitution rates or short-term strength [3]. General purpose Portland cement often takes the form of high-performance concrete that combines, as its admixtures, silica fume, slag and fly ash. It tests the mechanical and fresh properties of these combinations, for various applications. Although silica fume slightly decreased workability of GP cement mixes, silica fume greatly enhanced the mechanical properties of GP cement mixes and had a smaller effect on high slag cement mixes [4]. The primary practical use of SCM in concrete and cement is that, it is used to substitute some of the clinker in cement or cement itself, usually in order to make the material less expensive, greener, and more resistant to high loads and permanence. As of now, research is going on toward the identification of new materials, increasing replacement levels, enhancing testing procedures and changing the materials to incorporate performance enhancing additives [5]. HSC strengthened with steel fiber (SF) and carbon nanotube (HSCRSC) has excellent fluidity, strength, toughness, and longevity, and is suitable to be used in wellbores. Testing of 70 100 MPa HSCs indicated that fiber reinforcement increased compressive strength and switching of failure modes to ductile. Moreover, biaxial and triaxial compression strengths were greatly superior to the uniaxial compression strengths and the modified B3 offered the precise estimations of the shrinkage and creep at less than 6% error [6]. The rheological and compressive strength properties of SCC, as compared to the conventional, were also investigated in this study. V-funnel and L-box tests were used in assessing the flow properties of fresh concrete and its segregation resistance. SCC showed reduced early strength gain compared to traditional concrete, leading to reduced strength after 28 days although strength could be greater after 90 days [7]. The water/cement ratio exhibited a comparatively low influence on plastic properties of SCC in relation to the conventional concrete [8].

Durability of concrete is the ability to resist weather conditions and chemical exposure and abrasion resistance to its structural properties, is influenced by factors such as environmental conditions, intended use, and the nature of characteristic needed to enhance both strength and durability, RHA was applied partially to replace cement in M40 concrete (5-15%) in concrete. The workability tests showed that RHA had no influence on the concrete flow with 7.5% RHA giving optimal compressive strength. Durability tests such as resistance to acids, sulfates, alkaline as well as chlorides showed good performance. According to ASTM C1202, RCPT tests revealed that the penetration of chloride in the specimens was low to very low [11]. The experiment also evaluated the strength of SCC with the use of FAA as self-curing agents. FAA was pre-soaked 24 hours prior to mixing and scoria content was ranging between 5-20% and FAA at 15%. Tests were carried out to check the durability against salt attack, volume variation and UPV at 7, 28, 56, and 90 days on the hardened concrete. The findings indicated that substituting fine aggregate with pre-saturated scoria and FAA was effective in maintaining the strength as well as the durability of the concrete [12]. The industrial waste materials that can be utilized as alternative admixtures in concrete are silica fume, RHA, ferrochrome ash, and fly ash. The paper has determined that incorporation of small volumes of ferrochrome ash in 30 percent fly ash increases compressive strength extremely. Also, it also minimizes the emission of greenhouse gases and contributes to the management of waste, which facilitates sustainable construction [13]. Studied the durability properties of SCC, by adding the admixtures with ternary powder blends. SCC was produced with metakaolin, fly ash, and waste marble powder as replacements for cement and sand. Salhi et. al [14] investigated the influence of blended cements containing limestone and pozzolanic additions on the durability indicators of SCC. Use of fly ash-based Portland pozzolana cement with silica fume, metakaolin and

polycarboxylate ether were explored in SCC. They replaced SF, MK, or their mixtures with 5, 10, 15 and 20 percent of cement. Fresh properties were tested using a series of tests, which were relative to hardened properties, and some tests were followed by measurement of compressive and split tensile strength properties, and measurement of durability was then done [15]. Previous studies have examined the influence of these SCMs individually, showing that their inclusion significantly improves strength retention, reduces permeability, and enhances resistance to corrosive environments [2, 16,17]. Despite these advancements, further experimental validation is required to understand the individual contributions of each SCM to HSSCC's durability under varied exposure conditions [16][17].

1.1 Literature Review

Recent studies have explored the potential of various mineral and chemical admixtures to enhance the durability performance of self-compacting concrete (SCC). Pozzolanic cement significantly reduced sorptivity, water absorption, and porosity while improving resistance to sulfate and acid attack, demonstrating enhanced durability through microstructural refinement [14]. The inclusion of mineral powders significantly improved the concrete's resistance to permeability and chemical attack. Rajasekhar et. al [15] examined SCC produced with Portland Pozzolana Cement (PPC), supplemented with silica fume (SF), metakaolin (MK), and polycarboxylate ether (PCE) superplasticizer. The study involved partial replacement of PPC with MK, SF, and their combinations at 5–20% levels. Fresh property tests (slump flow, V-funnel, L- and U-box) and hardened property evaluations (compressive and split tensile strength) were conducted. Durability was further assessed through acid resistance, chloride penetration, and sulfate attack tests, with results confirming improved performance due to SCM inclusion. Murty et al. [18] stated that the combination of colloidal nano-silica in GGBFS-based SCC enhanced strength and durability over faster hydration, pore refinement as well as microstructural densification. Reddy et al. [19] recognized that nano-silica turns as both a nucleation site and a pozzolanic material, leading to improved C–S–H gel formation, condensed pore connectivity, and enhanced mechanical performance.

Table 1. Assessment of present study with previous literature

Study	Mix Composition	Tests Conducted	Key Findings	Advantage over Present Study
Nath & Sarker [2] – Cement & Concrete Composites	OPC + 20 % Fly Ash	Compressive strength, RCPT	Reported strength gain at later ages and moderate chloride resistance in high-volume fly-ash concrete.	Limited to a single SCM; no multi-admixture synergy studied.
Toutanji et al. [16]– Construction & Building Materials	OPC + 15 % Silica Fume	SEM, RCPT	Observed microstructural densification and very low chloride penetration due to pozzolanic reaction of silica fume.	Did not evaluate chemical or acid resistance.
Rajasekar et al. [17]– Journal of Building Engineering	OPC + 40 % GGBFS	RCPT, Sulphate resistance	Demonstrated improved chemical durability and long-term stability of GGBFS-based SCC mixtures.	Freeze–thaw and multi-environmental effects not considered.
Present Study	OPC + 15 % Fly Ash + 10 % Silica Fume + 20 % GGBFS	Compressive strength, RCPT, Chemical attack (HCl, H ₂ SO ₄ , MgSO ₄ , NaCl)	Exhibited enhanced strength retention, very low chloride ingress, and reduced weight loss under multiple stressors.	Evaluates combined SCM synergy and multi-environmental durability in HSSCC.

Gnanaraj et al. [21] developed SCC using fly ash and ultra-fine natural steatite powder (UFNSP) in ternary blends. Cement was replaced partially with fly ash (10% and 20%), and UFNSP was added

in varying dosages up to 25%. Durability measures were checked, such as porosity, water absorption, sorptivity, and rapid chloride permeability. The optimum blends minimized microcracks and pore connectivity, which were verified by SEM analysis as well as formation of magnesium silicate hydrate phases, which led to denser matrix. Aziz et al. [22] explored blended supplementary cementitious materials in self-compacting concrete to improve sustainability and engineering performance. The findings indicated that fine-tuning these materials improved workability, strength development, and durability through enhanced particle packing and matrix densification, decreasing cement usage. The comparative analysis (Table 1) shows that earlier researchers mainly studied the effects of individual supplementary cementitious materials like fly ash, silica fume, or GGBFS on the durability performance of concrete at a given exposure condition. Conversely, the current experiment incorporates various SCMs in a combined framework, and it is possible to synergistically improve both mechanical and durability properties in different environmental stress conditions. This holistic approach offers greater practicality for applications to aggressive service environments like marine and industrial environments. Table 1 provides a comparative overview of the literature.

1.2 Statement of The Problem

Despite the number of studies examining the impact of individual SCMs on the characteristics of high-strength concrete, including fly ash, silica fume, and GGBFS, there is still a lack of understanding of the individual impact of each material on the durability in various environmental conditions. Most past studies have concentrated on compressive strength or chloride resistance individually rather than including other attributes of durability, like acid resistance, sulphate resistance, weight loss, or long-term permeability. Such a deficiency of systematic knowledge limits the optimal application of individual SCM to particular durability demands. Hence, this research is aimed at comparing and assessing the respective individual impact of fly ash, silica fume, and GGBFS on the mechanical and durability properties of HSSCC with special reference to the resistance to the penetration of chlorides and chemical attacks under controlled laboratory conditions.

1.3 Research Objectives

The current research intends to examine the stability and performance of High-Strength Self-Compacting Concrete (HSSCC) under extreme environmental aggressors using a number of short-term accelerated and simulated long-term exposure tests. With the identified research gap, the specific objectives were developed as follows:

- To establish and assess HSSCC mixes with optimal mix proportions to represent high strength and durability performance. To investigate the resistance of HSSCC to different stress is durability including:
- Chloride penetration test which is measured on Rapid Chloride Penetration Test (RCPT) and salt ponding.
- Sulphate attack test was determined by the loss of mass and expansion.
- Acid resistance has been examined on the basis of mass loss and visible surface degradation.
- To compare performance of the HSSCC and that of normal-strength self-compacting concrete (SCC) and conventional high-performance concrete (HPC) under the same testing conditions.

1.4 Novelty and Research Significance

This study develops an all-inclusive durability evaluation framework of High-Strength Self-Compacting Concrete (HSSCC) that has been subjected to various loadings in the environment. The research employs new methods to improve the accuracy of the service situation in the real world and the validity of performance assessment:

- Hybrid SCM blending: It is made using optimized ratios of fly ash, silica fume, and ground granulated blast-furnace slag (GGBFS), to refine the pore structure, lower permeability, and enhance chemical attack resistance.

- Multi-aggressor assessment: Durability testing is not limited to any single exposure conditions but includes chloride, sulphate and acid environments to mimic the realistic degradation and to give a comprehensive profile of durability.
- Comparative benchmarking: HSSCC behavior is equated with that of normal self-compacting concrete (SCC), and high-performance concrete (HPC) behavior under the same exposure conditions, which allows easy evaluation of the relative merits of the behavior in strength retention and degradation resistance.

Generally, this study will be helpful in developing a methodological approach to durability assessment of sustainable and high-performance concrete in strong environments like coastal area, industrial area and cold climate areas. Overall, this investigation contributes to the development of a systematic durability evaluation approach for sustainable, high-performance concrete suitable for aggressive environments such as coastal, industrial, and cold-weather regions.

2. Materials

In the study, OPC grade 53 was chosen to comply with the IS:12269-1987 (9) requirements, providing cement of a high quality, with similar performance when used in buildings. The fine aggregate was river sand which is categorized as zone-II as per IS:383-1970 (10). This sand is best suited in concrete because it offers a balanced grading which adds to overall workability and strength of the mix. Crushed granite (12 mm) was selected as coarse aggregate, and it meets the standard requirements of the hardness, durability, and angularity that are important in the design of a robust and stable concrete matrix in accordance with the standards of IS 383: 1970 (10). In order to improve the performance of concrete, the byproducts of the different industries were used as partial replacements to the cement. Silica fume, fly ash, and GGBFS were added to the mixture at different concentrations (between 5 and 30 percent) of thermal power plants, ferrosilicon and steel manufacturing plants, respectively. The materials have been selected due to the following positive qualities: fly ash enhances workability and heat of hydration, silica fume adds strength and durability by improving pore structure, and GGBFS prevents chemical attack and permeability. The angular shape of these cementitious additives adds a lot to the overall strength development of the high-performance concrete since the angularity improves the mechanical interlocking of the particles. Moreover, the blend also included a superplasticizer (Glenium B233) and a viscosity-modifying agent (Glenium Stream2). The superplasticizer enhanced fluidity of the concrete without raising the amount of water in it hence maintaining the low water-cement ratio needed to create high-strength concrete. The viscosity-modifying agent was employed to make the mix stick together to overcome such problems as segregation and bleeding that may happen with high-flow concretes. With this balanced mix of materials and additives, it was properly planned to include high-performance concrete with high strength and durability, and workability to satisfy the construction standards of the day.

3. Experimental Procedure

Standard tests were conducted on typical specimens using the Indian standards of durability to determine the behavior of HSSCC in different situations. The angular shape of these cementitious additives and sulfate attack and high temperatures, the RCPT and the water absorption. It concentrated on mixes that had good mechanical performances. The study involved conventional mixes for grades M60 and M70, as well as blends with 20% fly ash, 10% silica fume, and 15% GGBFS, corresponding to the conventional grades of concrete. The specific mixes tested were CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC-70, SCF70-20, SCS70-10, and SCG70-15. Water curing spectrum of the cube specimens for 28 days before immersion into acid, chloride, and sulfate solutions for 30, 60 and 90 days. Compressive strength and periodic weight for the specimens which were refreshed every two weeks to ensure the solutions were still effective. RCPT of the concrete was done on cylindrical discs and water absorption test done on cube specimens to determine the permeability of the concrete.

3.1 Rationale for Selection of SCM Proportions

The percentage of supplementary cementitious materials (SCMs) constituting 10% silica fume, 15% ground granulated blast furnace slag (GGBFS) and 20% fly ash was chosen in this paper in relation to their respective as well as combined effects on the concrete durability, workability and mechanical performance. Silica Fume (10%) is characterized by a high reactivity of pozzolana and the ultrafine particle size, which significantly leads to the improvement of the microstructure of concrete by decreasing the porosity and increasing the interfacial transition zone (ITZ). It has been demonstrated that a 10% replacement level is the optimal level to maximize these benefits and not to affect workability. Research has established that silica fume with this dosage is effective in enhancing the chloride penetration and acid attack resistance, and therefore, it can be used in hostile environments.

GGBFS helps in long-term strength equipment and improves sulfate resistance because of the latent hydraulic characteristics. A 15% replacement will provide a balance between long life and youthful power. It has been found that, GGBFS enhances the ability of the concrete to resist chemical attack and it also lowers the heat of hydration, which is advantageous in the mass concrete practices. Fly Ash (20%) is highly spherical in nature and its pozzolanic reaction is slow therefore it enhances workability and also provides long term durability. A replacement level of 20 percent has gained a lot of acceptance in high performance concrete with the aim of decreasing permeability and increasing resistance to chloride intrusion. The dosage is also in line with sustainable building practices since it minimizes cement. The chosen proportions are justified by the existence of the prior experimental projects, which tested such compositions and indicated the improvement of such durability parameters as the lower values of rapid chloride permeability (RCPT), higher compressive strength, and decreased water absorption rates.

3.2 Experimental Design and Control of Environmental Variables

To measure the high strength self-compacting concrete (HSSCC) durability in the aggressive conditions of the environment, the specimens were immersed in three different solutions: sulfuric acid (H_2SO_4) and sodium chloride (NaCl) and sodium sulphate (Na_2SO_4). These solutions had concentrations that were chosen with close care to reflect realistic exposures in the situations that are likely to occur in industrial, coastal and sulphate-rich environments. Particularly, the solution was made using sulfuric acid with concentration of 5% by volume, with the pH that was kept at the range of 2.0-2.5 during the soaking time. This was done by constant monitoring with a calibrated digital pH meter and by adding dilute acid to the solution when necessary to overcome the buffering effect of leaching concrete ions.

The sodium chloride solution was held at 3.5 percent/kg, which is a salt concentration where sea water is found and the sodium sulphate solution was set at 5 percent/kg to simulate the conditions found in the soil or ground water of a high-sulphate concentration. The chloride and sulphate solutions were replenished weekly so that there were constant ionic concentrations and that the solutions were not depleted by chemical reactions with the concrete matrix. Each specimen was completely covered in the solution of the solutions at ambient laboratory temperature ($25 \pm 2^\circ\text{C}$) for 90 days. The soaking containers were covered so as to reduce evaporation and contamination and the exposure regime was kept constant in all mixes so that reproducibility of results is possible and the effect of binder composition on the performance of the durability is isolated.

4. Results and Discussion

4.1 Acid Attack Test

This study paper involved an acid attack test on standard 150mm cube specimens to determine their ability to resist acidic environment. The cube compressive strength meant was measured following 28-day water curing (NWC) and subsequently after being exposed to an acid medium on 30, 60 and 90 days. Moreover, the proportional weight loss as a result of being exposed to acid was noted. The findings, shown in Table 2 provide information on the stability of concrete mixes in acidic conditions. Tests conducted on the specimens are CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC-70, SCF70-20, SCS70-10, and SCG70-15. These specimens had compressive strengths of 66.1

MPa, 67.8 MPa, 68.1 MPa, 67.9 MPa, 75.9 MPa, 76.8 MPa, 78.4 MPa, and 77.8 MPa after 28 days of water curing, respectively. After 30 days at an acid medium, compressive strengths dropped to 63.2 MPa, 64.9 MPa, 65.4 MPa, 65.1 MPa, 72.4 MPa, 73.4 MPa, 75.2 MPa and 74.5 MPa. These values show a decrease of 4.39, 4.28, 3.96, 4.12, 4.61, 4.43, 4.08 and 4.24 respectively of their strengths after normal curing.

The first decrease indicates the sensitivity of the material to acid attacks, albeit the reduction is not that strong. The compressive strength at acid immersion at 60 days was 57.9 MPa, 59.9 MPa, 60.9 MPa, 60.4 MPa, 66.7 MPa, 67.7 MPa, 70.2 MPa and 69.0 MPa. These decreases of these strengths, or 12.41, 11.65, 10.57, 11.05, 12.12, 11.85, 10.46, 11.31, are all lower than after normal cure. The greater reduction shows the further worsening of the concrete structural integrity as the time of exposure in the acidic environment increases. The compressive strengths were further reduced to the level of 52.1 MPa, 53.8 MPa, 54.6 MPa, 54.1 MPa, 58.5 MPa, 59.8 MPa and 61.5 MPa, of the first 90 days of acid immersion. The 21.18, 20.65, 19.82, 20.32, 22.92, 22.14, 21.56 and 21.98 MPa respectively, are strength of these results reduced by normal curing values of specimens. This is especially true in the case of structures that might be subjected to such conditions as the high propensity of a loss in strength through a long-term exposure to acid justifies the choice of material with higher resistance to acid attack.

The continuous loss of weight with time is a gradual deterioration of the concrete matrix by the action of the acid which gradually attacks the material. Overall, these findings suggest that there is a risk to select and compose concrete mixtures with a higher resistance to the assault of acid, in particular, in the situations when the material under consideration is exposed to the acidic climate over an extended period of time. The data also claim that tested concrete mixtures lose strength and mass over time and that the rate at which they contribute to the deterioration is not even, but introduction of SCMs can support in minimizing undesirable outcomes of the exposure on acids.

Table 2. Acid attack test results

Mix ID	Compressive strength in MPa (at the day of)				% of loss in compressive strength (at the day of)		
	NWC	30	60	90	30	60	90
CSC-60	66.1	63.2 ± 2.89	57.9 ± 8.2	52.1 ± 14	4.39	12.41	21.18
SCF60-20	67.8	64.9 ± 2.89	59.9 ± 7.9	53.8 ± 14	4.28	11.65	20.65
SCS60-10	68.1	65.4 ± 2.69	60.9 ± 7.2	54.6 ± 13.5	3.96	10.57	19.82
SCG60-15	67.9	65.1 ± 2.80	60.4 ± 7.50	54.1 ± 13.8	4.12	11.05	20.32
CSC-70	75.9	72.4 ± 3.5	66.7 ± 9.2	58.5 ± 17.4	4.61	12.12	22.92
SCF70-20	76.8	73.4 ± 3.39	67.7 ± 9.09	59.8 ± 17	4.43	11.85	22.14
SCS70-10	78.4	75.2 ± 3.2	70.2 ± 8.2	61.5 ± 16.9	4.08	10.46	21.56
SCG70-15	77.8	74.5 ± 3.3	69 ± 8.8	60.7 ± 17.1	4.24	11.31	21.98

Table 3. Percentage loss in weight test results due to acid attack

Mix ID	% of loss in weight at the day of			
	0	30	60	90
CSC-60	0	4.22	6.28	8.11
SCF60-20	0	4.13	6.01	7.98
SCS60-10	0	3.85	5.84	7.65
SCG60-15	0	4.03	5.96	7.82
CSC-70	0	4.38	6.36	8.18
SCF70-20	0	4.24	6.13	8.02
SCS70-10	0	4.02	5.95	7.81
SCG70-15	0	4.19	6.02	7.95

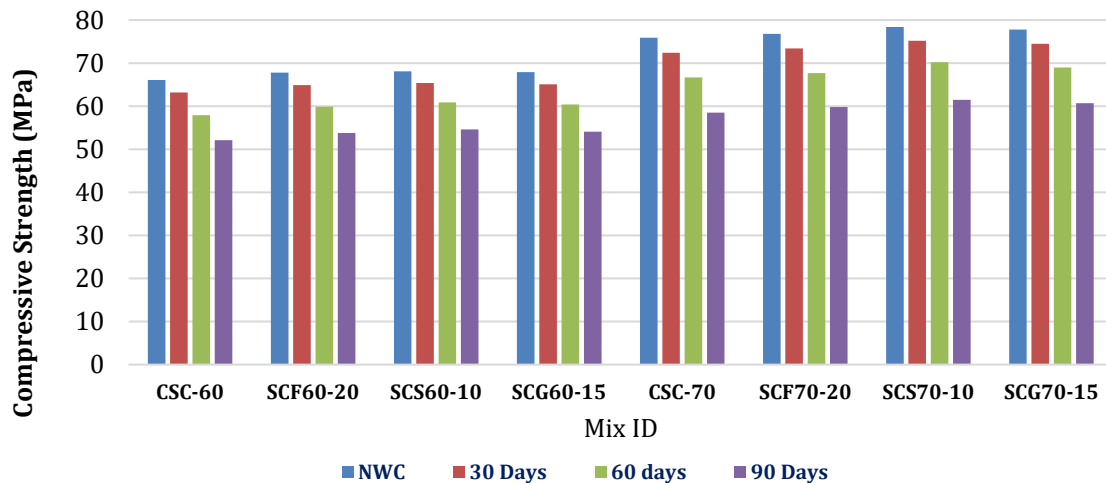


Fig. 1. Compressive strength variation with exposure duration for acid attack

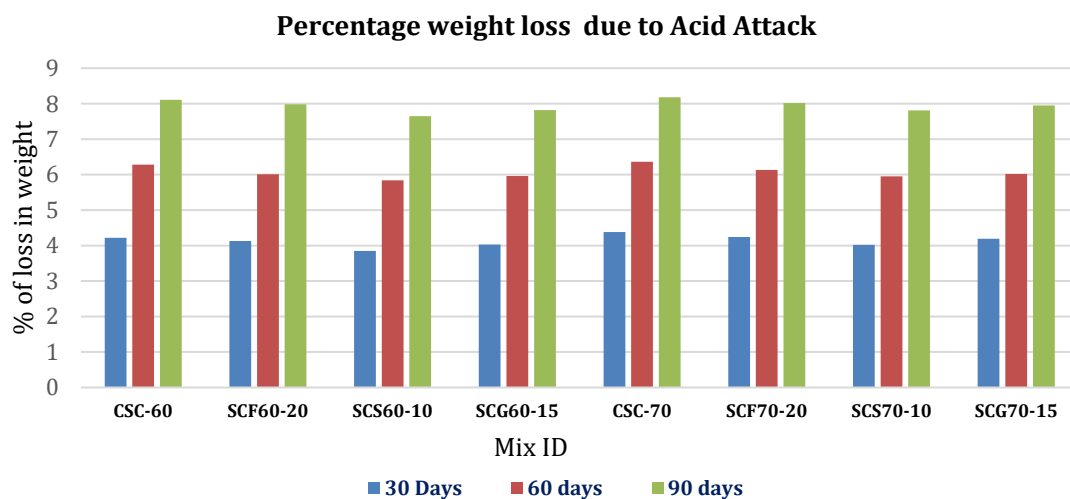


Fig. 2. Percentage weight loss with exposure duration for acid attack

4.2 Chloride Attack Test

The test was performed on standard 150 mm cube specimens to assess their performance regarding the resistance to degradation caused by chloride. The compressive strength of the specimens was determined after a normal 28 days water curing (NWC) and that after 30-, 60- and 90-days immersion under a chloride medium. The proportionality of the weight loss as a result of the chloride attack was also measured and the results are as presented in Table 4. The initial compressive strengths of CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC-70, SCF70-20, SCS70-10, and SCG70-15 specimens, 28 days water curing, and after 28 days, were 66.1 MPa, 67.8 MPa, 68.1 MPa, 67.9 MPa, 75.9 MPa, 77.1 MPa, 78.3 MPa, and 77.7 MPa. The compressive strengths reduced a little to 65.9 MPa, 67.6 MPa, 68.0 MPa, 67.8 MPa, 75.6 MPa, 76.6 MPa, 78.3 MPa, and 77.7 MPa after 30 days of being exposed to the chloride medium. These are relatively small, ranging between 0.13 and 0.40 per cent less than the respective compressive strengths following normal curing, suggesting that the effects of the chloride attack on the concrete as far as strength is concerned are relatively low. The compressive strengths of the chloride medium further reduced after 60 days to 61.8 MPa, 64.1 MPa, 65.3 MPa, 64.9 MPa, 71.6 MPa, 73.1 MPa, 75.2 MPa and 74.2 MPa. These values are however a greater decrease than in the instance of the strengths after normal curing, not at 4.08 but at 6.51. This reduction depicts that the length of exposure to the chloride has a more severe bearing on the integration of the concrete and perhaps with time, it leads to the erosion of the material. The compressive strength after 90 days in the chloride medium fell to another extent of 57.8 MPa, 60.3 MPa, 61.8

MPa, 60.8 MPa, 66.9 MPa, 68.4 MPa, 70.6 MPa, and 69.7 MPa. This continued degradation was confirmed by the now reduced range of compressive strength reduction from 9.25% to 12.56%. These results emphasize the need to develop concrete with better chloride resistance, especially for the structures in environments where the salt content is high or there is deicing salt.

Table 4. Chloride attack test results

Sl No	Mix ID	Compressive strength in MPa (at the day of)				% of loss in compressive strength (at the day of)		
		NWC	30	60	90	30	60	90
1	CSC-60	66.1	65.9±0.2	61.8±4.3	57.8±8.3	0.30%	6.51%	12.56%
2	SCF60-20	67.8	67.6±0.2	64.1±3.7	60.3±7.5	0.29%	5.46%	11.06%
3	SCS60-10	68.1	68±0.1	65.3±2.8	61.8±6.3	0.15%	4.11%	9.25%
4	SCG60-15	67.9	67.8±0.1	64.9±3	60.8±7.1	0.15%	4.42%	10.46%
5	CSC-70	75.9	75.6±0.3	71.6±4.3	66.9±9	0.40%	5.67%	11.86%
6	SCF70-20	76.8	76.6±0.2	73.1±3.7	68.4±8.4	0.26%	4.82%	10.94%
7	SCS70-10	78.4	78.3±0.1	75.2±3.2	70.6±7.8	0.13%	4.08%	9.95%
8	SCG70-15	77.8	77.7±0.1	74.2±3.6	69.7±8.1	0.13%	4.63%	10.41%

Table 5. Percentage loss in weight test results due to chloride attack

Sl No	Mix ID	% of loss in weight (%) at the day of			
		0	30	60	90
1	CSC-60	0	3.34	4.94	6.28
2	SCF60-20	0	3.14	4.68	6.12
3	SCS60-10	0	2.44	3.59	5.43
4	SCG60-15	0	2.91	4.08	6.01
5	CSC-70	0	3.41	4.89	6.31
6	SCF70-20	0	3.18	4.72	6.19
7	SCS70-10	0	2.53	4.01	5.73
8	SCG70-15	0	2.86	4.41	6.11

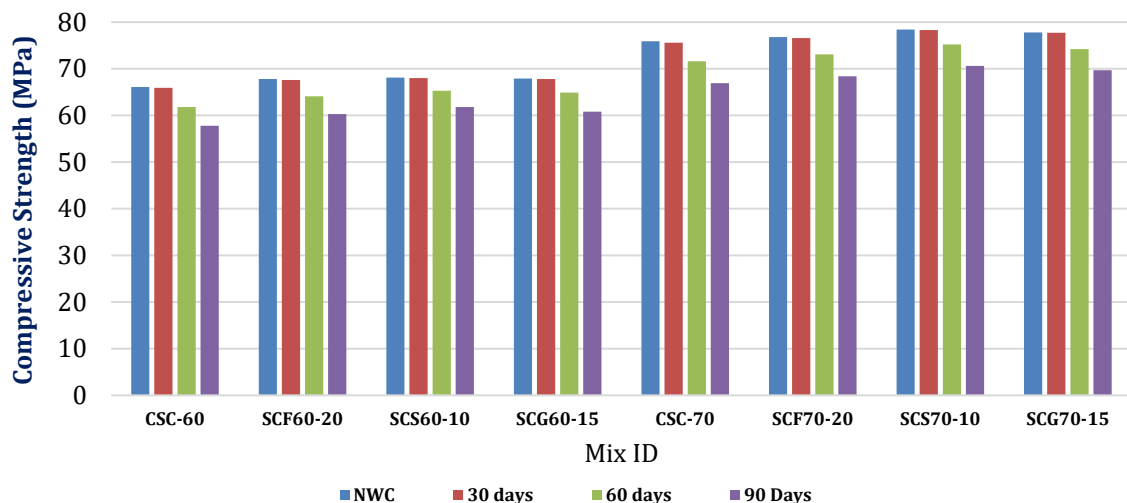


Fig. 3. Compressive strength variation with exposure duration for chloride attack

Table 5 includes monitoring of the weight loss due to chloride attack in addition to compressive strength. After 30 days of exposure, the percentages for the weight loss with the CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC70, SCF70-20, SCS70-10, and SCG70-15 specimens were 3.34%, 3.14%, 2.44%, 2.91%, 3.41%, 3.18%, 2.53%, and 2.86%, respectively, and remained modest. The initial loss of weight found is associated with the release of chloride-sensitive components of the concrete. Upon day 60, the weight loss was increased to 4.94%, 4.68%, 3.59%, 4.08%, 4.89%, 4.72%, 4.01%, and 4.41% respectively, indicating the continued deterioration of physical integrity of the concrete

due to chloride exposure. Over the course of 90 days, the percentage weight loss rose to 6.28%, 6.12%, 5.43%, 6.01%, 6.31%, 6.19%, 5.73%, and 6.11% and continued degradation of the concrete material due to chloride exposure.

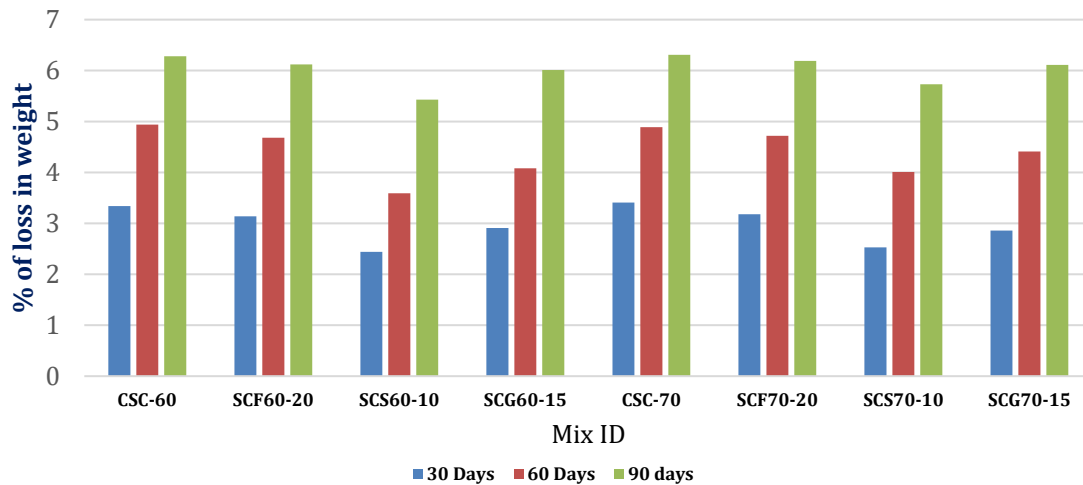


Fig. 4. Percentage weight loss with exposure duration for chloride attack

Figure 3 illustrates compressive strength variation with exposure duration, while Fig. 4. shows percentage weight loss trends due to chloride attack. Overall, the results of the chloride attack test indicate that the concrete specimens exhibit a former ability to resist chloride induced strength loss; however, prolonged exposure results in large decreases in both strength and in weight. Degradation extent is different between different concrete mixes and it is thought that some mixtures may be more resistant to chloride attack. Which emphasizes the necessity of materials and mix designs chosen carefully in environments where chloride exposure takes place.

4.3 Sulphate Attack Test

A sulfate attack experiment was conducted on standard 150mm cubes, in order to determine the durability of the cubes and their resistance to the effects of sulfate deterioration. It was conducted by measuring compressive strength of cubes at the end of standard 28 days of water curing (NWC), followed by 30, 60 and 90 days of the immersion in a sulfate solution. The %age of sulfate attack on weight loss was also reported. These tests were summarized in Table 6. First, compressive strengths of CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC-70, SCF70-20, SCS70-10, and SCG70-15 specimens were 66.1 MPa, 67.8 MPa, 68.1 MPa, 67.9 MPa, 75.9 MPa, 76.8 MPa, 78.4 MPa and 77.8 MPa. The compressive strengths of these specimens however reduced slightly after 30 days of exposure to the sulphate medium to 65.1 MPa, 66.8 MPa, 67.2 MPa, 67.0 MPa, 74.5 MPa, 75.7 MPa, 77.6 MPa, and 76.9 MPa respectively. The decreases in these strengths were between 1.02-1.84 %, which means that there was a slight initial effect of the exposure to sulphates on the compressive strength of the concrete after 60 days in the sulphate medium, and the compressive strengths of the specimens were further decreasing to 60.6 MPa, 62.2MPa, 63.4MPa, 62.8MPa, 69.1MPa, 70.3MPa, 72.1MPa, and their strength reduction was now between 6.90 and 8.96 % indicating a bigger corrosion of the concrete since it had been exposed to sulphates over a longer period. Lastly, the compressive strengths decreased to 53.8 MPa, 56.7 MPa, 58.2 MPa, 57.1 MPa, 62.5Mpa, 64.3Mpa, 66.6Mpa and 65.2Mpa after 90 days of immersion in the medium of sulphate. These reductions of 14.54% to 18.61% signify a significant reduction in the concrete strength, and the consequences of the long-term harmful impact of sulphate on the concrete matrix. Besides compressive strength, the weight loss as a result of sulphate attack of the specimens also measured and reported in Table 7.

The weight loss in the CSC-60, SCF60-20, SCS60-10, SCG60-15, CSC-70, SCF70-20, SCS-70-10, and SCG70-15 after 30 days exposure to sulphates was 3.58, 3.34, 2.94, 3.12, 3.68, 3.39, 3.01 and 3.28 respectively. This primary loss of weight is the first sign of chemical degradation of the concrete

since the sulphates start attacking the cementitious materials. The weight loss was also enhanced to 5.43, 5.02, 4.28, 4.91, 5.39, 5.13, 4.87 and 5.03 after 60 days of immersion in the sulphate medium. The weight loss increase is thus due to the continuous degrading of the concrete as the sulphate ions are still reactive to the components in the concrete and cause further degradation. The subjects had lost weight in the amount of 7.01, 6.82, 6.02, 6.39, 6.99, 6.56, 6.11, and 6.48 at the end of the 90-day exposure. The outcomes that have been achieved on the basis of these tests confirm that sulphates can indeed result in the loss of the material in the long run and hence compromise the structure of concrete.

Table 6. Sulphate attack test results

Sl No	Mix ID	Compressive strength in MPa (at the day of)				% of loss in compressive strength (at the day of)		
		NWC	30	60	90	30	60	90
1	CSC-60	66.1	65.1±1	60.6±5.5	53.8±12.3	1.51%	8.32%	18.61%
2	SCF60-20	67.8	66.8±1	62.2±5.6	56.7±11.1	1.47%	8.26%	16.37%
3	SCS60-10	68.1	67.2±0.9	63.4±4.7	58.2±9.9	1.32%	6.90%	14.54%
4	SCG60-15	67.9	67±0.9	62.8±5.1	57.1±10.8	1.33%	7.51%	15.91%
5	CSC-70	75.9	74.5±1.4	69.1±6.8	62.5±13.4	1.84%	8.96%	17.65%
6	SCF70-20	76.8	75.7±1.1	70.3±6.5	64.3±13.4	1.43%	8.46%	16.28%
7	SCS70-10	78.4	77.6±0.8	72.1±6.3	66.6±11.8	1.02%	8.04%	15.05%
8	SCG70-15	77.8	76.9±0.9	71.1±6.7	65.2±12.6	1.16%	8.61%	16.20%

Table 7. Percentage loss in weight test results due to sulphate attack

Sl No	Mix ID	% of loss in weight (%) at the day of			
		0	30	60	90
1	CSC-60	0	3.58	5.43	7.01
2	SCF60-20	0	3.34	5.02	6.82
3	SCS60-10	0	2.94	4.28	6.02
4	SCG60-15	0	3.12	4.91	6.39
5	CSC-70	0	3.68	5.39	6.99
6	SCF70-20	0	3.39	5.13	6.56
7	SCS70-10	0	3.01	4.87	6.11
8	SCG70-15	0	3.28	5.03	6.48

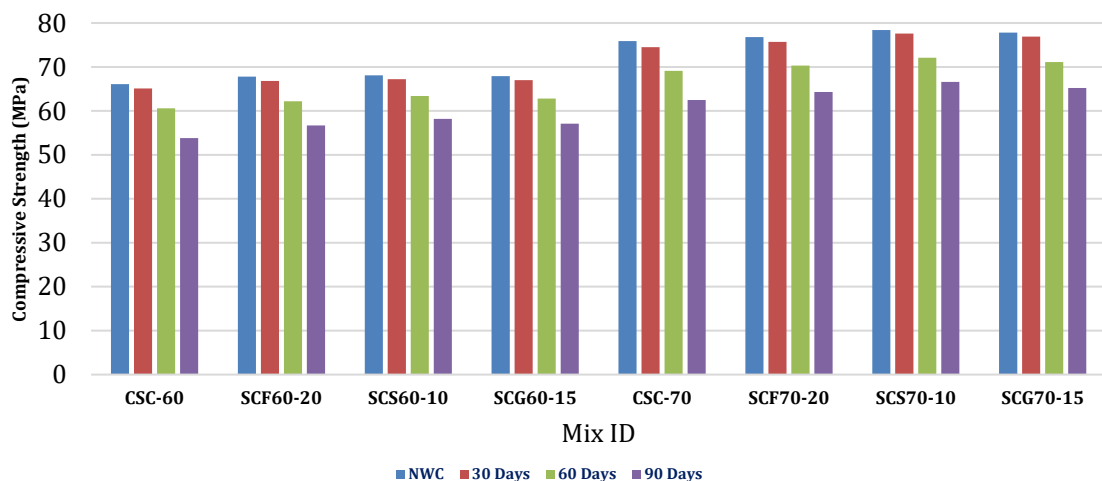


Fig. 5. Compressive strength variation with exposure duration for sulphate attack

Figure 5 demonstrates compressive strength variation with exposure duration, while Fig. 6. shows percentage weight loss trends due to sulphate attack. Results of the sulphate attack test indicate that concrete withstands initially the sulphate induced damage but in case of extended exposure, the compressive strength and weight parameters dropped decrease. The degradation in relation to concrete mix is also exhibited and it is concluded that the destruction caused by sulphate is highly

dependent on the materials used, whether they are sulphate resistant or not and also with regard to the mix design when sulphate is anticipated. This test emphasizes the necessity of the selection of proper chemical resisting materials and the mix design that will lead to a long-term durability and performance of concrete exposed to aggressive chemical environment.

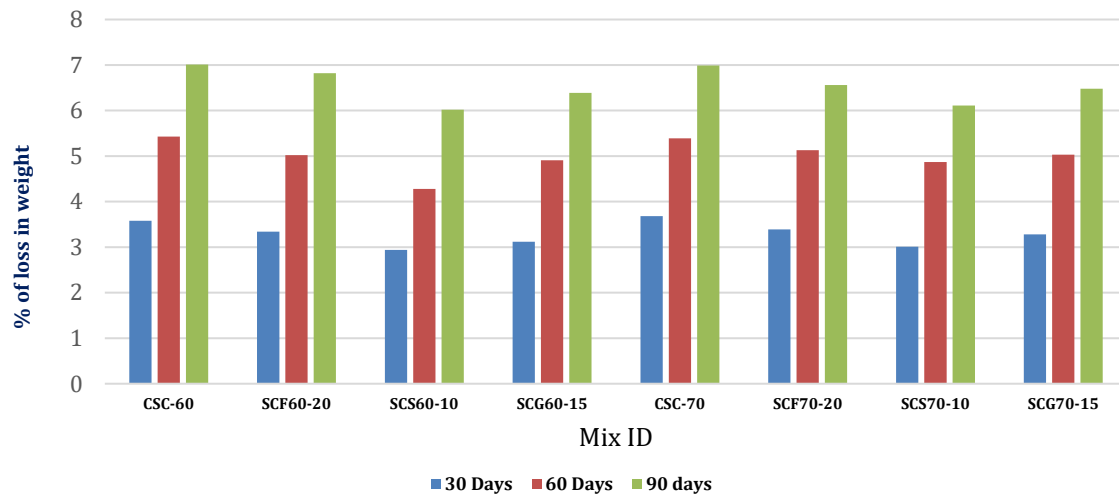


Fig. 6. Percentage weight loss with exposure duration for sulphate attack

The high-strength self-compacting concrete (HSSCC) in the real-world application and how it is suitable, in particular to marine infrastructure, fertilizer production facilities, and drainage systems, where chloride, sulphate and acidic environments are common. The increased durability in our investigation indicates that SCM-modified HSSCC may have a high potential to increase structural service life and decrease the frequency of maintenance thus lifecycle performance and cost-effectiveness. As well, the experimental findings verify that the SCM-based HSSCC has a high chloride ingress/chemical attack resistance. Its properties render it very appropriate in marine buildings (ports, jetties, offshore platforms), fertilizer plants and urban drainage systems, where exposure to aggressive ions is inevitable. The lower permeability guarantees the long life of use and reduced maintenance frequency, which is of huge economic and environmental advantage.

4.4 Rapid Chloride Penetration Test

The Rapid Chloride Penetration Test (RCPT) was performed on cylindrical disc specimens measuring 100 mm in diameter and 50 mm in thickness. This test evaluates concrete quality by quantifying chloride ion permeability through the measurement of electrical charge passed, expressed in coulombs. The presence of pores is high when the RCPT values are high and the concrete is designated under poor quality. When the RCPT values are low which means the presence of pores is also low in quantity and so the current voltage value is also low which indicates the concrete is made with good quality. The electrical charge passed during RCPT is reported in coulombs and classified according to ASTM C1202 and the results are summarized in Table 8. According to ASTM C1202, conventional Concrete and SCF70-20 specimens will be classified as Low. This categorization implies that the mixes of concrete used in this case have a comparatively high resistance to the penetration of chloride ions implying that the concrete mixes contain fewer pores, and therefore, more resistant to chloride-related degradation.

Table 8. RCPT values of HSSCC

Mix ID	Charge Passed in Coulomb	Recommendation as per ASTM C1202
CSC-60	1228 ± 110	1000-2000 - Low
SCF60-20	994 ± 85	100-1000 - Very Low
SCS60-10	768 ± 75	100-1000 - Very Low
SCG60-15	848 ± 82	100-1000 - Very Low
CSC-70	1109 ± 105	1000-2000 - Low
SCF70-20	1021 ± 88	1000-2000 - Low
SCS70-10	762 ± 74	100-1000 - Very Low
SCG70-15	821 ± 80	100-1000 - Very Low

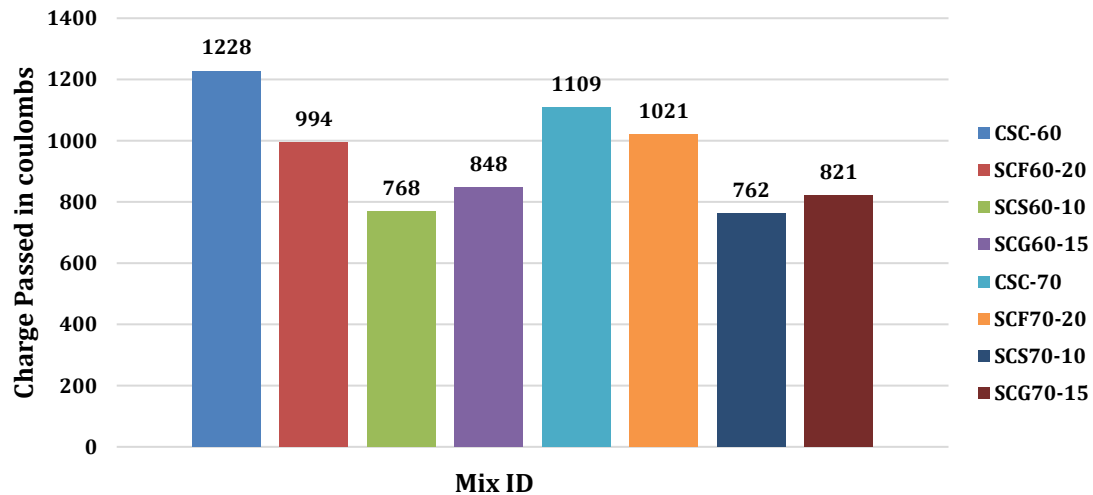


Fig. 7. RCPT results of HSSCC

Other Specimens are classified as very Low. This classification is an indicator of even improved performance with little penetration of chloride ion which depicts the very low presence of pores and high-quality concrete mix. RCPT findings indicate that the standard concrete and the SCF70-20 mix has sufficient resistance to the penetration of chloride ions and satisfies the requirements of the Low category. In the meantime, the other mixes have better performance as they have the best classifications of Very Low indicating that they are highly resistant to chloride attack. Chloride ion penetration was significantly lowered by the addition of silica fume and GGBFS. Fig. 7. provides a bar chart of the values of charges in all mixtures, making it evident that SCM-based HSSCC with an improved resistance is the illustration of the successful quality and durability of analyzed concrete samples. The findings of the study show that high-strength self-compacting concrete (HSSCC) with fly ash, silica fume and GGBFS exhibited a much higher strength and durability than that of the control mix. This increase in performance could be explained by the pozzolanic reaction and filler effect of the SCMs that polished the pore structure and formed more C-S-H gel.

The results of RCPT showed that total charge passed on SCM-blended concretes was significantly lower and they fell within the category of low to very low permeability per ASTM C1202. This proves the enhanced resistance towards chloride penetration and also signifies denser and less interconnected pore network. The peer-reviewed works have been cited in order to support these observations. Similar results were previously reported [2, 16,17] where silica fume and GGBFS were observed to lower the pore connectivity, interfacial transition zone strength and resistance to aggressive ions. These findings seem to confirm the current findings, indicating that the enhanced durability of HSSCC can be in large part attributed to the collective microstructural refinement that has been reached by the use of composite SCMs. Also, the chemical exposure tests (HCl, H₂SO₄, MgSO₄, and NaCl) demonstrated that the weight loss and strength retention were lower in the mixes prepared with SCM, which is why they can be used in the construction of structures exposed to severe conditions, including coastal, marine, and industrial environments. It was demonstrated in recent experimental studies that nano-silica and GGBFS enhance microstructural densification in SCC systems [18,19] and reviews underscore the importance of binder chemistry and refinement of pores on long-term durability [20].

4.5 Water Absorption Test

High quality concrete must have low permeability. To test HSSCC, a water absorption test was conducted on a cylindrical disc with a thickness of 50 mm and a diameter of 100.00 mm and as per ASTM C 140. Results are shown in Table 9. The absorption of water is monitored according to the ASTM C 140 requirements. The average water absorption should not be absorbed by the tested samples at an average of more than 5%. Besides this, it should be possible in no single specimen to achieve an absorption of not more than 7 percent of water. All the tested samples met these requirements implying that the rates of water absorption are acceptable. The findings indicate that

the concrete is high quality and hardy in addition to being of low permeability. The HSSCC samples also satisfy the standard requirements of the ASTM C 140 and as such, there is an overall water absorption of less than 5% and no single sample is higher than 7%.

Table 9. Water absorption values of HSSCC

Mix ID	Water Absorption (%)
CSC-60	2.84
SCF60-20	2.31
SCS60-10	1.46
SCG60-15	1.96
CSC-70	2.78
SCF70-20	2.52
SCS70-10	1.78
SCG70-15	2.03

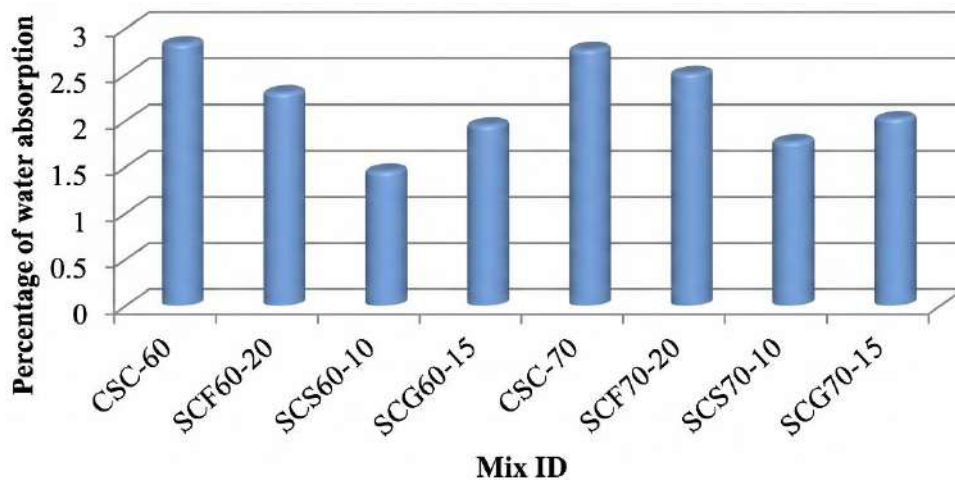


Fig. 8. Water absorption results of HSSCC

5. Limitations and Future Work

Although the findings are promising, certain limitations should be acknowledged:

- Freeze-thaw resistance was not evaluated, which is critical for cold-weather applications.
- Long-term field validation under real service conditions was not conducted; results are based on laboratory-scale exposure tests.
- Dynamic and cyclic loading effects on durability were not studied.

Future work should address these limitations by:

- Conducting freeze-thaw cycle testing in accordance with ASTM C666.
- Implementing field trials in marine and industrial environments.
- Evaluating performance under fatigue, seismic, or impact loading to simulate real structural demands.

6. Conclusions

This study shows that the use of silica fume, fly ash and GGBFS in HSSCC, has a great influence on the durability performance of the material under different stressors of the environment. The results reveal:

- Acid, sulphate and chloride attack tests observations and analysis: The improvement on the concrete quality with silica fume effect has been experienced during the tests of the concrete specimen submitted to the attack of acid, sulphate or chloride over the 90 days. It is full of

silica fume and calcium and it fills the pores in the cement matrix. This concrete building becomes denser and becomes more resistant to environmental pollutants.

- Compressive Strength Results

For M60 Grade:

- Acid Attack: 54.6 MPa (19.82% reduction from normal curing strength)
- Chloride Attack: 61.8 MPa (9.25% reduction from normal curing strength)
- Sulphate Attack: 58.2 MPa (14.54% reduction from normal curing strength)

For M70 Grade:

- Acid Attack: 61.5 MPa (21.56% reduction from normal curing strength)
- Chloride Attack: 70.6 MPa (9.95% reduction from normal curing strength)
- Sulphate Attack: 66.6 MPa (15.05% reduction from normal curing strength)

These values imply that, although reduced, the concrete specimens remain pretty strong, subject to good resilience properties under different environmental conditions. Better performance against any type of exposure is therefore indicated for the lower reduction percentages for chloride attack.

- Effect of Silica fume influences the voids in the concrete have been reduced by the presence of silica fume significantly. The reduction in porosity yields concrete specimens that are more resistant to adverse environmental conditions, and to chemical attack in particular, than other types of concrete specimens.
- Concrete specimens with silica fume and GGBFS mixtures contain very low chloride ion penetration. This performance is better than that of conventional concrete and concrete with fly ash, which demonstrates that the pores of silica fume and GGBFS specimens have been completely filled and improved performance of durability.
- The water absorption test results show that all the specimens conform to ASTM C 140, in water absorption percentage in the specified limits. This result further shows that concrete specimens, especially the silica fume reinforced concrete, has a much lower porosity & with a high quality.
- The incorporation of silica fume in the concrete mix improves density and decreases porosity and thus gives enhanced resistance to acid, sulphate and chloride attacks. Even under these conditions there are some reductions in compressive strength of concrete but overall, the durability of concrete is still high. This further corroborates the use of silica fume & GGBFS to produce high performance concrete as the chloride ion penetration is low & the water absorption standards also follow.

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Fresh and mechanical behavior of highly flowable concrete incorporating eggshell powder as partial cement replacement

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Article Info	Abstract
<p>Article History:</p> <p>Received 09 Oct 2025</p> <p>Accepted 11 May 2026</p> <p>Keywords:</p> <p>Highly flowable concrete;</p> <p>Eggshell powder;</p> <p>Fresh properties;</p> <p>Mechanical properties;</p> <p>Failure mode</p>	<p>This study explores the fresh and mechanical properties of Highly Flowable Concrete (HFC), that utilizes eggshell powder (ESP) as a cement substitute. HFC eliminates mechanical vibration during placement, minimizing segregation and honeycombs while improving construction quality. Considering the increased interest in sustainable alternatives, eggshell powder's calcium content makes it a viable substitute for cement. Thus, laboratory tests were conducted on HFC specimens with eggshell powder substitutions ranging from 5% to 20% by cement weight. The mixture's fresh properties were assessed using slump flow, T500, sieve segregation, and L-box tests. Mechanical performance was examined using compressive and split-tensile strength tests, with specimens tested to failure after 7 and 28 days of curing, respectively. The failure mode of the test specimens was also investigated. The experimental results show that all HFC mixtures containing eggshell powder met the EFNARC fresh properties requirements. Increasing percentages of eggshell powder replacement resulted in improved flowability, reduced segregation, and increased early age strength. The optimal replacement level was found to be 15%. This study provides vital insights into the potential of ESP in improving concrete performance while addressing environmental concerns associated with cement production and paving the way for more sustainable practices in concrete construction.</p>

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1. Introduction

The construction industry is expanding rapidly worldwide, especially in Malaysia, where concrete is extensively utilized. However, the usage of conventional concrete, primarily composed of Ordinary Portland Cement (OPC), results in substantial construction costs and environmental degradation owing to high CO₂ emissions from the production of cement [1-3]. In order to alleviate these concerns, self-compacting concrete (SCC), also known as highly flowable concrete (HFC) has been developed as an alternative approach. HFC improves flowability, resists segregation and honeycombing, and minimizes the requirement for vibration during installation, which renders it beneficial for complex formworks and congested reinforcement areas [4].

To improve the performance and sustainability of HFC, researchers investigated the utilization of supplementary cementitious materials (SCMs) generated from industrial and agricultural waste. The incorporation of waste materials such as fly ash (FA), wastepaper sludge ash (WPSA), silica fume (SF), palm oil fuel ash (POFA), rice husk ash (RHA), and ground granulated blast furnace slag improves the workability of HFC while lowering the cement content [5-9]. These supplementary cementitious materials (SCMs) not only reduce the amount needed for OPC, but also boost particle packing, reduce concrete permeability, and enhance concrete durability [7].

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In accordance with this sustainable approach, the current study looks into the utilization of green cementitious materials, particularly eggshell powder, as partial substitutes for OPC in HFC. [10] indicated that GGBS can be employed as a partial replacement for naturally mined limestone while improving durability and speed up the concrete setting process. Similarly, because of the high content of calcium oxide (CaO), eggshell powder (ESP), although being an agricultural waste product, has emerged as a promising material due to its pozzolanic properties and can be utilized as supplementary cementitious materials [5,11]. Eggshell powder (ESP), produced from waste material, could improve the fresh and mechanical properties of concrete while lowering reliance on OPC.

The performance of ESP-modified HFC is equivalent to that of SCC that incorporates various environmentally friendly supplementary cementitious materials such as fly ash, ground granulated blast furnace slag, rice husk ash, palm oil fuel ash, wastepaper sludge ash, and silica fume [5, 6, 8, 10]. Similar enhancements in flow stability, viscosity modification, and mechanical response were observed across these SCM systems, demonstrating that ESP is competitive and has significant promise as a feasible substitute material in ecologically friendly concrete production. Furthermore, when incorporated into HFC, ESP has the potential to boost the fresh and hardened properties, aligning with the aims of sustainable and green construction practices. Incorporating waste materials as cement substitutes not only enhances concrete properties but also aids with the disposal of solid waste.

A significant aspect of the research is assessing the fresh properties of HFCs incorporating ESP. These properties such as T500, slump flow, segregation resistance and L-box were critical indicators of the concrete's fresh performance. The inclusion of ESP as a cement replacement alters the flow and workability of the concrete mixture. Understanding the unique properties of HFC which includes ESP, such as flowability, viscosity, and segregation resistance, is crucial for optimizing its performance and ensuring its appropriateness for construction applications [5,12].

Aside from fresh characteristics, the mechanical performance of HFC with ESP was explored. This includes assessing its compressive and tensile strength at various curing ages, as well as its durability and resistance to cracking. The experimental investigations conducted by [6] and [13] on other waste-based SCMs emphasizes the necessity of such assessments in establishing the practical usability of alternative materials for structural concrete. Extensive testing for mechanical properties can quantify the potential benefits of using ESP as a partial cement replacement, offering useful insights for designing and implementing sustainable and cost-effective concrete structures.

To shed light on the scope of this investigation, this study focuses on the primary rheological and mechanical properties proposed by EFNARC for identifying Highly Flowable Concrete (HFC). These consist of slump flow, T500, passing ability, and segregation resistance, as well as compressive and splitting tensile strength tests, which are considered as significant indicators of hardened performance. The research conducted stands out in the following aspects: (i) it uses an integrated rheological-mechanical framework for assessing eggshell powder (ESP)-modified HFC, providing a comprehensive understanding of material behaviour; (ii) it develops a predictive regression model for splitting tensile strength based on experimental data; and (iii) it determines optimal ESP replacement levels that provide a balance between workability and mechanical performance. Thus, the findings of this research could have significant implications for the construction industry, particularly in terms of cost reduction and sustainable practices. Through utilizing waste materials such as eggshells, the research aims to promote resource efficiency and waste management while improving concrete performance. This approach promotes resource efficiency and cost savings, aligning with global initiatives to decarbonize the construction industry.

2. Methods

The laboratory study focused on the fresh properties of highly flowable concrete (HFC) with various constituent ingredients. The slump-flow test, carried out in line with [14] was used to assess the flow characteristics and setting behaviour of HFC. The passing ability was assessed utilizing the L-Box test in accordance with [15], whereas segregation resistance was measured using [16]. Compressive strength testing for the hardened state was conducted on 100 × 100 mm

cube specimens following BS EN 12390-1:2012 and BS EN 12390-3:2012. Splitting tensile strength tests were performed on 50×100 mm cylindrical specimens with a length-to-diameter ratio of 2.0, using the procedure outlined in BS EN 12390-6: 2009. These mechanical tests were carried out at curing ages of 7, 28, and 90 days to evaluate the short- and long-term strength development of HFC, as discussed in the subsequent topics.

2.1 Materials and Mixture Proportions

The materials that were employed consisted Ordinary Portland Cement (OPC) conforming to Malaysian Standard MS 522, water, natural fine aggregate (≤ 5 mm), natural coarse aggregate (5–10 mm), eggshell powder (ESP) from discarded eggshells, and SP Master Glenium 8784 superplasticizer, as shown in Fig. 1. ESP was collected from a variety of sources, including local street suppliers, food processing industries, and cafeterias. The obtained eggshells were properly cleansed with tap water to eliminate any organic membranes or contaminants. The eggshell samples were oven-dried at 105 ± 5 °C for 24 hours to remove moisture content. After drying, the eggshells were manually crushed and then ground with a mechanical grinder to produce a fine powder. The ground material was sieved to ensure homogeneous particle size distribution for cement replacement applications. The resulting eggshell powder (ESP) has a fineness similar to cement, indicating its viability as a micro-filler material. It was ground and sieve to a fine powder (Fig. 2) and utilize as a partial cement replacement at 0%, 5%, 10%, 15%, and 20% by weight. The chemical compositions of OPC and ESP, evaluated via X-ray fluorescence (XRF), are displayed in Table 1.

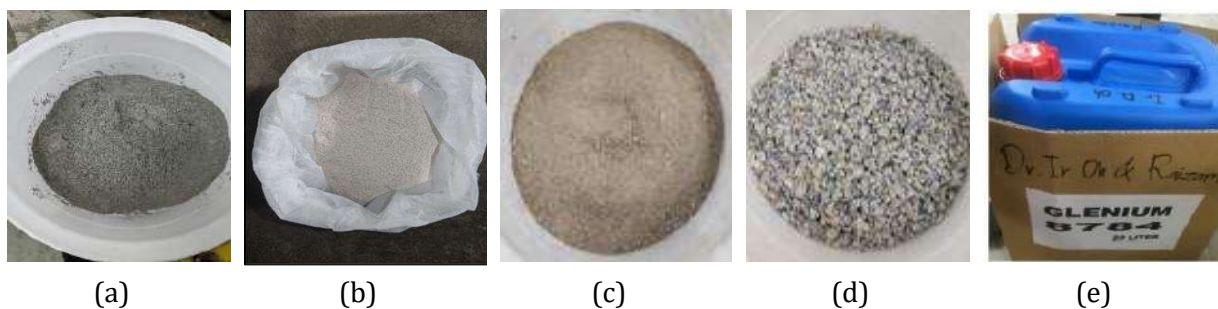


Fig. 1. HFC constituent materials: (a) cement, (b) grinded ESP, (c) fine aggregates (sand), (d) coarse aggregates, and (e) superplasticizer (Master Glenium 8784)



Fig. 2. Mechanical grinder used for processing eggshells into fine powder (ESP)

It is essential to note that the ESP used in this study was not subjected to calcination. The raw eggshells maintained their original calcium carbonate (CaCO_3) form during processing, and the CaO value reported in the XRF results represents the oxide-equivalent convention of XRF analysis, not actual decomposition. Thus, the value “CaO = 98.8%” in Table 1 corresponds to calcium carbonate content reported as CaO equivalents. A footnote was added to Table 1 to clarify this convention. The current experimental investigations sought to assess the beneficial effects of non-calcined ESP

as a sustainable micro filler in HFC. Furthermore, calcination produces free CaO, which can cause volumetric instability and increase the possibility of expansion-related microcracking. CaCO₃-based ESP provides stable workability and controlled hydration behaviour, both important in HFC systems. In addition, eliminating high-temperature processing improves economic feasibility, industrial scalability, and carbon efficiency, which aligns with the study's sustainability aims.

Table 1. Chemical composition of ESP and OPC (% by weight)

Compound	ESP (%)	OPC (%)
MgO	< LOD	2.021
Al ₂ O ₃	< LOD	3.7462
SiO ₂	< LOD	21.8349
P ₂ O ₅	0.7387	< LOD
SO ₃	< LOD	3.5414
K ₂ O	< LOD	0.2467
CaO	98.8053	64.4872
TiO ₂	0.0196	0.197
MnO	< LOD	0.0787
Fe ₂ O ₃	< LOD	3.8159

*Note: LOD denotes the limit of detection CaO represents the oxide-equivalent convention of XRF analysis

Meanwhile, tap water was utilized as the mixing medium, with a superplasticizer added to improve workability. Fine aggregate filled voids, whereas coarse aggregate ensured structural stability. The standard sieve analysis method was employed for assessing the particle size distribution (PSD) of aggregates, that involved measuring the amount of material retained on a series of sieves with varied apertures. The proportion retained on each sieve was then determined, and the cumulative distribution was plotted resulting in PSD curves for fine aggregate (FA) and coarse aggregate (CA), as shown in Figure 3. The findings revealed that the fine aggregates had a well-graded distribution within the standard limits, while the coarse aggregates had a relatively consistent grading pattern, indicating their appropriateness for use in SCC mixtures.

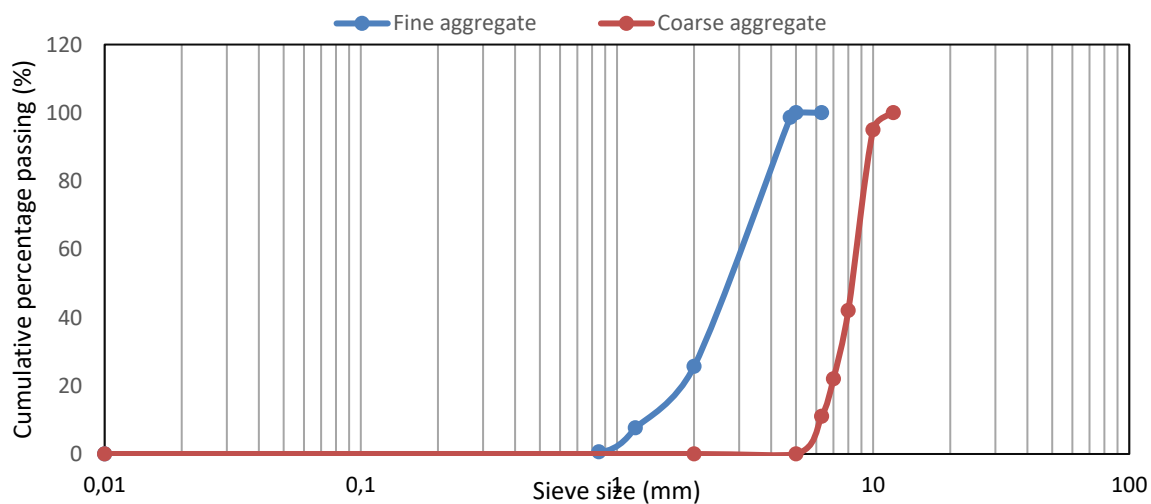


Fig. 3. Particle size distribution of fine and coarse aggregates

The mixtures were blended together to determine the effect of eggshell powder (ESP) on the fresh and mechanical performance of highly flowable concrete (HFC), as well as its suitability as a sustainable construction material. The mix proportions are indicated in Table 2. Thirty specimens had been produced for testing to determine the desired fresh and hardened properties.

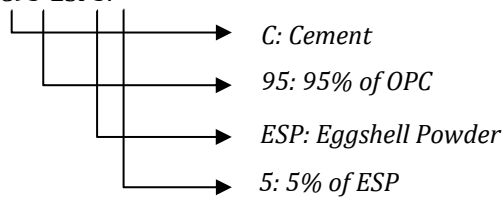
The process of mixing highly flowable concrete (HFC) with eggshell powder (ESP) as a partial substitute for Ordinary Portland Cement (OPC) started with the adding of fine and coarse aggregates to the mixing container (Fig. 4a). ESP and OPC were then carefully inserted and mixed with the aggregates to achieve a balanced distribution. The superplasticizer (SP) and water were

subsequently added, and mixing proceeded until a homogenous consistency was attained (Fig. 4b). The fresh concrete was immediately put to the required fresh properties tests (slump flow, T500, L-box and sieve segregation). Following completion of these tests, the concrete was cast into the designated moulds (Fig. 4c) and demoulded 24 hours later for water curing (Fig. 5a and 5b).

Table 2. Mix design for HFC incorporating 0-20% ESP as a partial replacement for OPC

Mixture ID	OPC (kg/m ³)	ESP (%)	ESP (kg/m ³)	Coarse Aggregate (kg/m ³)	Fine Aggregate (kg/m ³)	Water (litre/ m ³)	Super plasticizer (litre/kg/m ³)
C100	480	NA	0	760	890	215	9.6
C95-ESP5	456	5	24	760	890	215	9.6
C90-ESP10	432	10	48	760	890	215	9.6
C85-ESP15	408	15	72	760	890	215	9.6
C80-ESP20	384	20	96	760	890	215	9.6

*Note: C95-ESP5:



(a)



(b)



(c)

Fig. 4. Mixing and casting process for HFC: (a) addition and blending of materials in the mixer; (b) homogeneous HFC mix; (c) freshly cast HFC specimens in moulds



(a)



(b)

Fig. 5. HFC specimens: (a) after demoulding at 24 hours; (b) undergoing water curing.

All mixing, casting, curing, and testing were performed under controlled laboratory conditions (23 ± 2 °C and 60-70% relative humidity). These controlled environmental settings assured that the fresh and hardened performance of the HFC mixtures was primarily determined by material

composition rather than external changes, thus improving the experimental results' reliability and uniformity. Besides that, to make sure methodological consistency and traceability, every phase of experimentation adhered entirely to internationally recognized standards, BS EN 12350 for fresh properties [14, 15, 16], BS EN 12390-3 for compressive strength [17], and ASTM C496 for splitting tensile strength [18], which aligning the testing procedure with worldwide concrete evaluation practices. In addition, each mechanical properties test was carried out on three replicate specimens per mix, and all values provided include the mean and standard deviation to demonstrate statistical confidence and repeatability.

Based on this validated testing framework, the current study's experimental scope was specifically restricted to first-phase essential assessments in order to gain a controlled comprehension of ESP's direct influence on early-age rheology and strength advancement. Other characterization methods such as U-box flow assessment, V-funnel viscosity assessment, flexural strength, UPV, water absorption, and durability indicators are designed for a subsequent extended study as part of the HFC-ESP system evaluation.

2.2 Fresh Properties of the HFC Mixtures

The fresh properties of HFC, specifically flowability, passing ability, and segregation resistance, were assessed in compliance with the EFNARC guidelines using the standardized approaches outlined in [14], [15], and [16].

2.2.1 Slump Flow and T500 Test

Flowability and flow rate were assessed using the slump flow test and T_{500} measurement as stated in BS EN 12350-8:2010 [14]. The slump flow test assesses HFC's ability to spread under its own weight, whereas T_{500} examines the time needed for the spread to achieve a 500 mm diameter in the presence of obstacles. The set-up had a 900 × 900 mm base plate and a typical slump cone, as shown in Fig. 6. After mixing, the clean cone was placed in the center of the base plate and filled with fresh concrete. Any spills were cleared prior to testing. The cone was then raised vertically, and the stopwatch was started when it lost touch with the base plate. The T_{500} time was recorded once the concrete spread reached a diameter of 500 mm. The final slump flow diameter was measured in two perpendicular directions (d_1 and d_2), and the average value was estimated as given in Equation (1).

$$D = \frac{d_1 + d_2}{2} \quad (1)$$



Fig. 6. Equipment and measurement procedure for the slump flow and T500 test: (a) Experiment set-up; (b) diameter measurement (d_1 and d_2)

2.2.2 L-Box test

The L-Box test, as specified in BS EN 12350-10:2010 [15], was used to determine passing ability. This approach analyzes HFC's ability to flow into restricted spaces such as reinforcement gaps,

without segregation. The apparatus is L-shaped, with a vertical part (inlet) coupled to a horizontal section that simulates restricted flow (Fig. 7). Fresh HFC was introduced into the vertical part, and the gate was lowered to enable free flow under gravity (Fig. 8a). Concrete heights were calculated at the end of the horizontal segment (H_2) and vertical part (H_1) (Figs. 8b and 8c). The passing ability (PA) was evaluated using Equation (2) as follows:

$$PA = \frac{H_2}{H_1} \quad (2)$$

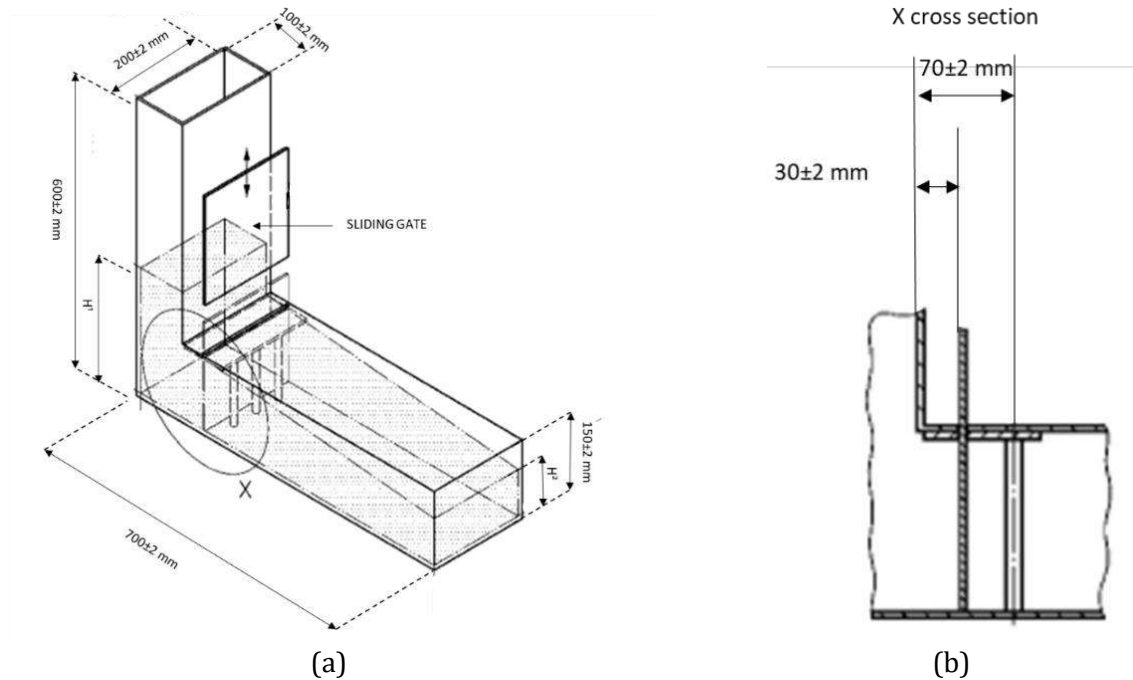


Fig. 7. Illustration of the L-Box test: (a) full dimensions and equipment used, (b) cross-section of the L-Box. [15]

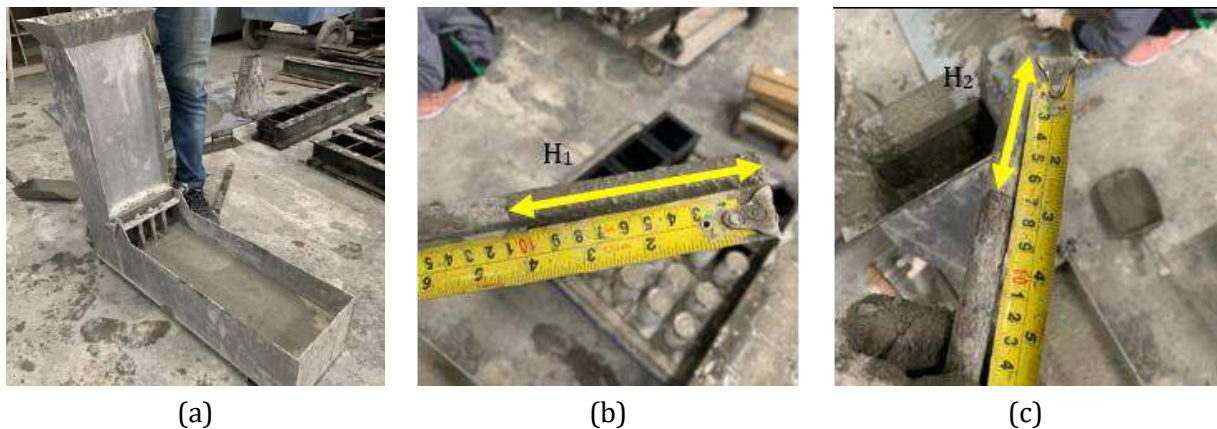


Fig. 8. L-Box test: (a) full view of the L-Box; (b) H_1 measurement; (c) H_2 measurement

2.2.3 Sieve Segregation Test

Segregation resistance was measured in line with BS EN 12350-11:2010 [16], used to examine the resistance of HFC to segregation. The initial mass of fresh concrete (m_c) was measured before it was placed on a 5 mm sieve over a sieve receiver (Fig. 9). After two minutes, the sieve was carefully removed upwards, and the combined mass of the receiver and passing material (m_{ps}) was measured. The mass of the sieve receiver (m_p) was also recorded. The segregation ratio (SR) was computed as given in Equation (3):

$$SR = \frac{(m_{ps} - m_p) \times 100}{m_c} \quad (3)$$

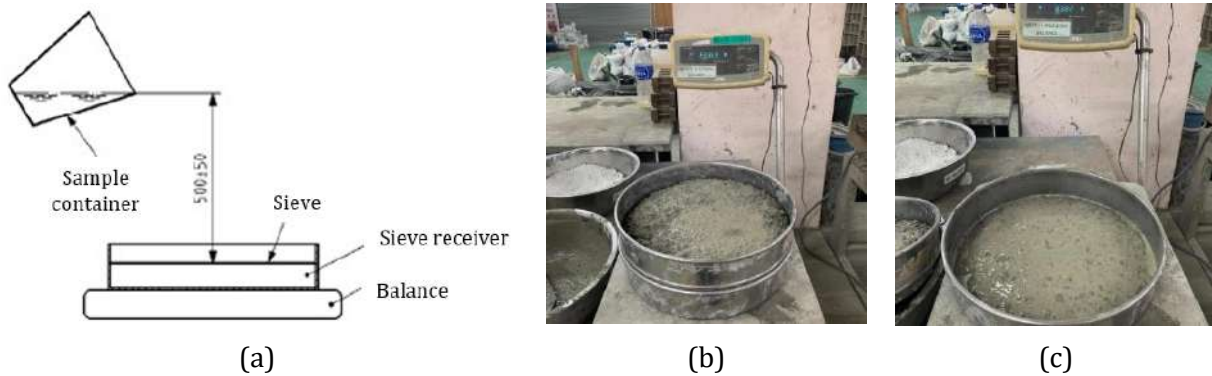


Fig. 9. Sieve segregation test: (a) experimental set-up [16]; (b) fresh concrete poured onto the sieve; (c) weighing the mass of the sieve receiver with the passed concrete

2.3 Hardened Properties of The HFC Specimens

Compressive and split tensile strength tests were performed to assess the hardened properties of HFC. The compressive strength test was carried out in line with [17], a basic approach for measuring the ultimate load capacity and structural integrity of concrete samples, as shown in Fig. 10. Under compression test, this study utilized 100 mm × 100 mm cube specimens that were water-cured for 7, 28 and 90 days. Testing was conducted using a 1000 kN compression testing equipment, which applied an axial compressive load until the specimen failed. The optimal blend was chosen by considering the highest compressive strength obtained from all HFC specimens.

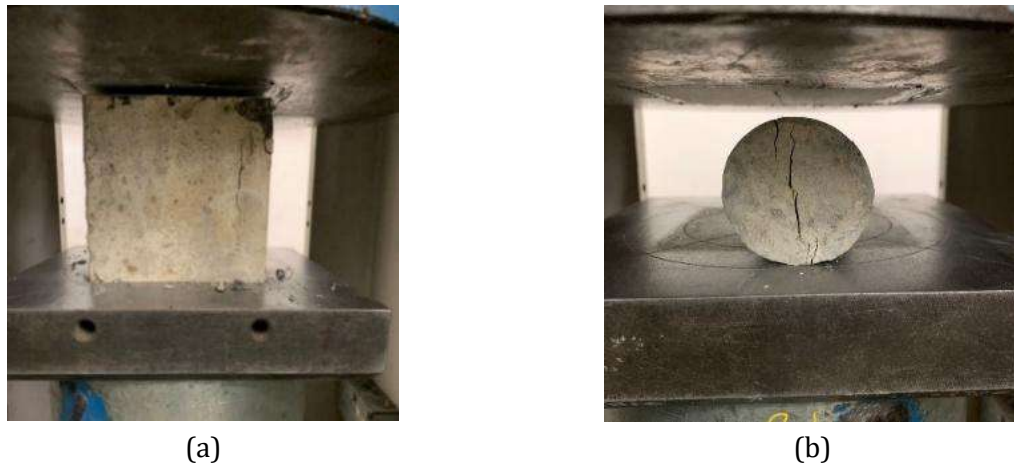


Fig. 10. Experimental setup of HFC specimens under compressive loading and split tensile strength: (a) cubic specimen; (b) cylindrical specimen

Meanwhile, the split tensile strength test was carried out in line with [18] to determine the concrete's tensile resistance. Cylindrical specimens with dimensions of 100 mm in height and 50 mm in diameter were produced and water-cured for 7, 28 and 90 days. Each specimen was placed horizontally between two loading platens, and a compressive force was exerted across its vertical diameter until failure occurred. This generated tensile strains in the specimen's cross-section, causing it to break. The splitting tensile strength (T_s) was estimated as shown in Equation (4) after calculating the maximum load at failure.

$$T_s = \frac{2P}{\pi DL} \quad (4)$$

In which, T_s denotes the splitting tensile strength in kPa, P embodies the maximum applied load indicated by the testing machine in kN, D as diameter of the sample and L represents the length of the specimen in m.

3. Results and Discussion

Considering that additional fresh, hardened, microstructure and durability-related tests such as U-box, and V-funnel, flexural capacity, SEM, EDX, UPV, sorptivity and chloride penetration, are needed for a more in-depth structural evaluation, the present findings are a crucial initial phase in offering a foundational understanding of ESP's influence on the flowability, viscosity, stability, and mechanical behavior of HFC. Identifying this baseline performance requires attention before proceeding to more extensive performance-based assessments. In line with this aim, the current study conducts a systematic investigation into the fresh properties of HFC mixes, the hardened properties of the resulting specimens, the correlation between compressive and splitting tensile strengths in comparison to previous studies, and the typical modes of failure. All of these aspects are further explained in detail within the subsequent sub-sections.

3.1 Fresh Properties of The HFC Mixes

Table 3 exhibits the experimental findings for highly flowable concrete (HFC) rheological properties tests such as slump flow, T500, L-Box, and sieve segregation.

Table 3. Results of the rheological properties of HFC containing ESP

Sample	Slump flow (mm)	T500 (sec)	L-Box, PA	Sieve Segregation, SR%
C100	626	7	0.90	10.22
C95-ESP5	610	8	0.89	8.82
C90-ESP10	580	11	0.83	6.74
C85-ESP15	563	13	0.81	5.28
C80-ESP20	680	6	0.42	15.21
EFNARC, 2005 [19]	550-650 (SF1)	<2 (VS1)	≥ 0.80 with 2 rebars (PA1)	≤ 20 (SR1)
	660-750 (SF2)	≥ 2 (VS2)	≥ 0.80 with 3 rebars (PA2)	≤ 15 (SR2)
	760-850 (SF3)			

Table 3 depicts the five investigated mixtures (C100, C95-ESP5, C90-ESP10, C85-ESP15, and C80-ESP20), with four of them (C100, C95-ESP5, C90-ESP10, and C85-ESP15) falling into the SF1 category based on the [19] (slump flow). C80-ESP20, on the other hand, is classified as SF2 since its slump flow value falls between 660 and 750 mm. These categories relate to diverse uses, with SF1 being advised for unreinforced or minimally reinforced slabs cast from the top, and SF2 better suited for vertical structural elements that include walls and columns. Fig. 11 shows a minor decline in flowability with an increase in ESP substitution up to 15%, followed by a significant improvement when 20% ESP was added. The comparable T500 measurements (Fig. 12) reveal that at 20% ESP, the slump flow time increased significantly from 7 to 13 s before decreasing to 6 s. This trend indicates the influence of ESP on the rheological behavior of the SCC, where gradual replacement in the range of 5-15% results in increasing viscosity, but over this threshold (20%) the mix exhibits increased fluidity. These experimental findings are consistent with previous studies [11,20,21,22], which have consistently shown that the inclusion of ESP modifies the workability of HFC, resulting in comparable flow characteristics. The agreement across various tests supports the evidence that ESP is a promising supplementary material for HFC manufacturing, with flow performance suitable for a wide range of structural applications.

The addition of ESP to the HFC mix has a considerable influence on segregation resistance, as shown by the sieve segregation outcomes (Fig. 13). Segregation was reduced when ESP content increased up to 15%, but segregation resistance fell substantially at 20% replacement. EFNARC classifications divide segregation ratios (SR) into two categories, which are SR1 and SR2. Fig. 13 demonstrates that mixes C100, C95-ESP5, C90-ESP10, and C85-ESP15 fall under the SR2

classification. This classification is deemed appropriate for vertical elements with flow distances higher than 5 m and confinement gaps greater than 80 mm, in which increased segregation resistance is necessary to ensure homogeneity throughout placement. Meanwhile, C80-ESP20 has been designated as SR1, suitable for thin slabs and vertical uses with lower flow distances (< 5 m) and comparable confinement circumstances. These categories emphasize the significance of properly choosing ESP content to balance flowability and segregation resistance in various structural applications. Increasing ESP amounts by up to 15% enables improved resistance to segregation, improving the reliability and quality of SCC in challenging placement situations. [23] published similar findings, showing that mixes containing 5%, 10%, and 15% ESP consistently attained SR2 categorization.

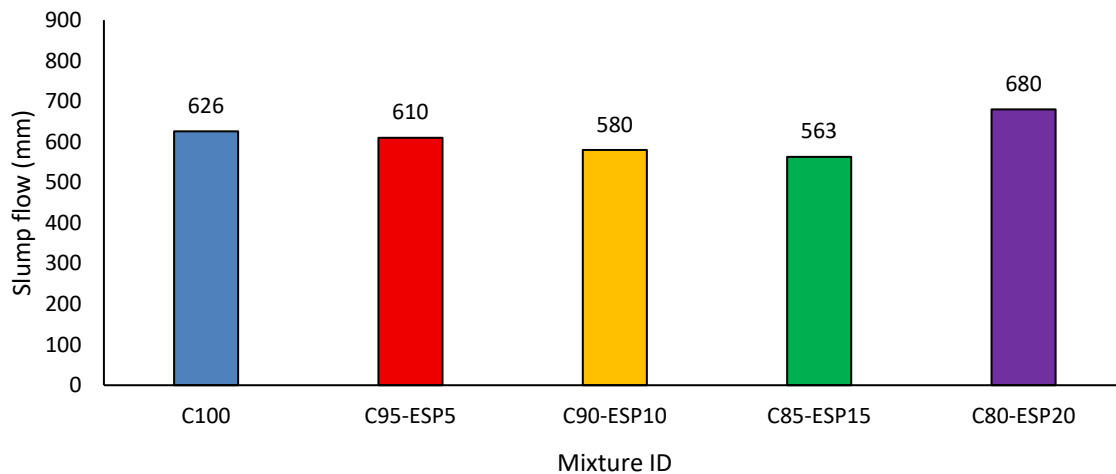


Fig. 11. Slump flow experimental results of HFC mixes

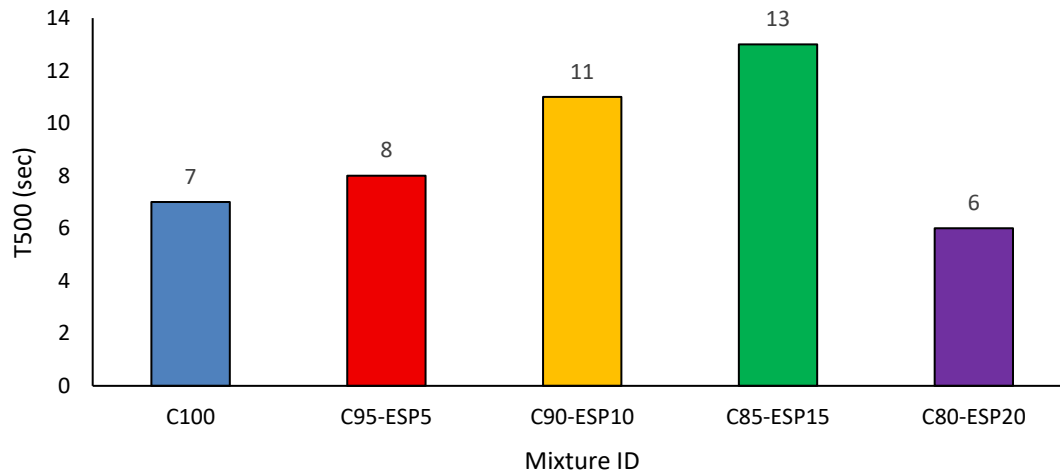


Fig. 12. T500 experimental results of HFC mixes

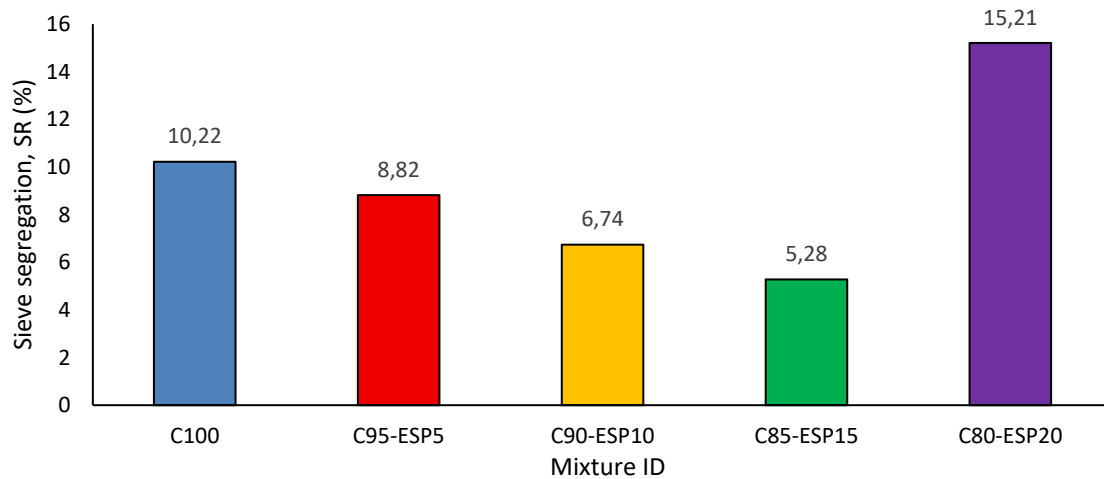


Fig. 13. Sieve segregation test results of HFC mixes incorporating varying ESP contents

The study's results indicates that partial ESP substitution improves segregation resistance in HFC, especially for vertical applications with longer flow distances and wider confinement gaps. In addition, these findings provide strong evidence that ESP is a feasible supplemental material to generate HFC with improved segregation control in a variety of practical settings.

On the other hand, Fig. 14 shows a gradual decrease in the passing ability (PA) ratio as ESP replacement increases in 5% increments. The PA ratio decreased from 0.90 to 0.42, demonstrating that the ability of HFC mixtures to flow through confined spaces becomes increasingly constrained as ESP level increases. All mixes were evaluated with three reinforcing bars as obstacles. Despite only the mix of C80-ESP20, all mixes' PA ratios remained over 0.80, meeting the PA2 categorization based on EFNARC criteria. This classification indicates mixes suited for complicated structural elements with reinforcing gaps of less than 60 mm. However, it should be noted that full-scale mock-up tests might need to be performed to verify what was found under actual construction settings.

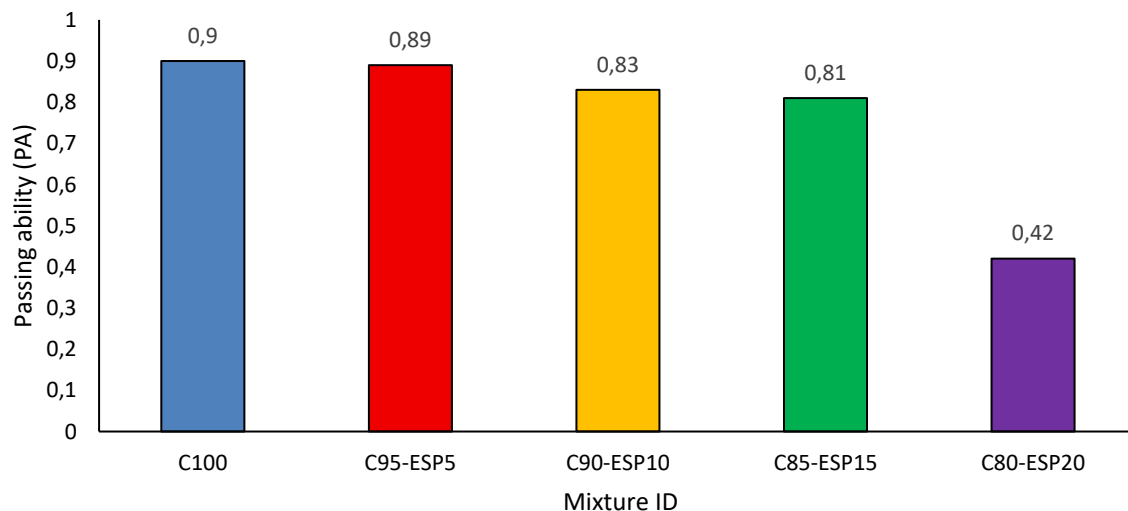


Fig. 14. L-box (passing ability) test results of HFC mixes incorporating varying ESP

The trend seen in this study (Fig. 14) is consistent with earlier research. [7], [11] and [22] all reported similar results, with HFC blends encompassing ESP showing PA2 classification with decreasing PA ratios. Additionally, [24] found a deviation to this general pattern, emphasizing the potential impact of combination proportions and testing conditions on passing ability outcomes. The observed decrease in PA ratio with increasing ESP concentration demonstrates the sensitivity of HFC rheology to fine supplementary materials. Greater ESP replacement limits the mix's ability

to handle congested reinforcement zones, which affects flowability and passing performance. Thus, the results obtained give compelling evidence that maximizing ESP concentration is critical for balancing flow and passage properties in highly flowable concrete, ensuring its use in complicated reinforced structural systems.

3.2 Hardened Properties of The HFC Specimens

All HFC specimens were loaded progressively until failure occurred. Table 4 displays the compression test results for the HFC specimens, with average values calculated from the three specimens. The strength progression pattern presented in Table 4 clearly indicates the effect of ESP concentration on HFC behavior at both early age and long term. At 7 days, the control mix (C100) had an average compressive strength of 29.23 MPa, whereas adding ESP at 5%, 10%, and 15% resulted in increases of 33.83 MPa, 45.90 MPa, and 46.47 MPa, respectively. The substantial improvement at 10-15% replacement suggests that finely ground ESP's micro-filler effect enhanced particle packing and reduced internal voids, promoting early hydration and matrix densification. Nevertheless, at 20% replacement (C80-ESP20), the 7-day compressive strength dropped to 38.93 MPa, implying that excessive cement dilution starts to outweigh its beneficial filler effect.

Table 4. Comparative compressive strength (CS) test results of HFC specimens (7 and 28 days)

Mix ID	7 days			28 days			90 days		
	CS (N/mm ²)	Mean (N/mm ²)	STD	CS (N/mm ²)	Mean (N/mm ²)	STD	CS (N/mm ²)	Mean (N/mm ²)	STD
C100	27.66	29.23	1.36	37.88	38.85	0.86	80.13	77.75	2.212
	29.89			39.16			75.76		
	30.13			39.51			77.35		
C95-ESP5	32.90	33.83	0.81	48.44	49.55	1.43	55.65	54.08	2.338
	34.38			49.05			55.19		
	34.22			51.16			51.39		
C90-ESP10	45.03	45.90	0.77	53.54	55.53	1.75	47.11	49.75	2.288
	46.16			56.21			50.98		
	46.51			56.84			51.16		
C85-ESP15	44.48	46.47	2.42	56.41	58.47	1.82	38.74	43.58	4.193
	45.78			59.86			45.81		
	49.16			59.13			46.18		
C80-ESP20	39.29	38.93	2.29	49.52	49.71	0.86	42.54	43.93	2.722
	41.02			50.64			47.07		
	36.48			48.96			42.19		

A comparable pattern of compressive strength can also be seen after 28 days. The control mix attained 38.85 MPa, while C95-ESP5, C90-ESP10, and C85-ESP15 recorded 49.55, 55.53, and 58.47 MPa, respectively. The maximum strength performance at 15% ESP demonstrates that, at modest replacement levels, ESP improves microstructural compactness and load transfer efficiency inside the hardened HFC matrix. The decrease in the amount at 20% ESP (49.71 MPa) contributes support to the concept that beyond an optimal threshold, clinker content dilution restricts the creation of primary hydration products (C-S-H), thus lowering the progression of strength.

The 90-day findings offer a better understanding of long-term hydration behavior and matrix stability. The control mix's strength increased significantly to 77.75 MPa, demonstrating that cement particles hydrated continuously throughout time. Additionally, the 90-day strength of HFC specimens incorporating 5% and 10% ESP (54.08 MPa and 49.75 MPa, respectively) did not exceed the control, and higher replacement levels resulted in a significant reduction (43.58 MPa for 15% and 43.93 MPa for 20%). This behavior indicates that, while ESP improves early- and medium-age strength through physical densification, its contribution to long-term strength is primarily due to the filler effect rather than a strong pozzolanic reaction. ESP is primarily CaCO₃-based, compared

to supplementary cementitious ingredients like GGBS or silica-rich pozzolans, which increase long-term strength via secondary C-S-H production. As a result, its long-term contribution is mostly dependent on microstructural refinement and nucleation effects, instead of ongoing chemical response. Higher replacement levels ($\geq 15\%$) cause more dilution over time, resulting in less clinker available for hydration and lower 90-day strengths.

The results of these studies suggest that the ideal ESP replacement level for balancing early-age enhancement and long-term performance is between 10-15%. At this level, enhanced particle packing and ITZ refinement allow for partial cement reduction without affecting long-term hydration potential. Yet, substitute levels exceeding 15% could adversely impact long-term hardened performance owing to inadequate cementitious binder concentration. These findings emphasize ESP's potential as a supplementary cementitious material to improve the mechanical properties of HFC. Previous studies by [7,11,20, 21,22,23] and [25] have all reported consistent results, particularly in terms of strength development at early (7-day) and standard (28-day) curing periods.

The presence of 90-day data from this study thus offers an extensive understanding of ESP-modified HFC behavior, demonstrating that ESP acts mainly as a micro-filler and hydration accelerator instead of a highly reactive pozzolan. The subsequent research will extend curing times to 56 days and beyond 90 days and add durability assessments (e.g., permeability, sorptivity, and microstructural analysis) in order to further clarify long-term performance mechanisms.

On the other hand, the consistently low standard deviations (STD) recorded for all mixtures show that the compressive strength data are resilient, indicating a good level of experimental precision and reproducibility. Furthermore, the strength growth of the ESP-modified mixtures is consistent with established behaviour patterns for SCC with CaO-dominant additional materials, as recorded in earlier research [10,11,23]. The congruence of the current findings with a wider body of literature offers external validation and strengthens confidence in the reliability of the observed mechanical benefits at moderate ESP substitute levels.

The split tensile strength (STS) test yields similar findings as compressive strength. This correlation was anticipated considering both tests utilize a similar type of concrete. The split tensile strength in Table 5 was computed using the empirical formula, which is given by Equation 4.

Table 5. Results of the split tensile strength of HFC

Mix	7 days			28 days			90 days		
	STS, F_{st} (N/mm ²)	Mean (N/mm ²)	STD	STS, F_{st} (N/mm ²)	Mean (N/mm ²)	STD	STS, F_{st} (N/mm ²)	Mean (N/mm ²)	STD
C100	2.44	2.53	0.10	3.09	3.15	0.06	4.38	4.31	0.29
	2.50			3.16			4.56		
	2.64			3.20			4.00		
C95- ESP5	2.75	2.85	0.11	3.55	3.69	0.13	2.25	2.65	0.45
	2.84			3.72			2.55		
	2.97			3.81			3.14		
C90- ESP10	3.03	3.08	0.04	3.74	3.97	0.22	3.49	3.53	0.33
	3.09			3.99			3.22		
	3.11			4.18			3.87		
C85- ESP15	3.48	3.65	0.17	3.82	4.07	0.22	3.31	3.31	0.10
	3.65			4.15			3.22		
	3.81			4.23			3.41		
C80- ESP20	3.11	2.99	0.30	3.96	3.68	0.30	3.19	3.62	0.60
	3.22			3.71			4.31		
	2.65			3.37			3.37		

Table 5 demonstrates the splitting tensile strength findings for the concrete specimens at various curing ages, offering information on their long-term mechanical characteristics. In general, all HFC

specimens containing ESP showed an enhancement in splitting tensile strength after 7 days of curing in contrast to the control mix (C100). C85-ESP15 had the maximum strength (3.65 MPa), followed by C90-ESP10 (3.08 MPa), and C95-ESP5 had the lowest (2.85 MPa). At 28 days, the performance trend was consistent. C85-ESP15 also had the highest splitting tensile strength (4.07 MPa), followed by C90-ESP10 (3.97 MPa). In contrast, C80-ESP20 and C95-ESP5 had substantially lower strengths of 3.68 MPa and 3.69 MPa. These results show that ESP addition had a substantial effect on the splitting tensile strength of the concrete specimens.

The increase in splitting-tensile strength can also be attributed to ESP's micro-filler effect, which refines pore structure, increases matrix cohesion, and leads to a more uniform stress distribution under splitting-tensile loading. The greater CaO concentration in ESP may potentially induce secondary hydration products within the ITZ, increasing the relationship between paste and aggregates which contributes to the higher splitting-tensile strengths measured at moderate ESP amounts. Comparisons across specimens illustrate that moderate ESP amounts, such as 10-15%, improve splitting tensile strength, typically exceeding the control mix and low ESP replacement (C95-ESP5). However, the results drop noticed in C95-ESP5 after 28 days implies that excessively reduced ESP inclusion may not be beneficial to tensile strength growth. Meanwhile, higher ESP amounts, such as 20%, led to a decline in strength, suggesting the presence of an optimal substitution threshold.

The 90-day findings reveal additional details about the long-term split-tensile performance of the ESP-modified HFC. The strength of the control mix (C100) increased steadily to 4.31 MPa, indicating continuous cement hydration and matrix densification. For HFC specimens, C90-ESP10 (3.53 MPa) and C80-ESP20 (3.62 MPa) exhibited comparable split-tensile strength at 90 days, while C85-ESP15 recorded a slight reduction to 3.31 MPa. Meanwhile, C95-ESP5 demonstrated a more noticeable decrease (2.65 MPa), indicating that lower ESP content may not be sufficient to sustain long-term tensile enhancement. The measured 90-day performance shows that, whereas ESP successfully improves early and intermediate split-tensile strength via micro-filler densification and increased ITZ cohesion, its long-term contribution is limited by cement availability and hydration continuity. As a result, at greater substitution levels, cement dilution diminishes long-term hydration products, potentially slowing split-tensile strength development when compared to the control mix. Nonetheless, the 10-15% replacement range exhibits rather steady long-term tensile performance, demonstrating that ESP integration does not degrade structural tensile resistance when utilized within an appropriate range.

In general, the findings show that adding ESP boosted splitting tensile strength up to an optimum level, beyond which its effectiveness gradually decreased. The observed trend highlights the occurrence of an optimal ESP range for maximizing tensile strength, which is consistent with earlier findings in compressive strength development. Furthermore, the correlation between compressive strength and splitting tensile strength was found to be similar across specimens, with higher compressive strength often resulting in higher splitting tensile strength. This correlation highlights the synergistic effect of ESP incorporation on the hardened performance of concrete specimens.

3.3 Experimental Observations and Literature-Based Comparisons on The Strength Correlation in HFC Specimens

There was a significant relationship between compressive strength and splitting tensile strength in the investigated HFC specimens. In order to estimate splitting tensile strength from compressive strength, numerous empirical equations formulated by previous studies were investigated and compared to the experimental findings gathered during this study. The inclusion of the splitting tensile test serves as crucial to ensuring a more thorough evaluation of the mechanical properties of HFC with ESP as a partial substitute for cement. Furthermore, using well-established prediction correlations from previous studies, the splitting tensile strength can be estimated without the need for significant laboratory testing.

In this study, a power-type relationship has been proposed using regression analysis to assess the correlation among compressive and splitting tensile strengths. Fig. 15 indicates that the splitting tensile strength increased proportionally with the compressive strength for all HFC

specimens incorporating ESP. Based on this relationship, a prediction model was developed and expressed as Equation (6), which resulted from regression analysis of the experimental results gained after 28 days of curing.

$$F_t = 0.3095f_{ck}^{0.6343} \quad (6)$$

In which, F_t denotes the split tensile strength and f_{ck} signifies the compressive strength of HFC specimens.

Fig. 15 exhibits a significant correlation ($R^2 = 0.9987$) between compressive and splitting tensile strengths, supporting the proposed power-type regression model. Table 6 summarizes the estimated splitting tensile strength values obtained from this regression equation, as well as the comparison results from previous studies.

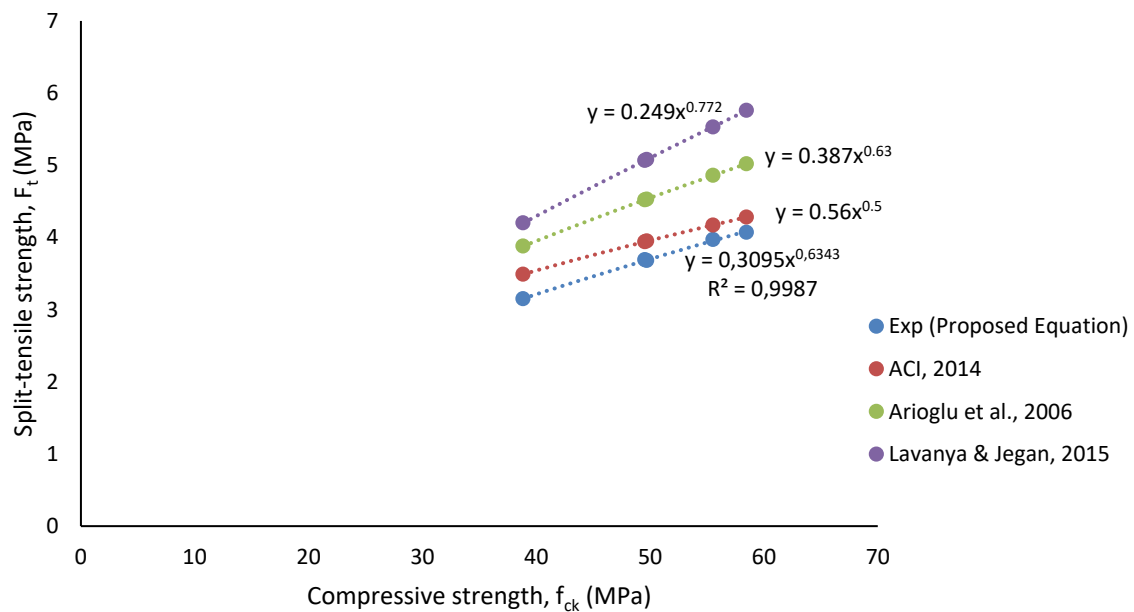


Fig. 15. Correlation between splitting tensile and compressive strengths of HFC incorporating ESP after 28 days of water curing

Table 6. Comparison of splitting tensile strength of hardened HFC specimens with previous studies

Mix	Compressive strength (MPa) (28 days), f_{ck}	Split tensile strength (MPa) (28 days), f_t	Proposed Equation $F_t = 0.3095f_{ck}^{0.6343}$ (Equation 6)	[26] $F = 0.56f_{ck}^{0.5}$	[25] $F = 0.387f_{ck}^{0.63}$	[22] $F = 0.249f_{ck}^{0.772}$
C100	38.85	3.15	3.15	3.49	3.88	4.20
C95-ESP5	49.55	3.69	3.68	3.94	4.52	5.067
C90-ESP10	55.53	3.97	3.96	4.17	4.86	5.53
C85-ESP15	58.47	4.07	4.09	4.28	5.02	5.76
C80-ESP20	49.71	3.68	3.69	3.95	4.53	5.08

The comparison of the results in Table 6 reveals that the proposed regression equation has the best agreement with the experimental splitting tensile strength values for all specimens. As an instance,

in the C95-ESP5 and C90-ESP10 specimens, the estimated strengths from the proposed model (3.68 MPa and 3.96 MPa, respectively) are almost identical to the experimental data (3.69 MPa and 3.97 MPa). In contrast, the [26] equation continually provides somewhat higher values, with discrepancies ranging from 5 to 10%. Even though the difference is small, it implies that the ACI model, which was designed largely for conventional concretes, does not fully represent the effect of ESP on the tensile behavior of HFC. Prediction drawn from [25] exhibit higher variations, overestimating splitting tensile strength by 20% or more, as seen in the C90-ESP10 specimen, where the model gives 4.86 MPa versus the experimental 3.97 MPa. Even bigger splitting tensile values are observed with the equation of [22], which provides the highest predicted values among all models. For instance, the C85-ESP15 specimen has a predicted value of 5.76 MPa compared to the experimental 4.07 MPa, reflecting a discrepancy of over 40%. These disparities demonstrate that empirical data derived from conventional or alternative concrete systems may not be immediately applicable to ESP-modified HFC.

Besides that, to assess the predictive accuracy of Equation (6), the 28-day experimental dataset was employed for calculating statistical indicators such as coefficient of determination (R^2), mean absolute error (MAE), and root mean square error (RMSE). The proposed model had a R^2 value of 0.999, with MAE and RMSE values of 0.010 and 0.012 MPa, respectively, suggesting a strong correlation between projected and experimental splitting tensile strength values. For comparison, previously developed empirical models produced substantially greater prediction errors. Model [26] produced $R^2 = 0.35$ with MAE = 0.254 MPa and RMSE = 0.259 MPa. Models [25] had significantly bigger deviations with MAE = 0.846 MPa and RMSE = 0.865 MPa. While model [22] with MAE = 1.411 MPa and RMSE = 1.428 MPa. These models typically overestimated splitting tensile strength, implying a limited applicability to non-calcined ESP-modified HFC. Equation (6) performs well because it was calibrated using experimental data relevant to the examined material system, capturing the effect of ESP-induced microstructural densification on tensile behaviour.

This study emphasizes the fundamental relationship of compressive and splitting tensile strengths and shows an effective predictive method for calculating splitting tensile strength from compressive strength of HFC encompassing ESP. The regression model developed in this study provides a useful tool for both the development and the performance assessment of ESP-based concrete. However, additional experimental validation under various mix proportions, curing settings, and material compositions would be beneficial to verify the proposed model's strength and practicality. However, despite its good predictive capability in the examined range, the model that was suggested is empirical and relies on a small dataset (ESP replacement 0-20%). Extrapolation beyond the given range or usage to calcined ESP systems, different binder compositions, or various conditions of curing needs to be done with caution. The model also does not adequately take into consideration microstructural features including porosity evolution, hydration degree, and ITZ properties, all of which might have an impact on splitting tensile performance. In order to enhance generalizability, additional experimental validation under various mix proportions, curing settings, and material compositions would be beneficial to verify the proposed model's strength and practicality.

3.4 Mode of Failure

In the present study, cracking patterns resulting from compressive and splitting-tensile strength tests were systematically examined, as shown in Fig. 16 and Fig. 17. Concrete fails in different ways based on the type of loading and stress conditions used. These modes are primarily driven by fracture formation and propagation inside the cementitious matrix and at the aggregate-paste contact. A better understanding of this overall failure mechanism lays the basis for the following explanation of the various fracture mechanisms that are discovered under compressive and splitting-tensile loads.

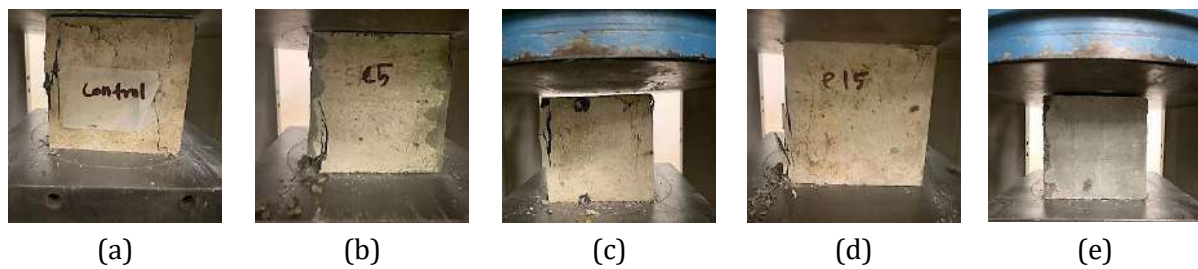


Fig. 16. Cracking patterns of HFC specimens under compressive loading: (a) C100 (control), (b) C95-ESP5, (c) C90-ESP10, (d) C85-ESP15, and (e) C80-ESP20

Fig. 16 (a-e) and Table 7 depict the cracking patterns of 100×100 mm HFC specimens under compressive loading. The control specimen (C100) showed substantial and critical vertical cracking, implying a brittle fracture mode, as depicted in Fig. 16(a). Vertical edge cracks were also observed in specimens containing 5% and 10% ESP (C95-ESP5 and C90-ESP10). However, their initiation points varied, where C95-ESP5 showed cracking along the lower left edge, whereas C90-ESP10 showed cracks at the upper left side, as can be seen in Fig. 16(b-c). On the contrary, specimens containing greater ESP replacement levels (C85-ESP15 and C80-ESP20) exhibited only small surface fractures, with C80-ESP20 having the least apparent damage (Fig. 16(d-e)). Overall, the severity and amount of cracking decreased as ESP content increased. The addition of ESP reduces crack propagation in HFC by densifying the matrix and lowering stress concentrations. Nonetheless, the overall cracking pattern was consistent with that of control specimen (C100), which is characterized by vertical fracture formation under compressive stress.

Table 7. Type of crack recorded on HFC cube specimens under compressive loading

Specimen designation	Type of cracks	
	Vertical	Edge
C100	Yes	Yes
C95-ESP5	Yes	Yes
C90-ESP10	Yes	Yes
C85-ESP15	Yes	Yes
C80-ESP20	Yes	Yes

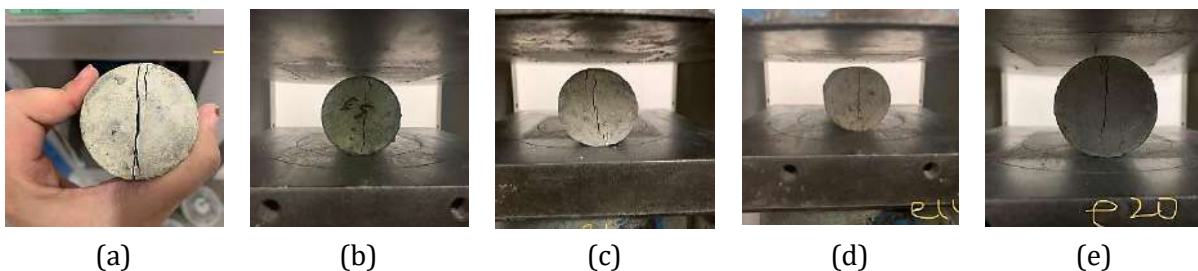


Fig. 17. Cracking patterns of HFC specimens under splitting tensile strength test: (a) C100 (control), (b) C95-ESP5, (c) C90-ESP10, (d) C85-ESP15, and (e) C80-ESP20

Table 8. Crack patterns observed in cylindrical specimens subjected to the splitting tensile strength test

Specimen designation	Type of cracks
	Vertical
C100	Yes
C95-ESP5	Yes
C90-ESP10	Yes
C85-ESP15	Yes
C80-ESP20	Yes

Fig. 17(a-e) and Table 8 exhibit the pattern of cracks of 50 × 100 mm cylindrical specimens subjected to the splitting tensile test for various HFC specimens. Different crack formation patterns were detected based on the ESP content (5%, 10%, 15% and 20%). The control specimen (C100), showed a noticeable and continuous crack pattern, indicating a typical brittle tensile failure, as depicted in Fig. 17(a). In contrast, C95-ESP5 exhibited more severe cracking, showing a decrease in tensile resistance with reduced ESP incorporation (Fig. 17(b)). In addition, cracks in C90-ESP10 were mostly found throughout the middle part of the specimen, indicating a more localized failure pattern, as shown in Fig. 17(c). Remarkably, the specimen with the highest ESP concentration (C80-ESP20) exhibited the least obvious cracking, with cracks reaching only to approximately two-thirds of the specimen's height, as can be seen in Fig. 17(e). C85-ESP15 showed much more cracking than C80-ESP20, which indicates that larger ESP replacement levels could lead to enhanced crack resistance via the micro-filler effect and increased matrix densification, as observed in Fig. 17(d).

The crack patterns that were noticed are comparable with the compression test results (Fig. 16), which demonstrated that the use of ESP reduced the severity of cracking. This shows that ESP improves HFC tensile performance by refining pore structure, reducing stress concentrations, and delaying crack propagation. [27] also discovered that partial ESP substitution improves cementitious materials' mechanical properties by serving as a micro-filler and densifying the matrix. Furthermore, several studies show that utilizing up to 20% ESP can increase concrete strength and durability by refining the pore structure and lowering stress concentrations [28]. However, more research is needed to understand the underlying mechanisms at the microstructural level and to determine the ideal ESP content for maximum tensile strength as well as durability in practical applications.

4. Conclusions

This study carefully explored the potential utilization of ESP as a partial cement substitution in HFC. The results demonstrate that ESP has a considerable impact on both the fresh and hardened properties of HFC, highlighting its ability to serve as a sustainable alternative binder. In accordance with the experimental findings and analysis, several significant conclusions can be drawn concerning the use of eggshell powder (ESP) as a supplementary cementitious material in highly flowable concrete (HFC):

- Increased ESP content resulted in variations in fresh properties such as slump flow diameter and T500 time. A significant increase in flowability was observed with 20% ESP substitution. According to EFNARC classification, the resulting SF1 and SF2 classifications show that HFC incorporating ESP can be adapted for a variety of structural applications, covering unreinforced or moderately reinforced parts to widespread structural use.
- For segregation resistance, ESP generally reduced sieve segregation, with the exception of 20% replacement, where segregation increased significantly. The calculated segregation ratio (SR) values attributed to the SR1 and SR2 categories, providing practical recommendations on permitted flow distances and confinement gaps.
- The addition of eggshell powder (ESP) greatly improved the hardened properties of HFC after 7, 28, and 90 days. Compressive strength (CS) increased significantly with 10-15% substitution, especially at early and intermediate ages, owing to better particle packing and microstructural densification. Even though the control mix exhibited the highest 90-day strength, ESP-modified mixes in the optimum range exhibited steady and structurally sufficient performance over the long term.
- Splitting tensile strength (STS) showed a similar trend, with 10-15% ESP improving fracture resistance and paste-aggregate bonding after 7 and 28 days. STS remained steady after 90 days, with no indication of long-term deterioration. ESP functions largely as a micro-filler and nucleation agent, with a 10-15% substitute level suggested to maximize strength development while maintaining long-term mechanical strength. To accurately measure long-term strength evolution and durability properties, extended curing times (56 and beyond 90 days) could be employed in the subsequent phase of this study.

- The addition of ESP in HFC enhances crack propagation resistance under both compressive and tensile loading, with higher ESP replacement levels (particularly 15-20%) enhancing the micro-filler effect and matrix densification, which leads to lesser crack severity and more localized or less noticeable failure patterns in contrast to the brittle and continuous cracks found in the control specimen.

Therefore, incorporating ESP with HFC provides considerable benefits in terms of sustainability and usage. ESP minimizes the consumption of cement and CO₂ emissions while promoting waste recycling, consistent with sustainable construction and circular economy concepts. While the present investigation shows that ESP improves the fresh and hardened characteristics of HFC, the results are mostly focused on early-age performance. In order to enhance ESP's usability as a sustainable cement substitute, future study should look into long-term durability in harsh conditions, microstructural behaviors, and optimal adjustment of ESP particle size and percentages of substitution. Even though a thorough techno-economic analysis was not within the scope of the present investigation, preliminary findings indicate that substituting 10-15% of cement with non-calcined ESP may lower binder-related material costs, given that cement accounts for a significant amount of concrete production prices. The reduction of calcination lowers processing energy requirements and operational costs. Furthermore, valorizing industrial eggshell waste could decrease disposal costs and promote sustainable economic practices. Future research should include life-cycle cost analysis (LCCA) and large-scale feasibility studies to quantify economic and environmental advantages, contribution to low-carbon and sustainable construction methods. Recognizing these needs, the study also recognized that further investigations, such as flexural behaviour, ultrasonic pulse velocity (UPV), absorption properties, flow assessments including the U-box and V-funnel and long-term durability tests like microstructure, shrinkage, chloride ingress, and carbonation, could enhance the overall understanding of HFC incorporating ESP. These concerns will be tackled in a forthcoming supplementary study as part of the second phase of this research effort.

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Cracking behavior of partially prestressed concrete pier caps: Field evidence, nonlinear modeling, and implications for code-based crack control

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Abstract

Partially prestressed concrete (PPC) is widely used in bridge substructures because it combines the stiffness of prestressing with the ductility of reinforced concrete. However, current crack control provisions are largely derived from reinforced or fully prestressed concrete and have not been validated for PPC members under real service conditions. This study investigates the cracking behavior of 113 in-service PPC pier caps from the Ben Luc – Long Thanh Expressway in Vietnam through field measurements, nonlinear finite element analysis, and code-based evaluation. Field inspections revealed that cracking occurred in 87 pier caps, with maximum crack widths reaching 0.545 mm and crack depths exceeding the 50 mm concrete cover, indicating a potential durability concern despite full compliance with AASHTO LRFD and ACI 318 crack control requirements. A three-dimensional nonlinear finite element model incorporating concrete fracture mechanics and prestress–crack interaction predicted a maximum crack width of 0.448 mm and reproduced the observed crack localization at pier–cap junctions and bearing regions. Comparisons with ACI 224R and AASHTO LRFD formulations show that standard empirical models may underestimate crack widths, in some cases by more than 40%. This discrepancy is attributed to the distinctive cracking mechanism of PPC, in which prestressing tends to suppress distributed multiple cracking while promoting strain localization and wider individual cracks. The results suggest that current crack control provisions may be non-conservative for PPC pier caps and that nonlinear fracture-based analysis offers a more reliable basis for serviceability assessment.

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1. Introduction

Partially prestressed concrete (PPC) was introduced as a hybrid structural system combining the crack tolerance and ductility of reinforced concrete (RC) with the stiffness and serviceability advantages of fully prestressed concrete (FPC) [1-5]. By allowing limited tensile cracking under service loads, PPC members can reduce excessive camber and creep typical of FPC while maintaining superior post-cracking stiffness compared with RC [2-4, 6, 7]. These advantages have led to the increasing use of PPC in bridge components such as beams, pier caps, and deep members.

Despite its practical importance, the serviceability design of PPC remains less well defined than that of RC or FPC. While RC design permits cracking and FPC design aims to avoid it, PPC occupies an intermediate regime in which cracking is allowed but must be strictly controlled to prevent

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durability problems such as reinforcement corrosion and surface deterioration [8, 9]. However, current design standards, including ACI 318 and AASHTO LRFD, do not provide crack control provisions specifically developed for PPC members. Instead, they rely on crack width formulas and reinforcement spacing rules originally derived for RC or fully bonded prestressed concrete.

Experimental and analytical studies have demonstrated that PPC exhibits cracking behavior that is distinct from that of RC. Witchukreangkrai [8] showed that cracking in PPC beams is governed by the interaction between prestressing and conventional reinforcement. Harajli and Naaman [3] and Chern et al. [2] reported that progressive cracking and stress redistribution in PPC depend on the balance between prestress and applied loads. Furthermore, the partial prestressing ratio (PPR) has a distinct impact on cracking behavior: a decrease in PPR results in reduced crack spacing and narrower crack widths [10]. More recent studies have shown that standard crack width formulas and serviceability models fail to predict cracking and deflection once decompression is exceeded, particularly under repeated loading [10-12]. Advanced research has established a correlation between the flexural stiffness degradation of cracked sections and crack width. Newly proposed equations facilitate crack width prediction by integrating strain compatibility, the partial prestressing ratio (PPR), the location of the non-prestressed reinforcement centroid, and the concrete cracking stress, thereby demonstrating improved accuracy when validated against experimental data [10]. In addition, deep members have been shown to exhibit crack development governed by internal strain energy rather than simple stress limits [13], and higher-strength concrete may even lead to wider cracks due to increased brittleness [14].

Advances in numerical modeling further indicate that cracking in prestressed and partially prestressed members is governed by nonlinear fracture processes that cannot be captured by empirical equations. Parametric studies [15] and fracture-based simulations [16] have demonstrated large discrepancies between code predictions and actual crack widths, while Accornero et al. [17] highlighted the importance of modeling the transition from distributed cracking to localized fracture in prestressed concrete. In addition, Liu et al. [18] developed a machine learning model utilizing a boosting framework to predict the maximum crack width in partially prestressed concrete (PPC) beams. Building upon these predictive capabilities, the authors introduced a rapid design methodology governed directly by crack width limitations.

However, most of the available evidence is derived from laboratory tests on beams under controlled loading conditions [3, 10, 14, 18]. Field-scale data from in-service PPC bridge components remain extremely limited, particularly for members designed to be crack-controlled according to ACI and AASHTO provisions. As a result, the reliability of current design rules has not been verified under real service conditions. Consequently, it remains unclear whether the crack control provisions embedded in ACI 318, ACI 224, and AASHTO LRFD [19] are conservative, adequate, or unsafe when applied to real PPC bridge structures.

This study addresses this gap by investigating the cracking behavior of 113 partially prestressed concrete pier caps from the Ben Luc – Long Thanh Expressway in Vietnam. These pier caps were designed in accordance with AASHTO LRFD and ACI-based crack control provisions, providing a unique opportunity to evaluate the real-world performance of current design rules. A detailed field survey was conducted to measure crack locations, widths, and depths under service conditions. In addition, a three-dimensional nonlinear finite element model based on concrete fracture mechanics was developed to simulate crack initiation, propagation, and reopening under combined prestress and service loads.

The objectives of this study are to (i) establish a large-scale field dataset describing the actual cracking behavior of PPC pier caps, (ii) evaluate the ability of nonlinear fracture-based finite element modeling to reproduce observed cracking patterns and magnitudes, and (iii) assess the adequacy of current ACI and AASHTO crack control provisions for PPC bridge substructures. By linking field observations, numerical simulations, and code-based predictions within a unified framework, this study demonstrates that the cracking mechanism of PPC differs fundamentally from that assumed in current design standards, and that spacing-based crack control may be insufficient for PPC members. The findings provide new physical insight into PPC serviceability

behavior and support the need for fracture-based design approaches for durability-critical bridge components.

2. Description of the PPC Pier Caps and Field Survey

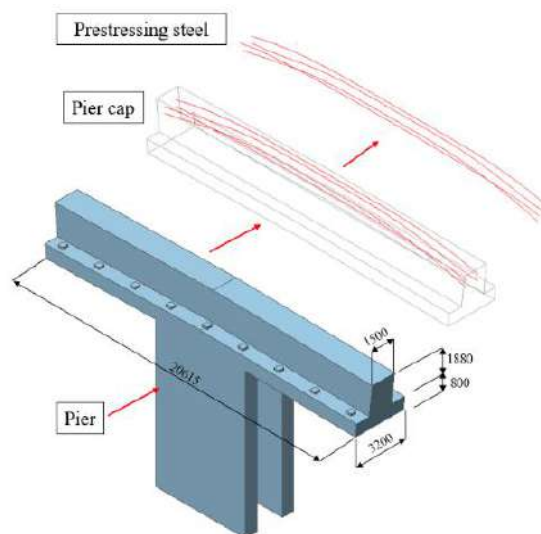
2.1 Structural Configuration of the PPC Pier Caps

The present study investigates 113 PPC pier caps forming part of the elevated viaduct of the Ben Luc – Long Thanh Expressway in Vietnam. The viaduct consists of 113 simply supported spans and one continuous bridge with a span arrangement of (86 + 140 + 86) m. The investigated pier caps were categorized into three categories: 92 piers supporting link slabs, 19 piers located at expansion joints, and two transition piers accommodating both Super-T girders and cast-in-place cantilever segments. During construction, visible cracking was identified in 87 of the 113 pier caps, which motivated the detailed field investigation reported in this paper.

An overview of the surveyed PPC pier caps is presented in Fig. 1. The layouts of the prestressing tendons and conventional reinforcement are illustrated in Figs. 2 and 3, respectively. The pier caps were designed using bonded post-tensioned tendons in combination with conventional reinforcement, with the amount and configuration of steel varying according to the structural function of each pier cap. In general, pier caps supporting link slabs were provided with twenty-three 32-mm-diameter reinforcing bars and four prestressing tendons, each consisting of twelve 7-wire strands with a nominal diameter of 15.2 mm. In contrast, pier caps at expansion joints or supporting girder segments were equipped with twenty-one 28-mm-diameter reinforcing bars and five identical prestressing tendons to accommodate higher force demands. The material properties of the concrete, reinforcing bars, and prestressing tendons used in the pier caps are summarized in Table 1. These properties were taken from the project specifications and were applied consistently in both the field interpretation and the numerical analyses.



(a) Overview of investigated pier



(b) 3D view of investigated pier and PPC pier cap

Fig. 1. Typical piers and PPC pier cap at expansion joints in Ben Luc – Long Thanh Expressway in Vietnam

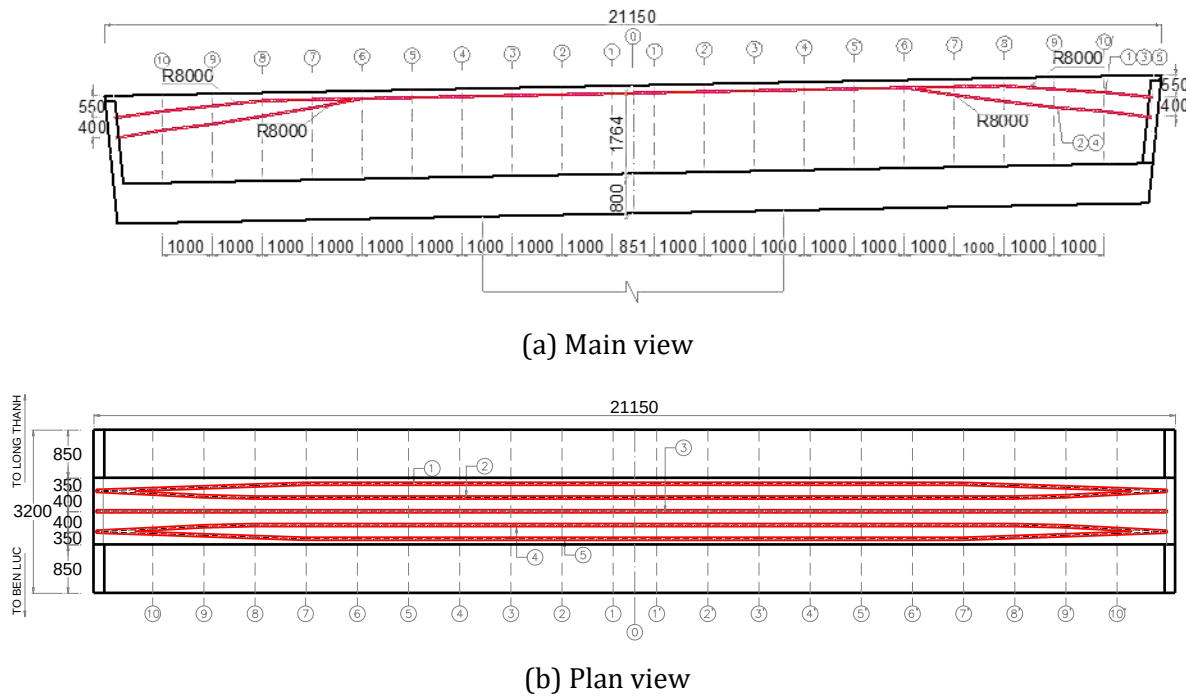


Fig. 2. Layout of prestressing tendons in a typical PPC pier cap

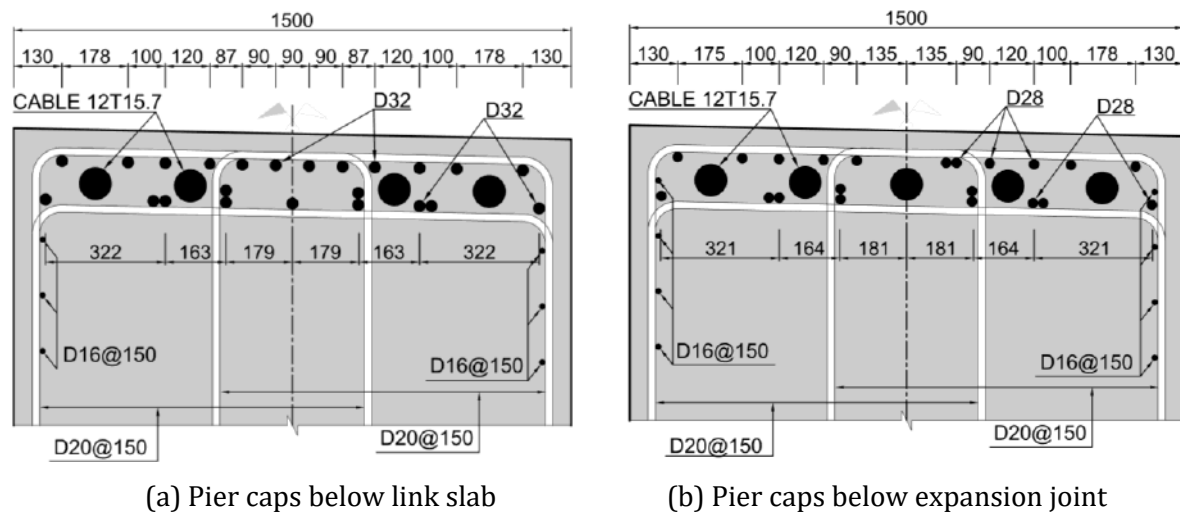


Fig. 3. Layout of conventional reinforcement in a typical PPC pier cap

Table 1. Material properties of concrete, reinforcing bars, and prestressing tendons

Material	Type used	Density (kN/m ³)	Yield strength, f_y (MPa)	Ultimate strength, f_u (MPa)	Elastic Modulus, E (MPa)
Concrete	Compressive strength, $f'_c = 30$ MPa	24.5	-	-	29440
Reinforcing bars	CB400-V	78.5	400	570	200000
Prestressing tendons	Twelve 7-wire/low relaxation strands	77.1	1674	1860	197000

2.2 Field Inspection and Crack Measurement

Once the superstructure had been fully installed, visible cracks on the pier caps were systematically investigated to identify the characteristic cracking patterns of partially prestressed concrete

members. To provide a rigorous assessment of the cracking behavior of the partially prestressed pier caps, a detailed field survey was conducted on 24 piers selected randomly from the 87 pier caps in which visible cracking had been identified during the preliminary inspection stage. It should be noted that, due to limitations in site accessibility and inspection time, the selected sample represents approximately 25% of the cracked pier caps and was intended to reduce potential subjectivity and selection bias in the field investigation. Consequently, 24 out of the 87 cracked pier caps were examined in detail. Crack widths were measured using a specialized crack width gauge (ZBL-F130), as shown in Fig. 4(a), while crack depths were determined using an ultrasonic tester (C369N), as illustrated in Fig. 4(b).



(a) Measurement of surface crack width using ZBL-F130



(b) Measurement of crack depth using ultrasonic tester (C369N)

Fig. 4. Measurement of crack width and crack depth in the pier caps

2.3 Classification of Crack Patterns

Three representative crack types were identified, as illustrated in Fig. 5.

- Type 1 – Cracks at the cantilever–pier interface (ledge region):

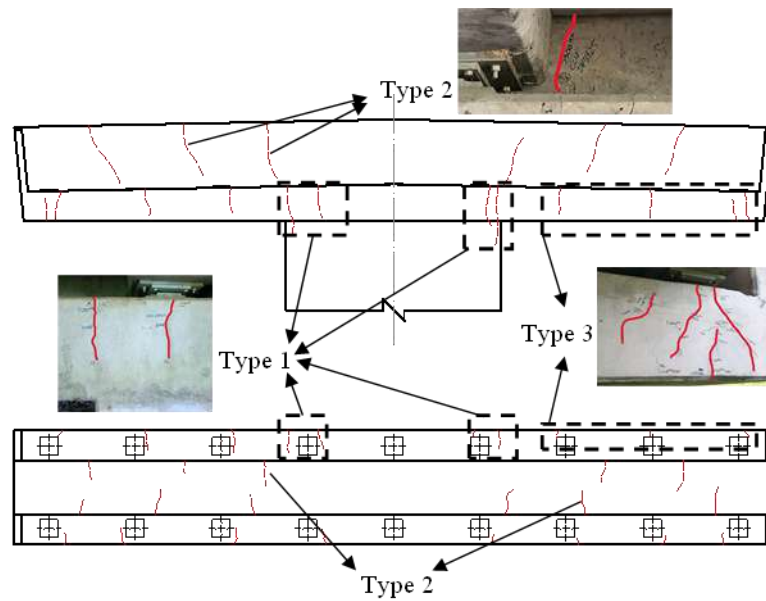
Type 1 cracks develop along the vertical interface between the pier column and the overhanging cantilever part of the pier cap, where they are concentrated in the ledge region immediately above the pier shaft. These cracks propagate predominantly in a near-vertical direction and often extend upward into the body of the pier cap. Structurally, this region corresponds to the zone of maximum negative bending moment and high shear transfer between the pier and the cantilever arm. The combined effects of prestressing force, superstructure dead load, and column restraint generate high tensile stresses normal to the interface, making this region particularly susceptible to crack initiation and widening.

- Type 2 – Cracks in the stem of the pier cap above the pier:

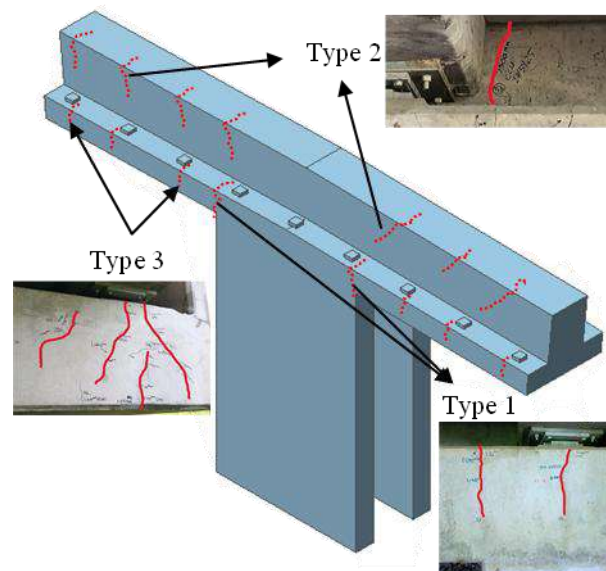
Type 2 cracks occur in the stem (web) of the pier cap, which is located slightly above the cantilever–pier interface and extending upward into the main body of the pier cap. These cracks are generally inclined or vertical and are less frequent than Type 1 cracks. This region is governed by stress redistribution and local bending within the deep-beam zone of the pier cap. The presence of prestressing suppresses distributed cracking but promotes tensile strain localization in this transition zone, leading to the formation of discrete cracks in the stem, as confirmed by the nonlinear finite element results presented later.

- Type 3 – Cracks beneath bearing locations:

Type 3 cracks are vertical cracks directly beneath the bearing seats on the top surface of the pier cap. They initiate at the soffit of the bearing region and propagate downward into the pier cap. These cracks result from high local bearing pressures, transverse tensile splitting, and concentrated reactions transmitted from the superstructure. Under service loads, the combination of bearing pressure and restrained deformation generates tensile stresses perpendicular to the load path, producing vertical splitting cracks beneath the bearings.



(a) Detailed crack patterns



(b) Three-dimensional schematic view of crack patterns

Fig. 5. Observed crack patterns in PPC pier caps

Among the 24 surveyed cracked pier caps selected from the total population of 113 pier caps, a total of 426 cracks were recorded. Cracks occurring at the cantilever–pier interface (Type 1) and beneath the bearing zones (Type 3) were dominant, accounting for 74.9% and 20.7% of the total cracks, respectively, and were observed in nearly all of the pier caps. In contrast, Type 2 cracks represented only 4.5% of the total and were identified in just 5 of the 24 surveyed piers. Notably, these pier caps were all located at positions with link slabs, suggesting that restraint from the superstructure may play an important role in promoting crack development in this region.

2.4 Field Observations and Statistical Analysis of Cracking

2.4.1 Crack Patterns and Spatial Distribution

The overall distribution of measured crack widths is presented in Fig. 6. The results indicate that most cracks exhibited small crack openings, with 43.9% falling in the range of 0.1–0.2 mm and 29.6% being smaller than 0.1 mm. Cracks wider than 0.3 mm were relatively uncommon,

accounting for only 8.7% of all observations, while cracks exceeding 0.5 mm were negligible, representing just 0.2% of the total (a single crack).

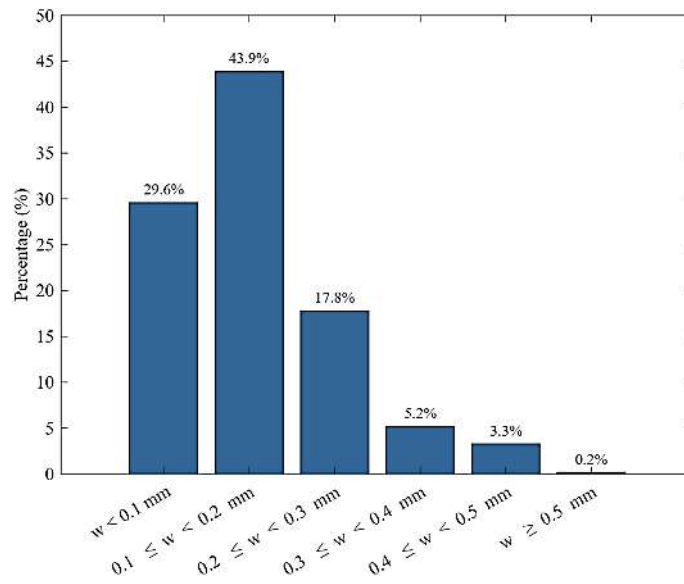


Fig. 6. Overall distribution of field-measured crack widths

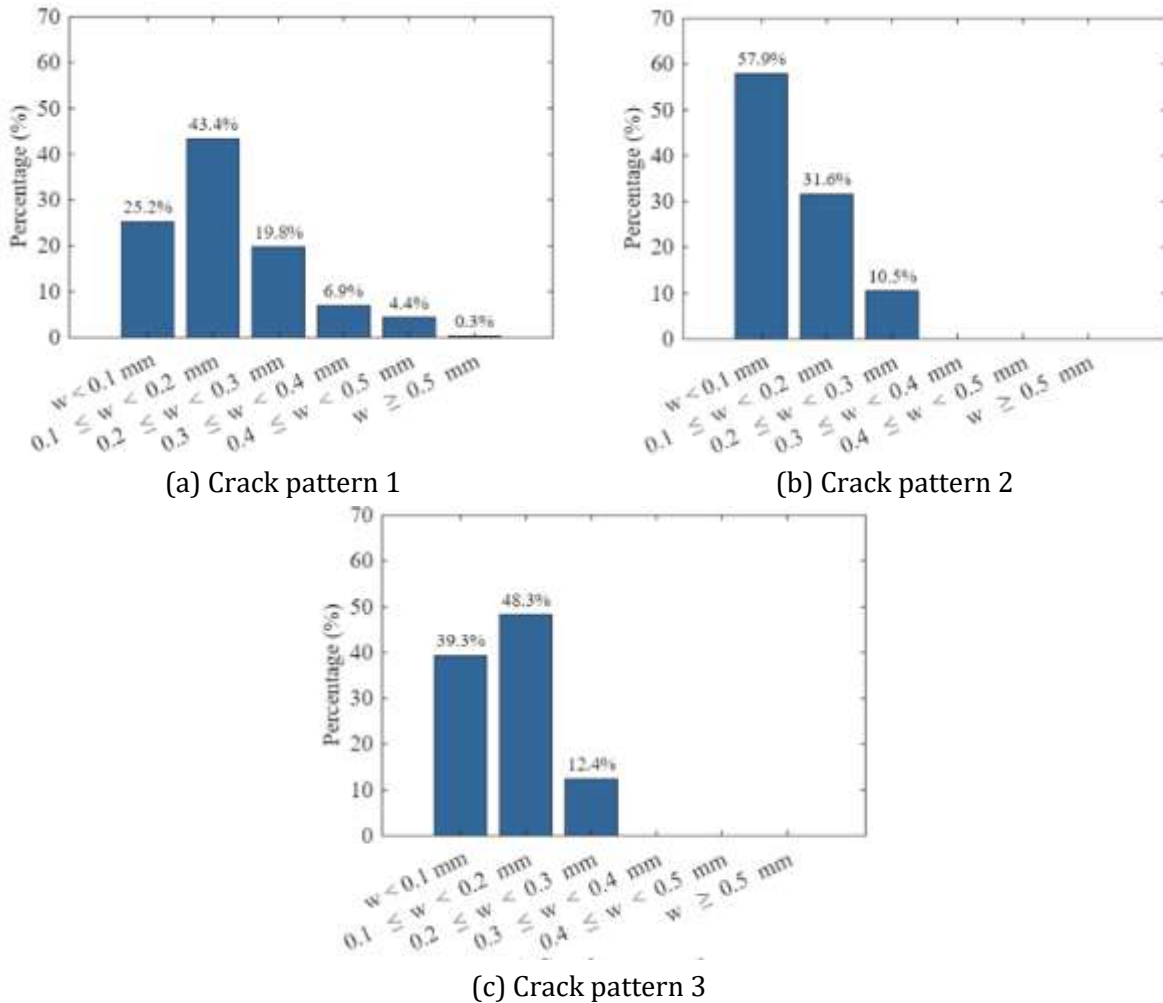


Fig. 7. Crack width distributions for the three crack patterns

The crack width distributions for the three crack types are shown in Fig. 7 and reveal clear differences in cracking behavior among the three regions of the pier caps. Cracks wider than 0.3 mm occurred exclusively in Type 1 cracks at the cantilever–pier interface; however, they accounted for less than 12% of this group, while the majority of Type 1 cracks (43.4%) remained in the 0.1–

0.2 mm range. This indicates that although wide cracks are more likely to develop in this highly restrained load-transfer zone, most cracks remain within moderate widths.

Type 2 cracks, located in the stem of the pier cap above the pier, were dominated by very small crack openings, with 57.9% of cracks smaller than 0.1 mm and 31.6% in the 0.1–0.2 mm range. Only 10.5% of Type 2 cracks exceeded 0.2 mm, reflecting the absence of strong strain localization in this region. A similar trend was observed for Type 3 cracks beneath the bearing zones, where 39.3% of cracks were smaller than 0.1 mm and 48.3% fell within the 0.1–0.2 mm range. Cracks wider than 0.2 mm accounted for only 12.4% of the population. This distribution suggests that although the bearing regions are subjected to high local stresses, the surrounding concrete provides sufficient confinement to limit crack opening.

Overall, the crack width distributions confirm that the cantilever–pier interface (Type 1) acts as the primary crack localization zone in PPC pier caps, whereas the stem and bearing regions tend to exhibit more distributed and finer cracking. The observed crack-width distribution exhibits a generally decreasing statistical trend, with the majority of cracks remaining below 0.2 mm and only a very limited number approaching or exceeding 0.5 mm. Although a single crack wider than 0.5 mm was identified, several additional cracks exhibited widths close to this threshold, indicating that the largest measured crack does not represent an isolated anomaly within the surveyed population.

2.4.2 Crack Depth Characteristics

Regarding crack propagation, the data summarized in Table 2 indicate that, for every surveyed pier, the maximum measured crack depth exceeded the nominal concrete cover thickness of 50 mm. In several pier caps, crack depths reached more than five times the cover thickness, with a maximum recorded value of 264.9 mm, indicating that cracks not only penetrated the cover layer but extended deep into the structural core of the pier caps.

Table 2. Maximum measured crack depths for the surveyed PPC pier caps

Pier Cap No.	Number of cracks measured	Number of vertical cracks	Number of inclined cracks	Maximum crack depth (mm)
P15	7	4	3	135.5
P16	6	6	0	155.9
P17	7	5	2	136.1
P19	5	5	0	172.3
P21	6	2	4	132.4
P22	5	2	3	187.5
P25	5	2	3	92.4
P26	5	2	3	69.2
P29	5	3	2	106.1
P30	5	3	2	125.7
P50	5	0	5	n.a
P51	7	6	1	129.9
P52	6	6	0	169.0
P55	5	4	1	122.6
P56	6	3	3	118.9
P59	8	2	6	168.9
P70	5	5	0	264.9
P71	7	7	0	207.0
P81	4	1	3	65.4
P82	5	3	2	114.3
P83	10	6	4	150.5
P85	7	6	1	173.9

This behavior is particularly critical for partially prestressed concrete members. Although prestressing reduces the number of surface cracks, it promotes crack localization, allowing individual cracks to propagate deeply once decompression is exceeded. Consequently, even cracks with relatively small surface widths can penetrate far beyond the cover depth, creating continuous

transport paths for moisture and chlorides toward the prestressing strands and conventional reinforcement. The field data therefore demonstrate that crack control based solely on surface crack width is insufficient to ensure durability in PPC pier caps. While a large proportion of the measured cracks had surface widths below 0.2 mm, their penetration depths frequently exceeded the protective cover by large margins, indicating a substantially higher corrosion risk than would be inferred from crack width alone.

The distinction between vertical and inclined cracks was introduced because depth measurements obtained from ultrasonic testing are considered more reliable for predominantly vertical cracks than for inclined cracks with irregular propagation paths. In addition, the reported crack depths are presented as absolute measured values and were primarily intended to assess whether the cracks had penetrated beyond the nominal concrete cover. A more detailed mechanics-based investigation relating crack propagation to member thickness and the effective tensile depth of the section is beyond the scope of the present study and is recommended for future research.

3. Numerical Study

To predict the structural response of the partially prestressed pier caps and to clarify the mechanisms governing crack development, a three-dimensional nonlinear finite element model was developed in this study. The model was used to simulate the effects of prestressing, dead load, and service load on crack initiation, propagation, and opening in the pier caps.

The analysis incorporates nonlinear material behaviour of concrete and steel and allows explicit simulation of cracking and tension stiffening degradation. This enables reproduction of the observed crack patterns and crack widths measured in the field, particularly in critical regions such as the cantilever–pier interface and beneath the bearing zones.

By comparing the numerical results with the field measurements, the finite element model provides both a predictive tool for evaluating crack development and a mechanistic framework for interpreting the cracking behaviour of PPC pier caps under service conditions.

3.1 Finite Element Model and Meshing

Fig. 8 shows the geometry and mesh of the finite element (FE) model. The concrete components were discretized using three-dimensional solid elements generated by extruding planar meshes along the longitudinal axis of the structure. Reinforcing bars and prestressing tendons were represented by one-dimensional embedded bar elements, whose material properties and cross-sectional areas were defined within the surrounding concrete elements to ensure strain compatibility and force transfer.

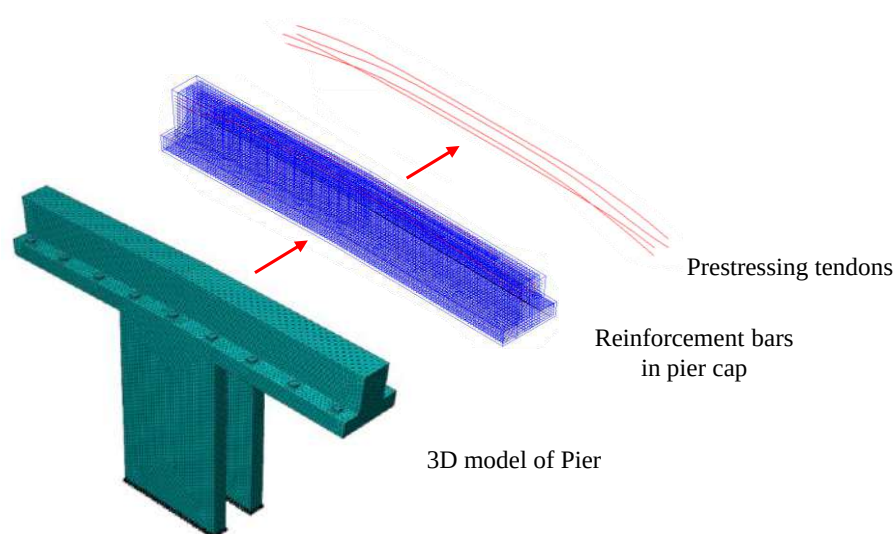


Fig. 8. Finite element model and mesh of the PPC pier cap

The computational domain was discretized using the automated Tetra Mesher in MIDAS FEA [20], which generates a variable-density tetrahedral mesh with smooth transitions between refined and coarse regions. To ensure adequate element quality, the meshing procedure employed an Optimization Level of 7, a Refinement Factor of 1.2, and a Shape Quality Weight of 7. The final FE model comprised 37,383 nodes and 177,871 elements. This mesh resolution was selected to be sufficiently fine in critical regions to accurately capture crack initiation, localization, and opening within the fracture-based concrete model.

3.2 Material Models

All structural components were modelled as isotropic in their initial (undamaged) state. The linear elastic properties of concrete and steel, including Young's modulus, Poisson's ratio, and mass density, were taken from the project design specifications as shown in Table 1.

To represent the nonlinear cracking and fracture behavior of concrete, the total strain crack model implemented in MIDAS FEA was adopted. Within this formulation, concrete is treated as initially isotropic, while anisotropy induced by cracking is captured through progressive stiffness degradation normal to crack planes. The compressive response of concrete was described using the Thorenfeldt stress-strain model [21], whereas the post-cracking tensile behavior was governed by the Hordijk softening law [22], which controls crack opening, tension stiffening, and energy dissipation (Fig. 9). The parameters defining these two constitutive relationships are provided in Table 3. It should be noted that in the Hordijk softening law, h denotes the crack band width, commonly taken as the representative element size in the fracture process zone. In this study, h is taken as 50 mm.

Reinforcing bars and prestressing tendons were modelled using a linear elastic constitutive relationship, which is appropriate for the serviceability limit state (SLS) considered in this study, where steel stresses remain well below yield and inelastic steel behavior is not expected to influence crack development. Under these conditions, crack initiation, propagation, and opening are governed primarily by the fracture behavior of concrete rather than by steel yielding.

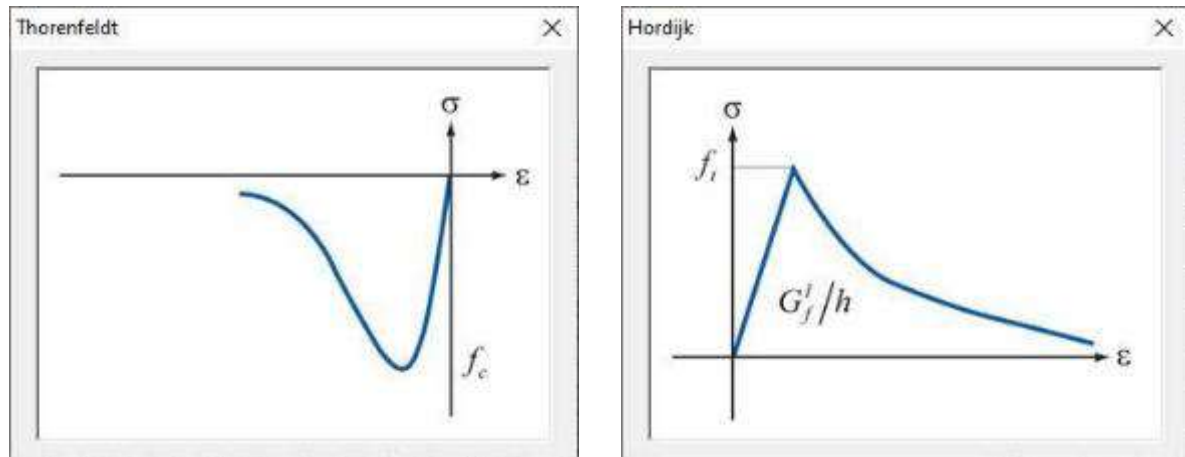


Fig. 9. Hordijk and Thorenfeldt models in Midas FEA

Table 3. Parameters for Thorenfeldt and Hordijk models

Parameters	Thorenfeldt's model	Hordijk's model		
	Compressive strength f'_c , N/mm ²	Tensile strength f_t , N/mm ²	Fracture energy G_f , N/mm	Crack band width h , mm
Value	35	3.2	0.084	50

3.3 Load and Boundary Conditions

Fig. 10 presents the boundary conditions, loading scheme, and interaction definitions adopted in the FE model. A perfect bond was assumed between concrete and both reinforcing bars and prestressing strands, enforcing full kinematic compatibility by tying the reinforcement nodes to the

surrounding concrete elements and preventing slip. This ensures that tensile forces are transferred to the steel as cracking develops in the concrete. The base of the pier was fixed to represent the foundation restraint. Self-weight was automatically applied based on the defined material densities. Superstructure loads, including dead and live loads, were obtained from an independent structural analysis and applied as pressure loads at the bearing areas. The prestressing action was introduced as an equivalent distributed pressure acting over the anchorage zones of the pier cap, representing the effective transfer of tendon forces into the concrete.

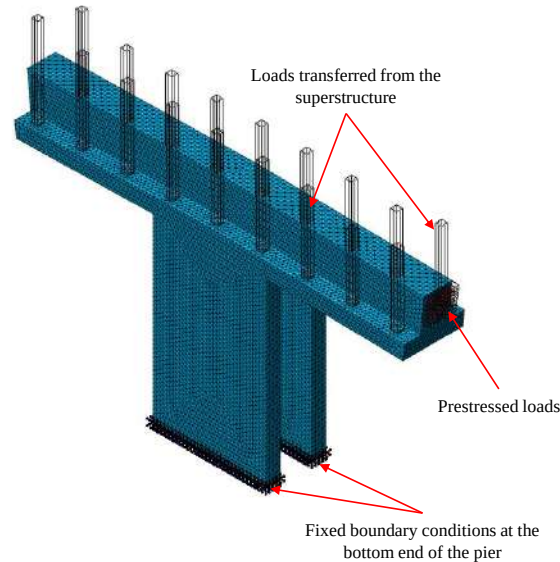


Fig. 10. Loads and boundary conditions

3.4 Numerical Validation and Analyses

Figs. 11 and 12 jointly illustrate the stress field and fracture response predicted by the fracture-based FE model. Fig. 11 shows the distribution of the first principal tensile stress in the concrete, revealing strong tensile stress concentrations at the cantilever–pier interface and beneath the bearing zones, which exhibit a strong spatial correlation with the cracking patterns observed in the field survey.

Fig. 12 presents the crack status predicted across the pier cap elements. In this representation, red regions denote fully open cracks, corresponding to zones undergoing tensile softening and progressive strength degradation. Blue regions indicate partially open cracks, where cracking has occurred but the material is in an unloading or reloading state, or within a transition regime of the fracture model. White regions represent uncracked elastic concrete, in which the tensile stress has not exceeded the cracking threshold. The presence of partially open cracks suggests that the prestressing force contributes to limiting crack opening by inducing localized unloading in the surrounding concrete, a behavior that is consistent with strain compatibility and tension-stiffening effects in prestressed members.

The FE analysis predicts two main regions of crack concentration: (i) the cantilever–pier interface and the bearing zones, where stress concentrations are expected to be high, and (ii) a more distributed cracking pattern along the pier cap stem. Field observations confirm the pronounced cracking at the pier cap ledge and bearing regions, whereas the stem cracking predicted by the model was observed less frequently, likely due to limited accessibility during on-site inspection.

Fig. 13 shows the predicted crack opening response under service loading, with a maximum crack width of 0.448 mm. This value is in good agreement with the field measurements, where most cracks were below 0.2 mm and only a single crack exceeded 0.5 mm, indicating that the fracture-based FE model captures the overall magnitude of crack opening reasonably well.

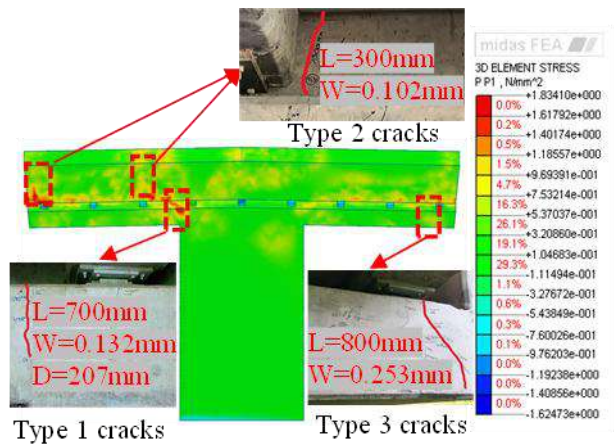


Fig. 11. First principal tensile stress distribution predicted by FE analysis

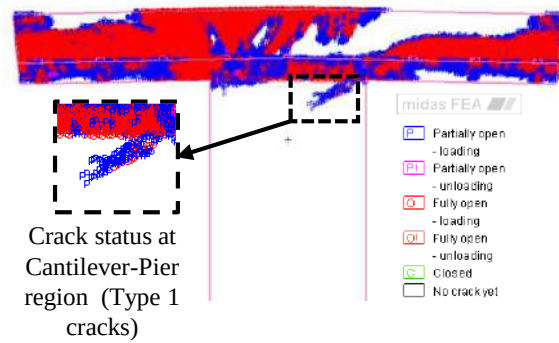


Fig. 12. Predicted crack status in the pier cap

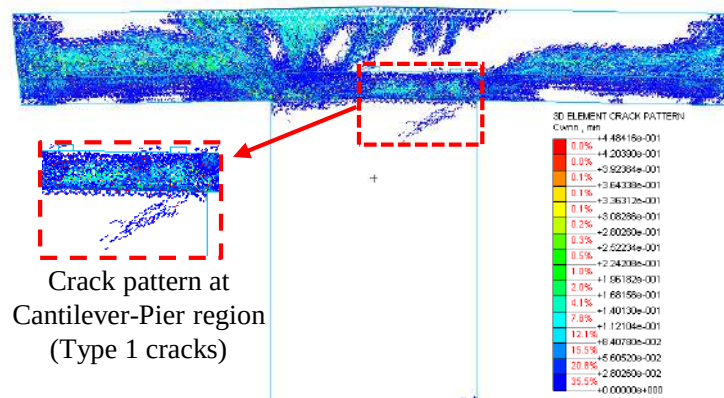


Fig. 13. Predicted crack opening width under service loading

Overall, the fracture-based FE analysis provides a physically consistent representation of crack initiation, localization, and opening in the PPC pier caps under service loading. While the model does not aim to reproduce every individual crack, it captures the dominant cracking zones and the upper bound of crack widths observed in the field, thereby offering a reliable framework for interpreting the measured crack patterns. This validated numerical representation forms a sound basis for assessing the adequacy of existing code-based crack control provisions, which is examined in the following section.

It should also be noted that the FE results represent a continuous crack-opening field throughout the concrete volume, including internal crack development, whereas the field survey is limited to discrete crack widths observable at the concrete surface. In addition, the field-measured cracks represent the accumulated structural response after construction and during service, including the effects of prestress transfer, self-weight, long-term restraint, creep, shrinkage, and traffic loading. Consequently, the FE model is not intended to reproduce the exact instantaneous in-situ crack state, but rather to capture the dominant crack localization mechanisms and upper-bound crack-opening behavior under service-level actions.

4. Crack Control According to Design Specification

The bridge investigated in this study was originally designed in accordance with TCN 272-05 [23], which is equivalent to the AASHTO LRFD bridge design specification [24]. In the present work, the code-based comparison is conducted using the ACI framework (ACI 318, ACI 224R, and ACI 350). This choice is motivated by the observation that the crack control provisions adopted in AASHTO LRFD are largely derived from, or calibrated against, ACI formulations. Accordingly, the ACI models provide a consistent and theoretically grounded basis for evaluating both the field measurements and the fracture-based FE predictions.

4.1 Background on Crack Control Provisions in the ACI Framework

Within the American design system, crack control in concrete structures is primarily governed by ACI 318 (Building Code Requirements for Structural Concrete). This standard is supported by a series of technical documents that provide the theoretical basis and interpretative guidance for cracking behavior, notably ACI 224R (Control of Cracking in Concrete Structures) and ACI 224.1R (Causes, Evaluation, and Repair of Cracks). For structures exposed to aggressive environments or requiring enhanced durability, more stringent crack control criteria are prescribed in ACI 350 (Code Requirements for Environmental Engineering Concrete Structures).

Together, these documents define the conceptual framework used in ACI-based design to limit cracking for durability and serviceability. However, the manner in which crack width is estimated and controlled within this framework has evolved significantly over time, particularly within ACI 318. This evolution is central to understanding the limitations of current crack control provisions when applied to partially prestressed concrete members.

4.1.1 Crack Control Philosophy in ACI 318

The crack control provisions in ACI 318 have undergone a fundamental conceptual shift over the past several decades. Prior to the 1999 edition, ACI 318-95 [25] adopted a direct crack width prediction approach based on the statistical model developed by Gergely and Lutz [26], in which the maximum surface crack width was expressed as a function of reinforcement stress and the geometry of the surrounding concrete. In this formulation, crack width was governed by the tensile strain in the reinforcement and the thickness and effective area of the concrete cover, reflecting a fracture-based view of cracking in reinforced and prestressed concrete:

$$w_{max} = 0.011 \times 10^{-3} \times \beta f_s^3 \sqrt{d_c A} \quad (1)$$

where f_s is the steel stress at service (MPa), d_c is the concrete cover to the nearest bar (mm), A is the effective tension area of concrete per bar (mm²), and β accounts for the strain gradient. Under ACI 318-95, allowable crack widths were specified as 0.33 mm (0.013 in.) for exterior exposure and 0.41 mm (0.016 in.) for interior exposure, directly linking serviceability to crack opening.

However, subsequent research by Frosch [27] demonstrated that explicit crack width formulas exhibit large statistical scatter and limited predictive reliability. As a result, ACI 318-99 [28] abandoned the direct crack width calculation and replaced it with a spacing-based crack control model, in which crack width is no longer treated as an explicit response variable but is instead indirectly controlled through reinforcement spacing. In this physical interpretation, the average crack width is approximated as

$$w_{max} = 2 \frac{f_s}{E_s} \beta \sqrt{d_c^2 + \left(\frac{s}{2}\right)^2} \quad (2)$$

leading to reinforcement spacing limits of the form

$$s = \frac{95000}{f_s} - 2.5c_c \leq 300 \left(\frac{252}{f_s}\right) \quad (3)$$

where s is the bar spacing and c_c is the concrete cover (mm). This approach implicitly assumes that limiting crack spacing is sufficient to control crack opening, without explicitly modeling fracture localization or tension softening.

This spacing-based philosophy remains the basis of crack control in ACI 318-19 [29], including for Class C partially prestressed members. For bonded prestressed reinforcement, the code limits the maximum spacing of steel closest to the tension face as

$$s = \frac{5}{6} \left[380 \left(\frac{280}{\Delta f_{ps}} \right) - 2.5c_c \right] \quad (4)$$

or

$$s = \frac{5}{6} \left[300 \left(\frac{280}{\Delta f_{ps}} \right) \right] \quad (5)$$

with the intent of maintaining crack widths below approximately 0.40 mm. Here, Δf_{ps} is the increase in prestressing steel stress beyond decompression (MPa).

4.1.2 Crack-Width Philosophy in ACI 224 and ACI 350

ACI 224R-01[30] provides a comprehensive framework for serviceability crack control by consolidating provisions from ACI 318, CEB-FIP, and Eurocode 2. In addition to code-based spacing rules, the document compiles several empirical and semi-mechanistic models for estimating maximum flexural crack widths in reinforced and prestressed concrete members. These include the formulations proposed by Meier and Gergely (1982) and by Nawy and Huang (1977) and Nawy (1989a), which are intended to relate crack width to steel stress, concrete cover, and effective tension area [31-33].

Meier and Gergely [31] proposed the following expressions for the maximum flexural crack width:

$$w_{max} = C_1 \frac{f_{ct}}{E_c} d_c \quad (6)$$

$$w_{max} = C_2 \frac{f_{ct}}{E_c} d_c \sqrt[3]{A} \quad (7)$$

where C_1 and C_2 are bond-related coefficients depending on the type of steel nearest the tension face; f_{ct} is the nominal tensile stress at the concrete tension (MPa); E_c is the concrete elastic modulus (MPa); d_c is the concrete cover to the centroid of the nearest reinforcement (mm); and A is the effective concrete tension area per bar (mm²) as defined in ACI 318 [28].

For prestressed members, ACI 224R-01 also includes simplified crack-width formulations derived by Nawy and Huang (1977) [32] and Nawy (1989a) [33], expressed in terms of the increase in steel stress beyond decompression:

Pretensioned members:

$$w_{max} = 5.85 \times 10^{-5} \frac{A_t}{\Sigma_0} \Delta f_s \quad (8)$$

Post-tensioned unbonded members:

$$w_{max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma_0} \Delta f_s \quad (9)$$

Where $\Delta f_s = f_{nt} - f_d$ is the increase in steel stress (ksi) relative to the decompression state, Σ_0 is the sum of reinforcement perimeters (in), and A_t is the effective concrete area in uniform tension (in²).

In the subsequent revision ACI 224.1R-07 [34], these empirical expressions were re-evaluated and largely superseded by the physically based crack model proposed by Frosch (1999), which relates crack width to reinforcement spacing and steel stress. ACI 350M-06 [35] adopts the same crack-width philosophy but introduces additional durability constraints by limiting the effective concrete cover used in crack-control calculations to a maximum of 50 mm.

4.1.3 Crack Control in AASHTO LRFD

The crack control provisions in the AASHTO LRFD Bridge Design Specifications closely follow the evolution of crack control philosophy in ACI 318. In the 2nd Edition (1998), AASHTO 1998 [24] adopted a stress-based crack width control approach derived from the Gergely-Lutz formulation [26], in which the allowable tensile stress in reinforcement was limited as a function of concrete cover and effective tension area in order to restrict surface crack widths to approximately 0.40 mm.

Beginning with the 3rd Edition, AASHTO 2004 [36] replaced this statistical crack-width formulation with the spacing-based physical model proposed by Frosch, consistent with the transition introduced in ACI 318. In the later version of AASHTO LRFD provisions [19, 37], crack control is governed primarily by the maximum spacing of reinforcement closest to the tension face, expressed as a function of steel stress, concrete cover, and an exposure factor. Crack width is no longer calculated explicitly but is assumed to be indirectly controlled through reinforcement spacing.

This evolution indicates that, as in ACI 318, AASHTO crack control for bridge structures has shifted from a fracture-based crack-width concept to a geometric spacing-based proxy, a simplification that may not fully capture the cracking behavior of partially prestressed bridge pier caps, where cracking, unloading, and re-opening are intrinsic to serviceability performance.

4.2 Comparison and Discussion

Table 4 summarizes the crack control verification of the investigated pier caps according to the maximum reinforcement spacing provisions in AASHTO LRFD and ACI 318. For all the investigated pier caps, the calculated bar spacing and steel stresses satisfy the code requirements. Under these provisions, compliance implies that surface crack widths should remain within approximately 0.40 mm at the serviceability limit state.

However, the direct crack-width evaluation presented in Table 5 for Pier P17 (the pier cap exhibiting the largest measured crack width) reveals a markedly different picture. Although this pier fully satisfies the spacing-based requirements of both AASHTO and ACI (Table 4), the fracture-based finite element model predicts a maximum crack width of 0.448 mm, while the field survey recorded a maximum surface crack width of 0.545 mm. Both values exceed the nominal serviceability limit of 0.40 mm, indicating that compliance with spacing-based crack control does not necessarily guarantee satisfactory crack performance for these partially prestressed pier caps. It is noted that the reported field maximum corresponds to the largest accessible crack, and additional cracking at the top surface of the pier cap may not have been fully captured.

Although cracks wider than 0.5 mm represent only a very small fraction of the total crack population (approximately 0.2% according to the distribution in Fig. 6), their occurrence is critical from a durability and serviceability standpoint. Crack control provisions in AASHTO and ACI are intended to limit the maximum crack width, rather than the average or most probable crack. The presence of even a single crack exceeding 0.5 mm in a pier cap that fully satisfies all code requirements indicates that the upper tail of the crack-width distribution is not adequately controlled by current spacing-based provisions. Nevertheless, several additional cracks exhibited widths close to the 0.5 mm threshold, and no abnormal cracking characteristics or visible construction deficiencies were identified in the corresponding pier caps during the field survey. This suggests that the occurrence of relatively large cracks is associated with localized structural response rather than with an isolated measurement anomaly.

In contrast, most of the traditional crack-width formulas summarized in Table 5, including those of Gergely–Lutz [26], Frosch [27], and Nawy & Huang [32], predict crack widths between 0.219 mm and 0.294 mm, well below the observed and numerically predicted values. This systematic underestimation is consistent with the empirical nature of these models, which were primarily calibrated using laboratory beams with relatively uniform stress fields and limited restraint, conditions that differ significantly from the deep, highly restrained, and prestressed pier caps examined in this study.

The only formulations that do not underestimate cracking are those proposed by Meier and Gergely [31], which predict crack widths of 1.097 mm and 2.270 mm, respectively. These values substantially exceed both the FEA and field measurements. This discrepancy is likely related to the fact that the Meier and Gergely equations were calibrated using laboratory beam tests characterized by moderate stress levels and distributed flexural cracking. In contrast, the investigated PPC pier caps exhibit highly restrained behavior and localized tensile stress concentrations near the crack localization zones. Based on elastic transformed-section analysis, the equivalent tensile stress associated with service-level cracking reaches approximately 32.3 MPa,

which lies well beyond the stress range represented in the original calibration database [31]. Consequently, direct application of these empirical equations to the investigated PPC pier caps may lead to unrealistic crack-width predictions.

More generally, the comparison between numerical predictions, code-based evaluations, and field observations should be interpreted within the limitations associated with in-service cracking measurements. Although the field-measured cracks represent accumulated in-service behavior rather than an instantaneous loading condition, the comparison remains valuable because both the FE analysis and the code-based crack-control checks were performed using consistent serviceability load combinations. The comparison therefore aims to assess whether current crack-control provisions can reasonably represent the magnitude and localization tendency of cracking observed in real PPC pier caps.

Taken together, the results in Tables 4 and 5 demonstrate a fundamental inconsistency between spacing-based crack control rules and the actual cracking behavior of partially prestressed pier caps. While current AASHTO and ACI provisions indicate full compliance, both field observations and fracture-based finite element simulations reveal crack widths that approach or exceed serviceability limits. This discrepancy highlights the limitations of indirect crack control based solely on reinforcement spacing when applied to large, deep, and highly restrained PPC members, and supports the need for fracture-informed or mechanics-based approaches to evaluate serviceability performance in such structures.

Table 4. Crack control verification according to ACI 318 and AASHTO LRFD

		Piers below Link Slab	Piers below Expansion Joint	P39
Reinforcement	Diameter (mm)	32	28	32
	Space (mm)	125	125	125
	Number	23	21	11
	d_c (mm)	84	84	84
	A (mm ²)	16663	18250	18273
	Z (N/mm)	23000	23000	23000
	M (kN.m)	30695	30584	26406
Service limit state	f_s (MPa)	209.1	217.6	185.5
	AASHTO 1998	f_{sa} 240	237	237
	Check	Compliant	Compliant	Compliant
		β_s 1.046	1.048	1.028
	AASHTO 2007	γ_e 1	1	1
	s_{max} (mm)	394	371	477
	Check	Compliant	Compliant	Compliant
	ACI 318-99	s_{max} (mm) 244	227	302
	Check	Compliant	Compliant	Compliant
	ACI 318-19	s_{max} (mm) 249	232	303
	Check	Compliant	Compliant	Compliant

Table 5. Comparison of crack width predictions and measured values for Pier P17

	Gergely and Lutz approach ACI 318- 95	Frosch approach ACI 318- 99	Meier and Gergely Equation (6)	Meier and Gergely Equation (7)	Nawy and Huang	Maximum crack width by FEM	Maximum crack width by on-site inspection
w (mm)	0.268	0.219	1.097	2.270	0.294	0.448	0.545

5. Conclusions

This study investigated the cracking behavior of partially prestressed concrete (PPC) pier caps based on an extensive field survey of 113 in-service bridge pier caps, complemented by nonlinear fracture-based finite element analysis and code-based crack control evaluation. The following conclusions can be drawn:

- Field inspections revealed that cracking is widespread in PPC pier caps designed in accordance with AASHTO LRFD and ACI provisions, with 87 out of 113 pier caps exhibiting visible cracks under service conditions. Although most cracks exhibited relatively small surface widths, the maximum measured crack width reached 0.545 mm, and crack depths consistently exceeded the nominal concrete cover thickness of 50 mm, indicating a potential durability concern.
- Three dominant crack patterns were identified: cracks at the cantilever–pier interface, cracks beneath bearing zones, and cracks in the pier cap stem. Among these, the cantilever–pier interface acts as the primary crack localization zone, accounting for the majority of observed cracks and the largest crack widths, highlighting the critical role of restraint and force transfer in PPC pier caps.
- The fracture-based finite element model successfully reproduced the dominant cracking zones observed in the field and predicted a maximum crack width of 0.448 mm under service loading. While the model does not aim to capture every individual crack, it provides a physically consistent representation of crack initiation, localization, and opening, and offers a reliable framework for interpreting field observations.
- Code-based crack control checks according to AASHTO LRFD and ACI 318 indicate full compliance for all investigated pier caps when assessed using reinforcement spacing limits. However, both field measurements and numerical simulations indicate that localized crack widths may approach or, in isolated cases, exceed the commonly adopted serviceability limit of 0.40 mm despite full code compliance. These findings suggest that spacing-based crack control may not always provide a reliable representation of crack localization behavior in PPC pier caps.
- Comparisons with empirical crack-width formulations compiled in ACI 224 show that most traditional models tend to underestimate crack widths for the investigated PPC pier caps, whereas some formulations calibrated outside the relevant stress range significantly overestimate cracking. These discrepancies reflect the limitations of empirical crack-width models when applied to large, deep, and highly restrained PPC members operating at high service-level stresses.

Overall, the results suggest that the cracking behavior of PPC pier caps may differ significantly from the assumptions embedded in current code-based crack control provisions. The combination of crack localization, deep crack penetration, and prestress-induced unloading cannot be fully captured by reinforcement spacing rules alone. These findings support the need for more mechanics-based approaches to assess serviceability and durability in partially prestressed bridge substructures.

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Bond behavior of deformed steel bars in fly ash-based geopolymer concrete: Test results and development-length implications

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Abstract

Fly ash geopolymer concrete (GPC) is a low-carbon alternative to Portland cement concrete. A reliable bond between steel reinforcement and GPC is essential for structural design. This study investigates the bond behavior of deformed steel bars embedded in fly ash GPC. The GPC mixtures are activated with a sodium hydroxide-sodium silicate solution and heat-cured at 60 °C for 24 h. Pull-out tests are conducted to obtain bond stress-slip curves. The peak bond stress, the slip at peak, and the residual bond stress are reported. The influence of compressive strength and confinement conditions is examined. The test results are compared with common bond models developed for Portland cement concrete. The comparisons show systematic differences in peak bond and post-peak softening. A simple calibration is proposed to adapt existing models for fly ash GPC. The calibrated model is used to discuss development-length implications for reinforced GPC members.

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1. Introduction

Concrete remains the most widely used structural material, but recent development is increasingly driven by decarbonization, durability, and performance-based design. As research expands from conventional OPC to low-carbon binders (including geopolymer/alkali-activated systems) and high-performance matrices such as UHPC, there is a growing need to reassess bond and anchorage behavior beyond OPC-based assumptions [1,2]. In particular, changes in matrix composition and cracking response may alter bond stress transfer and splitting sensitivity, which motivates the present investigation on fly ash-based geopolymer concrete (GPC) [3,4]. GPC is a promising low-carbon alternative to ordinary Portland cement concrete (OPC) [5-7]. GPC replaces Portland clinker with an aluminosilicate precursor that is activated by alkaline solutions. It can reduce embodied CO₂ and can valorize industrial by-products [8]. It can also achieve compressive strengths that are suitable for structural applications [9]. These advantages support increasing interest in using GPC in reinforced concrete members [10-12].

Structural use of GPC requires reliable design information [13,14]. The bond between deformed steel bars and the surrounding concrete is a governing mechanism. Bond controls govern the transfer of load from steel to concrete. Bond also controls crack spacing and crack width. Bond determines development length and lap splice length. Bond affects stiffness and rotation capacity in flexural members. A reinforced member can be unsafe if bond demand exceeds bond capacity. A reinforced member can be uneconomical if the development length is overestimated [15].

Bond in OPC is well studied [16,17]. Bond resistance is commonly described by a bond stress-slip relationship. The relationship reflects adhesion, friction, and mechanical interlock of ribs. The

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relationship depends on concrete strength and cover. The relationship depends on bar diameter and bar surface geometry. The relationship depends on confinement from transverse reinforcement or external pressure. The relationship also depends on casting position and local bleeding effects. Bond failure in OPC is typically classified as pull-out failure or splitting failure [18]. Pull-out failure occurs when ribs crush the surrounding concrete and the bar is extracted with limited radial cracking. Splitting failure occurs when radial stresses induce longitudinal cracks, leading to cover separation [19]. Splitting failure is common when the cover is small or the confinement is weak. Many design codes embed this knowledge through empirical rules and simplified bond models [20]. Several widely used analytical bond laws and code expressions were calibrated on OPC databases [21,22]. These models are widely adopted for development length and lap splice provisions. They also support nonlinear analysis via τ - s laws and tension-stiffening models.

The direct use of OPC-based bond models for GPC is not necessarily justified [23]. GPC differs from OPC in binder chemistry and microstructure. GPC often exhibits a different elastic modulus at a given compressive strength. GPC can show different pre-peak nonlinearity and post-peak softening in compression. GPC can also exhibit different interfacial transition zones and pore structures, depending on precursor type, activator composition, and curing regime. These differences can influence rib bearing, concrete crushing around ribs, and splitting resistance. The failure mode can shift with curing and mixture design. The post-peak bond response can also change due to different fracture processes and different stiffness of the surrounding matrix.

Published studies on bond in geopolymer and alkali-activated concretes show mixed trends [24], [25,26]. Some studies report peak bond stress comparable to or higher than OPC at similar compressive strength [27,28]. Some studies report lower bond capacity under certain curing regimes or for specific precursors. Many results show that confinement strongly influences the failure mode in GPC, as it does in OPC. Several studies indicate that post-peak softening and residual bond stress can differ from OPC [29,30]. However, the available evidence remains fragmented. Many studies use different precursors and different activators. Many studies use different curing temperatures and durations. Many studies use different specimen geometries and embedment lengths. They report only peak bond stress without full τ - s curves. Many studies do not provide sufficient information for design calibration. As a result, there is no widely accepted bond model for fly ash-based GPC that can be used with confidence in reinforced concrete design [31,32].

This gap is relevant for practical implementation. Fly ash-based GPC commonly requires heat curing to achieve stable strength development in many formulations [33]. Heat curing can densify the matrix and can modify early-age cracking behavior. Heat curing can also influence shrinkage and microcracking near the steel interface. These effects can directly influence bond and splitting resistance. Design provisions for anchorage and splicing typically assume OPC-based behavior. A systematic bias in bond capacity would translate into a systematic bias in development length. This bias can be unconservative if the bond is overestimated. This bias can also be overly conservative if the bond is underestimated. Both outcomes are undesirable. Unconservative design threatens safety. Overly conservative design penalizes material adoption and increases reinforcement congestion.

A focused experimental dataset is therefore needed. The dataset should target a well-defined material system. The system should use fly ash as the precursor. The system should use a sodium hydroxide-sodium silicate activator, which is common in practice. The system should employ a curing regime that is typical for fly ash GPC. The dataset should cover a practical strength range for structural applications. The dataset should include full bond stress-slip curves. The dataset should document failure modes. The dataset should also allow comparison against conventional OPC-based bond models and code expressions.

This paper addresses these needs. The study investigates the bond behavior of deformed steel bars embedded in fly ash-based geopolymer concrete [34]. The geopolymer mixtures are activated by a sodium hydroxide-sodium silicate solution. The specimens are heat-cured under a controlled temperature regime. Pull-out tests are performed to obtain complete bond stress-slip curves. The study reports peak bond stress and slip at peak. The study reports residual bond stress at larger

slips. The study classifies failure modes as pull-out or splitting. The study examines the influence of compressive strength and confinement conditions, as permitted by the test matrix. The study then compares the measured bond response with representative bond models that were originally developed for OPC. The comparison focuses on peak bond capacity and post-peak softening characteristics. The study finally proposes a simple calibration approach. The approach adapts an existing OPC-based bond model to fly ash GPC through a small number of parameters. The calibrated model is then used to discuss development-length implications for reinforced fly ash GPC members.

2. Experimental Program

2.1 Materials and Mixture Design

This study used low-calcium fly ash to produce fly ash-based geopolymer concrete (GPC). The fly ash was obtained from Pha Lai thermal power plant (Vietnam). The fly ash satisfied the requirements for Class F fly ash. Its chemical composition is summarized in Table 1. The main oxide components were SiO_2 , Al_2O_3 , and Fe_2O_3 , while the CaO content was low, which is consistent with the classification of the material as a low-calcium Class F fly ash [35].

Table 1. Chemical composition of the fly ash used in this study

Component	Content (mass %)
SiO_2	51.74
Al_2O_3	24.53
Fe_2O_3	5.59
CaO	0.81
Loss on ignition (LOI)	8.98

The alkaline activator consisted of sodium hydroxide (NaOH) and sodium silicate (Na_2SiO_3). The main properties of the activator components are summarized in Table 2. Solid NaOH flakes with a purity of 98% were dissolved in water to prepare solutions with target molarities of 12 M, 14 M, and 16 M. The sodium silicate solution had a silicate modulus of $\text{SiO}_2/\text{Na}_2\text{O} = 2.5$. The mass ratio $\text{Na}_2\text{SiO}_3/\text{NaOH}$ was kept constant at 2.5 for all mixtures

Table 2. Main properties of the alkaline activator used in this study

Component	Parameter	Value
NaOH flakes	Purity	98%
NaOH solution	Target molarity	12 M, 14 M, 16 M
NaOH solution	NaOH required for 1 kg of solution	361 g (12 M), 404 g (14 M), 444 g (16 M)
Na_2SiO_3 solution	Silicate modulus, $\text{SiO}_2/\text{Na}_2\text{O}$	2.5
Na_2SiO_3 solution	Na_2O content (mass %)	11.8
Na_2SiO_3 solution	SiO_2 content (mass %)	29.5
Na_2SiO_3 solution	Water content (mass %)	58.7
Combined activator	$\text{Na}_2\text{SiO}_3/\text{NaOH}$ mass ratio	2.5

A conventional aggregate skeleton was adopted. The aggregate content was selected to be comparable to ordinary concrete. The target fresh unit weight was 2400 kg/m^3 . The total aggregate mass was taken as 1848 kg/m^3 . Fine aggregate was set to 30% of the total aggregate mass. The sand content was therefore 554.4 kg/m^3 . Mixture design focused on two key parameters. The first parameter was NaOH molarity (M). The second parameter was the activator-to-fly ash mass ratio (Alkaline Activator Solution - AAS/FA). The investigated range of AAS/FA was 0.40–0.50. The ratio was selected because the aggregates were used in a dry condition. Lower ratios produced mixtures that were too stiff for mixing and casting. Three mixtures were selected for structural-strength levels. The mixtures targeted nominal compressive strengths of 30, 40, and 50 MPa. The mixtures are denoted as G30, G40, and G50. $\text{Na}_2\text{SiO}_3/\text{NaOH}$ was fixed at 2.5. NaOH molarity was 12 M, 14 M,

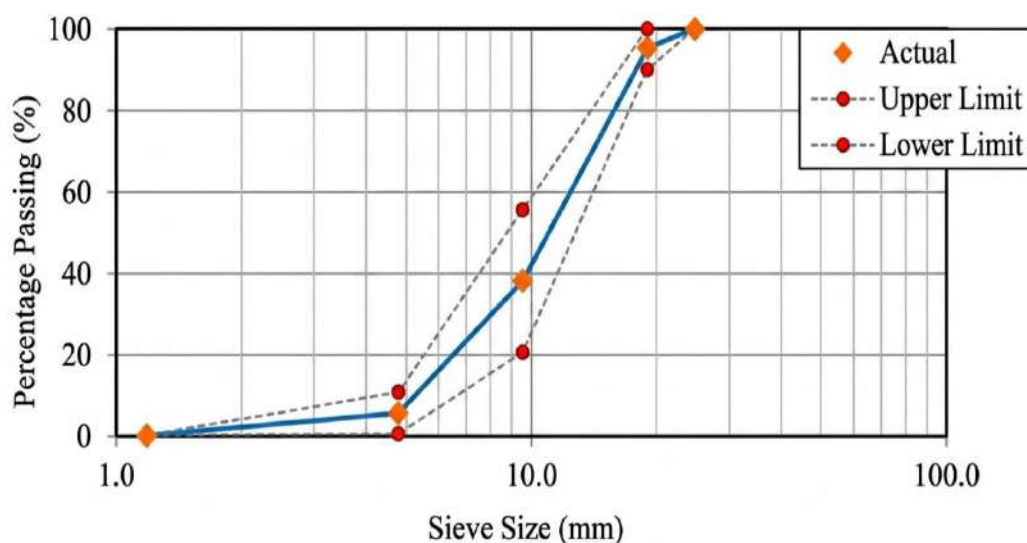
and 16 M for G30, G40, and G50, respectively. All specimens were heat cured at 60 °C for 24 h. The mixture proportions for the three GPC mixtures (G30, G40, and G50) are summarized in Table 3.

Table 3. Mixture proportions of fly ash-based GPC (kg/m³)

Component	G30	G40	G50
Fly ash	372.52	375.84	390.35
Fine aggregate (sand)	554.0	554.0	554.0
Coarse aggregate 2.36–4.75 mm	64.68	64.68	64.68
Coarse aggregate 4.75–9.50 mm	420.42	420.42	420.42
Coarse aggregate 9.50–19.0 mm	743.82	743.82	743.82
Coarse aggregate 19.0–25.0 mm	65.68	65.68	65.68
NaOH solution	51.28 (12 M)	50.33 (14 M)	46.18 (16 M)
Na ₂ SiO ₃ solution	128.20	125.83	115.46
Na ₂ SiO ₃ /NaOH (mass ratio)	2.5	2.5	2.5
AAS/FA (mass ratio)	0.4818	0.4687	0.4141
Heat curing	60 °C, 24 h	60 °C, 24 h	60 °C, 24 h

All specimens were heat-cured at 60 °C for 24 h. This regime was not intended to represent a universal optimum for all Class F fly ash geopolymers; rather, it was chosen as a practical curing condition for the material system adopted in this study. Previous studies have shown that heat curing for about 24 h is commonly used for fly ash-based geopolymer concrete, and that 60 °C can provide effective early strength development, while higher temperatures mainly accelerate the reaction. In the experimental basis underlying the present mixtures, curing at 60 °C for 24 h produced strength levels close to those obtained at 80 °C and 100 °C after the same curing duration, with lower energy demand. For this reason, 60 °C for 24 h was selected for all specimens in the present study. The granulometric distributions of the coarse and fine aggregates used in this study are presented graphically in Figure 1. The coarse aggregate grading was selected to satisfy ASTM C33, while the fine aggregate grading was determined based on sieve analysis in accordance with ASTM C136.

As shown in Table 4, all mixtures were designed as normal-density geopolymer concretes with the same target fresh unit weight. The principal difference among the three mixtures was their compressive strength, which increased from G30 to G50. Therefore, the variation in bond behavior observed in this study is interpreted mainly as a consequence of matrix-strength development rather than of density variation.



(a)

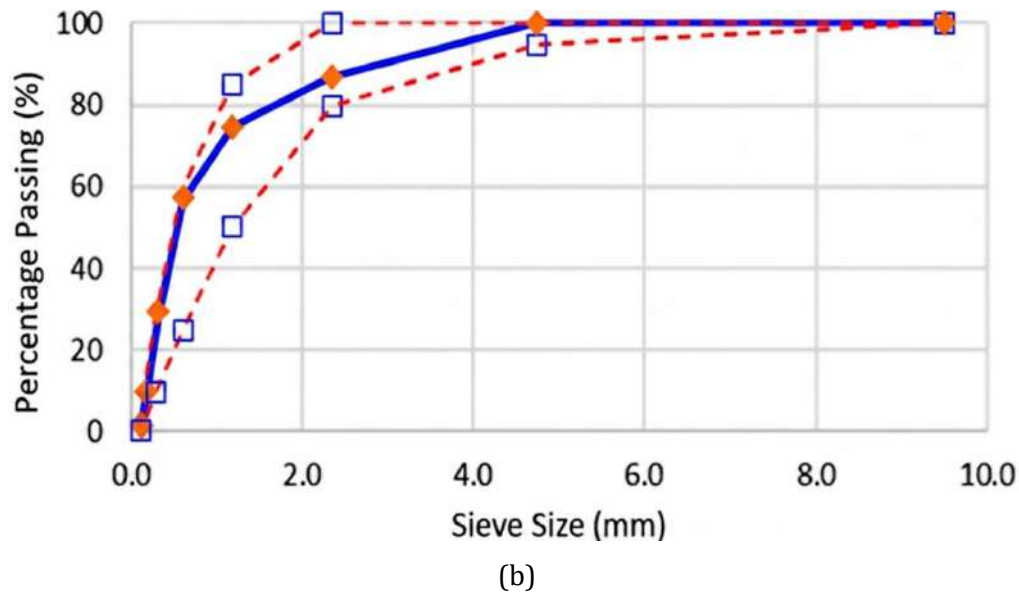


Fig. 1. Aggregate granulometry: (a) coarse aggregate grading; (b) fine aggregate grading

Table 4. Basic physical and mechanical characteristics of the tested GPC mixtures

Mixture	Target fresh unit weight, γ_t (kg/m ³)	Reference normal-density range, ρ_{ref} (kg/m ³)	Mean compressive strength, f_{cm} (MPa)	Characteristic compressive strength, (f_c) (MPa)
G30	2400	2360 ± 60	39.46	31.67
G40	2400	2360 ± 60	49.86	40.46
G50	2400	2360 ± 60	59.81	52.06

Note: The target fresh unit weight was fixed at 2400 kg/m³ for all mixtures. A typical hardened density of approximately 2360 ± 60 kg/m³ has been reported for fly ash-based geopolymer concrete with granite aggregates after heat curing at 60 °C for 24 h. Direct hardened-density measurements of the present batches were not reported; therefore, the density information is presented here as a reference normal-density range rather than as measured values.

2.2 Specimen Details

Pull-out specimens were prepared in accordance with the specimen concept recommended in EN 10080:1995. The concrete specimen was a cube with a side length of 200 mm. A single deformed reinforcing bar was embedded concentrically in the cube. The reinforcing bar had a nominal diameter of 22 mm ($\phi 22$). The bar grade was CB400V steel.

Table 5. Pull-out specimen geometry and test matrix

Item	Value / description
Concrete specimen	200 × 200 × 200 mm cube
Reinforcing bar	Deformed bar, $\phi 22$ mm, grade CB400V
Bonded length, lb	5db = 110 mm
Debonding method	Plastic sleeve on the non-test portion of the embedded bar
Number of specimens	18 total
Specimens per mixture	6 per mixture (G30, G40, G50)
Reference procedure	EN 10080:1995 specimen concept

The bonded (anchorage) length was set to five times the bar diameter. The bonded length was therefore 110 mm. The remaining embedded bar length inside the concrete was intentionally debonded using a plastic sleeve. This arrangement ensured that bond stresses were mobilized only over the target bonded zone. The protruding bar length above the concrete surface was reserved for gripping and for slip measurement. A total of 18 pull-out specimens were cast. Six specimens

were prepared for each geopolymer mixture (G30, G40, and G50). The corresponding compressive strength levels used in the bond tests were within the intended range for the three mixtures (Table 4).



Fig. 2. Schematic of pull-out specimen

2.3 Test Setup and Instrumentation

Pull-out tests were performed using a direct tension configuration consistent with the EN 10080:1995 specimen concept. The concrete cube was positioned on a rigid steel support plate. A circular hole was provided at the center of the plate. The hole diameter was set to $2d_b$. Fig. 3 presents a schematic diagram of the pull-out specimen used in the bond-strength test. The specimen consisted of a $200 \times 200 \times 200$ mm concrete cube with a centrally embedded $\phi 22$ deformed steel bar, in which the bonded length was limited to $5d_b$ (110 mm) and the remaining embedded portion was debonded by means of a plastic sleeve. The figure is intended to provide a clear representation of the specimen geometry and the main parameters governing the test.

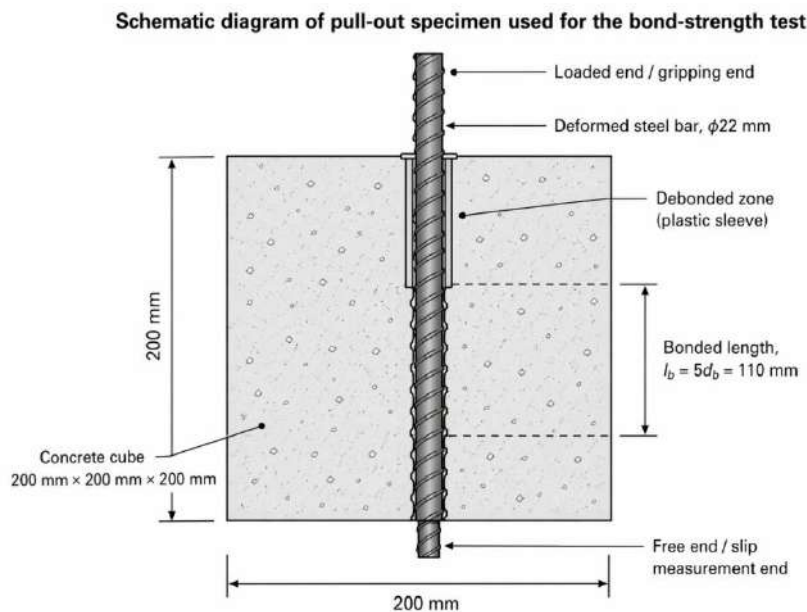


Fig. 3. Schematic diagram of the pull-out specimen used for the bond-strength test



Fig. 4. Test setup and instrumentation

The reinforcing bar passed through the hole and was clamped by the testing machine grips. The tensile load was applied at the longer protruding end of the bar. The slip was measured at the shorter end of the specimen, which represents the loaded-end slip definition adopted in the test procedure. During testing, the concrete cube was placed on a rigid steel support plate. A circular hole was provided at the center of the plate to allow the reinforcing bar to pass through without bearing directly on the plate. The hole diameter was $2d_b$. The loaded end of the protruding reinforcing bar was clamped by the grips of the testing machine, while the concrete specimen remained supported on the steel plate. Therefore, when tensile load was applied by the machine, the force was transferred to the embedded bar and bond stresses were mobilized along the bonded length inside the concrete. The slip was measured at the free end of the specimen, corresponding to the shorter protruding end of the reinforcing bar below the concrete cube. The loading rate was defined as $V = 0.56 d^2$. For $d_b=22$ mm, the corresponding rate was $V=271$. The applied load was recorded continuously by the testing machine.

2.4 Data Processing

The test data consisted of the applied tensile force P and the measured bar slip s . The bond stress was computed by assuming uniform bond stress along the bonded length. The bond stress τ was calculated as:

$$\tau(s) = \frac{P(s)}{A_b} = \frac{P(s)}{\pi d_b l_b} \quad (1)$$

where A_b is the bonded surface area of the bar, $d_b=22$ mm is the bar diameter, and $l_b=5d_b=110$ mm is the bonded length. The τ - s curve was obtained by pairing $\tau(s)$ with the measured slip s throughout the loading history.

A characteristic bond stress was extracted at $s = 1$ mm. This value was used because, for normal concrete, the characteristic slip s_1 is approximately 1 mm in the MC90 bond model. In this study, the average bond force F_b recorded at $s = 1$ mm was used to compute the corresponding bond stress. The peak bond stress $\tau_{\max}$ was taken as the maximum value on the measured τ - s curve. The slip at peak, $s(\tau_{\max})$, was recorded for each specimen. For comparison with conventional OPC-based design, the reference peak bond stress in “good” bond conditions was computed using the CEB-FIP MC90 expression:

$$\tau_{\max,MC90} = 2\sqrt{f_{ck}} \quad (2)$$

where f_{ck} is the characteristic compressive strength.

3. Results and Discussion

3.1 Failure Modes

Two principal bond failure modes are commonly reported in pull-out-type tests. One mode is pull-out failure. The other mode is splitting failure. Bond deterioration progresses in stages. The first stage is the loss of chemical adhesion at the steel-concrete interface. The second stage is the formation of transverse microcracks in the surrounding concrete. The final stage is the development of radial stresses at the ribs. These radial stresses can exceed the tensile capacity of concrete and trigger splitting.

In the present test series, all specimens failed by concrete splitting in the bonded region. The concrete cube split into two parts along the bar line. The failure was sudden. The behavior was brittle. This observation is consistent with the failure pattern reported for the test specimens.



Fig. 5. Failure modes observed after testing

3.2 Bond Stress-Slip Response (τ -s Curves)

The bond stress-slip response was derived from the measured pull-out force-slip curves of the three mixtures. The response is expressed in terms of the average bond stress τ and the measured bar slip s . The τ -s curves show an initial ascending branch. This branch corresponds to the progressive loss of chemical adhesion and the mobilization of rib-bearing and frictional surfaces. The bond stress then approaches a peak value $\tau_{\max}$. The peak is reached after a limited amount of slip.

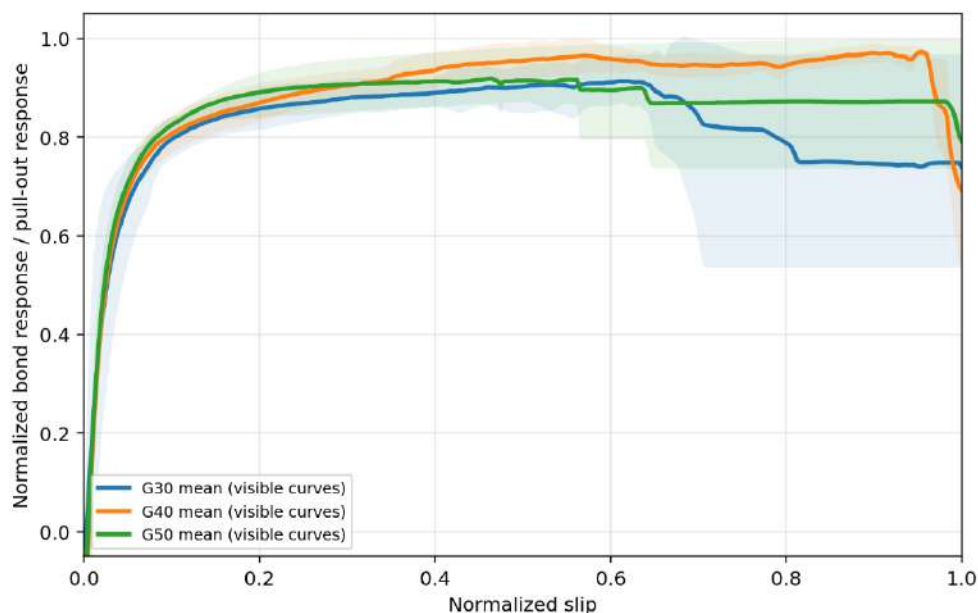


Fig. 6. Mean normalized bond stress-slip response curves of G30, G40, and G50 mixes

After the peak, the response softens. The softening is abrupt because the governing failure mode is concrete splitting. The post-peak branch, therefore, exhibits a steep drop in bond stress with increasing slip. A characteristic bond level was evaluated at $s=1\text{mm}$. This reference slip was adopted because it is commonly used in the MC90 bond model for ordinary concrete. The bond stress at this slip provides a consistent basis for comparing mixtures and for comparing fly ash GPC against OPC-based bond models.

3.3 Effect of Key Parameters

The test results clearly show that the bond response of the fly ash-based geopolymer concrete improved with increasing compressive strength. The average bond stress at a slip of 1 mm increased from 16.07 MPa for G30 to 16.69 MPa for G40 and 18.57 MPa for G50, while the corresponding compressive strengths increased from 31.67 MPa to 40.46 MPa and 52.06 MPa, respectively. This trend indicates that a higher concrete strength enhanced the resistance of the surrounding matrix against the local bearing stresses generated by the ribs of the deformed bar, thereby delaying the onset of longitudinal cracking and increasing the attainable bond level. Although the increase was not perfectly proportional to compressive strength, the overall tendency was consistent and confirms that the compressive strength of geopolymer concrete is an important parameter governing bar–concrete interaction in pull-out type bond tests.

The observed trend is in good agreement with previous studies on reinforced fly ash-based geopolymer concrete. Sarker reported that the bond strength of reinforcing steel embedded in fly ash-based geopolymer concrete was higher than that of comparable OPC and that bond resistance increased with concrete strength [36]. More recently, Yuan et al. [27] also found that the ultimate bond strength of steel bars in unconfined fly ash-based geopolymer concrete increased with increasing concrete compressive strength. Taken together with the present results, these findings suggest that the beneficial effect of compressive strength on bond is not limited to OPC but is also characteristic of fly ash-based geopolymer systems. This may be attributed to the denser geopolymer matrix and its stronger resistance to local splitting stresses around the ribs, which improve stress transfer at the steel–concrete interface.

At the same time, the present tests showed that all specimens failed by splitting rather than by a pure pull-out mechanism. This indicates that the ultimate bond resistance was governed primarily by the tensile capacity of the surrounding concrete cover and not solely by the adhesion and mechanical interlock at the steel surface. Once the radial stresses induced by rib bearing exceeded the splitting resistance of the unconfined concrete, longitudinal cracks propagated rapidly along the embedded bar, leading to a sudden post-peak degradation in bond stress. This interpretation is consistent with the failure patterns reported in previous bond studies on geopolymer concrete, where splitting has been identified as the dominant failure mode under limited confinement. Therefore, the present increase in bond stress with compressive strength should be understood as an increase in splitting-controlled bond capacity rather than as evidence of a fundamental change in the failure mechanism.

3.4 Comparison with OPC-Based Models

A comparison with previous studies indicates that the present bond levels are realistic for fly ash-based geopolymer concrete and are not anomalously high. The measured bond stresses exceeded the MC90 reference values for good bond conditions by approximately 13–43%, depending on the concrete strength level. This result supports the view that bond models originally developed for OPC may be conservative when applied to geopolymer concrete. A similar conclusion was drawn by Sarker [36], who showed that the bond strength of reinforcing steel in fly ash-based geopolymer concrete was higher than that in comparable OPC and suggested that existing OPC-based expressions could be conservatively used for geopolymer concrete. Recent studies and reviews have also emphasized that bond behavior in geopolymer concrete is often at least comparable to, and in many cases better than, that of OPC systems, particularly when the geopolymer matrix exhibits high splitting tensile resistance and a dense interfacial structure [25,27,32].

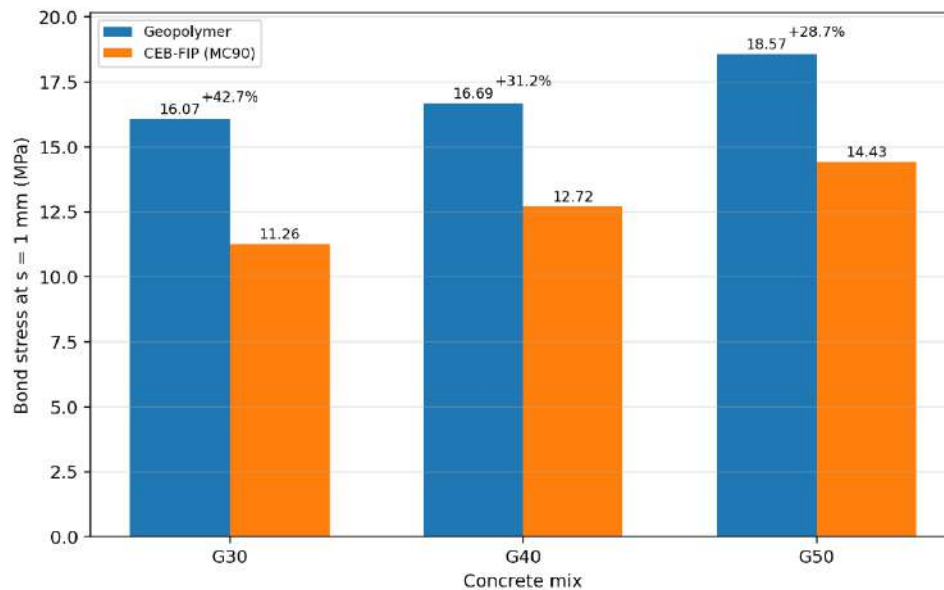


Fig. 7. Bond stress versus compressive strength for geopolymer concrete compared with the CEB-FIP Model Code 1990 (MC90)

The higher bond performance of geopolymer concrete reported in the literature has been linked to microstructural features of the geopolymer matrix. Compared with OPC, fly ash-based geopolymer concrete may develop a denser binder phase and a more compact interfacial transition zone, which enhance the transfer of shear and bearing stresses from the ribs of the bar to the surrounding matrix. However, the present results also show that a higher bond stress does not necessarily imply a more ductile bond response. Because all specimens failed by splitting, the bond capacity remained strongly dependent on the tensile resistance of the concrete cover and on the confinement condition. In this sense, the present results are consistent with the broader literature: geopolymer concrete can exhibit high bond resistance, but the governing failure mode may still be brittle when confinement is insufficient.

From a design perspective, the present results suggest that MC90 provides a conservative estimate of bond strength for the tested fly ash-based geopolymer concrete, and this conservatism explains why the development lengths derived from the experimental bond stresses are shorter than those obtained from the code-based prediction. Nevertheless, this implication should be interpreted with caution. The present study used a single bar diameter, a fixed bonded length, and unconfined pull-out specimens that consistently failed by splitting. Therefore, the reduction in calculated development length should not be generalized to all geopolymer concretes or all anchorage conditions. Rather, the main implication of the present results is that fly ash-based geopolymer concrete may warrant a dedicated calibration of bond and development-length expressions, especially for cases where the cover, confinement, and curing regime are similar to those adopted in the present experimental program.

4. Model Development and Design Implications

The significance of the present model development lies in converting the experimental bond results into a form that is directly usable for structural analysis and preliminary design. Experimental bond stress-slip curves provide important physical insight, but by themselves they remain specific to the tested materials, geometry, and curing regime. A calibrated bond model is therefore needed to represent the measured response in a compact analytical form, so that it can be used to estimate bond resistance, compare fly ash-based geopolymer concrete with OPC-based assumptions, and discuss practical implications for anchorage and development length. In this study, the objective is not to propose a fully general design law for all geopolymer concretes, but rather to provide a simple calibration of an established OPC-based model that captures the main features of the measured bond response of the investigated fly ash GPC.

4.1 Baseline Model and Calibration Objective

This study adopts the CEB–FIP MC90 bond framework as a baseline model. The baseline peak bond stress for good bond conditions is computed as $\tau_{\max,MC90} = 2\sqrt{f_{ck}}$. The experimental bond stress is evaluated from the pull-out force and the bonded surface area. The bond stress at $s=1$ mm is selected as a characteristic value because MC90 commonly uses $s \approx 1$ mm for ordinary concrete.

The test results show that the bond stress at $s=1$ mm for fly ash GPC is higher than the MC90 prediction. The manuscript-level summary indicates an increase of about 13–43% relative to MC90. This systematic bias motivates a simple calibration factor that preserves the MC90 functional form and improves agreement for fly ash GPC.

4.2 Proposed Calibrated Bond Level for Fly Ash GPC

A practical calibration is introduced through three parameters (a, b, k). The parameter a scales the bond stress level. The parameter b scales the slip axis. The parameter k scales the post-peak level. The calibrated law is written in a compact form as:

$$\tau_{GPC}(s) = a\tau_{MC90} \frac{s}{b} \quad (3)$$

and

$$\tau_{GPC,post}(s) = k\tau_{GPC}(s) \text{ for the descending branch} \quad (4)$$

The available dataset provides robust information on bond stress levels at $s=1$ mm and at peak. The dataset does not provide sufficient landmark points to uniquely re-fit the full curve shape for each mixture. Therefore, the calibration focuses on the stress level. The slip and post-peak shape are preserved. This choice is consistent with using MC90 as a baseline and applying a design-friendly correction.

Table 6. Proposed calibration parameters and fitting errors

Parameter	Meaning	Value
a	stress scaling factor for MC90 (τ level)	1.34
b	slip scaling factor (shape preservation)	1.00
k	post-peak / residual scaling factor (shape preservation)	1.00
Fitting errors (for τ at $s = 1$ mm across mixes)		
	Metric	Value
	MAE (MPa)	0.71
	RMSE (MPa)	0.75
	MAPE (%)	4.18

The proposed calibration parameters and fitting errors are summarized in Table 6. The value $a=1.34$ represents the average amplification of the bond level relative to MC90 at $s=1$ mm over the investigated mixtures. The parameters b and k are set to 1.00 to preserve the MC90 curve shape.

4.3 Design Implications for Development Length and Lap Splice

Bond-controlled anchorage and lap splice lengths are inversely related to the effective bond stress that can be mobilized along the bar. This relationship provides a direct link between the calibrated bond model and practical detailing. A first-order implication can be expressed using a normalized ratio:

$$\frac{l_{d,GPC}}{l_{d,OPC}} \approx \frac{\tau_{OPC}}{\tau_{GPC}} \quad (5)$$

If the OPC reference is MC90 and the GPC level is determined from the measured τ at $s=1$ mm, the ratio can be evaluated for each mixture. The results are reported in Table 7. The computed ratios range from about 0.70 to 0.78 for the investigated strength levels.

Table 7. Normalized comparison of development length (MC90 vs calibrated for GPC)

Mix	f'_c (MPa)	$\tau_{\text{GPC,mean}}$ at $s=1$ mm (MPa)	$\tau_{\text{MC90,max}}$ (MPa)	$\tau_{\text{GPC}} / \tau_{\text{MC90}}$	$l_{d,\text{GPC}} / l_{d,\text{OPC}}$
G_30	31.67	16.07	11.26	1.43	0.70
G_40	40.46	16.69	12.72	1.31	0.76
G_50	52.06	18.57	14.43	1.29	0.78

The trend of $l_{d,\text{GPC}}/l_{d,\text{OPC}}$ with compressive strength is illustrated in Figure 8. The ratio increases slightly as f'_c increases in the investigated range. This occurs because the relative exceedance of GPC over MC90 decreases with increasing strength in the present dataset.

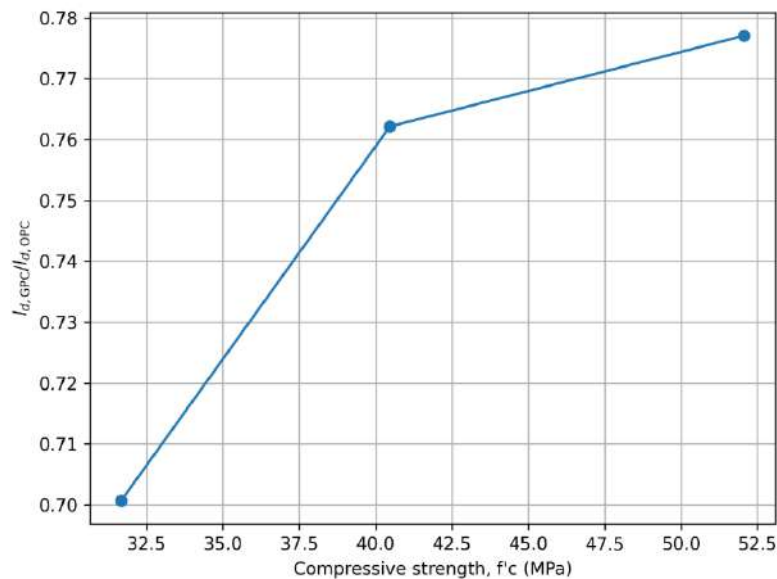


Fig. 8. Effect of compressive strength on the development-length reduction factor for fly ash geopolymer concrete

The above implications must be interpreted together with the observed failure mechanism. All pull-out specimens failed by splitting. Splitting is sensitive to concrete cover and confinement. The proposed reduction in development or splice length is therefore applicable only when the cover and confinement are comparable to those of the tested configuration. A conservative design approach can still use MC90 directly because MC90 provides a lower-bound estimate for the investigated fly ash GPC system at the reference slip level.

From an engineering perspective, the value of this calibration is that it allows the experimental findings to be interpreted in a design-oriented manner. Instead of treating the measured bond behavior only as an isolated set of test results, the calibrated model provides a practical means of quantifying how much the bond level of the investigated fly ash GPC differs from that assumed in the conventional MC90 formulation. This, in turn, enables a first-order assessment of development-length demand and shows how existing OPC-based provisions may be conservatively adapted for reinforced fly ash-based geopolymer concrete under conditions comparable to those examined in this study.

5. Conclusions

This paper investigated the bond performance of deformed steel bars in fly ash-based geopolymer concrete (GPC) using pull-out tests with a 22 mm bar and a bonded length of 5db (110 mm). The following conclusions are drawn.

- All tested specimens failed by concrete splitting in the bonded region. The failure was sudden and brittle.

- The measured bond response exhibited a rapid post-peak drop. This trend is consistent with splitting-controlled behavior rather than a stable pull-out mechanism.
- The bond level of fly ash GPC at the reference slip $s = 1$ mm was higher than the OPC-based MC90 prediction for good bond conditions. The exceedance ranged from about 13% to 43% for the investigated mixtures.
- A simple calibration that preserves the MC90 curve shape and scales the stress level was proposed. The recommended parameters are $a=1.34$, $b=1.00$, and $k=1.00$.
- The calibration errors for τ at $s=1$ mm are low ($MAE \approx 0.71$ MPa, $RMSE \approx 0.75$ MPa, and $MAPE \approx 4.18\%$).
- The higher bond level implies shorter anchorage demand in a first-order sense. The normalized ratio $l_{d,GPC}/l_{d,OPC}$ derived from the experimental bond level is about 0.70–0.78 for the investigated strength range.
- The above design implication must be interpreted together with the observed splitting failure. Bond capacity is sensitive to cover and confinement. Therefore, any reduction of development or splice length should be adopted only when detailing conditions are comparable to those in the tests.
- The main significance of the proposed model is that it bridges the gap between experimental bond characterization and practical structural application by enabling a first-order calibration of development-length demand for reinforced fly ash-based geopolymer concrete.

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Seismic frequency content as a key parameter in the dynamic buckling behavior of tall thin-walled steel tanks

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Abstract

The dynamic buckling behavior of anchored thin-walled steel storage tanks subjected to seismic excitation is investigated, with particular emphasis on the influence of seismic frequency content. Nonlinear analyses are performed on a representative anchored cylindrical tank with an aspect ratio $H/D=2$, selected for its practical relevance and known susceptibility to diamond-shaped local buckling. Three recorded ground motions with distinct frequency characteristics - Boumerdes, -Parkfield, and -San Fernando, are employed to evaluate their effects on seismic response and buckling of the tank. The numerical results reveal the development of diamond-shaped local buckling modes concentrated in the upper shell courses, with buckling occurring at PGA levels of 0.16 g, 0.11 g, and 0.76 g for the Boumerdes, Parkfield, and San Fernando records, respectively. These findings demonstrate that earthquake frequency content plays a governing role in the dynamic stability of thin-walled steel storage tanks and should be explicitly considered in seismic response and buckling assessments.

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1. Introduction

Earthquakes are among the most destructive natural hazards, although only a limited number of seismic events lead to severe structural damage. Experience from past earthquakes has shown that liquid-containing steel storage tanks are particularly vulnerable to seismic excitation, often suffering extensive damage that renders them inoperative. Post-earthquake investigations following major seismic events in Turkey and India documented critical failure modes in petroleum steel storage tanks, including shell buckling, roof and anchorage damage, foundation distress, and hydrodynamic pressure-induced effects. These studies further indicate that steel tanks exhibit greater vulnerability than reinforced concrete tanks when subjected to comparable seismic demands [1,2]. Among the observed damage mechanisms, dynamic buckling of the tank wall has consistently been identified as the most frequent and severe failure mode, governing the seismic safety of slender tank configurations. This behavior has been repeatedly observed during major earthquakes, including the Long Beach (1933), Kern County (1952), and Kobe (1995) events, which further revealed the inherent susceptibility of steel tanks to dynamic instability when subjected to seismic loading [3].

These observations emphasized the dominant role of fluid-structure interaction effects, in which the coupled response of structural vibrations and hydrodynamic pressures is highly sensitive to the frequency content of the seismic excitation. Consequently, significant research efforts have been devoted to understanding the dynamic response, instability mechanisms, and seismic fragility of thin-walled steel tanks under earthquake excitation.

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Accordingly, the dynamic behavior of cylindrical shells containing fluid has been the subject of numerous analytical, numerical, and experimental investigations. Zhang et al. [4] developed a finite element formulation based on the classical shell theory and potential flow theory, which was validated against with published experimental results. The influence of flow velocities, hydrostatic pressures and boundary conditions on natural frequencies has also been addressed. Shen et al. [5] for instance, investigated cylindrical shells under combined axial compression and external pressure, demonstrating the sensitivity of buckling response to initial imperfections. In addition, comprehensive reviews by Virella et al. [6–8] synthesized research on buckling behavior, foundation settlement, and fire resistance of steel tanks, with particular emphasis on dynamic loading conditions such as wind and earthquakes.

Several numerical investigations have focused on the seismic instability of steel storage tanks and their assessment within the framework of existing design codes. Buratti and Tavano [9] examined the seismic fragility of fixed-base tanks using an added-mass approach to represent fluid–structure interaction, confirming its effectiveness in predicting buckling behavior. Djermane et al. [10] evaluated seismic design provisions prescribed by AWWA D100 and Eurocode 8, reporting satisfactory performance for broad tanks while highlighting notable limitations when applied to slender configurations. Furthermore, the seismic response of anchored tanks subjected to combined horizontal and vertical ground-motion components was investigated by Sobhan [11,12], who demonstrated that explicit consideration of vertical acceleration leads to more accurate predictions of buckling capacity and deformation demand.

Beyond code-based evaluations, recent investigations have increasingly emphasized the fundamental mechanisms governing dynamic buckling of thin-walled structures under extreme excitation. Çelik and Köse [13] numerically examined the seismic buckling response of thin-walled steel tanks, highlighting the influence of roof geometry and shell thickness on deformation patterns and instability onset. Wind-induced dynamic buckling was investigated experimentally and numerically by Yasunaga and Uematsu [14], showing that shell vibration plays a crucial role in instability, while inertial effects remain secondary under certain loading conditions. The influence of vertical seismic excitation on raised conical tanks was explored by Hadj Djelloul et al. [15], underscoring the importance of vertical acceleration components in seismic stability assessments.

In parallel, the effects of anchorage conditions and geometric slenderness on seismic stability have been widely investigated. Through three-dimensional nonlinear finite element analyses, it was demonstrated by Moreno et al. [16], analyses that anchorage configuration strongly affects stress distribution and buckling capacity, particularly for slender tanks, for which existing design codes may be unconservative. A marked decrease in buckling strength with increasing radius-to-thickness ratio was reported by Ullah and Mamaghani [17], confirming the governing role of geometric slenderness in seismic stability.

Complementing these investigations, the seismic response of cylindrical storage tanks subjected to ground motions with varying frequency content was examined by Abd-Elhamed et al. [18], with explicit consideration of fluid–structure–soil/foundation interaction. It was demonstrated that low-frequency ground motions generally induce a smaller response than high-frequency motions, although soil–tank interaction can still amplify the resulting response. More recently, a comparative review of recent research, design recommendations, standards, and seismic codes for large liquid storage tanks was provided by Nadjuri et al. [19], in which the limitations of existing guidelines in capturing complex tank–fluid–soil interactions and dynamic buckling behavior were highlighted.

Collectively, these studies indicate that current design codes and simplified seismic evaluations may not fully account for the frequency-sensitive dynamic behavior of anchored thin-walled steel tanks, highlighting the need for detailed numerical investigations of critical seismic loading and associated buckling mechanisms. The seismic performance of these tanks is strongly influenced by both structural geometry and ground motion characteristics: tall cylindrical tanks are particularly susceptible to local diamond-shaped buckling, whereas broader tanks tend to develop elephant-foot deformations, as confirmed by experimental studies and post-earthquake observations, such as the 1980 Livermore event. To address these challenges, the present study employs nonlinear

numerical simulations to investigate the dynamic buckling behavior of a representative anchored tank with an aspect ratio (H/D) of 2. The investigation aims to determine the critical horizontal acceleration associated with the initiation of buckling and to assess the influence of seismic frequency characteristics on structural response. The selected tank geometry reflects a commonly adopted and economically efficient design, while its susceptibility to diamond-shaped local buckling provides a relevant benchmark for future studies on mitigation strategies, including the potential implementation of stiffening rings.

2. Seismic Modeling of Steel Storage Tanks

2.1 Tank Design, Geometry, and Hydrodynamic Assumptions

To investigate the influence of seismic motion frequency characteristics on the dynamic response of tall steel storage tanks, a representative slender cylindrical tank with a height-to-diameter ratio (H/D) of 2 was selected for analysis (Fig. 1 (a)). The adopted tank configuration is explicitly defined, and its selection is justified by both economic factors and site space requirements, while its susceptibility to diamond-shaped local buckling under seismic excitation is clearly identified [7,10].

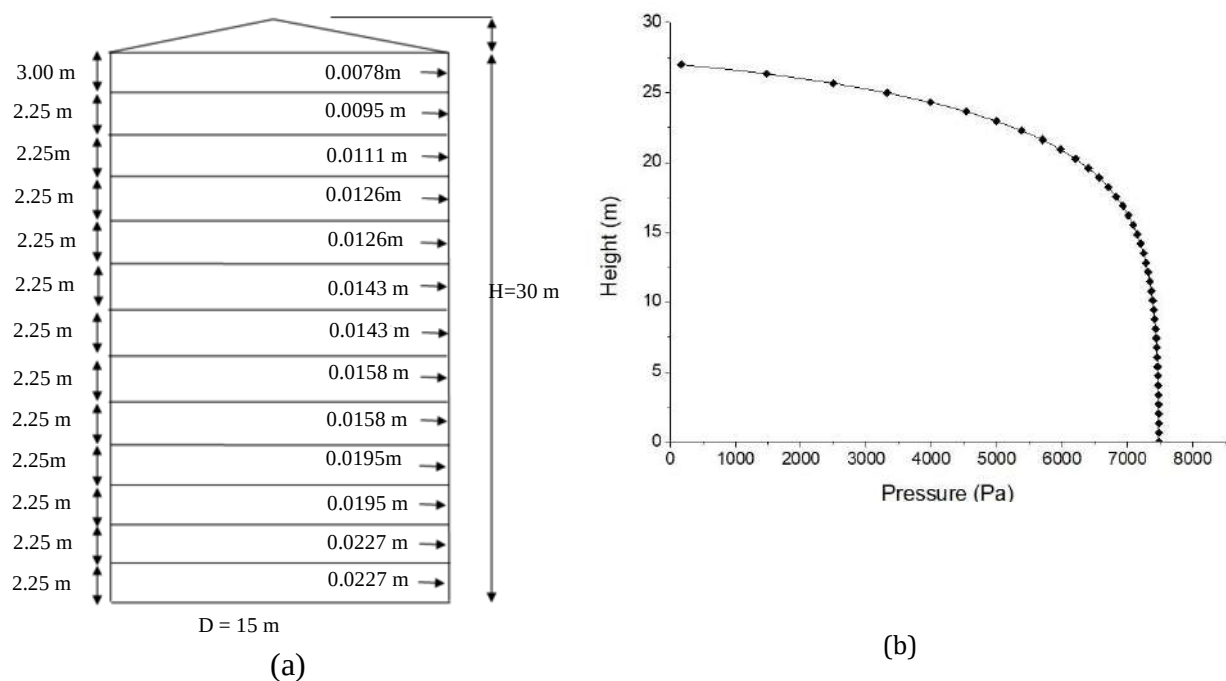


Fig. 1. Schematic of a tall cylindrical tank ($H/D = 2$) showing the (a) geometry and (b) variation of impulsive hydrodynamic pressure

The modeling assumptions were carefully defined to capture the coupled fluid–tank interactions and to provide an accurate representation of the tank’s dynamic response under seismic excitation. Following Housner’s formulation, adopted in Eurocode 8 (EN 1998-4) [20] and API 650 [21], the hydrodynamic response of liquid-filled tanks is represented by impulsive and convective components. Due to the large separation of their natural periods, these components were treated as uncoupled, with the response dominated by the impulsive component. A three-dimensional finite element model was developed considering only the impulsive interaction, while convective effects were neglected, consistent with previous studies [6]. All translational and rotational degrees of freedom at the tank base were restrained, except for horizontal displacement in the direction of seismic excitation, and horizontal ground motion was applied along the x-axis to simulate the earthquake.

The tank shell thickness was determined following API 650 provisions, assuming a fill level of 90% of the total tank height. The contained fluid was modeled using an equivalent density approach, in which the hydrodynamic added mass of the liquid was incorporated into an effective density assigned to the tank shell. Equivalent density values were evaluated based on the impulsive

hydrodynamic pressure, following the formulations proposed by Veletsos and Shivakumar and subsequently adopted by Virella et al. [7], as illustrated in Fig. 1 (b).

$$P_i = (\eta, \theta, t) = c_i(\eta) \rho R \ddot{x}_g(t) \cos \theta \quad (1)$$

According to Virella et al., the function $c_i(\eta)$ which describes the distribution of impulsive pressure over the cylinder height is calculated as follows:

$$c_i(\eta) = 1 - \sum_{n=1}^{\infty} c_{cn}(\eta) \quad (2)$$

$$c_{cn}(\eta) = \frac{2}{\lambda_n^2 - 1} \left(\frac{\cosh[\lambda_n(H/R)\eta]}{\cosh[\lambda_n(H/R)]} \right) \quad (3)$$

Where the symbol λ_n corresponds to the n th solution of the equation obtained from the derivative of the Bessel function. The first three roots of this equation are: $\lambda_1 = 1.841$, $\lambda_2 = 5.311$, and $\lambda_3 = 8.536$. $\eta = z/H$; z is defined as the height taken from the tank base upward.

A simplified technique to account for fluid-structure interaction was proposed by Barton et al. (1987), in which an equivalent density was assigned to the tank shell section submerged beneath the liquid level. This equivalent density was defined to represent both the mass of the steel shell up to that level and the impulsive fluid mass as described by Housner [6,10,15]. The method was subsequently adopted and refined by Nachtigall et al. [8], enabling the equivalent density to be evaluated for the tank wall [10] as follows:

$$\rho^* = \frac{\text{Mass of the liquid} + \text{Mass of the wall shell of the tank}}{\text{the volume of the wall of the tank}} \quad (4)$$

2.2 Analytical Approach

Assuming a 3D FE approach for structural simulation, the dynamic performance characteristics of the system can be described by the equation governing the system's equilibrium:

$$[M] \left\{ \frac{d^2 u}{dt^2} \right\} + [C] \left\{ \frac{du}{dt} \right\} + [K] \{u(t)\} = -F(t) \quad (5)$$

Where the matrices $[M]$, $[C]$ and $[K]$ refer to the mass, damping and stiffness properties of the analyzed structure, respectively. When the matrices K and C are to be identified. According to Eurocode 3 part 1-6 correspond to Strength and Stability of Shell Structures[22], the structural analyses should be performed with consideration of both geometric and material nonlinear effects. To determine the elasto-plastic behavior of an isotropic steel material, the formula given below is used [23].

$$\sigma_u = \sigma_y + \varepsilon_{pl1} * k \quad (6)$$

Where k is adopted as $0.0036 * E$ and elastic modulus E equal to 210 GPa. The material is linear-elastic up to a first yield stress of $\sigma_y = 250$ MPa, followed by 0.36% linear strain hardening up to an ultimate stress of $\sigma_u = 400$ MPa. Rayleigh damping was used to attempt in the nonlinear analysis, where the damping matrix $[C]$ is formulated as follows:

$$[C] = (\alpha_M) [M] + (\beta_K) [K] \quad (7)$$

Where α_M and β_K denote the mass-proportional and stiffness-proportional damping coefficients respectively. The value of α_M and β_K , which correspond to a constant ξ_i , can be calculated from two modes and so on for the other modes. This procedure preserved the value of damping for modes whose natural periods range between the limits corresponding to the highest participation factors along the translational axes. Generally, for steel tanks, ξ_i is expected to be of the order of 2% for

impulsive mode [23]. The additional effect of radiation damping is neglected in this study where the foundation is not included in the simulation:

$$\xi_i = \frac{1}{2\omega_i} \alpha_M + \frac{\omega_i}{2} \beta_K \quad (8)$$

The external force applied to the storage tanks containing fluids corresponds to the combined contribution of the structural and liquid masses, each subjected to seismic loading.

$$F(t) = [M_s] \{r_x\} \ddot{x}g(t) + \{P_i\} \ddot{x}g(t) \quad (9)$$

$$\ddot{x}g(t) = PGA * f(t) \quad (10)$$

In Eq. (5), $\{u(t)\}$, $\{\dot{u}(t)\}$ and $\{\ddot{u}(t)\}$ denote the displacement, velocity and acceleration vectors respectively, some hypotheses are made to solve the equation (5), with respect to the nature and direction of the incident motion. Where $[r_x]$ is an n-dimensional influence vector, with entries of '1' assigned to the degrees of freedom corresponding to the direction of ground excitation, and '0' elsewhere, $\{P_i\}$ denotes the vector of nodal forces induced by an acceleration of 1g, representing the impulsive component of the hydrodynamic pressure acting on the system. Here, $[M_s]$ stands for the shell mass matrix, and $f(t)$ indicates the earthquake time history.

2.3 Modeling and Finite Element Mesh

The continuous advancement of computational power and numerical techniques has enabled robust software tools to solve highly nonlinear equilibrium and instability problems in structural systems. In this study, nonlinear dynamic analyses were performed using the explicit time integration scheme available in ABAQUS/Explicit [24]. Unlike implicit algorithms, which require iterative procedures and may encounter difficulties with strong nonlinearities, the explicit approach is conditionally stable and free from convergence issues, making it particularly suitable for problems involving severe material and geometric nonlinearities, despite potentially longer computation times. The solver was employed to investigate the seismic buckling behavior of slender steel storage tanks.

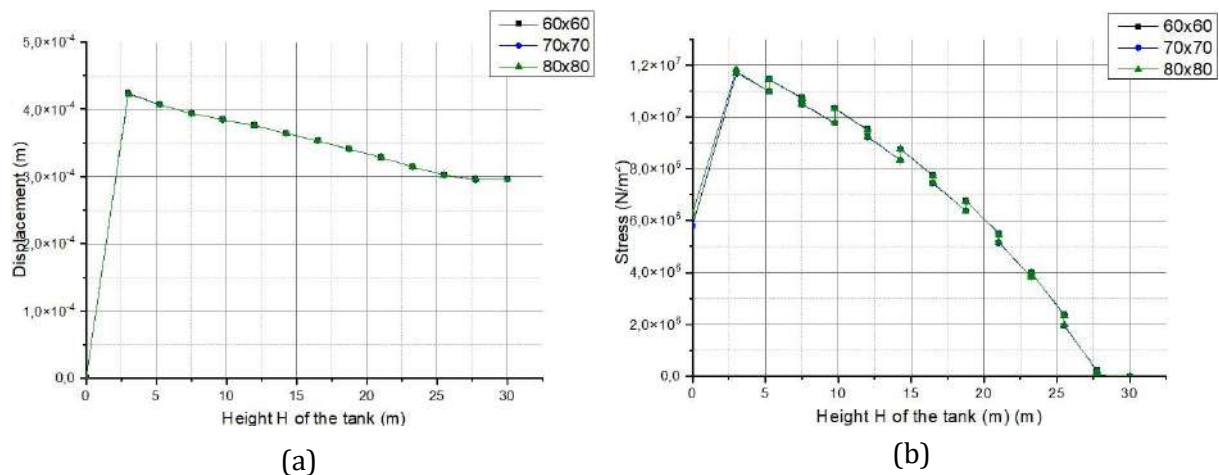


Fig. 2. Mesh sensitivity analysis under static loading: (a) displacement distribution along the tank height; (b) axial stress distribution for different mesh densities

The adequacy of the finite element discretization was verified through a mesh sensitivity analysis conducted under both static (hydrostatic) and dynamic (seismic) loading conditions, with an empty tank considered for the dynamic analysis. In the static analyses, a mesh sensitivity study was performed by varying the characteristic element size along the tank height, adopting 60×60, 70×70, and 80×80 discretizations. The resulting displacement and stress distributions were compared in order to assess convergence. As shown in Fig. 2(a) and (b), only minor differences were observed

between the responses obtained with the refined meshes, indicating adequate convergence of the numerical solution.

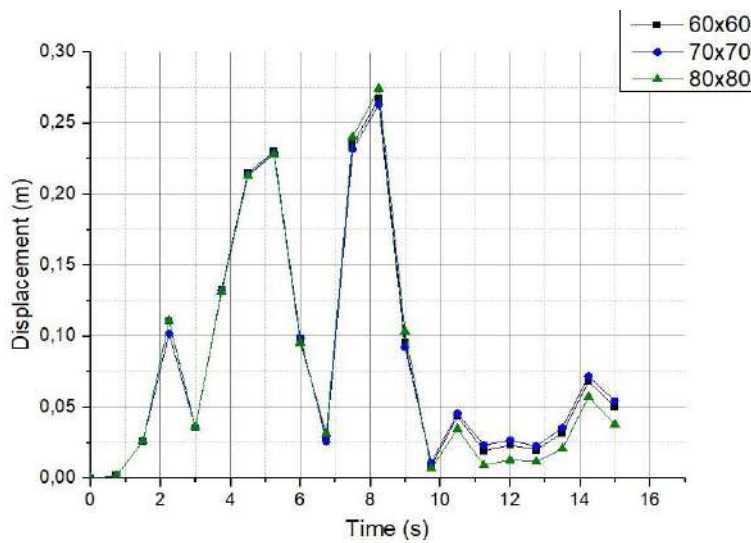


Fig. 3. Mesh sensitivity analysis under dynamic loading: comparison of shell displacement time histories obtained using 60×60 , 70×70 , and 80×80 meshes

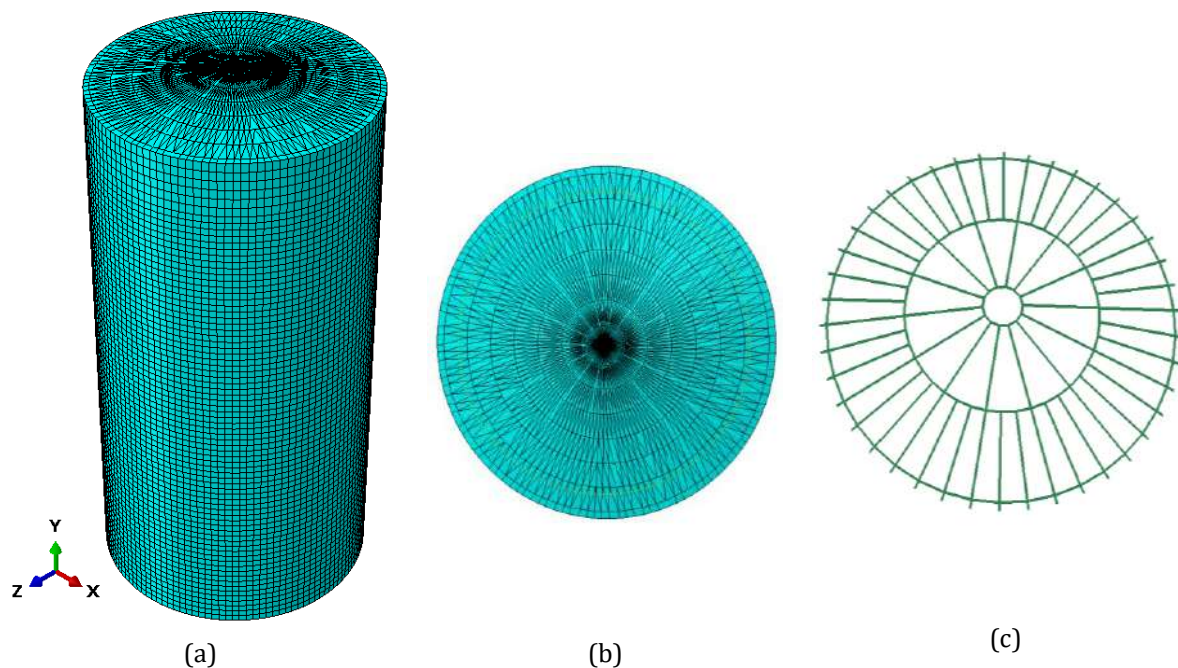


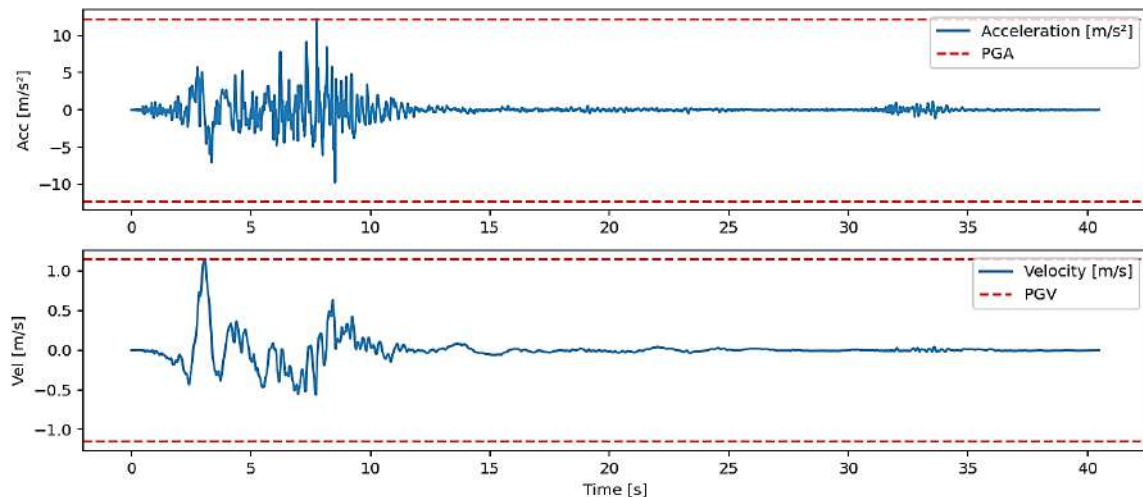
Fig. 4. Finite element mesh geometry of the tank showing: (a) Cylindrical steel shell discretized with four-node reduced-integration shell elements (S4R), (b) Roof modeled with triangular shell elements (S3), and (c) Rafters represented using beam elements (B31)

For the dynamic analyses, the displacement vs time histories corresponding to the different mesh densities exhibited nearly identical response trends and peak amplitudes (Fig. 3). Based on these findings, the 70×70 mesh was selected for subsequent simulations, as it ensures sufficient accuracy while maintaining reasonable computational efficiency. Based on the selected mesh, the finite element discretization was composed of 12,026 elements, including 10,080 four-node reduced-integration shell elements (S4R) for the cylindrical shell, which are particularly suitable for thin-walled structures; 1,638 three-node triangular shell elements (S3) for the roof; and 308 two-node beam elements (B31) for the roof rafters (Fig. 4). Both geometric and material nonlinearities were considered. Large-deformation effects were accounted for by activating the NLGEOM option in ABAQUS, allowing the stiffness matrix to be continuously updated as the geometry evolved.

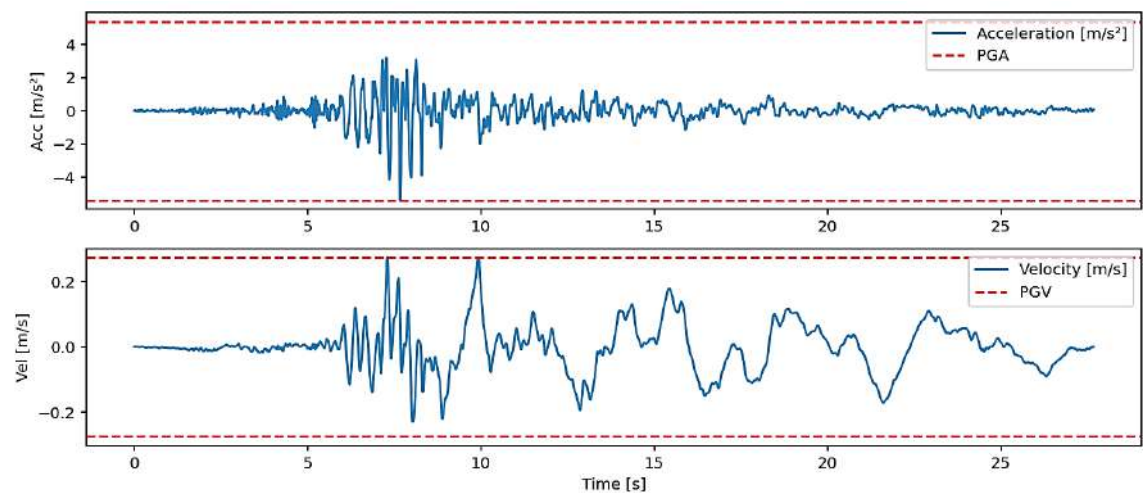
2.4 Seismic Input Motions, Frequency Characteristics and Scaling Procedure

Seismic records were selected based on key characteristics, including peak ground acceleration (PGA), duration, and frequency content. To reduce computational cost in nonlinear dynamic analyses, short-duration records capturing essential amplitudes and frequencies were preferred. It has been demonstrated in previous studies that the buckling behavior of tanks can be adequately evaluated using three carefully selected ground motions while maintaining computational efficiency [10,18,25,26]. Accordingly, three representative earthquakes were adopted to cover intermediate- and high-frequency excitations: San Fernando (1971), Boumerdes (Dar El Beida, 2003), and Parkfield (1966) (Fig. 5).

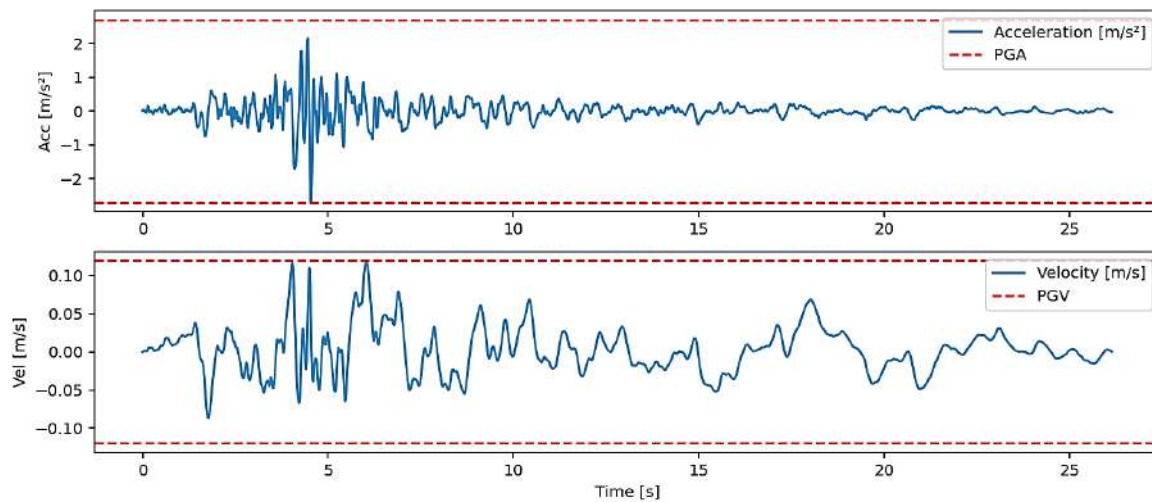
To quantitatively characterize the frequency content of the selected records, the PGA/PGV ratio—defined as the ratio between peak ground acceleration and peak ground velocity—was employed as a basic indicator of earthquake frequency characteristics. Records with PGA/PGV ratios greater than 1.2 are classified as high-frequency motions, whereas those with ratios lower than 0.8 are categorized as low-frequency motions. Motions with PGA/PGV ratios between 0.8 and 1.2 are considered to exhibit intermediate-frequency content. Based on this classification, the selected ground motions span distinct frequency ranges, as summarized in Table 1, enabling a systematic investigation of the influence of ground-motion frequency content on the dynamic buckling behavior of thin-walled steel storage tanks, beyond the effects of peak ground acceleration (PGA) alone.



(a)



(b)



(c)

Fig. 5. Acceleration and velocity time histories of the selected earthquake ground motions considered in this study: (a) San Fernando earthquake (1971), (b) Boumerdes earthquake (2003, Dar El Beida station), and (c) Parkfield earthquake (1966)

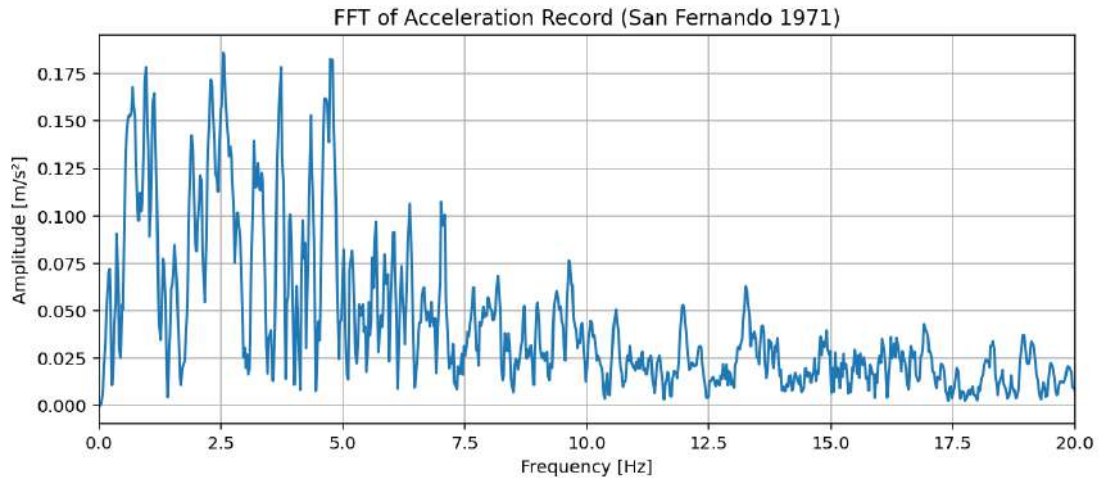
Table 1. Seismic intensity measures

Earthquake Record	Δt (s)	Duration (s)	PGA (m/s^2 , g)	PGV (m/s)	PGA/PGV ($\text{g}\cdot\text{s/m}$)	Dominant Frequencies
San Fernando (1971, S-W)	0.010	40.48	12.26 (1.25 g)	1.15	1.09	Intermediate
Boumerdes (2003, E-W)	0.005	27.68	5.40 (0.55 g)	0.275	2.00	High + some long
Parkfield (1966, NOAA Usaca)	0.020	26.14	2.70 (0.27 g)	0.120	2.30	High / short-period

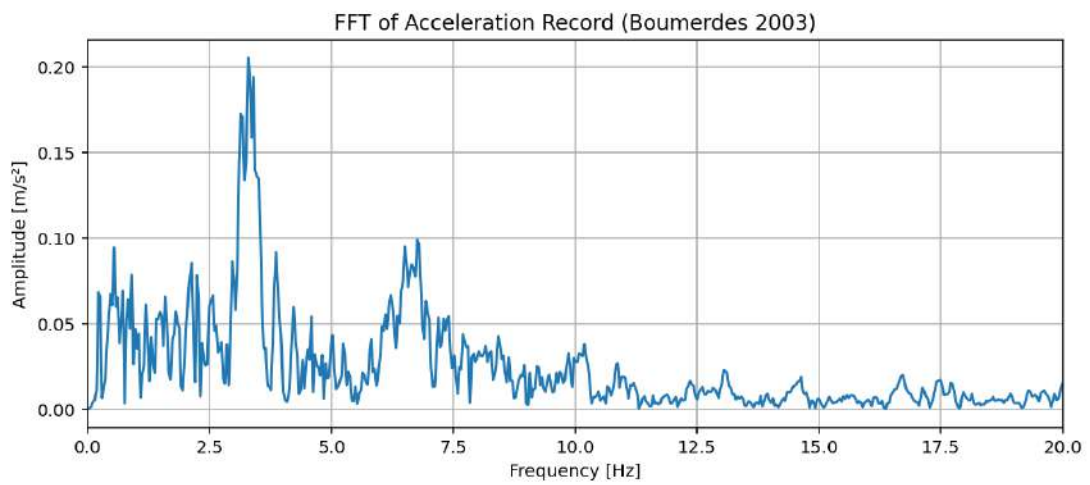
A comparative overview of three seismic records-the San Fernando (1971, S-W), Boumerdes (2003, E-W), and Parkfield (1966)-highlights differences in frequency content and PGA/PGV ratios. The San Fernando record is dominated by intermediate frequencies with a low PGA/PGV ratio of 1.09. Boumerdes exhibits a mixed spectrum containing both high- and long-period energy, with a moderate PGA/PGV ratio of 2.00. The Parkfield record is rich in high-frequency, short-period content and has the highest PGA/PGV ratio of 2.30. These distinctions emphasize how the frequency content and PGA/PGV ratio vary among seismic events, providing a clear comparative characterization of these ground motions.

The corresponding Fourier amplitude spectra of these ground motions are shown in Fig. 6. Consistent with the previous characterization, the San Fernando (1971) record exhibits dominant energy at low to intermediate frequencies, with most spectral amplitudes concentrated below approximately 5 Hz and a principal peak around 2–3 Hz. The Boumerdes (2003) record is dominated by high-frequency content and long-period components, mainly between 2 and 4 Hz, with a dominant peak near 3 Hz, reflecting pronounced regional near-field effects. In contrast, the Parkfield (1966) ground motion is rich in high-frequency, short-period content, with most energy concentrated above 5 Hz and several smaller peaks spread across 5–10 Hz, generally showing sharper, impulsive characteristics compared to the San Fernando and Boumerdes records. These spectral distinctions complement the PGA/PGV-based comparison, highlighting the variability in frequency content among the three seismic events.

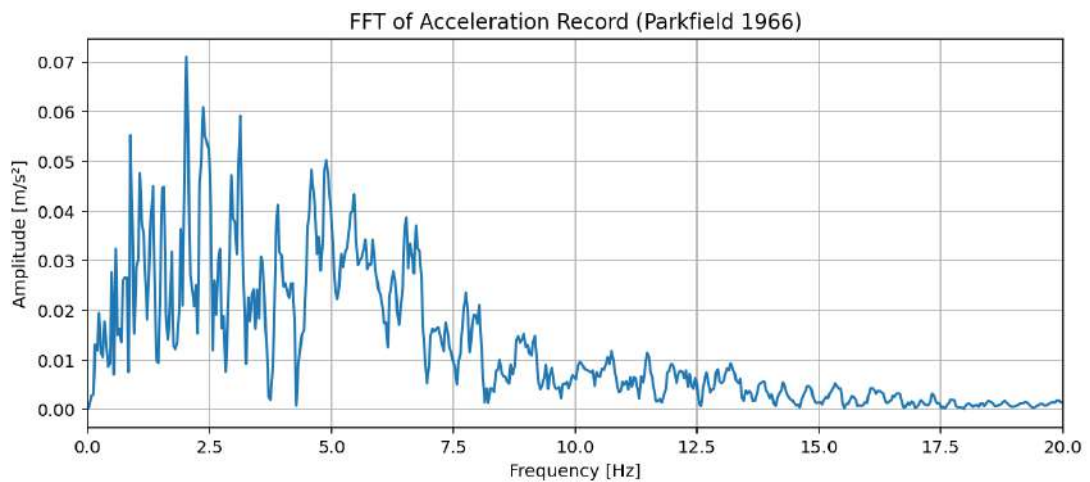
The recorded earthquake ground motions were scaled using a PGA-based normalization and scaling procedure. Each accelerogram was first normalized by its original peak ground acceleration (PGA) by applying a constant scale factor equal to the inverse of the original PGA, resulting in a normalized record with unit PGA.



(a)



(b)



(c)

Fig. 6. Fourier amplitude spectra of the accelerograms: (a) San Fernando 1971, (b) Boumerdes 2003 (Dar El Beida), and (c) Parkfield 1966

The normalized accelerograms were then multiplied by a target PGA value to achieve the desired intensity level. Incremental scaling was subsequently applied by progressively increasing the target PGA until the onset of buckling was observed. The PGA corresponding to the first occurrence of buckling was identified as the critical PGA associated with each earthquake record. This two-

step scaling procedure preserves the original temporal characteristics, frequency content, and duration of the ground motions, ensuring that only the amplitude of the excitation is modified.

3. Results of Analysis and Discussion

3.1 Buckling Onset and Critical PGA

Seismic-induced buckling of steel tanks is generally classified as either elastic or elasto-plastic dynamic buckling, as confirmed by experimental and numerical studies [10,15]. The critical dynamic load can be evaluated using three commonly applied criteria. The first, proposed by Budiansky and Roth [27], is based on Liapunov stability: time–displacement curves are plotted for increasing PGA values, and the critical PGA is identified as the curve exhibiting a sudden “jump” relative to neighboring curves, indicating the onset of dynamic buckling. The second criterion employs phase-plane trajectories ($U, \dot{U}$), where stable trajectories remain confined near the static equilibrium point, which acts as a center of attraction; at the critical load, trajectories diverge from this equilibrium point without oscillating around it, indicating the onset of instability. The third, the Pseudo-Equilibrium Path method, originally proposed by Budiansky and Roth and later generalized by Ari-Gur and Simonetta, relates the peak deflection to the pulse intensity. In the present study, elastic buckling was observed at the upper portion of the cylindrical shell, and the critical peak ground acceleration (PGAc) was determined using all three criteria.

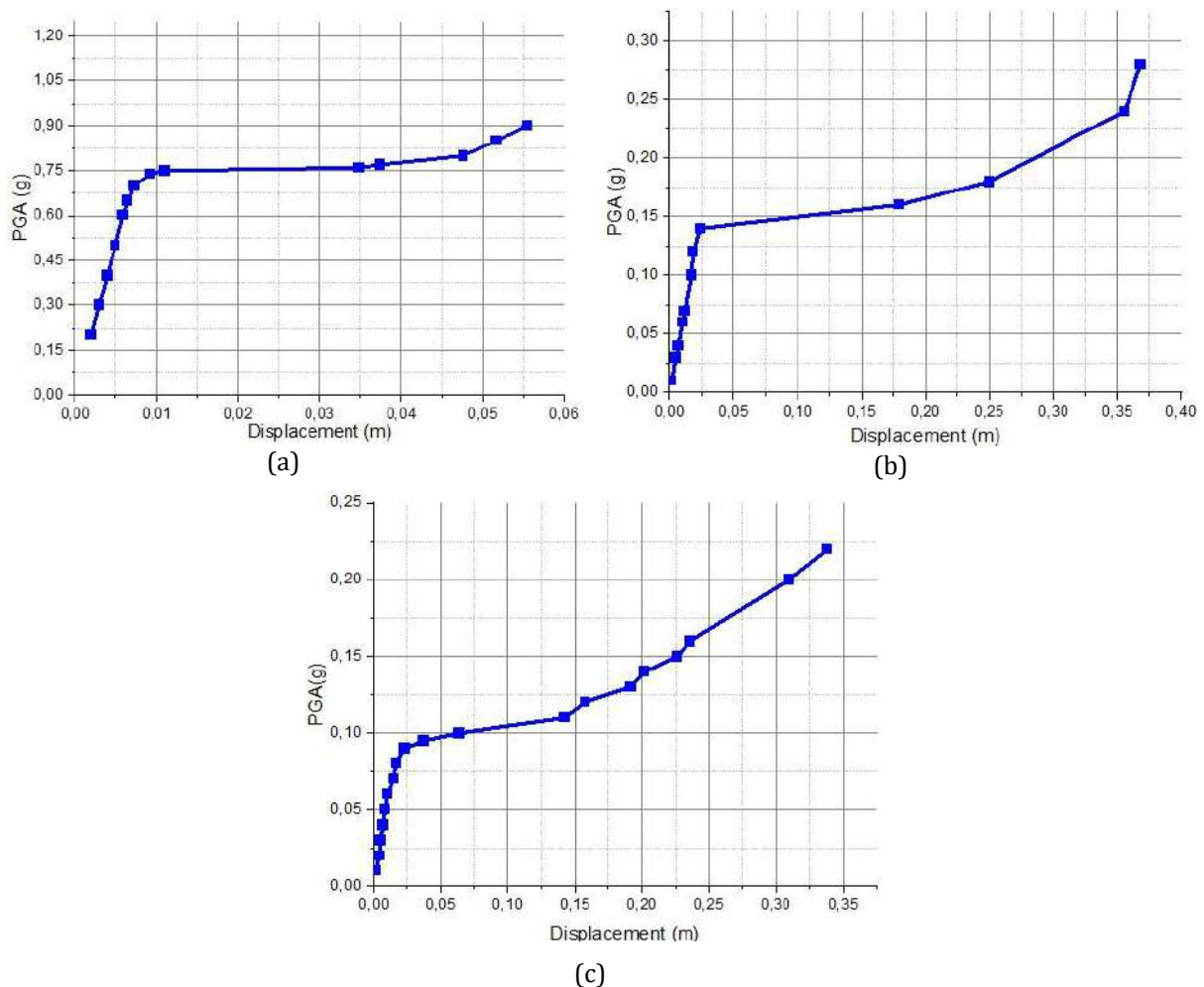


Fig. 7. Dynamic displacement response curves under selected earthquake ground motions: (a) San Fernando 1971, (b) Boumerdes 2003 (Dar El Beida), and (c) Parkfield 1966

The peak response is illustrated in Fig. 7(a), which presents the maximum radial displacement as a function of increasing PGA, evaluated according to the Ari-Gur and Simonetta criterion. For the model subjected to the San Fernando earthquake, the critical buckling PGA is identified at

approximately 0.75 g. A sudden reduction in the slope of the displacement PGA curve marks the onset of shell buckling once the excitation exceeds this critical value.

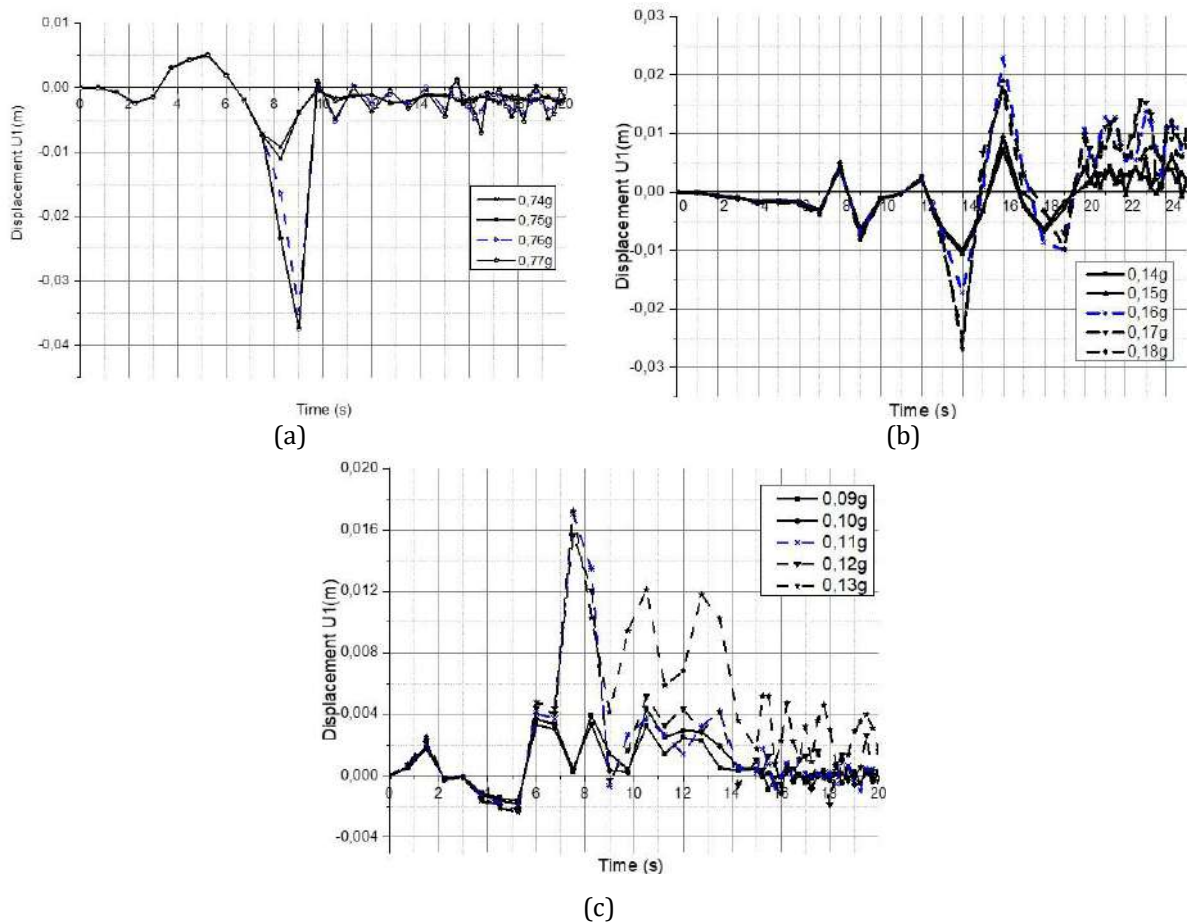


Fig. 8. Displacement Time Histories before and after PGAc under selected earthquake ground motions: (a) San Fernando 1971, (b) Boumerdes 2003 (Dar El Beida), and (c) Parkfield 1966

Figure 8(a) present the displacement time histories obtained at the critical PGA level of 0.76 g, based on the Budiansky and Roth criterion, corroborating the observations from Fig. 7(a), where a pronounced jump in displacement is observed near the critical PGA, indicating the initiation of buckling. Further verification is provided in Fig. 9(a), which presents the phase-plane diagram (velocity–displacement) of the tank shell under the San Fernando earthquake. This diagram depicts the transition from stable to unstable structural behavior, confirming the identified onset of buckling.

For earthquakes characterized by dominant high-frequency content, the dynamic displacement response curves are presented in Figs. 7(b) and 7(c) for the Boumerdes (Dar El Beida, 2003) and Parkfield (1966) records, respectively. Critical buckling PGA values of approximately 0.15 g and 0.10 g are identified for the Boumerdes and Parkfield records, respectively. The same procedure for several values of the PGA for critical PGAc values and their corresponding deformed shape under earthquake of Boumerdes and Parkfield is shown in Figs. 7, 8 and 9 (b) and (c) respectively. The jump of peak response is observed at critical value 0.16g under earthquake of Boumerdes and at 0.11g under earthquake of Parkfield.

Figure 10 illustrates the evolution of the Von Mises stress at the critical location where buckling initiation occurs within the tank shell. These stress time histories are presented for increasing PGA levels across the San Fernando (1971), Boumerdes (Dar El Beida, 2003), and Parkfield (1966) earthquake records. For each ground motion, the stress time histories are shown for several PGA levels in the vicinity of the critical buckling acceleration. For the San Fernando earthquake (Fig. 10(a)), the stress response increases gradually with PGA up to approximately 0.75 g. At this level, a pronounced amplification of Von Mises stress is observed, characterized by a sharp peak followed

by stress redistribution. This behavior coincides with the critical PGA previously identified from displacement-based criteria and indicates the onset of shell instability.

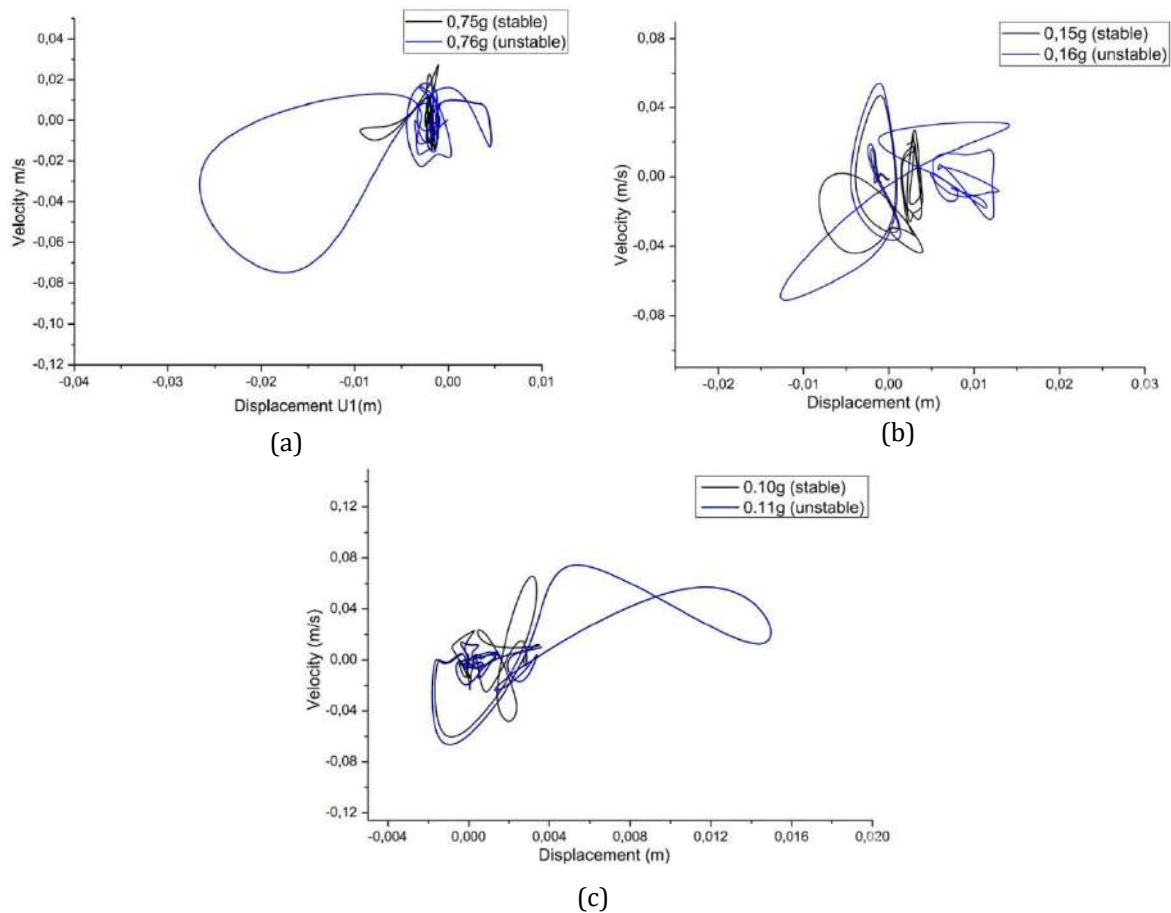


Fig. 9. Phase-plane representation before and after PGAc under: (a) San Fernando earthquake, (b) Boumerdes earthquake, (c) Parkfield earthquake

In contrast, the Boumerdes earthquake (Fig. 10(b)) exhibits a sharp increase in Von Mises stress at significantly lower PGA levels. A sudden rise in stress is observed near 0.16 g, corresponding to the critical buckling PGA identified from displacement jumps. This reflects the stronger interaction between the seismic frequency content of the Boumerdes record and the dominant vibration modes of the tank, leading to earlier stress concentration and instability despite the lower PGA level.

Similarly, the Parkfield earthquake (Fig. 10(c)) produces the earliest onset of critical stress conditions, with a marked stress amplification occurring at approximately 0.11 g. The sharp stress peak at this PGA indicates a highly dynamic response dominated by high-frequency components, which are particularly effective in triggering local shell vibrations. The lower critical PGA observed for Parkfield highlights the strong influence of frequency content on dynamic buckling susceptibility.

Overall, the Von Mises stress responses are consistent with the displacement Time Histories buckling criteria and confirm that dynamic buckling is governed not only by seismic intensity but also by the frequency characteristics of the input motion. Earthquakes with higher frequency content induce earlier stress localization and buckling at lower PGA levels, whereas records dominated by lower frequencies require higher PGA to reach instability.

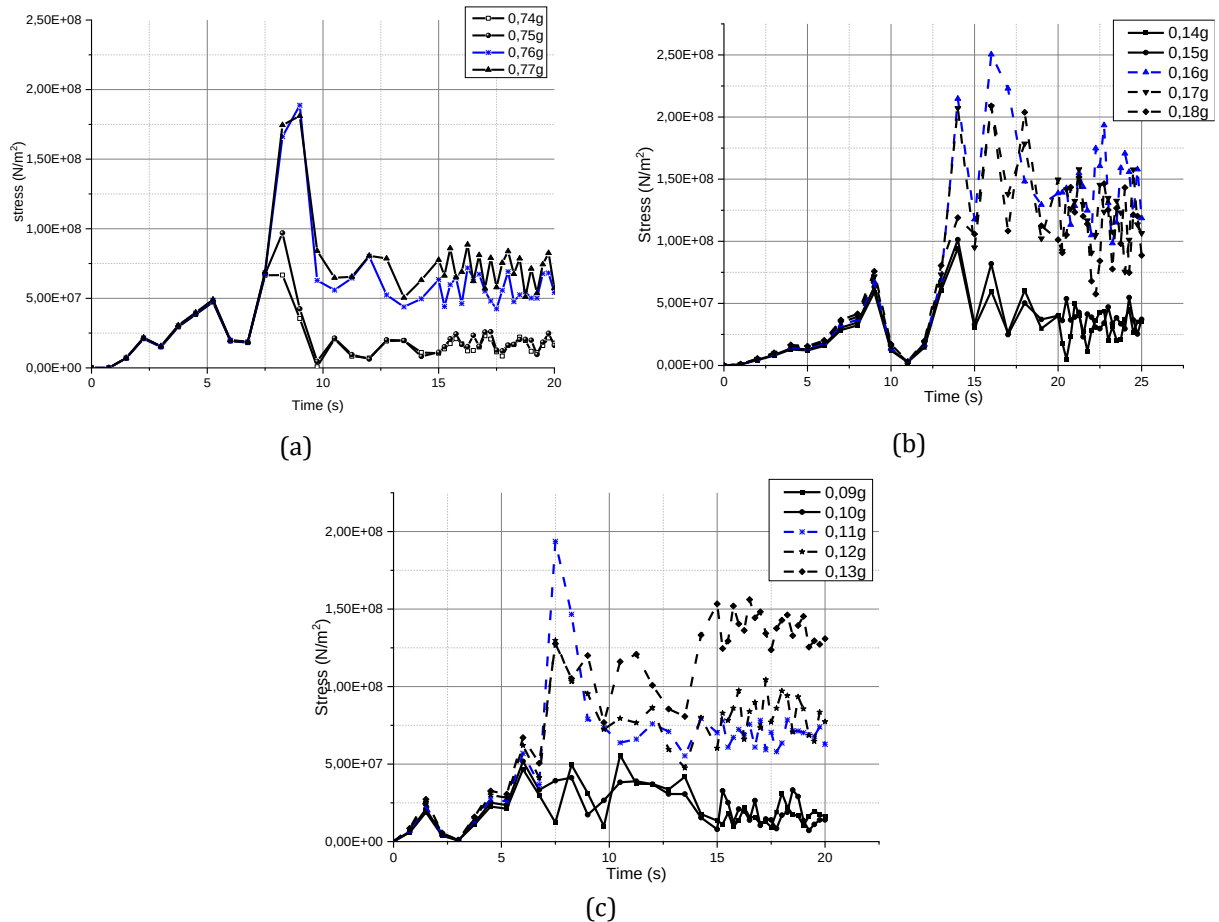
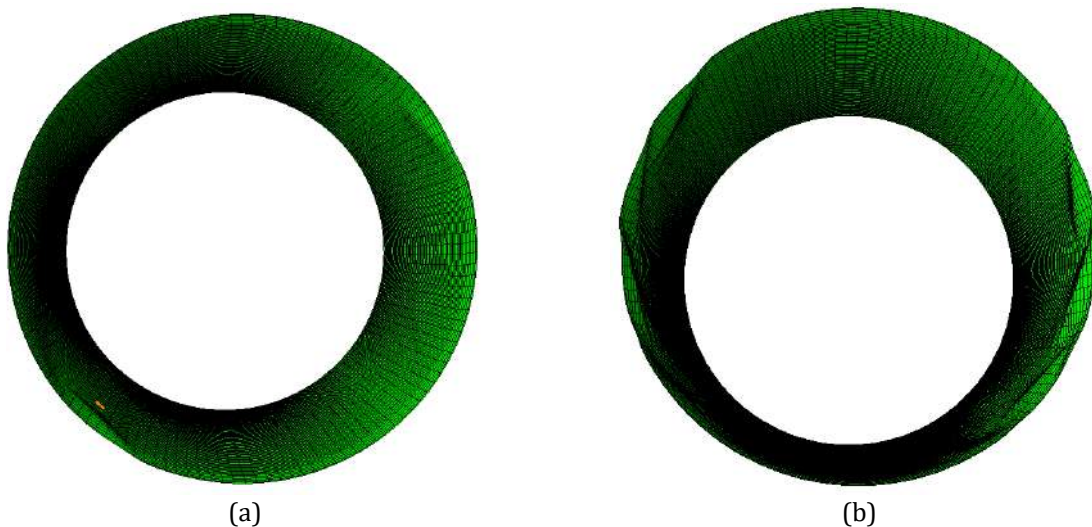
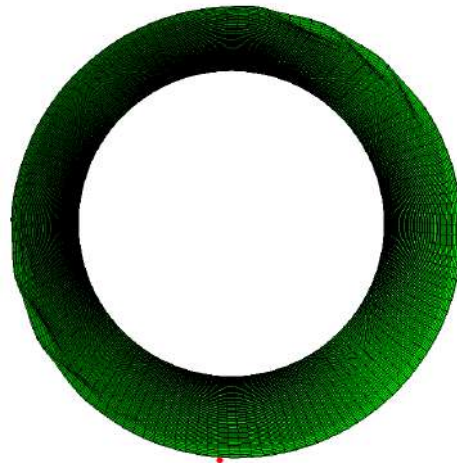


Fig. 10. Von Mises stress under of the selected earthquake ground motions: (a) San Fernando 1971, (b) Boumerdes 2003 (Dar El Beida), and (c) Parkfield 1966.

3.2 Buckling Patterns and Stresses

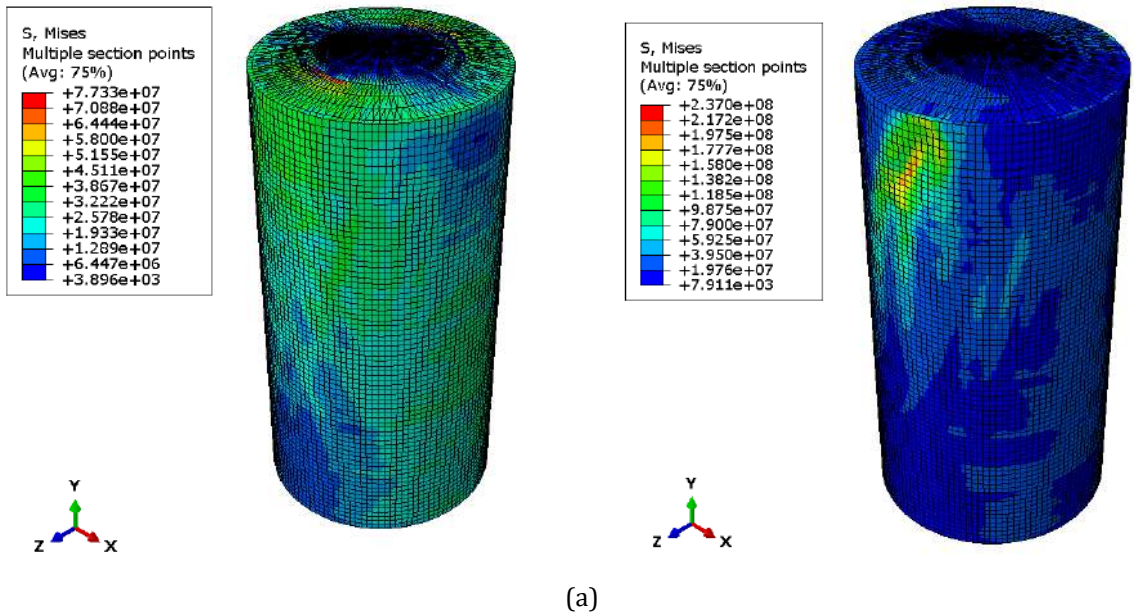
The nature of the dynamic buckling can be characterized by examining the deformed shape as well as the iso-stress contours at the critical level of excitation. Figure 11 shows the deformed tank exhibiting a partial depression at the top, which confirms a diamond-type buckling. The deformation patterns observed in diamond-type buckling are illustrated through the corresponding deformation shapes and contour plots (Figs. 11 and 12). The deformation patterns of the tank under dynamic loading clearly exhibit diamond-type buckling.



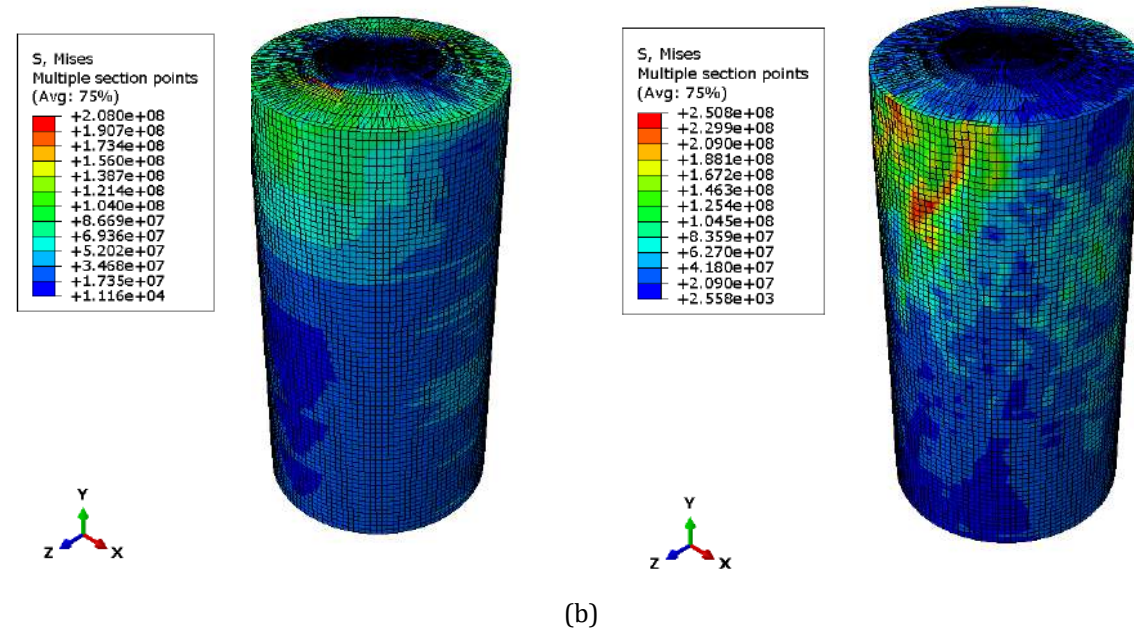


(c)

Fig. 11. Deformed shapes near the critical PGA for the (a) San Fernando, (b) Boumerdes, and (c) Parkfield earthquakes.



(a)



(b)

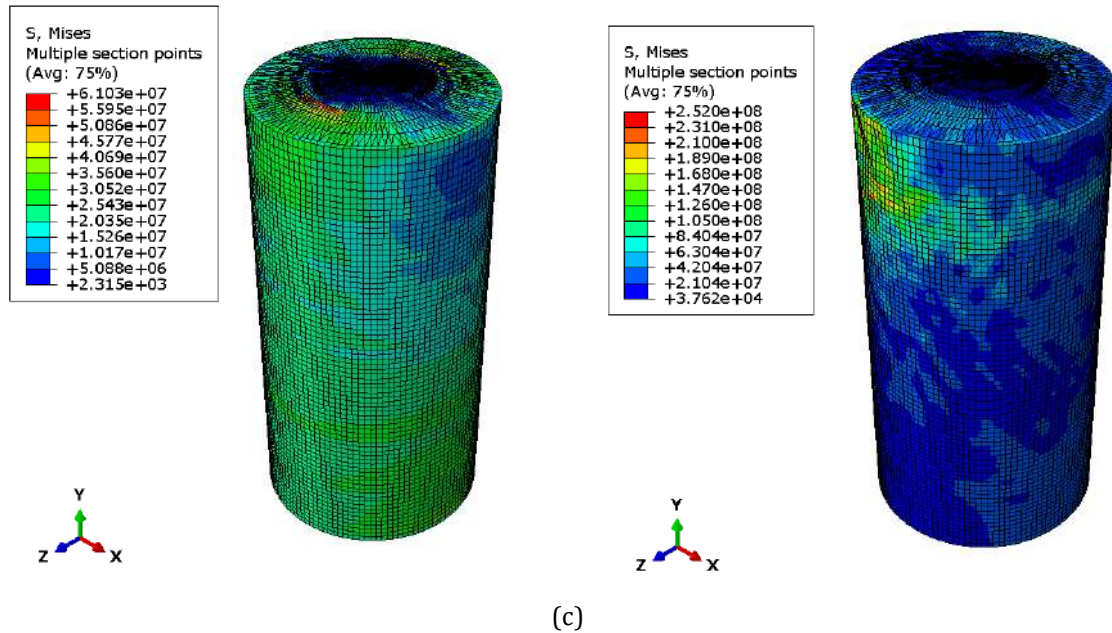


Fig. 12. Von Mises stress distribution before and after the critical PGA for the (a) San Fernando, (b) Boumerdes, and (c) Parkfield earthquakes, shown at the time of maximum peak response.

The shell shows characteristic inward and outward displacements forming circumferential lobes around the cylindrical surface, which are most pronounced at the instant of peak radial displacement. This instant represents the maximum response, when geometric instability is most apparent. The observed deformation remains consistent throughout the response, indicating that the buckling mode is governed by elastic shell behavior rather than post-yield phenomena such as elephant-foot buckling, which would require extensive plastic localization near the base.

The corresponding von Mises stress (see Fig .12) distribution highlights regions of stress concentration, typically at mid-height and near the top and bottom edges of the tank. At peak deformation, local stresses may approach or slightly exceed the steel's yield stress; however, yielding remains confined to small areas and does not modify the overall diamond-shaped deformation pattern. These stress contours provide valuable insight into the critical locations for potential material failure and serve as a key tool for validating numerical models against code predictions.

3.3 Effect of Seismic Frequency

The effect of seismic frequency on the dynamic response of thin-walled steel tanks is clearly observed when comparing the San Fernando (1971), Boumerdes (2003), and Parkfield (1966) earthquake records. For the San Fernando motion, characterized by intermediate-frequency content and a PGA/PGV ratio of 1.09, the phase-plane trajectory remains compact and nearly elliptical at low PGA, indicating stable oscillations of the tank. Critical dynamic buckling occurs around 0.75 g, where a sudden expansion of the trajectory is noted, indicating the transition to post-buckling behavior with nonlinear energy exchange between vibration modes. For the Boumerdes earthquake, which is characterized by high- and long-period frequency content and a moderate PGA/PGV ratio of 2.00, instability is triggered at a much lower PGA of approximately 0.16 g. Beyond this threshold, the phase-plane trajectory is observed to become distorted and scattered, indicating early loss of stiffness and asymmetric amplification of displacements and velocities, reflecting local dynamic effects. For the Parkfield record, which is dominated by high-frequency, short-period content with the highest PGA/PGV ratio of 2.30, the earliest onset of instability is observed at approximately 0.11 g. The phase-plane trajectory is observed to rapidly diverge from equilibrium, indicating amplified displacements and velocities associated with local shell vibrations and diamond-shaped buckling modes. These observations highlight that the frequency content of seismic excitations plays a critical role in determining the timing of dynamic buckling. Intermediate-frequency motions are observed to primarily activate local shell modes, leading to

higher critical PGA, whereas high-frequency motions are observed to primarily excite local shell instabilities, resulting in earlier buckling at lower PGA levels. Fourier amplitude spectra further confirm these distinctions, with San Fernando energy concentrated below 5 Hz, Boumerdes peaking around 3 Hz, and Parkfield distributing significant energy across 3–10 Hz. Overall, this analysis demonstrates that evaluating seismic response solely based on PGA may lead to an underestimation of the susceptibility of thin-walled tanks to high-frequency-induced local buckling, emphasizing the need to consider both amplitude and frequency characteristics in seismic design.

3.4 Comparison with Codes and Previous Studies

The discrepancies observed between design-code predictions and the numerical results obtained in this study can largely be attributed to differences in loading assumptions. Design codes typically determine elastic and plastic critical buckling stresses based on static analyses of cylindrical shells under uniform axial compression or a combination of axial compression and circumferential shear. These idealized conditions differ significantly from the dynamic seismic loading scenarios considered here, which helps explain the variations in critical buckling values. In addition, the results of the present study were compared with other numerical studies reported in the literature. Good agreement was observed, although the present study considers a slenderer tank (higher H/D ratio), which tends to buckle earlier than the tanks examined in previous studies. The critical buckling values adopted in Eurocode 8 [20], API 650 [21], and AWWA D100 [28] are summarized in Table 2.

Nonlinear dynamic simulations were conducted to determine the critical buckling PGA for the analyzed tanks and to compare these results with code-based estimates. Under the 1971 San Fernando earthquake, the simulations indicate a critical PGA of approximately 0.76 g. This value closely matches the results reported by Djermane [10] for a tank with H/D = 1. In the present study, the slightly earlier onset of buckling is due to the slenderer tank configuration (H/D = 2), which increases structural flexibility and promotes earlier instability. Design-code predictions for the same scenario are lower, with AWWA D100 estimating 0.27 g, while Eurocode 8 and API 650 provide 0.62 g and 0.55 g, respectively, reflecting the generally conservative nature of these standards.

Table 2. Comparison between numerical predictions and design guide values for critical stress and buckling

Standards	Numerical model	VIRELLA H/D=1	DJERMANE H/D=1	EC8	API	AWWA
Critical stress MPa	/	/	/	1.384E8	1.22E8	5.944E7
Critical buckling According to San Fernando	0.76g	/	1.00g	0.62g	0.55g	0.27g
Critical buckling According to Boumerdes	0.16g	/	/	0.09g	0.08g	0.04g
Critical buckling According to Parkfield	0.11g	0.20g	1.07g	0.08g	0.07g	0.04g

For the 2003 Boumerdes earthquake, the numerical analysis reveals critical PGA reductions of approximately –50% and –75% relative to API 650 and AWWA D100, respectively, further highlighting the conservative nature of code-based design approaches. In the case of the 2004 Parkfield earthquake, dynamic buckling occurs at 0.11 g. This value is slightly lower than that reported by Virella (Virella et al., 2006) for a tank with H/D = 1, owing to the increased flexibility of the slender tank configuration. Comparisons with code predictions indicate that Eurocode 8 yields 0.08 g, corresponding to a deviation of –27%, API 650 provides a similar estimate, and AWWA D100 gives a substantially lower value of 0.04 g. These results collectively emphasize that detailed nonlinear dynamic simulations provide a more realistic prediction of buckling behavior under seismic loading. In contrast, design-code-based approaches tend to be more conservative,

particularly for slender tanks, underscoring the importance of accounting for dynamic effects and geometry when evaluating structural stability under earthquake excitations.

4. Conclusions

The present numerical investigation has demonstrated that tall thin-walled cylindrical steel tanks exhibit a high sensitivity to both the intensity and frequency content of seismic excitations. A clear correlation was observed between critical buckling loads and the characteristics of the applied ground motions. In particular, the San Fernando (1971) earthquake, characterized by intermediate-frequency content and a PGA/PGV ratio of 1.09, was found to induce buckling at a PGA of approximately 0.76 g, which is in close agreement with predictions provided by Eurocode 8, exhibiting only an 18% deviation. In contrast, the Boumerdes (2003) and Parkfield (1966) ground motions, which contain substantial high-frequency components and exhibit higher PGA/PGV ratios, were associated with significantly lower critical PGA values of 0.16 g and 0.11 g, respectively. These findings indicate that the frequency content of seismic records exerts a pronounced influence on the dynamic stability and local buckling behavior of tall thin-walled tanks, often resulting in the onset of instability at lower PGA levels than predicted by static or quasi-static code-based formulations.

The observed elastic diamond-shaped local buckling near the top region of the tank walls was found to be consistent with previously reported experimental and numerical studies, confirming that instability tends to initiate in regions where bending moments are high and axial stiffness is reduced. Fourier analysis of the earthquake records further demonstrates that local shell instabilities and diamond-shaped buckling are triggered under both frequency ranges. However, the buckling of the tank is predominantly delayed under intermediate-frequency motions. In contrast, earlier triggering of the same buckling patterns is observed under high-frequency excitations, with critical thresholds being reached at significantly lower PGA values. It is therefore demonstrated that PGA alone is insufficient for the accurate assessment of seismic-induced buckling risk, and that the spectral content and the duration of seismic motions must also be considered.

Finally, the present work provides a structured framework for future research aimed at enhancing the seismic performance of cylindrical steel tanks. It is suggested that subsequent investigations involve a larger set of seismic records, including more than twenty motions, to more comprehensively evaluate the influence of frequency characteristics on dynamic buckling. Additionally, mitigation strategies, such as the incorporation of stiffening rings, could be explored to reduce the susceptibility of tanks to diamond-shaped local buckling. By explicitly defining the tank configuration, identifying its vulnerable regions, and systematically examining the effects of seismic frequency content, this study establishes a solid foundation for performance-based design approaches that go beyond simplified static criteria, ultimately contributing to the improved safety and reliability of storage tanks under earthquake loading.

Nomenclature

D	Tank diameter(m)	$\ddot{x}g$	Ground acceleration m/s ²
R	Tank radius (m)	ρ	Density equivalent
H	Tank height (m)	PGA	Peak ground acceleration (g)
T	Tank shell thickness	PGA _c	Critical peak ground acceleration (g)
ν	Poisson ratio	t	Time (s)

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Computational fluid dynamics analysis of wind loading on antennas attached to a telecommunication tower

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Abstract

The rapid expansion of telecommunication infrastructure has increased the need for reliable structural assessments of monopole towers as antennas are added or reconfigured. Traditionally, these assessments assume independent wind loading exposure for each antenna using conservative force coefficients or wind tunnel testing, often leading to overdesign. This study investigates the application of Computational Fluid Dynamics (CFD) as a more accurate and cost-efficient alternative. Using ANSYS Fluent and CATIA V5, a variety of antenna configurations were modeled using and analyzed under steady-state wind conditions. The results demonstrated that bending moments predicted for clustered antenna arrangements were up to 52% lower than those obtained from traditional design standard codes such as TIA-222 and SANS 10225. These reductions are primarily due to aerodynamic shielding effects. The study highlights CFD as a reliable design tool for telecommunication towers, validated against wind tunnel data and supported by the mesh independence studies conducted. These findings suggest that CFD offers a more accurate and cost-efficient alternative to traditional conservative assumptions, with the potential to optimize structural design and reduce material usage. It confirms the potential of CFD, using a standard laptop to reduce conservatism and analysis time without compromising safety.

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1. Introduction

Telecommunication towers are critical infrastructure enabling mobile and wireless communication. These towers, which include monopoles, lattice structures, guyed masts, and camouflaged artificial trees, are designed to elevate antennas for optimal signal transmission to specific heights. As the telecommunications sector evolves rapidly, frequent updates to antenna configurations necessitate accurate structural assessments to ensure tower integrity.

Traditionally, registered professionals rely on the use of wind tunnel testing or conservative design standard codes to evaluate wind loading on towers. However, wind tunnel facilities such as those operated by the Council for Scientific and Industrial Research (CSIR) are not always accessible, and code-based methods may result in overdesign due to simplifications and conservative assumptions.

This study aims to:

- Evaluate the use of CFD to simulate wind effects on telecommunication towers with various antenna configurations, scale geometry modeled from CATIA V5.
- Compare CFD results against wind tunnel data and established standards design code.
- Determine the reduction in bending moment and conservatism achievable through CFD results analysis.

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- Demonstrate the practicality and efficiency of conducting such analyses on standard computing hardware.
- Assess the feasibility of extracting and processing force coefficients for antenna configurations, at different heights for design purposes.

While traditional methods such as wind tunnel testing and standard design codes remain invaluable. In this study, CFD has proven its ability, with a positive outcome that was successfully compared with the existing data. Additionally, CFD gives registered professionals more options to generate useful data if needed for telecommunications infrastructure design and analysis.

2. Methodology

Scaled 3D geometries of monopole towers with different antenna arrangements were created using CATIA V5, as shown in (Fig. 1). Five configurations were analyzed: a bare mast and mast with one, two, three, and six antennas arranged in radial and stacked formations as shown in (Fig. 2).



Fig. 1. Left Mast and 6 antennas wind tunnel, right Mast and 6 antennas CATIA V5 design

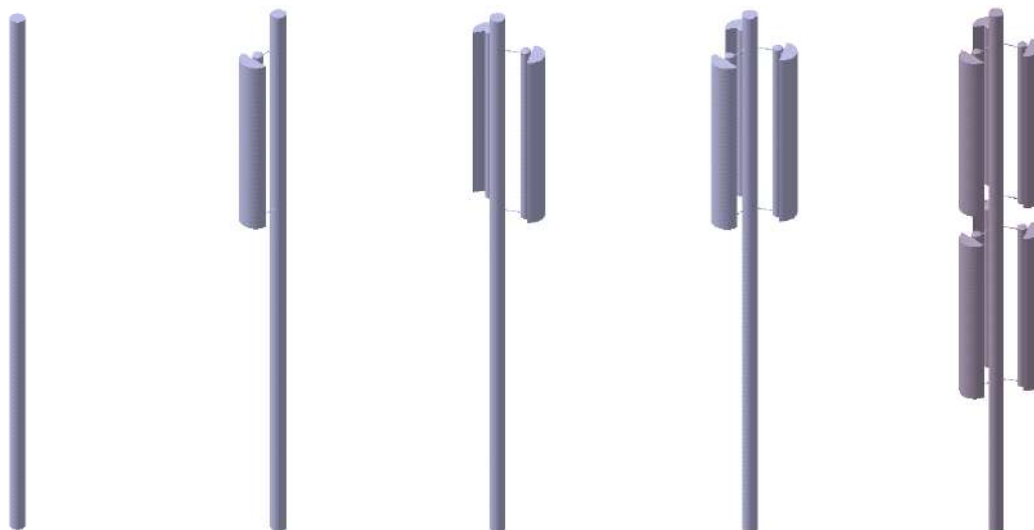


Fig. 2. CATIA V5 3D design, Mast only, Mast and 1 antenna, Mast and 2 antennas, Mast and 3 antennas, Mast and 6 antennas

Simulations were performed in ANSYS Fluent using the Reynolds-Averaged Navier-Stokes (RANS) method with the SST k- ω turbulence model. Several turbulence models were reviewed, with RANS and SST k- ω models identified as the most effective for bluff body simulations such as telecommunication towers. These models are well-suited for external aerodynamic analysis of bluff

bodies and allow for accurate prediction of separation and wake regions. To reduce RANS turbulence model uncertainty in outcomes prediction, a few measures were performed before each simulation in terms of geometry, fluid density, viscosity, grid, and boundary conditions [1].

The computational domain was defined following best practices, with domain extents based on mast height and antenna diameter using method of [2,3]. Five Body of Influence (BOI) regions were used to refine the mesh locally around critical geometry, including the antenna bodies and wake regions, according to [4] presented in (Fig. 3). The domain side view is shown in both (Fig. 4 and Fig. 5) and with all the necessary dimensions used. Boundary conditions included a uniform inlet velocity of 11.47 m/s and an outlet pressure set to atmospheric as per [5]. Air density and viscosity were matched to wind tunnel conditions ($\rho = 1.04068 \text{ kg/m}^3$, $\mu = 2.034\text{e-}5 \text{ Pa}\cdot\text{s}$). Probe lines were used in the solver to extract wind velocity.

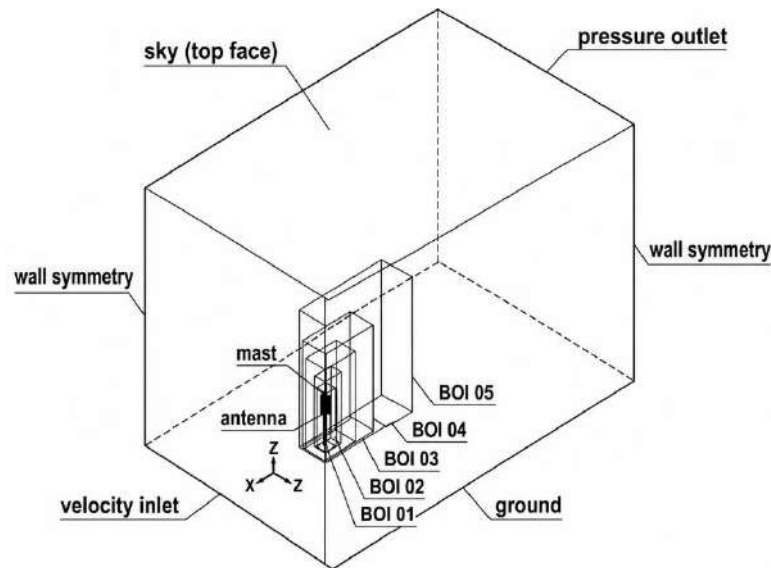


Fig. 3. CATIA V5 Domain Isometric View

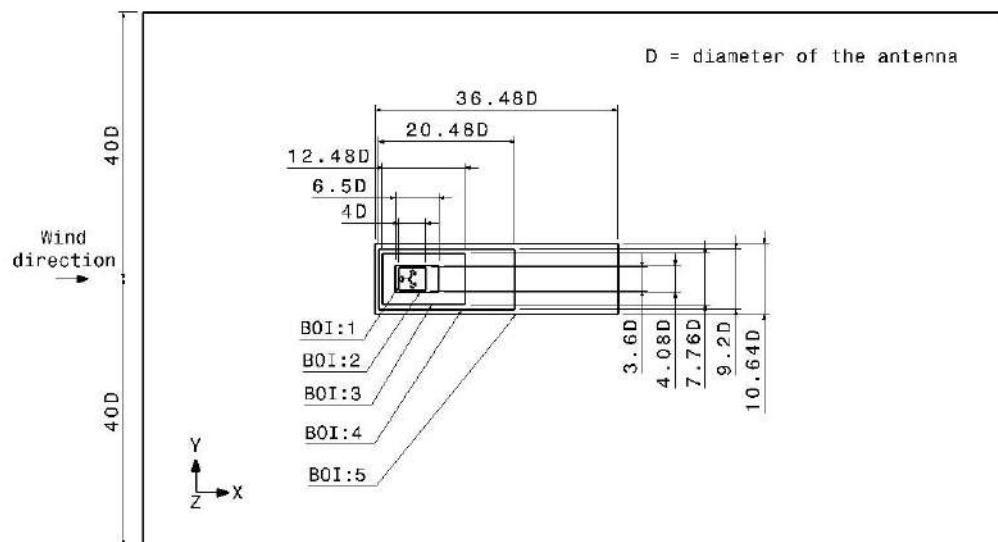


Fig. 4. CATIA V5 Domain Top View

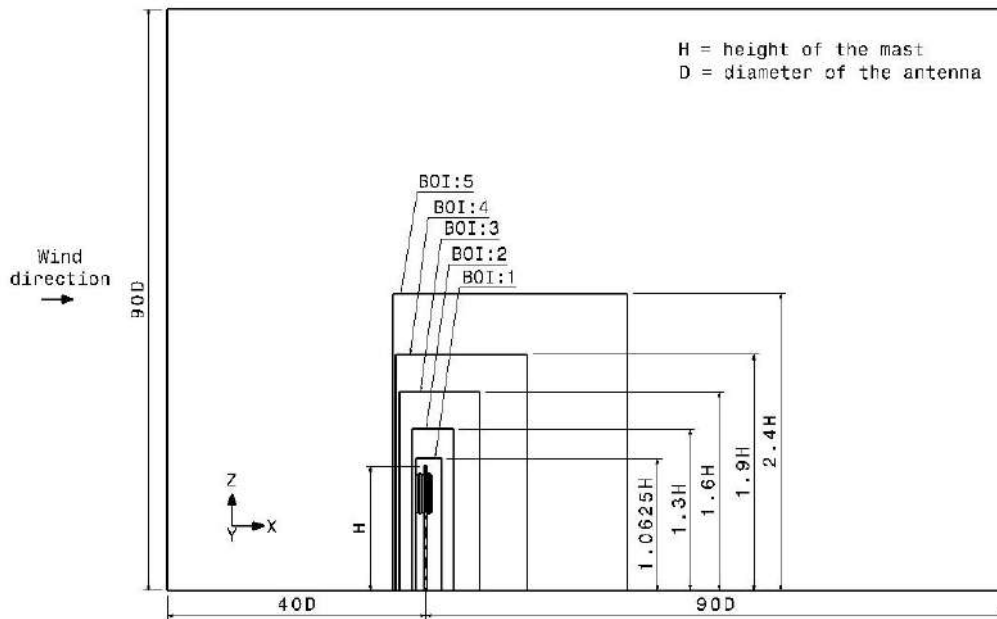


Fig. 5. CATIA V5 Domain Side View

2.1. Meshing Process in CFD

During the meshing process some guidelines from Qin were adapted in terms of meshing sizes, but each set of simulations in this study had unique sizes. With ANSYS® Fluent there are four types of mesh available in the watertight geometry interface, polyhexcore, hexcore, polyhedral and tetrahedral. Their availability is to do with different types of simulation that have to be done and adapted accordingly. If for example tetrahedral mesh type is used in the watertight geometry environment, once the geometry is transferred into solver interface there is an option to convert mesh to polyhedral [6]. In this study two types of mesh were adapted and used as suited, from polyhedral to tetrahedral (Fig. 6) illustrates the two types of mesh used in this study.

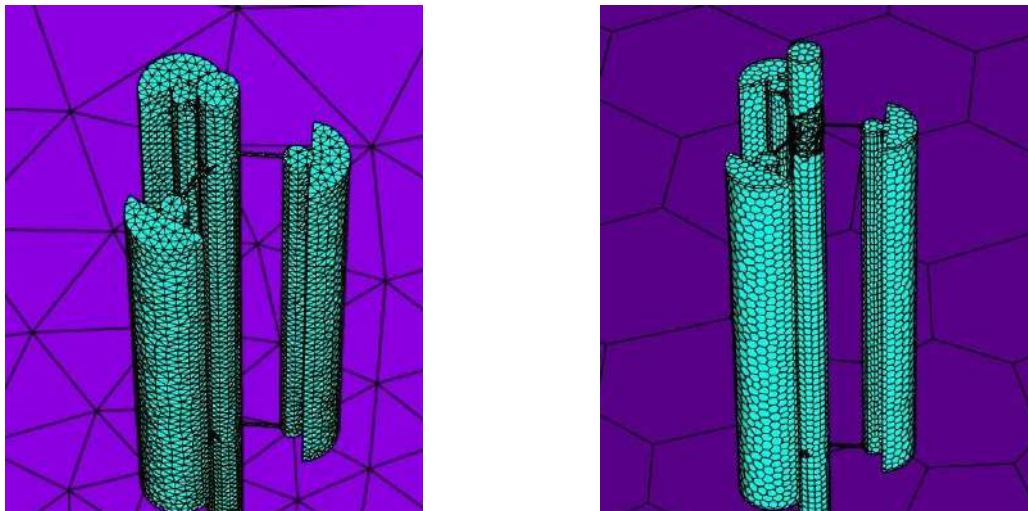


Fig. 6. Tetrahedral left and Polyhedral right mesh type mast with three antennas

Polyhedral meshes are generally associated with improved solver stability and reduced CPU time compared to tetrahedral meshes. Transitioning between mesh types within the solver is also efficient. Furthermore, [7] places more emphasis on the four types of mesh from watertight interface and in terms of mesh cells count, and preference. The combination of mesh type and inflation layers on the mast, either with or without antennas, varied throughout the experiment, although the domain or volume remained constant. This approach was necessary due to differences in geometry for each experiment. The introduction of bodies of influence (BOIs) enables localized mesh refinement in critical areas, such as the wake zone or antenna interfaces, while maintaining

coarser meshes in other areas of the domain. Standard near-wall treatment function correlation in Fluent Solver was adapted in this study, which is set as the default under SST k- ω models. Correlation: blends between analytical expressions for the viscous sublayer and the logarithmic region, allowing consistent accuracy regardless of mesh refinements described by [8].

2.2. Drag Force and Force Coefficient Settings in CFD

The settings to capture both the drag force and force coefficient in CFD were done for both axis in X and Y directions as standards practice. Once a set of simulations is completed, these two forces can be extracted using azimuth direction of 0° and 180° as shown in (Fig. 7) these two angles were adopted in this study.

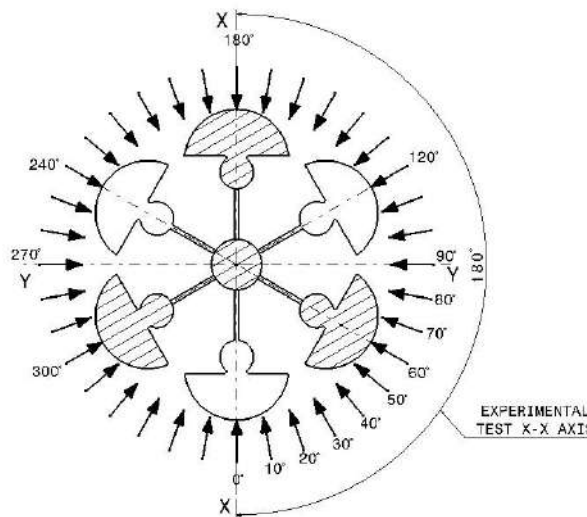


Fig. 7. Azimuth model set up in CFD domain

There is an advantage when using simulation software, especially in solver interface, a Notepad as a macro can easily be imported with coordinates and used to extract the required data. One aspect is referred to by several names; in a wind tunnel, it is known as the force coefficient, while in CFD settings, parameters, it is the drag coefficient and later becomes the pressure coefficient when the results are being processed in the solver. Mesh independence was verified through Grid Convergence Index (GCI) analysis over four mesh densities, with final simulations using polyhedral and tetrahedral mesh types ranging from 500k to 2.5 million cells, wind speed and dynamic pressure, and to determine force and moment coefficients. Table 1 displays the processed data from four simulations, prior to GCI.

Table 1. Results of four simulations, forces and force coefficients

Simulations	Iterations	Forces (N)	Force Coefficient	Mesh Elements Count
1	100	0.379	0.9980	265736
2	100	0.372	0.9424	528808
3	100	0.362	0.9168	1132456
4	100	0.356	0.9020	2466964

The GCI evaluation confirmed mesh independence, as the force values exhibited convergence requirements. The adaptation of the domain was based on consistent convergence behavior, force values, which aligned with physical wind tunnel results, and provided a robust foundation for subsequent analysis. The boundary layers on the mast were kept unchanged during these four simulation exercises. To account for the y plus value, the settings of boundary layers were taken as smooth layers, with the number of layers set at 20, transition ratio of 0.272, and with the growth rate of 1.2. Y+ value of the last set of completed simulations was extracted and was in the range of 0.0753 to 3.636 maximum.

The GCI was achieved using [9], method with 99.98 percent while using a polynomial line, an indication of good agreement during the experiment and continuation of using the same domain. Once the force and velocity pressure are extracted, it is easy to use the appropriate equations to find the force coefficient Table 2.

Table 2. Grid convergence study results for force and force coefficient

Performance parameter	Coarse	Medium	Fine		p	GCI ₁₂	GCI ₂₃	ϵ	Asymptotic range 23-12
	f_3	f_2	f_1	$f_{h=0(12)}$					
Force (N)	0.372	0.362	0.356	0.347	0.648	3.160	5.180	0.017	0.983
Force coefficient	0.9424	0.9168	0.902	0.8138	0.695	2.811	4.653	0.016	0.957
Mesh count	528808	1132456	2466964						

The extracted results were processed to verify the mesh converged solution by analyzing the variation of the drag force and drag coefficient in (Fig. 8) with increasing mesh densities. Parameters extracted from the solver solution, such as drag force, are both sensitive to mesh cell size and shape. The results are shown in (Fig. 9), in which the drag force stabilized beyond approximately 2.5 million elements, while the drag coefficient plateaued around 0.90.

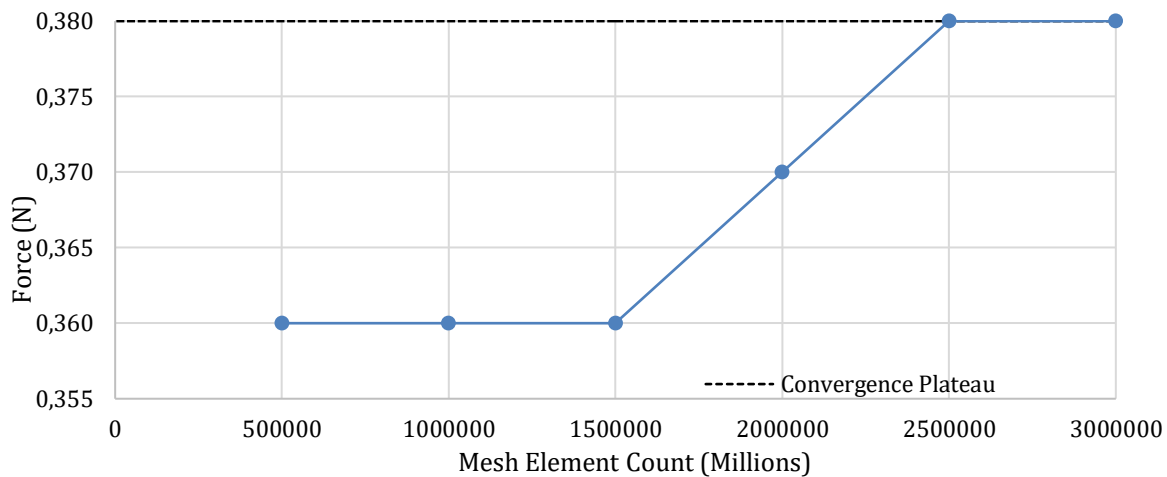


Fig. 8. Relationship between mesh element count and total drag force obtained from CFD simulations

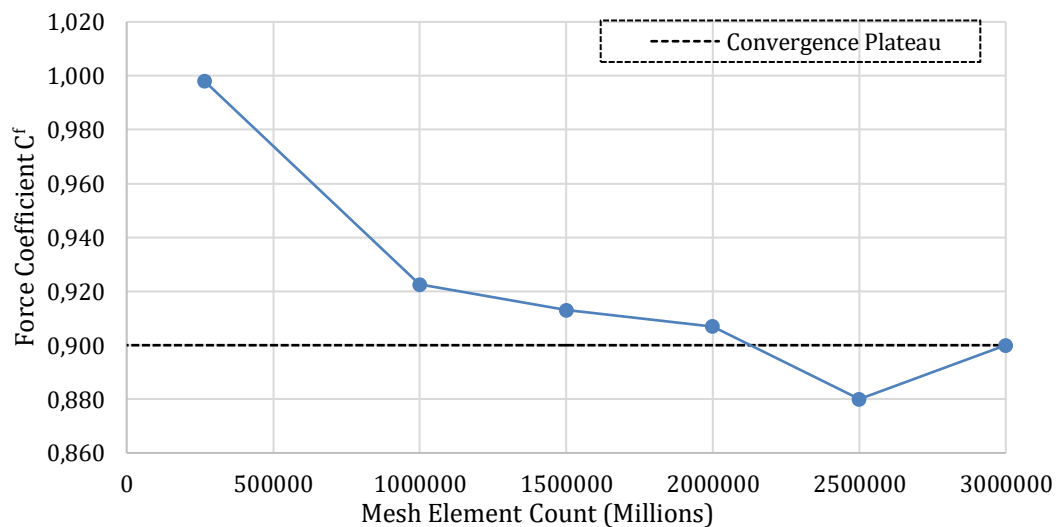


Fig. 9. Mesh element count versus drag coefficient C_d

Mesh convergence analysis confirmed that a mesh count above 2.5 million cells produced stable force and moment values, with minimal sensitivity to further refinement. Tetrahedral meshes resulted in nearly double the cell count compared to polyhedral meshes under equivalent refinement criteria. Results were compared with experimental data from CSIR wind tunnel conducted by Annandale.

3. Results

3.1. Parameters Extraction from Simulations Results

Once simulations have been completed in the CFD solver, all the elements like force, wind speeds, projected area, and velocity pressure were extracted from the solution file. Regarding the force coefficient, further steps were required that involved the use of Eqs (1), (2), (3), (4), and Eq (5) in a combination with standard codes and compared with wind tunnel.

- SANS 10255 (South Africa):

$$F = q_z \times A_{eff} \times C_g \times C_f \quad (1)$$

Where; F = resultant nominal wind force at height z (N), q_z = velocity pressure at height z (above atmospheric pressure) (N/m^2), A_{eff} = effective frontal area at height z (m^2), C_g = gust loading factor, C_f = force coefficient, z = height measured from the base to the centroid of the antenna (m)

- TIA-222-H (United States):

$$F = \frac{1}{2} \times \rho \times C_{dp} \times V^2 \times A \quad (2)$$

Where; F_w = wind force, ρ = air density, C_{dp} = profile drag coefficient, λ = length-to-width aspect ratio correction factor, V = wind velocity, A = cross-sectional area normal to wind direction

- EN 1993-3-1 and EN 1991-1-4 (Europe):

$$F_w = \frac{1}{2} \times \rho \times (C_f \times \Psi_r \times \Psi_\lambda) \times V^2 \times A \quad (3)$$

Where; F_w = wind load (N), C_{fo} = force coefficient for rectangular sections without rounded corners, Ψ_r = reduction factor for a rectangular section with rounded corners, Ψ_λ = end-effect factor, A = windward projected area (m^2), V = wind speed (m/s), ρ = air density (kg/m^3)

- AS 3995-1994 and AS/NZS 1170.2:2011 (Australia and New Zealand):

$$F = \left(\frac{1}{2} \times \rho_{air} \right) \times C_{fig} \times C_{dyn} \times (V_{des,\theta})^2 \times A_z \quad (4)$$

Where; F = wind force (N), ρ_{air} = air density (kg/m^3), $V_{des,\theta}$ = design wind speed in direction, C_{fig} = aerodynamic shape factor, C_{dyn} = dynamic response factor, A_z = reference area (m^2)

The refined equation used to isolate the antennas for the coefficient was:

$$C_{f,anenna} = \frac{F - (q_z \times A_{shaft} \times C_{f,shaft})}{q_z \times A_{antenna}} \quad (5)$$

Once all the data were processed, the comparison was conducted as the final phase. Table 3 represents the setup for the settings used for all four cases, including mesh sizes and the number of cells.

Time for simulations state: RANS Steady modelling was adopted, and Turbulence Mode: Viscous (SST k-omega) Menter adopted, wind speed 11.47 m/s, which remained static. The air density of 1.04068 kg/m^3 is derived from the wind tunnel data, whereas the viscosity of 0.00002034 is calculated through interpolation.

Probe lines as shown in (Fig. 10) were created and used in the solver to extract data results from the solver; their position did not change in all four simulation configurations. With probe lines using the function of line rake, to extract the wind pressure and wind speeds were extracted using the X-Y function in ANSYS solver.

Table 3. ANSYS Fluent parameter settings for Mast configurations

Parameters	Case 1	Case 2	Case 3	Case 4
	Mast with one antenna	Mast with two antennas	Mast with three antennas	Mast with two set of three antennas
Surface mesh size min	3.17383	3.17383	3.17383	3.17383
Surface mesh size max	100	81.22	200	125
Cell size max (mm)	100	103.5	262	100
Cell type	Tetrahedral	Polyhedral	Polyhedral	Polyhedral
Number of cells	371779	321263	313540	478120
Y+ range	0.055 to 16.6	0.072 to 61.179	0.187 to 352.233	0.060 to 90.713

CFD simulations produced results closely matching wind tunnel measurements. Deviations in wind speed and dynamic pressure were within 1%, while force coefficients differed by less than 6%. For the mast with three antennas, the calculated base bending moment from CFD was 0.18 Nm compared to 0.372 Nm from TIA-222-based calculations, indicating a 52% reduction due to shielding effects. Dynamic pressure was calculated using Eq (6) and used in conjunction with extracted force values to determine drag coefficients. Antenna-specific coefficients were derived by subtracting the mast-only contribution.

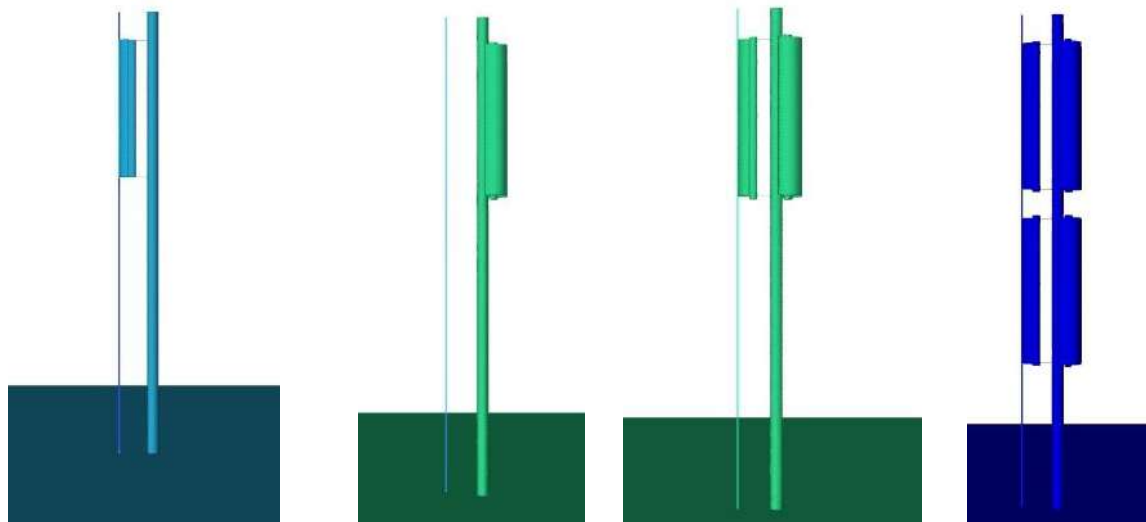


Fig. 10. Probe lines in the solver for Mast and different configurations

$$q_z = 0.5 \times \rho \times V^2 \quad (6)$$

3.2. Comparison of Standard Codes Force Coefficients

Using the standard codes from at least four regions, the force coefficients were processed while taking into consideration all their respective aspects and related equations. Each standard code has unique requirements that must be adhered to for the antenna force coefficient. The variation in the force coefficient of antennas results was easily noticeable due to the process of each standard code that had to be followed. In some of the cases results seem to be more conservative, when the number of antennas increases. The results from different standard codes were compared with those from wind tunnel results. The results from the wind tunnel, CFD, SANS 10255 and international codes: AS/NZS 1170.2, TIA-222, and EN 1991-1-4, of the cylinder without attachments, results are shown in (Fig. 11). Their variation is clear according to each code requirement, except for one that the Reynolds number must be accounted for EN 1991-1-4.

In (Fig. 12), are the force coefficients results for mast with antenna configurations from CFD, wind tunnel, and standard code. The difference in results for mast with one antenna is attributed due to standard codes elements aspect ratio assumptions, aero-dynamic shape factors, and wind exposure

classification. The variation for the force coefficient of the mast with two antennas attached differed in relation to the assumptions of elements highlighted, such as aspect ratio from TIA-222 and EN 1991-1-4, and the impact of aerodynamic factors from AS/NZS 1170.2, while the wind tunnel is still bound to SANS 10225, and CFD. For the mast three antennas attached, the force coefficient for both CFD and wind tunnel has shown a strong agreement. While for the standard code, variation still shows in the aspect ratio and shape factors. Looking at the force coefficients in of the six antenna configuration, there is agreement between wind tunnel and AS/NZS-1170.2, while there is a variation from CFD, TIA-222 and EN1991-1-4.

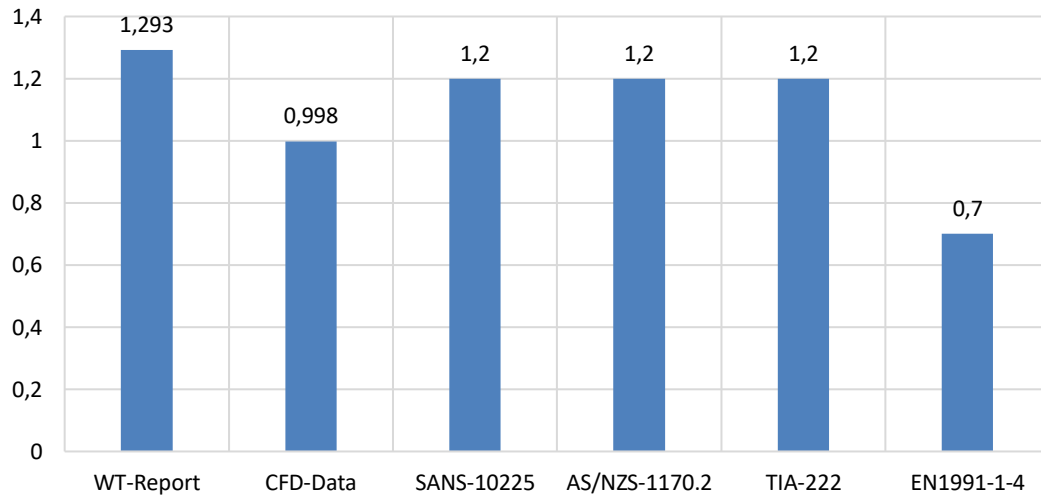


Fig. 11. Force coefficient for Mast only

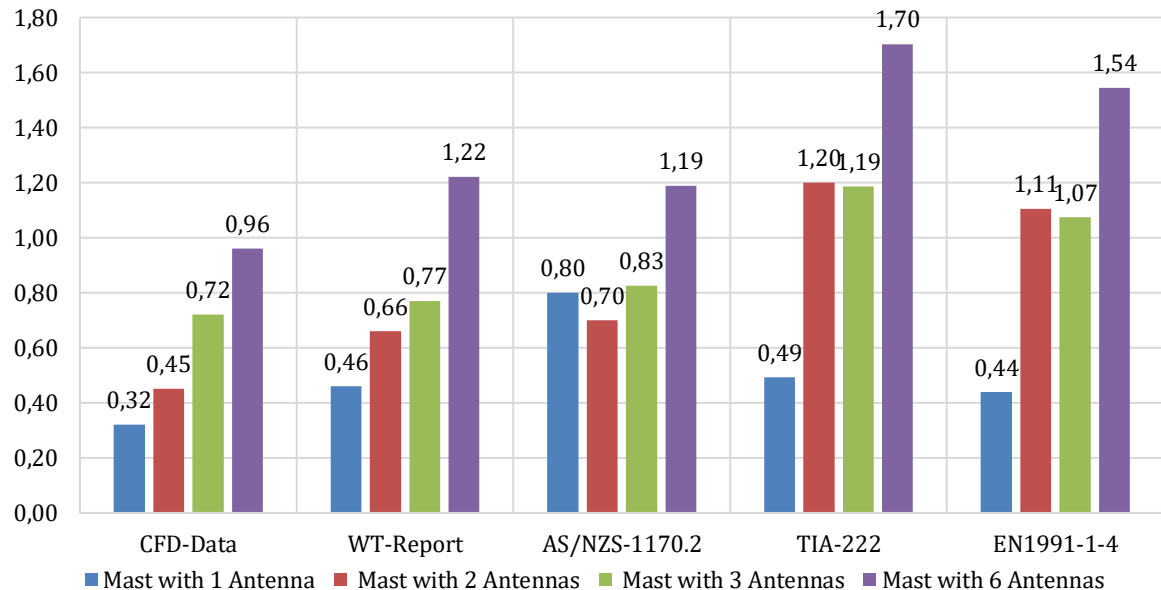


Fig. 12. CFD, wind tunnel and standard codes force coefficient for Mast with all configurations

3.3. CFD Force Coefficient for Mast with Three Antennas in Radial

ANSYS Fluent has several features that provide advantages which can be used up to a certain stage, as was the case in this study. The X, Y, and Z coordinates were generated in Excel, transferred to Notepad, and then imported into CFD solver to extract the force coefficients for statistical analysis at ten-degree intervals. The coordinates were generated following the principles and processes outlined by [10]. These were plotted using Excel, and the variances are visible at each 10-degree

interval. If desired, this process can be repeated at various heights of a telecommunication tower to obtain varied outcomes.

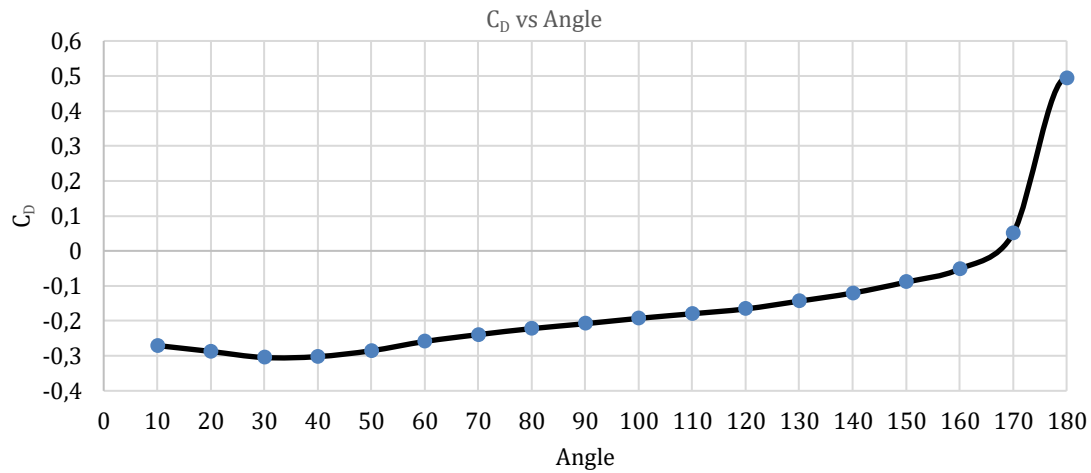


Fig. 13. CFD results for mast with one antenna; force coefficient at 10° intervals

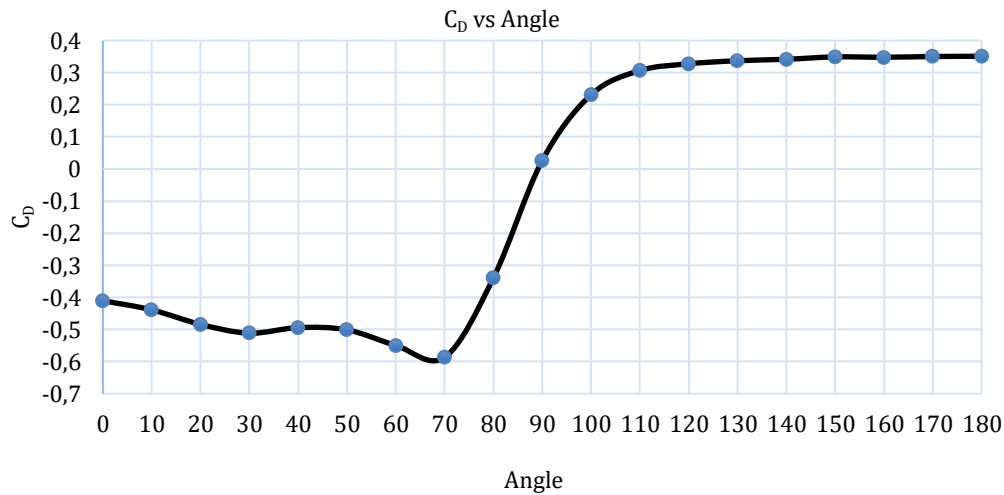


Fig. 14. CFD results for mast with two antennas; force coefficient at 10° intervals

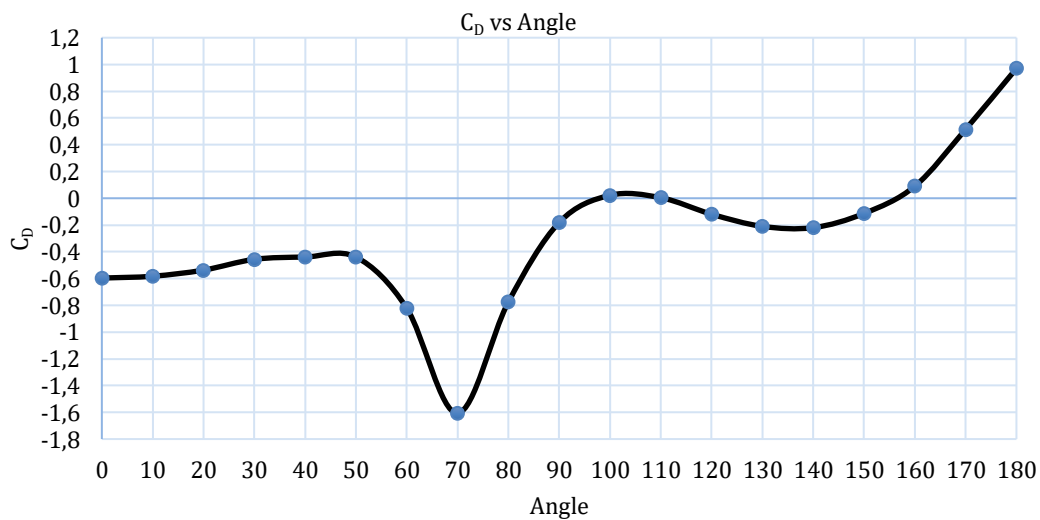


Fig. 15. CFD results for mast with three antennas; force coefficient measured at 10° intervals

This method resulted in an increased standard deviation in the force coefficient across the angles compared to previous setups, demonstrating that wind exposure for this arrangement is strongly orientation dependent. (Fig. 13) exhibits a unique fluctuation in force coefficient for a mast with a single antenna between 0° and 180° , representing the impact of geometrical profile. (Fig. 14) illustrate the force coefficient for a mast equipped with two antennas; the change in profile is unique due to the number of attachments. (Fig. 15) show this variance, with noticeable dips matching to the shielding position for mast with three antennas. (Fig. 16) demonstrate variation of force coefficient profile for the bottom profile, mast with six antennas showing a modest dip at 70° degrees when compared to the three attachments due to their height level positioning. In (Fig. 17), top force coefficient for the mast with six antennas this profile seems similar to the mast with three antennas in terms of shape.

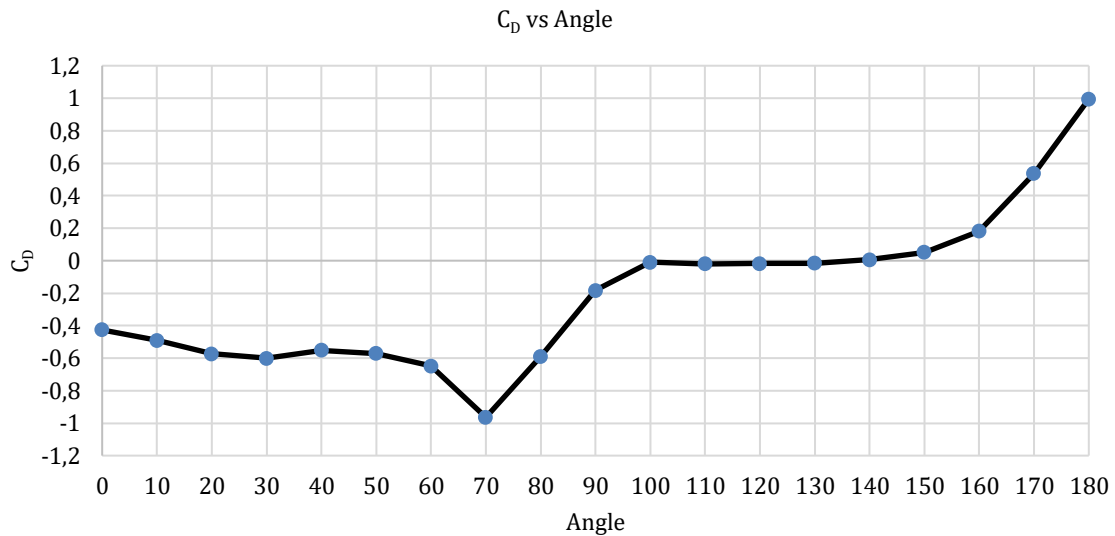


Fig. 16. CFD results for mast with six antennas; bottom force coefficient at 10° intervals

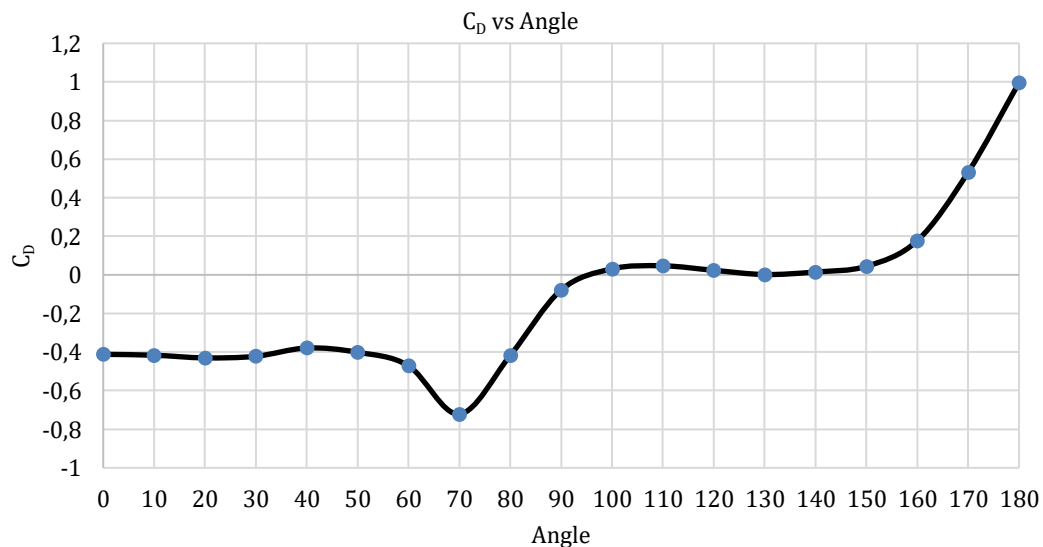


Fig. 17. CFD results for mast with six antennas; top force coefficient at 10° intervals

3.4. Bending Moments Comparison Between CFD and Design Code

The aim of this comparison was to evaluate the discrepancy between predicted bending moments from CFD and those obtained through conventional design code methods such as those in [11-16]. The procedure of design standards assumes that each antenna acts independently, contributes to full wind loads at a known height, whereas CFD accounts for aerodynamic shielding and interaction

effects. By quantifying this difference, the study tests the hypothesis that CFD can significantly reduce the design moment compared to traditional assumptions, particularly for the cluster antenna configurations as shown in Table 4. Table 5, represent the moment results for antenna configurations only.

Table 4. Moments for different mast configurations

System	Mast only (Nm)	Mast & 1 Antenna (Nm)	Mast & 2 Antennas (Nm)	Mast & 3 Antennas (Nm)	Mast & 6 Antennas (Nm)
WT	0.12	0.14	0.19	0.28	0.56
CFD	0.09	0.12	0.21	0.27	0.55

Table 5. Moments for antenna configurations

System	One Antenna (Nm)	Two Antennas (Nm)	Three Antennas (Nm)	Six Antennas (Nm)
WT	0.020	0.07	0.16	0.42
CFD	0.030	0.12	0.18	0.45

3.5. Governing equations for Design Code Wind Moments

Design codes typically estimate wind force on each attachment by using Eqs (1), (2), (3) and (4).

Where; $\rho = 1.04068 \text{ kg/m}^3$ (air density), $C_d = 1.3$ (for Semi-Circular Cylinder, Convex Face Upstream), A = frontal area of a single antenna ($0.025 \times 0.150 \text{ m}^2$), V = wind speed from simulation, F = wind force acting on one antenna

The bending moment at the base of the mast is:

$$M_{code} = \sum(F_i \times h_i) \quad (7)$$

For equal antenna heights:

$$M_{code} = N \times F \times h \quad (8)$$

Where; N: Number of antennas (1, 2, 3, or 6 depending on configuration), H: Height from mast base to the antenna center of pressure, this is the moment arm used in all calculations that follow.

3.6. Sample Calculation for Mast with Three Antennas

Each antenna set has:

$$A = 0.00375 \times 3 = 0.01125 \text{ m}^2 \quad (9)$$

Then;

$$F = \frac{1}{2} \times 1.04068 \times 1.3 \times 0.01125 \times (11.43)^2 \approx 0.992 \text{ N} \quad (10)$$

Then;

$$M_{code} = 1 \times 0.992 \times 0.375 = 0.372 \text{ Nm} \quad (11)$$

From Table 5 (CFD results) $M_{CFD} = 0.180 \text{ Nm}$

Reduction:

$$Reduction = \left(1 - \frac{0.180}{0.372}\right) \times 100 \approx 51.6\% \quad (12)$$

3.7. Summary of Moment Comparison

Table 6 and Figure 18 illustrate the percentage reduction in bending moments when using CFD simulations compared to traditional code-based computations in different antenna configurations. The reductions range from approximately 28% to 76%, highlighting the aerodynamic efficiency captured by CFD that is not considered in conventional design assumptions.

Table 6. Summary of moment comparison

Configuration	CFD Moment (Nm)	Code Moment (Nm)	Reduction (%)
One Antenna	0.030	0.125	76%
Two Antennas	0.120	0.249	52%
Three Antennas	0.180	0.372	52%
Two Sets of 3 Antennas	0.450	0.627	28%

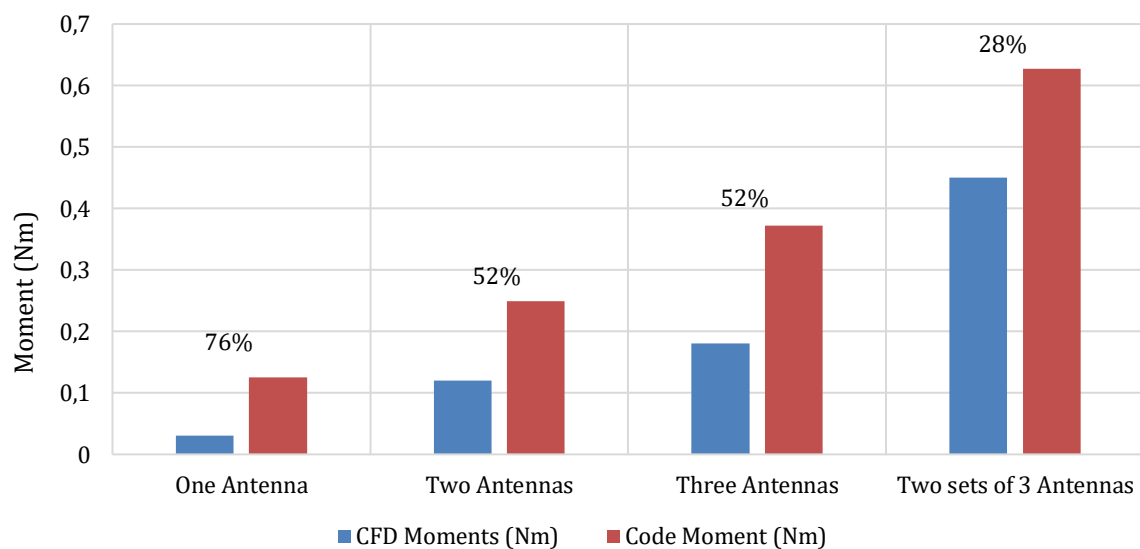


Table 7. Reduction in bending moments (CFD vs Code-Based) by antenna configuration

3.8. Projected and Effective Areas

Across all antenna configurations, the projected areas derived from CFD (via solver-based projection in either X or Y direction axis) and CAD (via geometric measurement) showed consistent alignment, with discrepancies typically under 5%. For example, the three-antenna configuration yielded CFD and CAD areas of 0.0062 m² and 0.0064 m², respectively. In the most complex setup with two tiers of three antennas, the solver and CAD returned 0.0089 m² and 0.0091 m², respectively.

4. Discussion

The results highlight the limitations of standard design codes in capturing aerodynamic interactions among multiple antennas. Design codes such as TIA-222, SANS 10225, and EN1991-1-4 treat antennas as fully exposed independent elements, ignoring aerodynamic shielding, which CFD captures effectively. In the six-antenna configuration, shielding led to a 28% reduction in predicted base moment.

Furthermore, projected and effective frontal areas derived from CFD matched within 5% of CAD-based measurements. This agreement confirms that CFD-based geometry projection is reliable for force coefficient computation. The ability to simulate full-scale geometries without scaling, combined with accurate turbulence modeling and refined meshing strategies, makes CFD a viable substitute for physical testing.

The integration of artificial intelligence in CFD is briefly discussed, noting its growing role in solver automation and predictive modeling by [17]. Although not directly applied in this study, the potential for AI-enhanced CFD to further reduce simulation time and increase accessibility is significant as discussed by [18-20].

An example of retrieved results following simulation for a mast and one antenna can be found in Appendix A.

- Scaled residuals
- Iterations vs Forces in X direction
- Iterations vs Forces in Y direction
- Velocity Magnitude
- Dynamic Pressure
- X-Y-Z Coordinates used to extract Force Coefficient

5. Conclusion and Recommendations

5.1 Conclusion

This study demonstrates the effectiveness of CFD in assessing wind loads on monopole telecommunication towers, for various antenna configurations. Key findings include:

- Mesh independence was achieved and validated using GCI methodology.
- ANSYS Fluent Academic Version 2023-R2 was available for simulations using, RANS SST k- ω turbulence modeling provides desired reliable results on standard laptop hardware.
- CFD findings were processed and compared to the wind tunnel report, taking into account the codes SANS-10255, AS/NZS-117.2, TIA-222, and EN1991-1-4. Five different designs configurations were tested: a bare mast with no attachments, a mast with one antenna, two antennas, a three-antenna radial layout, and a two-tiered arrangement with six antennas. Variations in the force coefficient comparison were obvious, owing to certain of the aspect criteria of each code.
- When compared to the wind tunnel, force coefficients could be derived at various heights of antenna designs using Notepad as a macro and processed in less amount of time, proving the advantage of CFD.
- CFD can reduce design code bending moment of estimates by up to 52% due to aerodynamic shielding effects between clustered antennas.
- CFD force coefficients aligned within 6% of wind tunnel results, validating simulation accuracy.
- The deviation in force coefficients between CFD and physical tests remained below 5.5% across all configurations, reinforcing confidence in the numerical approach.
- Projected areas from solver output agree closely with CAD measurements, ensuring accurate force predictions.
- The adoption of CFD enables more accurate, less conservative, and economically efficient structural assessments of monopole towers. Further it advocates for the integration of CFD into standard engineering practice, potentially in combination with AI tools for further optimization.
- Rapid and accurate examination of the structural integrity of existing telecommunications towers could also lead to the recommissioning of underutilized infrastructure. Furthermore, the implementation of CFD simulations during the design and site selection phases for new towers could enhance predictive capabilities, enabling registered professionals to anticipate environmental impacts and structural performance with greater precision before or during construction.
- Thus, the findings in this study have the potential to contribute meaningfully to the optimization and sustainability of telecommunications infrastructure. The design of telecommunication towers and antenna configurations could be optimized, analysis by registered professional can go in much deeper detail; giving a broader option when

designing. Important elements such as forces, wind speed, moments, dynamic pressure and force coefficient were successfully comparing to the wind tunnel data during this study.

5.2 Recommendations

Even though this study has shown how useful CFD is for designing telecommunication towers, there are still a number of areas that could be used in more studies:

- **Inclusion of Dynamic Wind Effects:** Steady-state wind simulations were the main focus of this study. Future study should use transient simulations to capture time-dependent phenomena, including vortex-induced vibration (VIV), galloping, and aeroelastic flutter, in order to further the understanding of fatigue behavior over time.
- **Expansion to Full-Scale Structures:** Although the simulations were scaled to match the wind tunnel models, full-scale simulations are recommended. This would necessitate high-speed computing resources but would yield insights into real-world aerodynamic performance at operating heights.
- **Incorporation of Terrain and Environmental Effects:** Realistic environmental factors, such as terrain roughness, atmospheric boundary layer profiles, and thermal gradients, should be included in the CFD model to simulate more accurate wind flow patterns.
- **Experimental Validation with Full-Scale Instrumentation:** In-situ measurements of actual towers under wind loading would offer further validation of CFD predictions. Sensor arrays mounted on towers could provide real-time force and displacement data for benchmarking.
- **Development of Simplified CFD-Based Design Guidelines:** The objective is to develop efficient, semi-automated CFD workflows and simplified adjustment factors that may be integrated into normal design practices, thereby connecting advanced simulations with routine engineering applications.
- **Exploration of AI-Augmented CFD:** With recent advances in artificial intelligence, future studies could investigate the use of AI to accelerate mesh generation, predict force coefficients, forces, or optimize tower geometries in real-time during simulation.
- **Evaluation of Additional Attachment Types:** The study focuses on cylindrical and planar antennas. Further study should evaluate the aerodynamic behavior of a broader range of attachments, including parabolic dishes, Remote Radio Units (RRU), and camouflaged (e.g., artificial tree-type) structures.

By pursuing these directions, future study can enhance the accuracy, applicability, and efficiency of CFD in structural engineering field. Such efforts will contribute toward modernizing design practices, reduce excessive conservatism, and ensure safer, more economical infrastructure across the telecommunications industry.

Acknowledgement

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Appendix

A. Extracted CFD simulation data for the mast and one Antenna

A.1. Solution Monitors Scaled Residuals

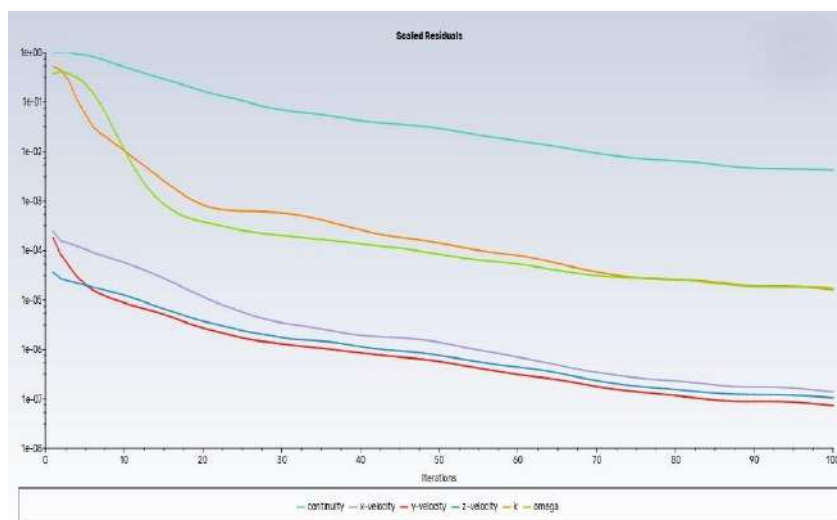


Fig. A. 1. Solution monitor scaled residuals

A.2. X-Direction Force vs Iterations

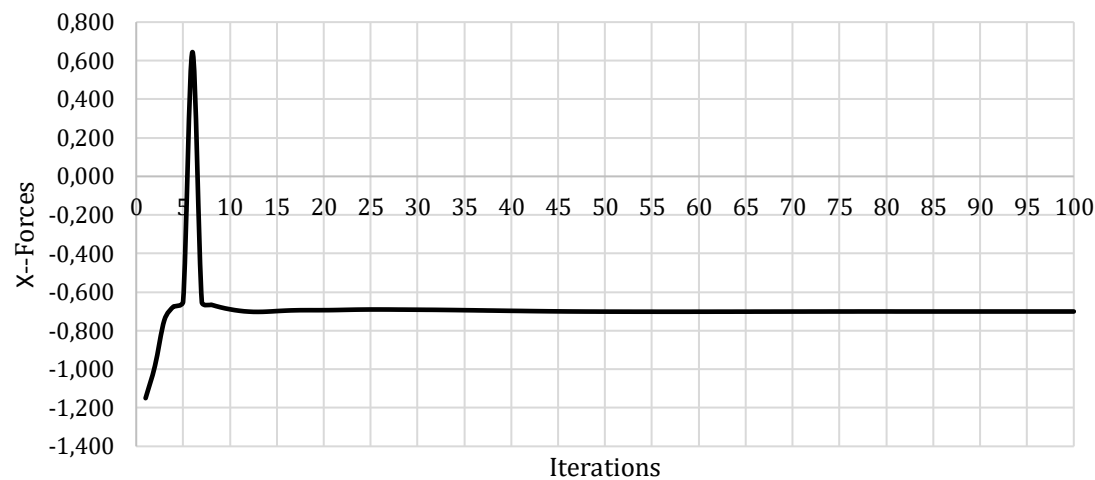


Fig. A. 2. X-Force vs Iterations

A.3. Y-Direction Force Iterations



Fig. A. 3. Y-Force vs Iterations

A.4. Velocity Magnitude

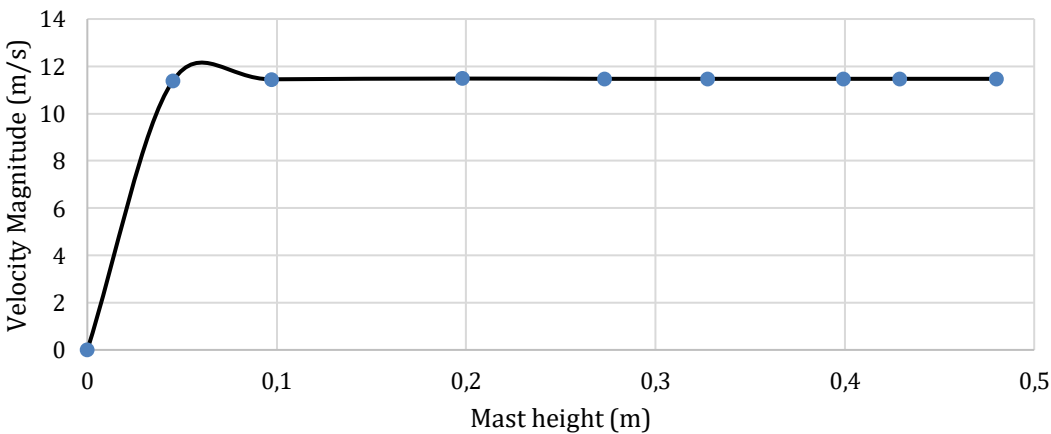


Fig. A. 4. Velocity magnitude vs Mast height

A.5. Dynamic Pressure

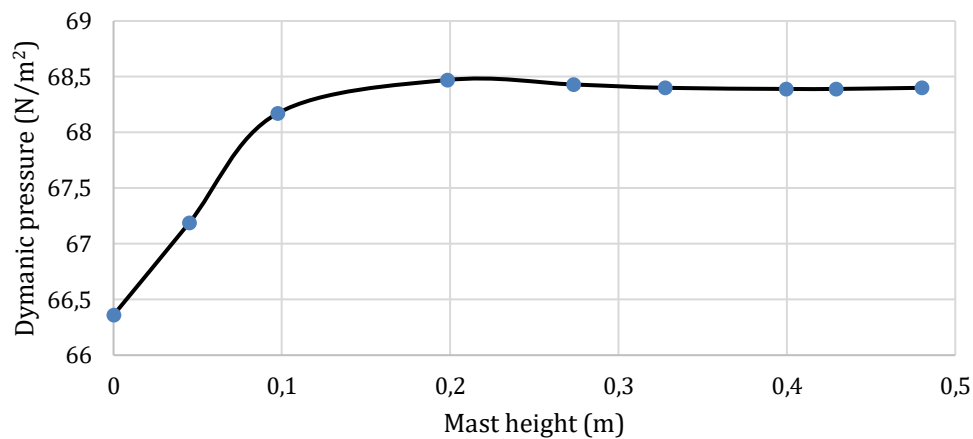


Fig. A. 5. Dynamic pressure vs Mast height

A.6. Notepad X-Y-Z Coordinates

Table A. 1. Coordinates points X-Y-Z

Points	Angle	X (m)	Y (m)	Z (m)
surface/point-surface pname0	0°	0,037	0	0,375
surface/point-surface pname1	10°	0,036438	0,006425	0,375
surface/point-surface pname2	20°	0,034769	0,012655	0,375
surface/point-surface pname3	30°	0,032043	0,0185	0,375
surface/point-surface pname4	40°	0,028344	0,023783	0,375
surface/point-surface pname5	50°	0,023783	0,028344	0,375
surface/point-surface pname6	60°	0,0185	0,032043	0,375
surface/point-surface pname7	70°	0,012655	0,034769	0,375
surface/point-surface pname8	80°	0,006425	0,036438	0,375
surface/point-surface pname9	90°	0	0,037	0,375
surface/point-surface pname10	100°	-0,00643	0,036438	0,375
surface/point-surface pname11	110°	-0,01266	0,034769	0,375
surface/point-surface pname12	120°	-0,0185	0,032043	0,375
surface/point-surface pname13	130°	-0,02378	0,028344	0,375
surface/point-surface pname14	140	-0,02834	0,023783	0,375
surface/point-surface pname15	150°	-0,03204	0,0185	0,375
surface/point-surface pname16	160°	-0,03477	0,012655	0,375
surface/point-surface pname17	170°	-0,03644	0,006425	0,375
surface/point-surface pname18	180°	-0,037	0	0,375



Valorization of plastic wastes (PVC) in sand concretes: Experimental approach and durability analysis

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Abstract

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Recent research highlights the use of eco-friendly waste materials, such as plastic, to improve concrete performance and sustainability. Similarly, sand concrete is being enhanced through waste reuse to increase its durability and competitiveness. For confirmation purposes, this study was conducted to investigate the recycling of Polyvinyl Chloride (PVC) plastic wastes, originating from sawing and cutting PVC profiles in carpentry workshops, in the form of fines. The aim was to partially replace limestone fines in the composition of sand concrete at substitution rates of 3%, 6%, 9%, and 12%. Likewise, the development of the properties of such concretes was studied in their fresh state (density, workability) and hardened state (compressive strength and flexural tensile strength, sclerometer test, ultrasonic pulse velocity). As for durability, it was investigated by testing water absorption through immersion and capillary action, water-accessible porosity, and behavior in aggressive environments (H₂SO₄, HCL). In consequence, the results obtained were compared with control samples with a 0% substitution rate. The results have clearly demonstrated that partial substitution led to a significant change in the properties of fresh and hardened concrete. In particular, replacing 6% of fine limestone with recycled plastic waste gave satisfactory results.

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1. Introduction

In reality, the world is experiencing economic growth in the production of various materials used in our daily lives, as plastic, particularly polyvinyl chloride (PVC), is the best example thereof, as it is widely used at present in various fields, mainly in the construction sector, such as pipes, windows, doors, and flooring. In addition, global polyvinyl chloride (PVC) production has increased significantly, rising from approximately 3 million tons in 1965 to over 20 million tons annually. This rapid growth contributes to the accumulation of plastic waste, highlighting the need for sustainable recycling solutions [1]. In 2017, its production reached 49 million tons, mainly used in construction, furniture and electronics [2]. Nonetheless, the production and use of PVC in various fields result in the generation of considerable amounts of waste, exceeding 25 million tons per year according to statistics. In Algeria, the National Waste Agency recorded in 2019 approximately 15.31% of plastic waste out of all household and similar waste, equivalent to 2.1 million tons. However, this waste includes polyethylene terephthalate (PET), polypropylene (PP), polystyrene (PS), and polyvinyl chloride (PVC) [3]. Above all, it is well known that this type of plastic is not easily biodegradable, which represents a major challenge for all countries in terms of waste disposal [4]. Basically, this issue has paved the way for research aimed at finding environmentally friendly alternatives to traditional waste disposal methods (such as incineration and landfill),

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particularly recycling in construction materials. In addition, this is all the more important given the irrational consumption of certain raw materials used in construction and concrete, such as sand, cement, and gravel.

In fact, a study conducted by Harshit Sangtani et al. [5] examined the replacement of ordinary sand with plastic waste sand (PVC) at varying rates. The results demonstrated a positive impact on the compressive strength of concrete with an optimal replacement rate of 10% at 7, 14, 21, and 28 days; nevertheless, the strength decreases due to the increase in PVC waste without significantly affecting the workability of fresh concrete. In another study conducted by Kou et al. [1] on various concrete mixtures, where river sand was partially replaced by plastic pipe waste aggregates (PVC) at different rates ranging from 5 to 45%. The study concluded that replacing the waste had a positive impact on certain properties (density and ductility) on the one hand and a negative impact on the workability, compressive strength and tensile strength of concrete, on the other. Concerning the study by Akila et al. [6], it focused on the strength required when replacing waste from plastic water containers and PVC. Moreover, five substitution rates were used: 5%, 10%, 15%, 20% and 25% replacement of sand. The results demonstrated that replacing sand with plastic waste as fine aggregate increases the compressive strength of concrete, but not to the extent where it can endure heavy loads compared to ordinary concrete. Furthermore, the replacement process resulted in a reduction in density. Youcef Ghernouti et al. [7] partially replaced the fine aggregates in concrete with fine aggregates from recycled plastic bags resulting from waste shredding at replacement rates of 10%, 20%, 30% and 40%. Thus, the results demonstrated a positive effect on the workability of fresh concrete, which increases with the waste substitution rate. However, the effect was negative on compressive and tensile strength in the hardened state. Consequently, this resulted in a reduction in strength and a decrease in the speed of sound propagation in the concrete. Sulaiman Mustafa Khazaal et al. [8] studied the possibility of partially replacing sand in concrete with plastic waste, such as Polyvinyl Chloride (PVC) and Polypropylene (PP). Tests were conducted through replacing 10%, 15%, 30%, 45% and 60% of ordinary sand. The results demonstrated a significant increase in compressive strength after 28 days when using 15% and 30% PVC, compared to the use of PP waste, which showed a decrease in strength. On the other hand, Bhosale Swapnil et al. [9] conducted a study on the partial replacement of fine sand in concrete using plastic waste such as Polyvinyl Chloride (PVC). In this respect, tests were conducted replacing 15%, 25% and 35% of the sand and tested the compressive strength at 7, 14, and 28 days. The incorporation of plastic waste into concrete helped to reduce its weight, with a slight decrease in compressive strength, where an optimal replacement value of 15% was observed, accordingly. Studies conducted by Choi et al. [10] and Marzouk et al. [11] confirmed the possibility of using plastic waste, particularly Polyethylene Terephthalate (PET) bottles, in concrete. Nonetheless, this leads to a decrease in the strength of the concrete, both in compression and tension, when the proportion of plastic waste exceeds 0.5% of the total weight of the concrete mixture. The majority of studies agree on the possibility of reusing various types of plastic waste in construction materials in general, and in concrete in particular. The recycling process is considered one of the best environmental, economic and ecological solutions proposed by many researchers. In another study conducted by da Silva A. M. et al. [12] on the effect of using plastic waste as a substitute for sand in mortar, the tested proportions ranged from 5%, 10% and 15% of the volume compared to the control mortar. The results were mixed, showing a reduction in the width of shrinkage cracks, a decrease in the modulus of elasticity and an improvement in impact strength, as well. As a counterpoint, an increase in capillary absorption and water vapour permeability was also observed. Furthermore, a study conducted by Bolat and Erkus in 2016 [2], examined the use of PVC waste in powder and granule form as a partial substitute for aggregates in concrete, at rates of 10%, 20% and 30%. The results demonstrated a decrease in density, capillary absorption and compressive strength, while, abrasion resistance improved with increasing substitution rate. Another study conducted by Boutlikht et al. [13] focused on the use of Polyvinyl Chloride (PVC) waste as a concrete manufacturing material, partially replacing natural sand with PVC fibres and fine sand at volume rates of 5%, 10% and 15%. Consequently, the results demonstrated that PVC reduced the workability of fresh concrete for both types. In addition, the PVC sand decreased compressive strength, unlike PVC fibres, which had a less negative effect.

On the other hand, sand concrete is one of the emerging concrete types due to its countless characteristics that make it an alternative to traditional concrete in certain construction works, such as underground structures, dams, pavements, airports, road infrastructure, and in the rehabilitation of old buildings particularly for injection works [14,15]. Along the same lines, sand concrete is considered one of the materials that attract the interest of researchers in the field of recycling various types of recyclable wastes, mostly those originating from the construction sector, with the aim of developing its properties for its adaptation for widespread use in construction and civil engineering, such as the pouring of foundations, walls, columns, the rehabilitation of old buildings, as well as injection works and repairs. Several studies have been conducted on the recycling of waste in sand concrete. Rihia et al. [16] conducted a study on the possibility of replacing fine natural sand with marble and ceramic wastes sand in a mixture of lightweight sand concrete reinforced with plant fibres. The results demonstrated positive indicators regarding the effect of this substitution on the physical and mechanical properties together with the durability of the concrete. Similarly, wastes can also be used in the form of fibres, as demonstrated by Sayadi et al. [17] in a study on the effect of reusing copper, aluminum and iron waste, transformed into fibres, on the properties of sand concrete, its durability and resistance to chemical attack. With rates of 0.4 to 1.2%, the results proved an improvement in certain characteristics, such as strength and durability, depending on the type and rate of wastes incorporated. In the same context, a few studies have been conducted on the reuse of plastic wastes in sand concrete, but they remain limited. In addition, Guendouz and Boukhelkhal [18] examined the recycling of rubber wastes in sand concrete and their impact on physical (workability, bulk density), mechanical (compressive and flexural strength) and thermal properties, with substitution rates of 10%, 20%, 30% and 40%. Consequently, the results demonstrated a decrease in compressive and tensile strength, along with an improvement in thermal performance. Guendouz et al. [19] studied the incorporation of two types of plastic wastes (Polyethylene Terephthalate (PET) and Low-Density Polyethylene (LDPE) used in the manufacture of bags) in the form of fibres and fine powder in sand concrete as a partial replacement for ordinary sand, at different proportions for each type of waste. Then, the results demonstrated that the use of plastic wastes, mainly at replacement rates of 10% and 20%, reduces the bulk density and air content, thus leading to an increase in compressive and flexural strength, accordingly. Along the same lines, Akinyele and Toriola [20], Davies and Olofinnade [21] conducted a study aimed at producing sand concrete blocks using shredded plastic waste as a replacement for fine aggregate and Polyethylene Terephthalate (PET) wastes at different substitution rates, respectively. They studied the impact of these proportions on compressive strength, tensile strength, density and water absorption. The results demonstrated that it was possible to achieve an increase in strength up to an optimal replacement ratio of 5%. Nevertheless, as the replacement rate increases, the density and water absorption decrease.



Fig. 1. Cutting PVC profiles

Most studies have demonstrated that the use of plastic waste in concrete and mortar is possible, with effects that can be both positive and negative on their properties, depending on the

substitution rates and the material replaced, whether cement or, more often, sand. Based on this research, Polyvinyl Chloride (PVC) plastic waste (Fig. 1) was reused as a substitute for limestone fines in sand concrete, at replacement rates of 3%, 6%, 9%, and 12%. Therefore, the aim is to study their impact on the physical and mechanical properties of concrete, together with their durability against chemical attack, as well.

2. Materials Used

2.1 Cement

The cement used in this study is CEM II 42.5, produced at the Hadjar Soud factory in Skikda, Eastern Algeria. It is composed of clinker and two types of additives: blast furnace slag and limestone. Its absolute density is 3.22 g/cm^3 whilst its Blaine specific surface area is $3550 \text{ cm}^2/\text{g}$.

2.2 Sand

In this study, the naturally rolled 0/1 mm fraction dune sand (DS) from the Oued Z'hour – Skikda region in Eastern Algeria was used.

2.3 PVC Plastic Wastes (Recycled Fines)

Polyvinyl Chloride (PVC) is a thermoplastic powder composed of carbon, hydrogen, and chlorine, containing approximately 56.7% chlorine. It is produced from Vinyl Chloride Monomer (VCM), an odorless and flammable compound, via suspension, bulk, or emulsion polymerization. In this polymer, the carbon and hydrogen are derived from petroleum, while chlorine is obtained from salt.



Fig. 2. Recycled fines from plastic waste

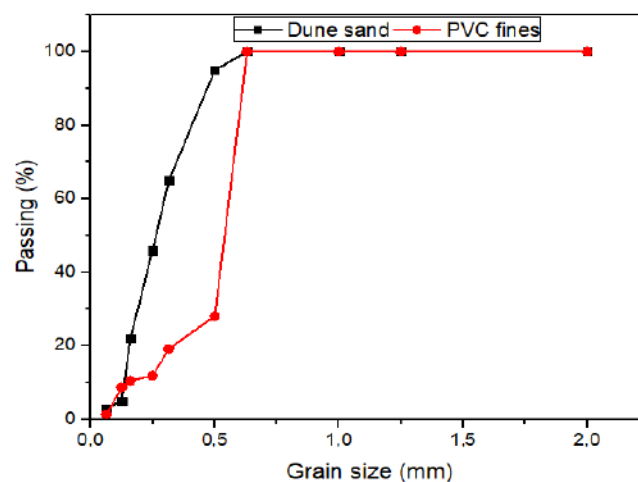


Fig. 3. Particle size distribution curve of dune sand and PVC fines

In this study, PVC wastes were recovered in two stages: the first involved collecting the cuttings from an aluminum joinery workshop (Fig. 2); the second involved obtaining fine particles by

sieving the material through a 1 mm sieve, resulting in particles smaller than 1000 μm . An analysis of the particle size distribution of Polyvinyl Chloride (PVC) was carried out (Fig. 3), which then revealed a percentage of fines (f) equal to 1.4%, with an estimated density of 1.24 g/cm^3 .

2.4 Limestone Fines

The fines (F) used in this study consist of finely ground white limestone powder obtained from the Ben Azzouz quarry, east of Skikda. More than 80% of the particles pass through an 80 μm sieve. The limestone fines have an absolute density of 2.741 g/cm^3 and a calcium carbonate (CaCO_3) content of 84.60%. Based on the characteristics obtained, it can be stated that:

- The lowest density is observed for PVC fines, which could reduce the overall density of the concrete. Nevertheless, the highest density is that of limestone fines;
- Since the fines content of PVC wastes (1.4%) is lower than that of dune sand; thus, incorporating it could lead to reduced workability, increased segregation, and reduced concrete compactness [22], as well;
- The limestone fines are primarily composed of calcium carbonate (CaCO_3) in high proportion, which contributes significantly to the cohesion of the cement matrix, thereby promoting early mechanical strength development [16];
- The dune sand has a fineness modulus below the optimum value, indicating a high proportion of fine and very fine particles, thus requiring an increase in water content to ensure good workability [22];
- The sand equivalent test indicates that the dune sand used is slightly clayey, with acceptable cleanliness. Moreover, it has a continuous particle size distribution (Fig. 3) and is suitable for high-quality concrete where shrinkage is not a major concern.

Table 1. Physical characteristics of materials

Physical characteristics	Bulk density (g/cm^3)	Absolute density (g/cm^3)	Sand equivalent (%)	Absorption (%)	Fineness modulus (%)	Fine content (%)
Dune sand	1.31	2.83	68.02	3.00	1.55	3.00
Limestone fines	1.05	2.74	--	--	--	--
PVC fines	--	1.24	--	--	2.51	1.40

Table 2. Chemical characteristics of materials

Chemical characteristics	CaO (%)	Al_2O_3 (%)	F_2O_3 (%)	SiO_3 (%)	MgO (%)	Na_2O (%)	K_2O (%)	Cl- (%)	SO_3 (%)	CaCO_3 (%)	SO_4 (%)
Cement	66.46	5.40	3.92	21.63	1.29	0.21	0.57	0.01	0.46	--	--
Dune sand	0.80	2.36	1.15	94.09	0.14	0.20	0.58	--	0.01	--	--
Limestone fines	--	--	--	--	--	--	--	0.21	traces	84.60	traces
PVC fines	--	--	--	--	--	--	--	56.70	--	--	--

2.5 Admixture

The superplasticizer (Glenium 26, BASF, Algeria) was used in this study. It is a high-range water reducer, compliant with the NF EN 934-2 standard, ready for use and formulated with modified polycarboxylates. This superplasticizer, which is chlorinated, comes in the form of a light brown liquid with a density of 1.07 ± 0.02 .

3. Experimental Program and Tests Performed

3.1 Cement

As part of the experimental program, a concrete made from cement, dune sand, limestone fines, admixture and water was selected as the reference concrete. The limestone fines were then partially replaced by PVC fines. For this purpose, five formulations were developed:

- SC-0%: Sand concrete composed of 0% plastic fines and 100% limestone fines;
- SC-3%: Sand concrete composed of 3% plastic fines and 97% limestone fines;
- SC-6%: Sand concrete composed of 6% plastic fines and 94% limestone fines;
- SC-9%: Sand concrete composed of 9% plastic fines and 91% limestone fines;
- SC-12%: Sand concrete composed of 12% plastic fines and 88% limestone fines.

Essentially, the mixtures were prepared according to the Sablocrete method [23], by keeping certain parameters constant, such as cement dosage, quantity of dune sand, superplasticizer content and water/binder ratio. The studied variables pertained to the substitution rates of limestone fines by plastic fines (PVC). Table 3 illustrates the different compositions of the five formulations.

Table 3. Composition of the mixtures

Formulations	Cement (kg/m ³)	Dune sand (kg/m ³)	Water (l/m ³)	Admixture (l/m ³)	Limestone fines (kg/m ³)	PVC fines (kg/m ³)
SC-0%	321	946	218	2.9	202.42	0
SC-3%	321	946	218	2.9	196.01	2.14
SC-6%	321	946	218	2.9	183.89	4.29
SC-9%	321	946	218	2.9	177.82	6.43
SC-12%	321	946	218	2.9	95.01	8.57

3.2 Tests Performed

The following tests were performed on the different formulations:

- The density of the fresh concrete was measured in accordance with the Standard NF EN 12350-6;
- The workability of the concrete was assessed by use of the Abrams cone slump test, in accordance with the Standard NF EN 12350-2;
- The flexural traction strength and compressive strength were measured at 02, 07, 14, 28 and 90 days on $4 \times 4 \times 16$ cm³ prismatic test specimens, stored in water, in accordance with the Standard NF EN 12390-5;
- The sclerometer test was carried out on $15 \times 15 \times 15$ cm³ test specimens stored in water at 28 days of age in accordance with the Standard NF EN 12504;
- The ultrasonic test was measured on cubic test specimens measuring $15 \times 15 \times 15$ cm³ after 28 days of curing in water, in accordance with the Standard NF EN 12504-4;
- The water-accessible porosity is measured in accordance with the Standard NF P18-459, on test specimens measuring $4 \times 4 \times 16$ cm³;
- The capillary absorption was measured on test specimens measuring $4 \times 4 \times 16$ cm³, in accordance with the Standard NF EN 480-5;
- The absorption by immersion was assessed on cubic test specimens measuring $5 \times 5 \times 5$ cm³, in accordance with the Standard NBN B 15-215;
- The chemical attack by acids and bases was carried out on $5 \times 5 \times 5$ cm³ test specimens stored during 28 days in water and then in the chemical solution. Then, weight loss and strength loss were measured according to the age of the test, in accordance with the Standard ASTM C-267-96;
- The chloride permeability was measured on $5 \times 5 \times 5$ cm³ test specimens stored during 28 days in water, then immersed in a solution containing 5% sodium chloride (NaCl). Afterwards, the samples were cut and then treated with a silver nitrate solution on the cut concrete surface, in accordance with the Standard NT BUILD 492-1.

4. Results and Discussion

4.1 Density

Based on the results shown in Fig. 4, it can be concluded that the use of recycled Polyvinyl Chloride (PVC) plastic wastes as a partial substitute for limestone fines in sand concrete resulted in a decrease in density compared to the control concrete. The minimum density value was observed for the maximum substitution rate of 12%, reaching approximately 2.142 g/cm^3 ; corresponding to a decrease of 3.86% compared to the reference mixture. Nonetheless, this decrease can be explained by the lower density of PVC fines compared to that of limestone fines. Therefore, these results are in agreement with those obtained by Senhadji et al. [24].

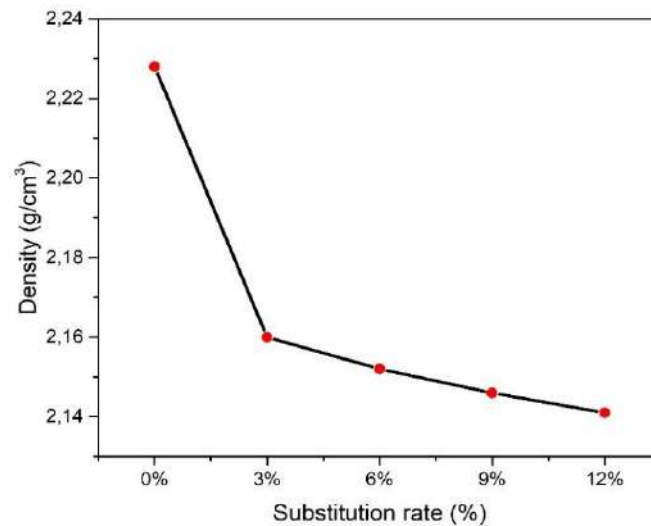


Fig. 4. Variation in density according to the rate of substitutions

4.2 Workability

Fig. 5 demonstrates that increasing the substitution rate of limestone fines with recycled plastic wastes in sand concrete, up to an optimal threshold of 6%, improves workability, with an increase of up to 30% compared to the reference concrete. This improvement is due to the low water absorption of plastic particles [25,26]; Nonetheless, above this threshold, workability gradually decreases, which can be attributed to the angular and irregular shape of the plastic fragments, mainly at higher substitution rates [27,28].

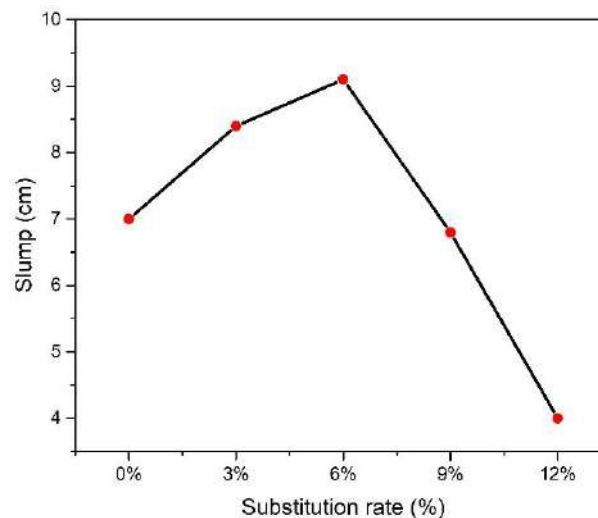


Fig. 5. Variation in workability according to the rate of substitutions

4.3 Compressive Strength

The results of compressive strength tests at 7, 28 and 90 days on sand concrete mixtures incorporating Polyvinyl Chloride (PVC), shown in Fig. 6, clearly demonstrated a decrease in compressive strength with increasing substitution rate of plastic wastes at all ages, compared to the reference concrete. Above and beyond, this decrease is particularly significant for the maximum substitution rate of 12% at 7 days, where a reduction of approximately 55% was recorded. The same downward trend is observed for the other substitution rates, although it is less pronounced, mainly at 3%. This behavior can be attributed to the low adhesion between the surface of the PVC grains and the cement paste. Furthermore, as PVC is a hydrophobic material, this characteristic may limit the penetration of water necessary for cement hydration through the structure of the samples during the curing period [2].

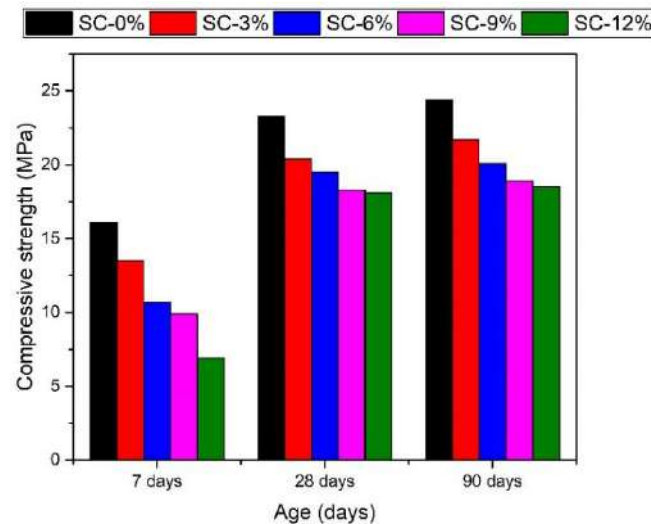


Fig. 6. Influence of the rate of substitutions on compressive strength

The results demonstrated that replacing limestone fines with plastic fines shows a trend of increasing compressive strength in sand concrete over time. Nevertheless, this strength remains lower than that observed for the reference concrete at 7, 28 and 90 days (Fig. 7). Additionally, the obtained experimental observations are consistent with those reported in numerous previous studies [29,30]. Along the same lines, this behavior could be explained by a decrease in the density of the concrete, leading to a potential increase in porosity, as well as the weak cohesion between the plastic particles and the other constituents of the concrete.

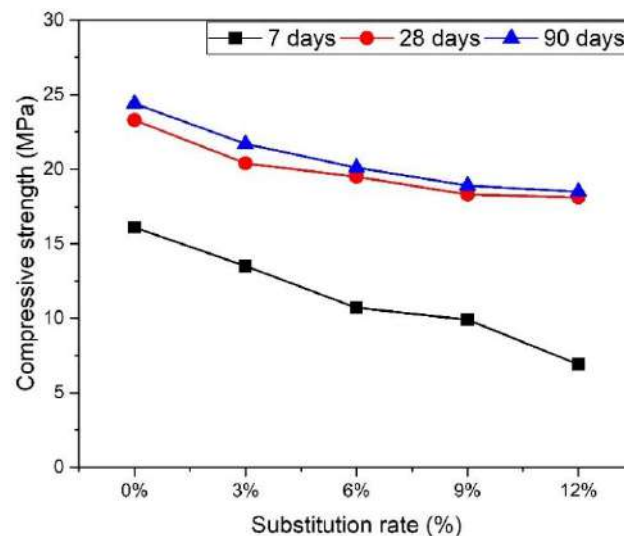


Fig. 7. Evolution of compressive strength according to the age

4.4 Flexural Tensile Strength

As shown in Fig. 8, the behavior of flexural tensile strength generally follows the same trend as compressive behavior, with a decrease in strength as the substitution rate increases across all curing ages. This reduction in strength is attributed to the low adhesion between the Polyvinyl Chloride (PVC) particles and the cement paste, as well as to the accumulation of bleeding water due to the low absorption rate of the plastic waste particles, which led to a weakened bond within the cement matrix [1]. Notably, the SC-3% mixture containing 3% recycled fines illustrated the best flexural strength performance at 7 days compared to the other formulations.

However, the mixture containing 9% plastic fines demonstrated a notable improvement in performance at 28 and 90 days, with tensile strengths of 6.78 MPa and 7.66 MPa, respectively. Moreover, these values are higher than those of other formulations containing plastic wastes, although they remain slightly lower than those of the reference concrete. This suggests that satisfactory tensile strength can be attained with an optimal substitution rate, despite the general trend towards decreased mechanical performance [31]. Indeed, tensile strength tends to increase over time, particularly at 7, 28 and 90 days, as illustrated in Fig. 9 for all formulations. Nonetheless, mixtures containing fines from plastic wastes have lower strengths than the reference concrete. Thus, a substitution rate of 9% can be considered an optimal value for attaining satisfactory strength at the ages of 28 and 90 days.

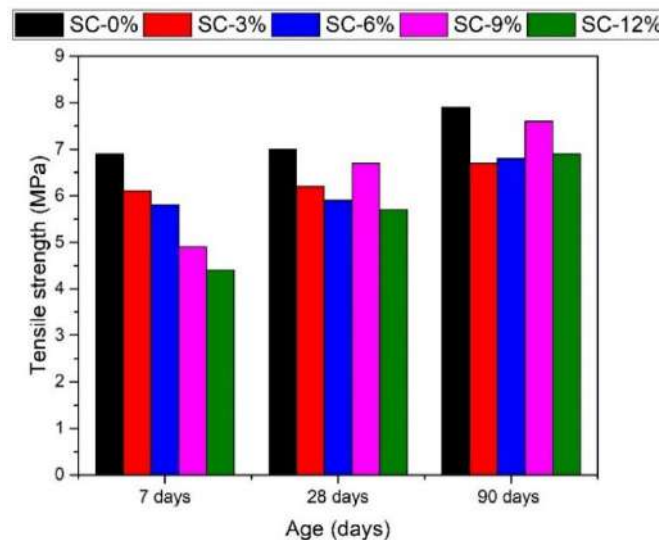


Fig. 8. Influence of the rate of substitutions on flexural tensile strength

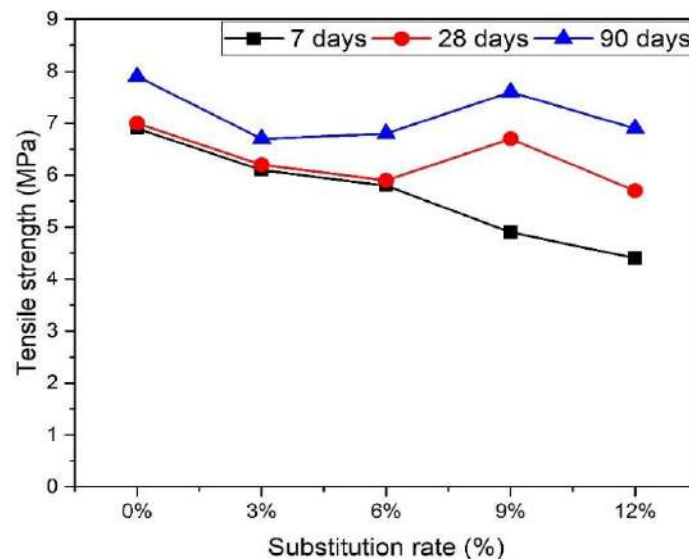


Fig. 9. Evolution of tensile strength in flexion according to the age

4.5 Compressive Strength Obtained by the Sclerometer Test

Based on the results obtained, it can be observed that the sclerometric index slightly increases compared to the control concrete, reaching values of 22 and 23 for substitution rates of 3% and 6%, respectively. Beyond these rates, the addition of recycled plastic fines becomes unfavorable. To better highlight the relationship between mechanical and surface properties, Fig. 10 presents the correlation between compressive strength and the sclerometric index, regardless of the substitution rates. A strong linear relationship is observed ($R^2 = 0.9995$), indicating that the sclerometric test can be used as a reliable indirect method for estimating compressive strength.

Similar trends are observed for compressive strength, where the maximum value was recorded at a substitution rate of 6%, reaching 16.53 MPa, which represents the best performance among all tested formulations. Consequently, replacing 6% of fine particles with recycled plastic significantly improves the compactness and strength of sand concrete mixture [32].

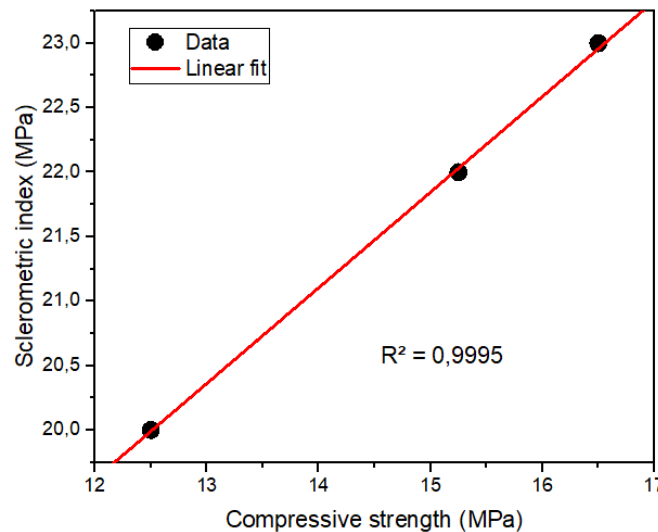


Fig. 10. Correlation between compressive strength and sclerometric index

4.6 Compressive Strength Obtained by The Ultrasonic Testing

Indeed, Fig. 11 demonstrates the ultrasonic wave velocities measured on concrete samples after 28 days of curing. Moreover, the results indicate a gradual decrease in ultrasonic pulse velocity (UPV) as the substitution rate of limestone fines by recycled fines increases. Specifically, this decreases ranges from 3472 m/s for the reference concrete to 3103 m/s for the concrete with a substitution rate of 12%.

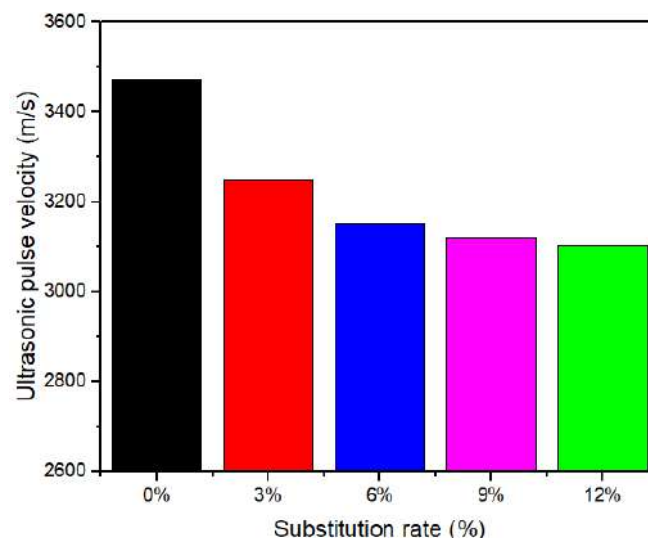


Fig. 11. Influence of substitution rate on propagation velocity

These observations indicate that the introduction of plastic waste fines into concrete reduces its density, which negatively affects the porosity thereof. Actually, the pores generated by these fines weaken the ultrasonic pulse velocity waves. Besides, this can be explained by the fact that these waves are partially reflected and partially transmitted when they pass through the PVC fines, voids and concrete, leading to a decrease in their speed [28]

4.7. Water Absorption by Capillarity

The results presented in Fig. 12 indicate that the capillary water absorption coefficient increases with the incorporation of recycled Polyvinyl Chloride (PVC) fines. This behavior can be attributed to the low adhesion between the matrix elements and the plastic fines, which makes this matrix more porous and therefore more likely to retain water. On the other hand, the low absorption capacity of PVC fines can lead to an increase in the amount of free water in the concrete, thus promoting the formation of interconnected voids following the evaporation of this water, which leads to an increase in the average size of capillary pores [21,16]. The capillary absorption rate decreases over time for all types of concrete, with the highest coefficient recorded for the SC-12% mixture.

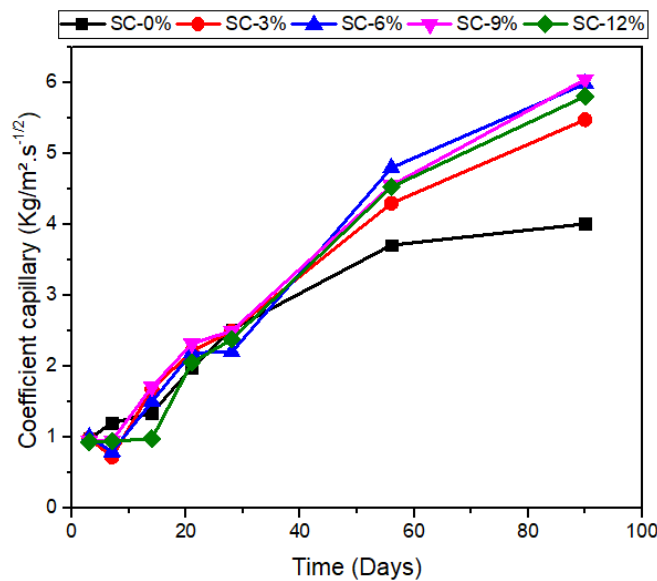


Fig. 12. Water absorption by capillarity vs time

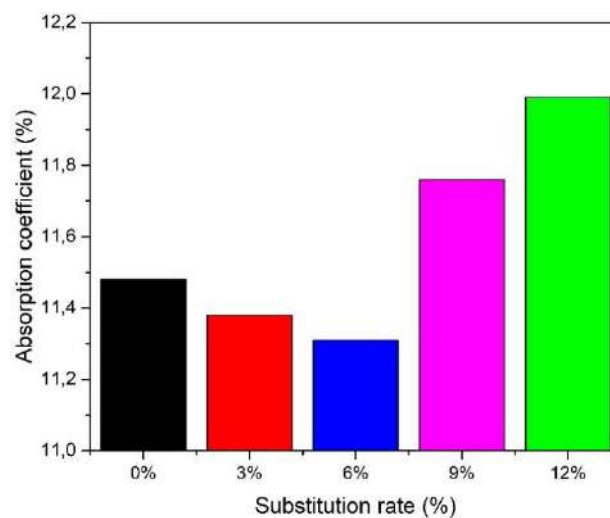


Fig. 13. Variation in the water absorption coefficient by immersion according to the rate of substitution

4.8 Water Absorption by Immersion

The results presented in Fig. 13 indicate that replacing limestone fines with recycled plastic fines leads to a decrease in absorption for substitution rates of 3% and 6%. Above this threshold, the effect is reversed and the absorption rate increases with the increase of substitution rate, attaining a maximum value at 12%. This behavior can be explained by the increase in specific surface area caused by the addition of Polyvinyl Chloride (PVC) relative to Portland cement. Consequently, the PVC particles partially cover the surfaces of fine aggregates, leaving some areas uncoated [33]. Moreover, the 6% substitution rate, which provided good concrete surface strength, indicates that this mixture is more homogeneous and of better quality, thus reducing the absorption process, accordingly [32,33].

4.9 Chemical Attacks

4.9.1 Chemical Attacks by A 4% Sulphuric Acid (H_2SO_4) Solution

Figure 15 illustrates the evolution of mass gain in the concretes studied when exposed to a Sulphuric Acid solution. It is evident that during the early stages (3 and 7 days), mass gain remains low for all formulations, particularly those incorporating Polyvinyl Chloride (PVC) fines. At intermediate ages (14, 21 and 28 days), the differences between the mixtures become more pronounced; the mixture containing 12% PVC shows almost constant mass stability, unlike the other formulations, which show more marked variations.



Fig. 14. Measurement of mass loss

Nevertheless, between 56 and 90 days, a notable increase in mass gain is observed, reaching a maximum value of 6.13% for the formulation with a substitution rate of 9%. In general, this mass gain can be attributed to the absorption of water or acid solution into the micro-voids between the polyvinyl particles and the cement matrix, as well as at the edges of the samples, which have shown not to be completely protected. This penetration promotes specific chemical reactions, in particular:

- The reaction Eq (1) between sulphuric acid and calcium hydroxide forming gypsum:



- Then the reaction Eq (2) between calcium sulphate and hydrated calcium aluminates, producing ettringite:



Indeed, the deposition of calcium sulphate or other salts in the fine pores leads to an increase in the total mass of samples. Moreover, the polyvinyl is known for its resistance to acidic environments, in respect such as wastewaters that can generate sulphuric acid; therefore, particles from polyvinyl wastes act as a protective barrier, limiting mass loss and concrete degradation over time [34,35,36].

This interpretation is consistent with the findings of Senhadji et al. [37], which illustrated that PVC aggregates have a relatively impermeable structure and a partial acid retention capacity in their pores. This reduces the penetration of acid into the cement matrix and limits surface reactions in a controlled manner. This results in the precipitation of products such as gypsum and ettringite inside the pores, rather than the dissolution of components, which contributes to an increase in mass and improved resistance of the concrete to long-term acid attack.

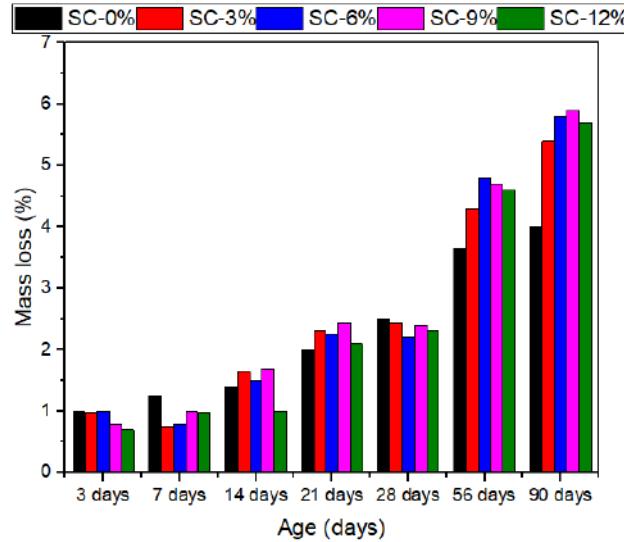


Fig. 15. Variation in mass loss of dune sand concrete as a function of immersion time in 4% H_2SO_4

4.9.2 Attacks by A 4% Hydrochloric Acid (Hcl) Solution

Fig. 16 illustrates the results relating to the mass loss of concrete subjected to a hydrochloric acid solution. It can be noticed that all formulations show a decrease in mass, a phenomenon attributable to the chemical reaction of hydrochloric acid with portlandite $Ca(OH)_2$ and lime $CaCO_3$ resulting from cement hydration. Further, these reactions produce calcium chloride ($CaCl_2$), a highly soluble compound that is particularly harmful to the mechanical and durability properties of concrete [17]. Hence, these processes can be represented by the equations Eq (2) and Eq (3) below:



The reference concrete records the greatest loss of mass up to 21 days. From 3 to 21 days, losses varied according to the substitution rate with polyvinyl wastes. At 28 days, the differences become more pronounced: the minimum value is observed for sand concrete containing 6% recycled plastic fines, whilst the maximum loss is recorded for concrete containing 12%, accordingly.

In effect, a substitution rate of 6% can be considered optimal for increasing resistance to chemical attack by hydrochloric acid. This improvement can be explained by a mechanism similar to that observed with sulphuric acid, where the polyvinyl particles retain some of the acid ions, thereby reducing the intensity of the attack and limiting the degradation of the cement matrix [37,34].

The SC-6% mixture illustrates good performance in immersion absorption and surface resistance tests. Further, the incorporation of this percentage reduces the absorption coefficient compared to other formulations, thanks to improved compactness and homogeneity of the matrix, as well as an increase in surface resistance [32]. Therefore, these improvements result in better long-term properties and increased resistance of concrete to chemical attack by hydrochloric acid [16].

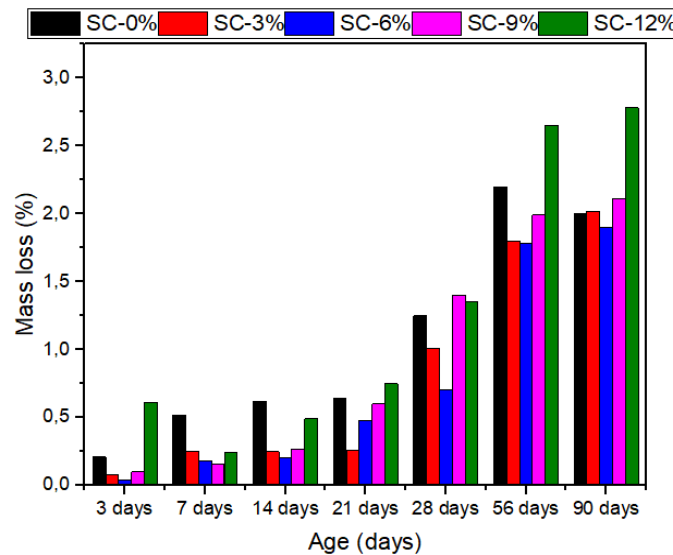


Fig. 16. Variation in mass loss of sand concrete according to the immersion time in 4% HCl

4.9.3 Chloride Penetration

According to the results obtained and presented in Fig. 17, the lowest chloride ion penetration depth was recorded for the SC-0% reference concrete. This finding indicates that this mixture is more homogeneous and compact, due to the filling effect exerted by the particles (cement, limestone fines and sand), which reduces the size of the internal pores of the concrete, as well as its SiO_2 and CaO contents, giving it better resistance in aggressive environments [16,22,38]. On the other hand, the incorporation of fines from plastic waste leads to an increase in the penetration depth of chloride ions. The maximum value was observed for a substitution rate of 3%, reaching approximately 31 mm, corresponding to a considerable increase of 416% compared to the reference concrete. Beyond this percentage, the increase in the content of recycled polyvinyl chloride fines leads to a decrease in the penetration depth, which nevertheless remains higher than that measured for the reference mixture.

This behavior reflects the concrete's lower resistance to chloride ion penetration. Further, these observations differ from those reported by Kou et al. [1] and Senhadji et al. [24]. In fact, the presence of polyvinyl in concrete could promote the formation of voids due to the low adhesion between the plastic particles and the cement matrix, a phenomenon that is particularly pronounced at low substitution rates [39].

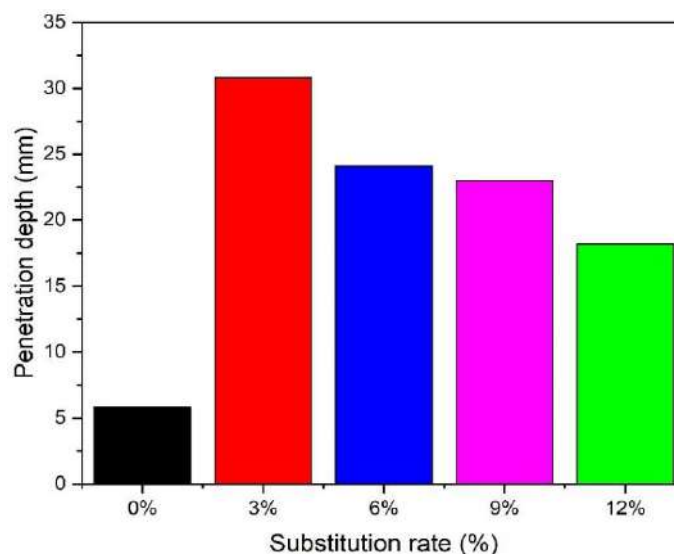


Fig. 17. Variation in chloride penetration depth according to the substitution rate

4.9.4 Water-Accessible Porosity

In general, the partial replacement of limestone fines with recycled polyvinyl fines (Fig. 18) leads to a decrease in water-accessible porosity, regardless of the rate of substitution, compared to the SC-0% reference mixture. The minimum value recorded in this test was observed for concrete with a substitution rate of 6%, with a water-accessible porosity of 21.6%.

This behavior can be explained by the fact that plastic is known to be hydrophobic, representing a low affinity for water. Further, the presence of plastic particles in the concrete composition thus reduces the voids accessible to water, while also limiting the amount of water retained between the concrete constituents [40,32,20]. Subsequently, polyvinyl fines occupy the spaces in the reference concrete matrix that were initially accessible to water, with a content of 6% considered optimal for achieving minimal porosity.

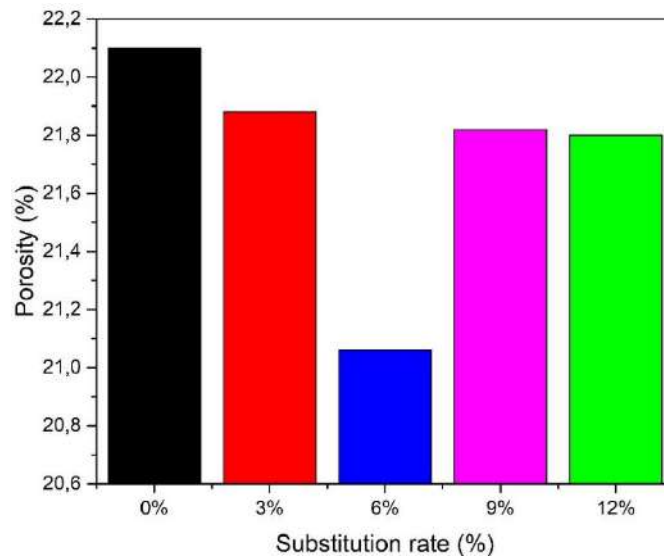


Fig. 18. Variation in water-accessible porosity according to the rate of substitution

5. Conclusions

In light of the results obtained in this experimental study, several important conclusions can be drawn, as follows:

- Incorporating the recycled polyvinyl wastes as a partial substitute for limestone fines reduced the density of the sand concrete, making it lighter;
- The best workability was attained with the mixture containing 6% recycled polyvinyl fines;
- The use of recycled polyvinyl waste as a partial replacement for limestone fines in sand concrete resulted in a decrease in compressive and tensile strengths, while maintaining acceptable values depending on the substitution rate;
- The best flexural tensile performance was observed for the SC-9% mixture, containing 9% recycled polyvinyl fines, compared to the other formulations, at 28 and 90 days;
- The maximum surface hardness was obtained for a 6% replacement of limestone fines with recycled polyvinyl fines;
- The ultrasonic pulse velocity decreases with increasing levels of the recycled PVC fines, indicating reduced homogeneity of the sand concrete;
- The incorporation of recycled PVC fines leads to an increase in the rate of water absorption by capillarity, with the mixture containing a 12% substitution rate recording the maximum absorption value;
- The 6% PVC fines content is considered the optimal percentage for achieving a lower immersion absorption rate than the other mixtures;
- The incorporation of polyvinyl fines improved durability and increased the mass of the concrete when exposed to sulphuric acid (H_2SO_4), with no loss of mass observed for all substitution rates;

- A substitution rate of 6% is optimal for limiting the penetration of hydrochloric acid (HCl) and, consequently, reducing the mass loss of sand concrete;
- Recycled polyvinyl plastic fines did not reduce chloride ion penetration, regardless of the substitution percentage;
- The incorporation of PVC fines, for all substitution rates, leads to a decrease in water-accessible porosity, with a minimum observed for a substitution rate of 6%.

These findings suggest that sand concrete incorporating PVC waste can be effectively used in non-structural applications such as paving blocks, masonry units and lightweight construction elements.

In conclusion, it can be said that recycling polyvinyl waste as a substitute for limestone fines has an overall positive impact, depending on the substitution rates and the targeted properties. This encourages further research and experimentation to optimize results and explore other characteristics.

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Performance evaluation of sustainable self-curing concrete with GGBS and rice husk ash

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Abstract

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In recent decades, rapid urbanization has increased the demand for high-rise construction and the continuous growth in population, the demand for water has significantly increased, leading to scarcity of water resources in construction, that leads to challenges for conventional curing practices in construction. However, extensive cement consumption contributes significantly to environmental degradation. To reduce this impact industrial and agricultural wastes are used as partial replacement. Furthermore, strength development is not possible without proper curing; hence, self-curing chemicals are used to guarantee that internal moisture is retained. The performance of self-curing concrete of M30 grade containing 30% Ground Granulated Blast Furnace Slag (GGBS), 10% Rice Husk Ash (RHA), and PEG-400 as a self-curing agent is examined in this research. Concrete specimens are evaluated for mechanical properties at different curing ages. The results show that the optimum dosage of 1% PEG-400 achieved a compressive strength of 40 N/mm² at 90 days along with improved tensile and flexural performance. An examination of the matrix's microstructure showed that it was denser, had better C-S-H formation, and had finer pore structure. Reduce cement use by 40% to help the environment.

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1. Introduction

The strength and adaptability of concrete make it a top choice for building infrastructure. Concrete consists of cement, fine aggregate, coarse aggregate and water. However, cement production has a significant environmental impact as it releases large amounts of CO₂ into the atmosphere. To reduce this issue partial replacement of cement with industrial and agricultural by-products has gained attention as a sustainable solution. Additionally, proper curing is important for the strength development of concrete, it ensures adequate moisture for continuous hydration. However, in many construction scenarios, especially in water-scarce regions, maintaining effective curing conditions is challenging. This study focuses on developing alternative methods for cement and achieve the desired strength without depending on external curing of concrete. Previous studies indicate that GGBS is a by-product that makes concrete stronger and lasts longer by changing the way its parts are arranged within. It interacts with cement compounds to form a denser matrix, which makes it harder for water and chemicals to get through. The best protection against acids and chlorides, providing 10-15% higher resistance, was found when 30% GGBS was used in place of cement, according to the research. Using GGBS makes concrete last longer and be better for the environment [1]. This study reveals that replacing 10-30% cement with Selangor Rice Husk Ash

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markedly enhances Grade 30 concrete durability, reducing permeability, water absorption, and chloride ingress, confirming RHA's sustainability potential [2]. Concrete with 5-30% RHA as a partial substitution of cement exhibits improved fresh, strength, properties. Finer RHA particles facilitate processing and boost pozzolanic activity, generating additional C-S-H gel for enhanced performance. Optimal incorporation of RHA addition lies within the range of 10% to 20%, which imparts improved resistance against water, chloride, and sulfate attacks. In addition to enhanced strength and higher density, RHA imparts better sustainability to concrete through the utilization of agricultural waste [3]. The use of particular agents, such as PEG inside the matrix, allows self-curing concrete to retain internal moisture, making it a form of durable concrete. PEG keeps concrete moistened and strong due to the absence of external addition of water inside the concrete for maintenance of hydration [4]. The admixture based on PEG generates hydrogen bonds with water molecules, which reduces evaporation and enhances hydration effectiveness overall. It lessens the likelihood of fracture formation and increases the material's durability. Studies have shown that 40-60% GGBS for cement, the strength of M20 concrete was increased by 10-15% with the addition of 0.5-1.0% PEG-400. Chemical durability was improved because of the pozzolanic and latent hydraulic reactivity of GGBS, leading to a denser and less permeable microstructure [5]. Incorporation of additional cementitious materials such as GGBS and RHA further improves the microstructure of SCC by inducing pozzolanic reactions and forming more C-S-H gel. The high concentration of amorphous silica in RHA increases the production of C-S-H when combined with calcium hydroxide, $\text{Ca}(\text{OH})_2$. This improves the pore structure and, in turn, increases the strength of the concrete matrix, as shown by [6]. GGBS provides improved chemical resistance and pore refinement, while RHA provides the material with fine reactive silica, which enhances further impermeable and long-lasting ability. Thus, the incorporation of PEG along with mineral admixtures results in a denser, less permeable, and more resistant matrix to chemical and chloride aggressiveness [7]. Due to the lack of readily available water, proper curing cannot take place in dry places, resulting in inadequate hydration and decreased durability. Cures on Its Own Concrete Newly developed self-curing concrete offers an innovative approach to addressing this issue. The internal feeding is provided by water-retaining compounds like propylene glycol (PG) and polyethylene glycol (PEG) rather than by external cures [8].

Water retention in self-curing systems based on PEG is a result of hydrogen bonding with water molecules, which inhibits water loss by slowing down the rate of evaporation. The addition of RHA to PEG at 10–20% increased water retention by up to 12%, while it reduced capillary porosity, thus enhancing resistance to chemicals and durability [9]. Experimental studies on self-curing concrete using propylene glycol showed that compressive strength improved by 13–18% at a 1.5% PG dose for M30 concrete after 28 days [10]. Concrete with 25–50% treated desert sand achieved enhanced compressive strength (up to 55.45 MPa) and reduced water absorption (3.59%), confirming TDS as a sustainable, high-performance aggregate [11]. Focusing on the confinement effect and structural engineering efficiency, this investigation examines the behavior of concrete filled steel tube stub columns subjected to axial loading. The study highlights that confinement significantly enhances strength and durability by improving stress distribution within the concrete core. In relation to the present study these findings emphasize the importance of internal structure and material interaction, which is similarly achieved through the use of GGBS, RHA and self-curing agents to enhance concrete [12]. Sophisticated models of ensemble neural networks for the detection of failure modes in flat slabs of reinforced concrete. The study demonstrated that machine learning techniques can accurately predict structural behaviour and failure patterns, understand crack development and failure behaviour is essential and similar insight are considered through experimental evaluation of flexure response and performance of self-curing concrete [13]. Few studies have examined the combined impact of GGBS, RHA, and PEG on self-curing concrete performance, despite many investigations into their separate effects. For example, there hasn't been nearly enough research on how GGBS, RHA and PEG 400 interact in self-curing circumstances. This study aims to investigate: to find out how these materials affect the mechanical properties and microstructural features of self-curing concrete. This research presents a revolutionary approach to concrete by using chemical self-curing agents in conjunction with industrial and agricultural waste materials. The result is a sustainable and practical solution. Furthermore, prior research indicates that a replacement level of 30% GGBS and 10% RHA is the most effective, outperforming

other proportions. Similarly, the PEG 400 dosage range of 0-1.5% was adopted based on the literature. These solutions contribute to hydration, making the microstructure denser; they improve mechanical properties with lower water consumption and lower CO₂ emission. Thereby, self-curing concrete is an encouraging development toward more environmentally friendly and long-lasting construction.

2. Materials and Methods

2.1 Cement

Cement serves as a fundamental binding material in construction, crucial for ensuring structural stability. During hydration, it hardens and functions as a bonding agent when mixed with fine and coarse aggregates, resulting in concrete. The binder sample exhibited a faint greenish hue and was free from hard lumps. It was evaluated as per Indian Standards IS 12269:2013, confirming compliance with prescribed quality requirements for construction applications. The cement's qualities are shown in Table 1.

Table 1. Measured properties of cement as per laboratory tests

Color	Grey
Specific gravity	3.14
Bulk density	1451 Kg/m ³
Fineness	329 g/cm ²
Density	3.276 g/cm ³

2.2 Aggregate

The aggregates that make up the bulk of concrete are both inexpensive and essential in establishing the material's mechanical strength and longevity. For around 70% of concrete's total strength, you may thank the mixture of coarse and tiny particles called aggregates. These particles exist in nature. The weathering of rocks produces natural aggregates, while the crushing of rocks produces artificial aggregates. Based on their capacity to flow through or be held by a 4.75 mm sieve, aggregates less than 4.75 mm are deemed fine, while those bigger than 4.75 mm are categorized as coarse, according to standard categorization. The placement of coarse aggregates in concrete creates voids, typically occupying 30–40% of the volume. These voids are filled by fine aggregates, and the remaining interstices are occupied by cement particles, ensuring a dense and cohesive matrix. The quality and grading of aggregates are governed by standards such as IS 383:1970 [14]. Table 2 shows properties of the aggregates.

Table 2. Laboratory test results of aggregates

Properties	Fine Aggregate (FA)	Coarse Aggregate (CA)
Specific gravity	2.67	2.89
density	1.68 Kg/m ³	1.58 Kg/m ³
Fineness modules	3.23	3.18
Maximum size	-	20mm
zone	II	-

2.3 Water

Concrete relies on water for hydration of cement, which in turn increases the material's strength, durability, and workability. However, excessive water can induce segregation and reduce structural integrity. To ensure optimal performance, water should be added incrementally, strictly maintaining the water-to-cement ratio as per IS 456:2000 [15]. Potable water with a pH between 6.2 and 7.5 is recommended for both mixing and curing, as specified in the standard. The water used in this process complies fully with IS 456:2000 [15] requirements for concreting, ensuring quality and reliability in construction applications.

2.4 PEG 400

Polyethylene glycol (PEG), and more especially PEG-400, will serve as the self-curing agent in this study. One example of a hydrophilic polymer is polyethylene oxide, or PEG. The conventional formula for it is $H(OCH_2CH_2)_nOH$, where n is often a figure between four and one hundred eighty, representing the number of ethylene oxide units that recur [16], [17]. Its ability to create strong hydrogen bonds with water molecules facilitated by its abundance of hydroxyl groups explains why it dissolves readily in water. [18]. PEG 400 is non-toxic, non-irritating, colorless, odorless, and viscous Fig.1.

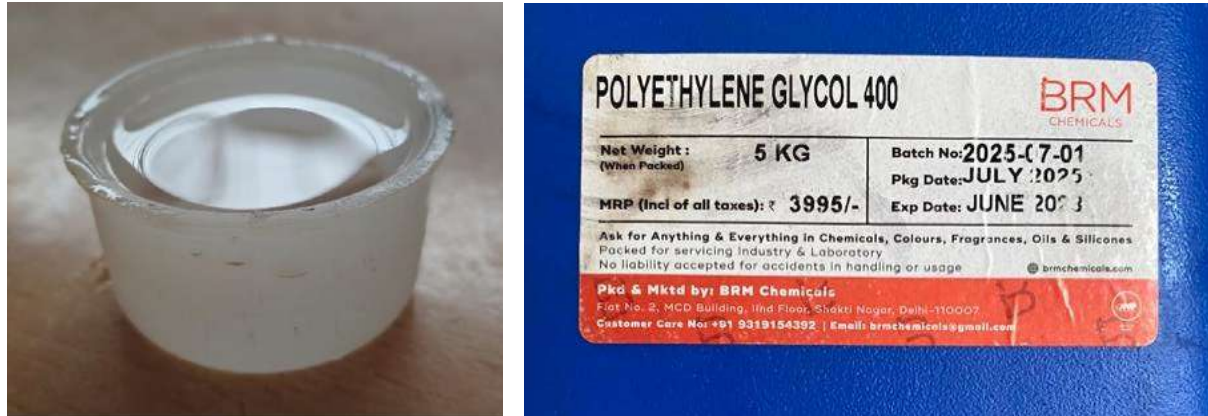


Fig. 1. Polyethylene Glycol 400

It also has a low volatility and a neutral PH, which makes it useful for a lot of different things, especially in pharmaceuticals and hygiene products [19]. PEG-400 is a plasticizing ingredient in concrete technology that improves workability and flow better, making them easier to put and compact [20]. It also helps keep water in the mix, which is important for getting the best hydration and strength growth. PEG-400 is used as a self-curing chemical admixture and not as a binding material. It functions by retaining internal moisture within the concrete through hydrogen bonding with water molecules, thereby ensures continuous hydration and improving strength development without external curing. PEG-400 also helps to evenly distribute the parts of the concrete matrix, which makes sure that the performance is consistent and homogeneous. PEG molecular weight may range from 300 g/mol up to 300 g/mol to 1×10^{78} g/mol. with PEG-400 representing one typical example of a molecule with an average molecular weight of about 400 g/mol. Based on the information provided by the manufacturer, Table 3 summarizes the key attributes of PEG-400.

Table 3. Properties of PEG-400

Properties	Values
Standard Molecular Weight	375-430 g/mol
Viscosity measured at 20 °C	86-106 cs
Maximum allowable Acid Value	0.045%
Density at 20 °C	Gm

2.5 Rice Husk Ash (RHA)

Rice husk is an agricultural by-product obtained from farming activities. Its utilization has increased significantly in recent times. Fig. 2 indicates that this is mainly due to its local availability and relatively low cost compared to other materials. It is used as a sustainable material for enhancing cementitious properties. Table 4 presents the characteristics of RHA.

Table 4. Characteristics of RHA

Property	Observed Value
Color	Black
Specific gravity	2.24
Bulk density	0.25 - 0.32 gm%
Silica (SiO ₂)	87.65%
Alumina (Al ₂ O ₃)	2.61%
Ferric oxide (Fe ₂ O ₃)	2.21%
Calcium Oxide (CaO)	0.18%



Fig. 2. Rice-husk ash

2.6 Ground Granulated Blast Furnace Slag (GGBS)

This study's GGBS came from the Bhilai steel factory in Chhattisgarh. The granular byproduct of quickly cooling molten blast furnace slag in water is known as ground granulated blast furnace slag. Fig. 3 illustrates that it forms a glassy product with minimal crystallization, exhibits high cementitious characteristics and is finely ground. Table 5 shows properties of GGBS



Fig. 3. GGBS

Table 5. Properties of GGBS

Color	Off white
Specific gravity	2.9
Bulk density	1200 Kg/m ³
Fineness	350 cm ²
Calcium oxide	40%
silica	35%
Alumina	13%

2.7 Superplasticizer

To make the concrete mix more workable, a superplasticizer admixture based on polycarboxylate ether (PCE) was used. The superplasticizer available under the brand name High bond, with a pH value of 6.8, shows near-neutral characteristics. It was discovered that the chloride concentration was 1.1%, which improved the concrete's overall performance.

3. Experimental Work

The concrete mix design for the M30 grade was carried out in accordance with IS 10262:2019. The mix proportions were obtained by taking into account the requirements of IS 456:2000 and the objective mean strength. The finalized mix proportions Kg/m³ are presented in Table: 6 and IS 1199:1959 was followed in the preparation of all specimen castings [21]. Five different mixtures were cast: plain concrete (PC) with no replacement, cement mixed with 30% GGBS and 10% RHA, and three other mixes (SC1, SC2, and SC3) with varying amounts of PEG-400 (0.5, 1%, and 1.5%) by weight of cementitious material. We evaluated the specimens at 7, 28, 56 and 90 days after curing to see how well they performed; we water-cured the PC and CM specimens and kept the self-curing mixtures SC1, SC2 and SC3 in an outdoor setting. The specimens were evaluated for compressive and flexural strength in accordance with IS 516 [22] and split tensile strength as per IS 5816 [23] using a compression testing machine under a constant and uniform loading rate without shock.

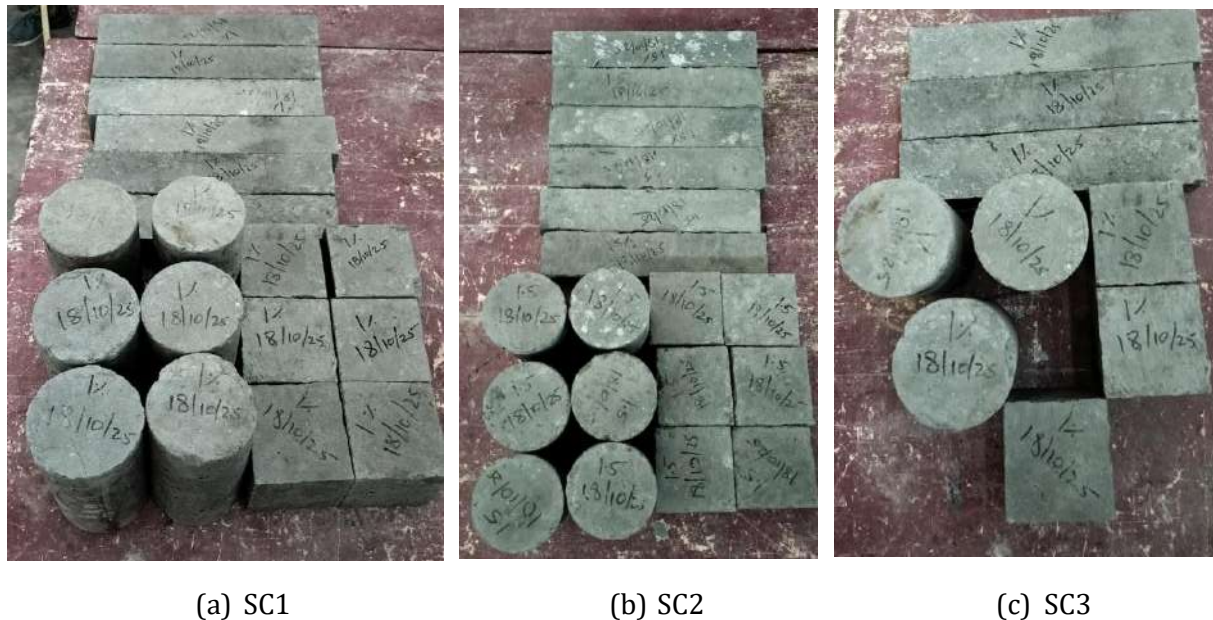


Fig. 4. Cast Specimens

Table 6. Details of mix proportion

Mix	w/c	Water Kg/m ³	Cement Kg/m ³	GGBS Kg/m ³	RHA Kg/m ³	Coarse Aggregate (20mm) Kg/m ³	Coarse Aggregate (10mm) Kg/m ³	Sand Kg/m ³	Admixture Kg/m ³	Self- Curing Kg/m ³
PC	0.46	180	385	-	-	866	442	555	-	-
CM	0.46	180	231	115.5	38.5	866	442	555	1.26	-
SC1	0.46	180	231	115.5	38.5	866	442	555	1.26	1.925
SC2	0.46	180	231	115.5	38.5	866	442	555	1.26	3.850
SC3	0.46	180	231	115.5	38.5	866	442	555	1.26	5.775

4. Results and Discussion

4.1 Mechanical Properties

4.1.1 Slump Cone Test

An essential workability test for determining how new concrete flows and feels is the slump test. It provides a rapid and reliable indication of the mixture's ease of placement, compaction, and finishing under site conditions. The increase in PEG-400 dosage leads to a higher slump because the polymer forms hydrogen-bonded interactions with mixing water, improving lubrication within the cementitious matrix. This reduces internal friction between particles, enhances workability, and results in greater flowability of fresh concrete. The slump value increased progressively with higher PEG-400 dosage, indicating improved workability, while maintaining acceptable consistency without segregation for all concrete grades tested [24]. The slump values are presented in Fig. 6. It is observed that parameter increase progressively with the rise in PEG-400 dosage. For M30 concrete, the slump shows an increasing trend.

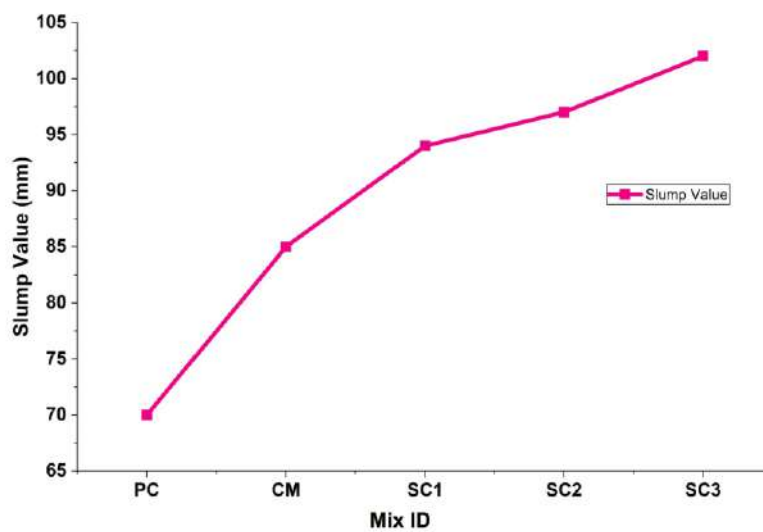


Fig. 6. Slump values for different mixes

4.1.2 Compressive Test

By acting as an internal curing agent, PEG-400 improves the overall mechanical performance of self-curing concrete by maintaining a consistent moisture content throughout hydration, increasing particle dispersion, and allowing for further strength increase [25]. According to the findings of the experiments, the strength properties of concrete are significantly improved when GGBS and RHA are used as additional cementitious materials. Although workability decreased as replacement levels increased, compressive strength rose steadily with curing time for every combination. The presence of RHA contributed to improved later-age strength, while GGBS further enhanced the overall strength development. These findings confirm that the combined use of GGBS and RHA effectively improves concrete strength while supporting sustainable construction practices [26]. The compressive strength performance of the proposed mixes was evaluated using standard cube specimens. Five concrete mixes were prepared in total, including one control mix for M30 grade concrete and four modified mix proportions. The findings shown in Fig. 7 show the outcomes of casting 12 cubes of each combination. Three specimens were assessed at 7, 28, 56 and 90 days. The research showed that the 1.0% PEG-400 dose significantly increased strength. Seven days later, the mix had reached 31.35 N/mm², 37.80 N/mm², 39.90 N/mm² and finally 40.07 N/mm² after 90 days. It surpasses all other percentages measured. When compared with the PC mix and the other self-curing variations, the 1.0% PEG 400 mix consistently registered the maximum compressive strength at both curing ages. The results indicate that this dosage level delivers the most effective performance within the experimental range. Thus, the 1.0% PEG 400 mix is identified as the optimum composition based on cube strength.

4.1.3 Split Tensile Test

Researchers found that adding a 1.0% self-curing chemical significantly increased the material's split tensile strength. While M35 concrete exhibited an increase from about 4.2 to 7 N/mm², demonstrating superior performance over conventional mixes [27]. The cylindrical specimens used for split tensile strength testing have a height of 300 mm and a diameter of 150 mm. The tests were conducted at 7, 28, 56 and 90 days, as shown in Fig. 8.

The cylinder specimen strength data show that out of all the tested proportions, the combination with 1.0% PEG-400 performed the best. On day 7, the mix reached a strength of 2.26 N/mm², on day 28, 3.11 N/mm², on day 56 3.42 N/mm², and on day 90 3.52 N/mm². Exceeding both the control mix and the remaining self-curing variations. The strength values show a consistent improvement at the 1.0% dosage across both curing ages.

4.1.4 Flexural Strength Test

A study reported that self-curing concrete incorporating GGBS exhibited improved flexural strength compared to conventional mixes. Optimal replacement levels enhanced matrix density and hydration, resulting in higher bending resistance. Excessive replacement reduced performance, confirming that balanced GGBS combinations provide superior flexural strength in self-curing concrete [4]. The strength behaviour indicates that mixes containing RHA and GGBS would show improved flexural performance at 28 days due to enhanced matrix densification. Higher RHA levels, especially with GGBS, are expected to increase bending resistance compared to conventional concrete [27]. Flexural strength test is performed for Prisms measuring 100 × 100 × 500 mm were used. The tests were conducted at 7, 28 and 56 days as illustrated in Fig. 9.

Flexural strength results for the beam specimens indicated a clear improvement for the mix incorporating 1.0% PEG-400. After 7, 28 and 56 days, this mixture reached strengths of 7.35 N/mm², 9.81 N/mm², and 10.79 N/mm², respectively. Noting the most elevated results across all proportions that were examined. The strength values show a consistent improvement at the 1.0% dosage across both curing ages. Based on the experimental flexural response.

4.2 Statistical Analysis of Mechanical Properties

The compressive strength results, along with the error bars representing standard deviation, indicate a consistent and reliable trend across all mixes and curing ages. The variability at 7 days is relatively higher, which is typical due to early-stage hydration. However, as the curing age increases to 28, 56 and 90 days, the error bars become smaller, indicating improved consistency and reduced variability. With a relatively low standard deviation and the best compressive strength throughout all curing ages, SC2 shows that it is the most stable and effective recipe. The statistical research confirms that adding PEG 400 and other ingredients to concrete improves its uniformity and strength.

The split tensile strength results demonstrate a consistent increase with curing age across all mixes. The variability, indicated by error bars, is relatively higher at early ages and decreases at later stages, reflecting improved stability. Among the mixes, SC2 shows the highest strength with comparatively controlled variation, indicating better performance. The coefficient of variation remains within acceptable limits for most cases, confirming reliability. In terms of improving tensile behavior and decreasing inconsistencies, the findings show that self-curing is successful.

The flexure strength results, along with the error bars representing standard deviation, indicate a consistent and reliable trend across all mixes and curing ages. At 7 days slightly higher variation is observed, particularly in SC1 indicating early-age variability at 28 and 56 days, the variation reduces significantly, showing improved consistency due to continued hydration. Among all mixes, SC2 exhibits the highest flexural strength with relatively low variability, indicating stable and reliable performance. Overall, the coefficient of variation remains within acceptable limits, confirming that the experimental results are statistically consistent.

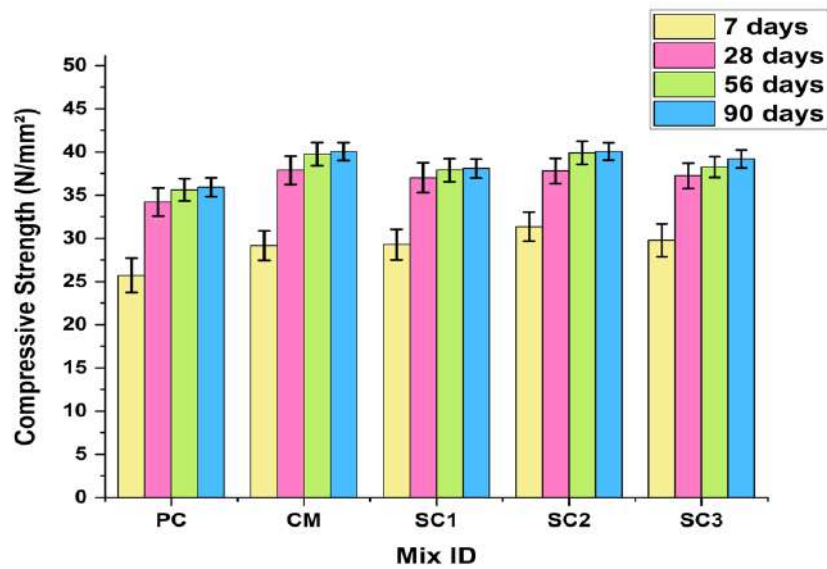


Fig. 7. Compressive test

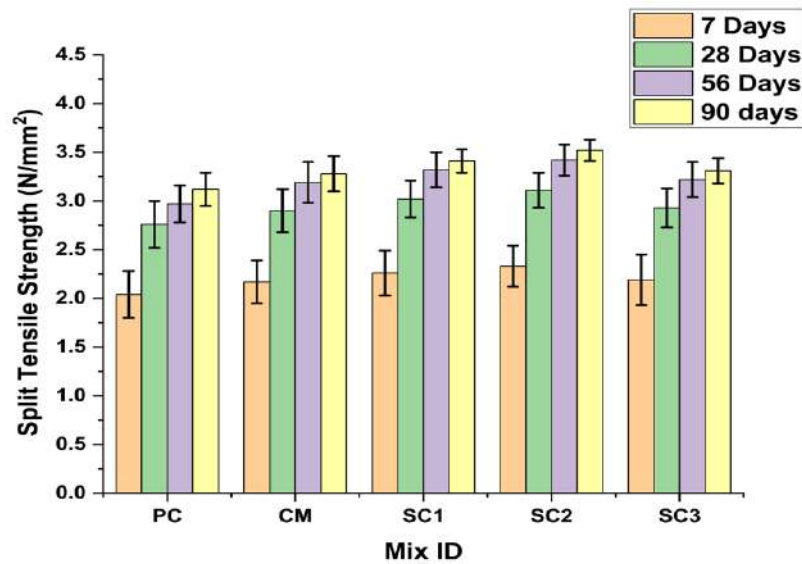


Fig. 8. Split tensile test

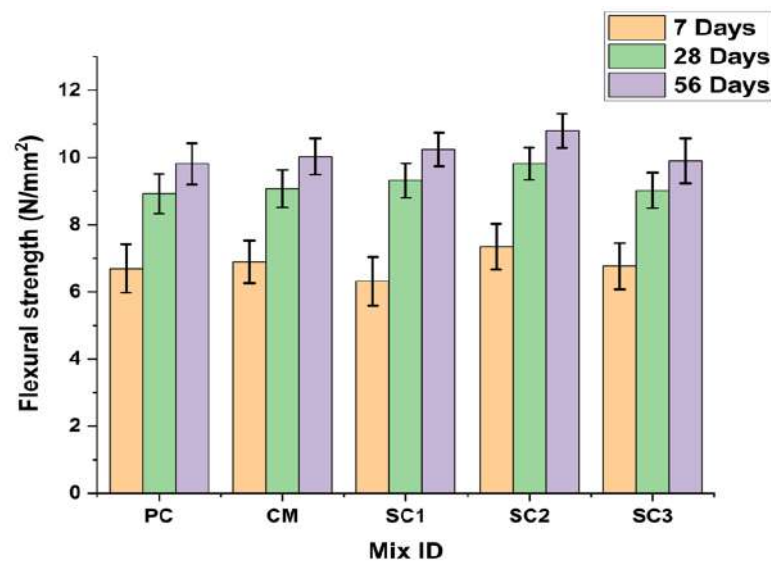


Fig. 9. Flexure strength

4.3 Co-Relation Graphs

To determine the link between the two important mechanical characteristics of concrete, a correlation graph is used. The graph shows the relationship between the compressive strength and the split tensile strength. The horizontal axis usually displays the values of compressive strength, while the vertical axis shows the equivalent values of split tensile strength. The figure enables the assessment of how variations in compressive strength influence the tensile capacity of concrete. The correlation analysis yielded $R^2 = 0.6289$, indicating a moderate relationship between the variables. This reflects the inherent variability in concrete behaviour and experimental conditions and therefore represents a meaningful trend rather than a strong correlation. Typically, as compressive strength increases, split tensile strength also increases in a similar fashion. This correlation suggests that enhancements to the concrete matrix and its holding properties lead to increased performance in both compression and tension. Such analyses are valuable for showing overall mechanical behaviour of alternate concrete mixtures.

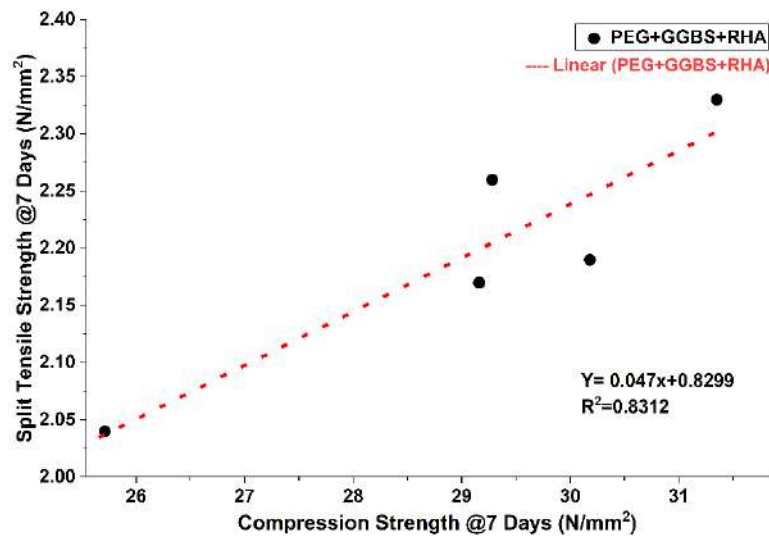


Fig. 10. Correlation between 7-day compressive strength and split tensile strength of concrete mixes

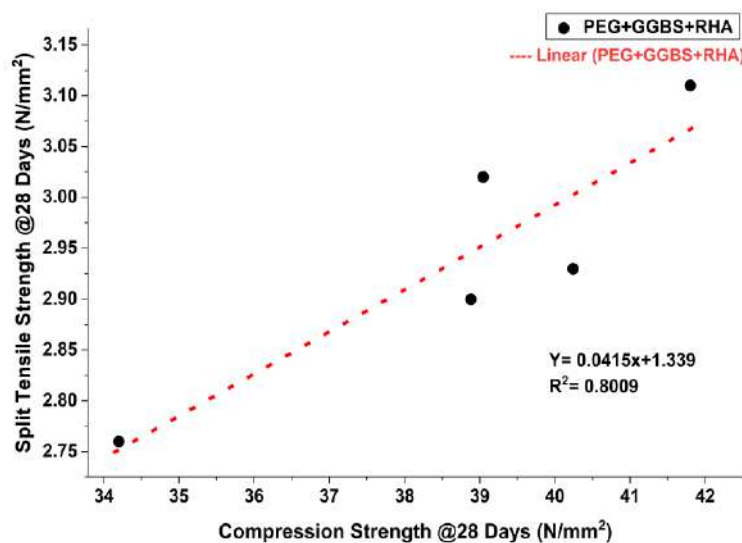


Fig. 11. Correlation between 28-day compressive strength and split tensile strength of concrete mixes

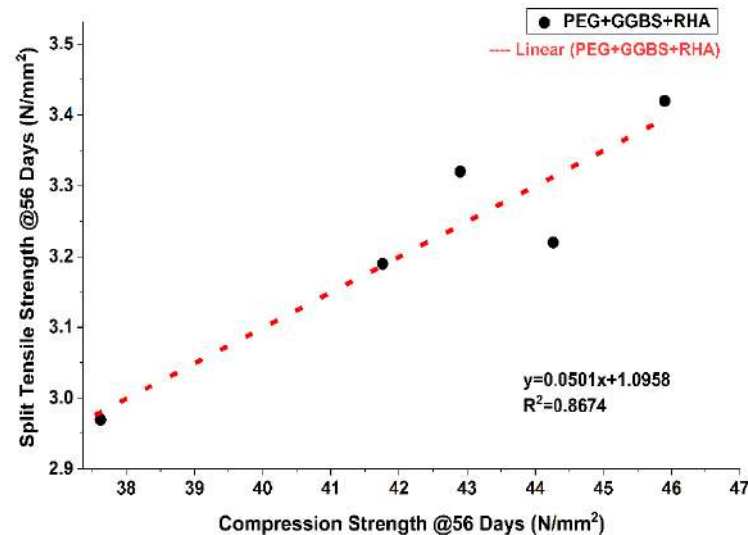


Fig. 12. Correlation between 56-Day compressive strength and split tensile strength of concrete mixes

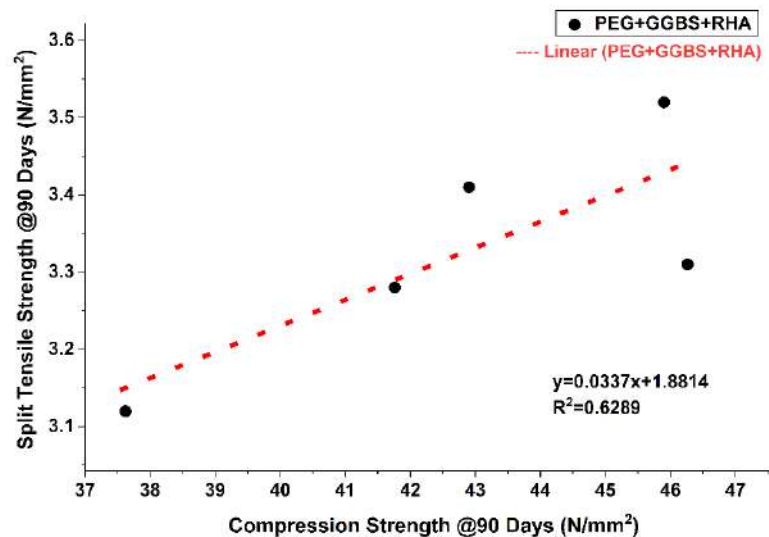


Fig. 13. Correlation between 90-day compressive strength and split tensile strength of concrete mixes

4.4 Microstructure Analysis

Microstructural characteristics were examined using SEM techniques. The SEM micrographs reveal significant changes in the microstructure with the incorporation of GGBS, RHA and PEG-400. The presence of GGBS and RHA increases pozzolanic and indirect hydraulic reactions leading to the formation of additional C-S-H gel which contributes to a denser matrix. The self-curing action of PEG-400 enhances internal moisture availability, facilitating continuous hydration and reducing microcracks. As a result, the voids structure becomes more refined with less voids and improved particle packing. These microstructural improvements are consistent with the observed enhancement in mechanical properties. The SEM patterns of CM, SC1, SC2 and SC3 mixes are presented in Fig.5. The mixes were selected based on their superior mechanical performance and fracture energy. The CM mix exhibited higher porosity and visible microcracks, indicating incomplete hydration. The C-S-H gel appeared as finer, loosely packed particles, reflecting a less dense cementitious matrix. The SC1 mix indicates a comparatively lower level of matrix densification. Although hydrate products are present, unbonded and loosely packed regions are observed, suggesting incomplete particle integration and reduced microstructural continuity.

compared to higher PEG content mixes. The inclusion of PEG as a self-curing agent in the SC2 mix contributed to its relatively thick and homogenous matrix, which in turn improved internal moisture retention and encouraged continual hydration. The SC3 mix, the concrete matrix became noticeably denser, and crack formation was effectively reduced, indicating improved crack control and enhanced structural integrity due to efficient self-curing action. The SC3 mix exhibited a compact matrix featuring significant C-S-H gel formation. This refined microstructure ensures superior interfacial bonding with ferrochrome slag aggregates, effectively minimizing porosity and optimizing mechanical strength.

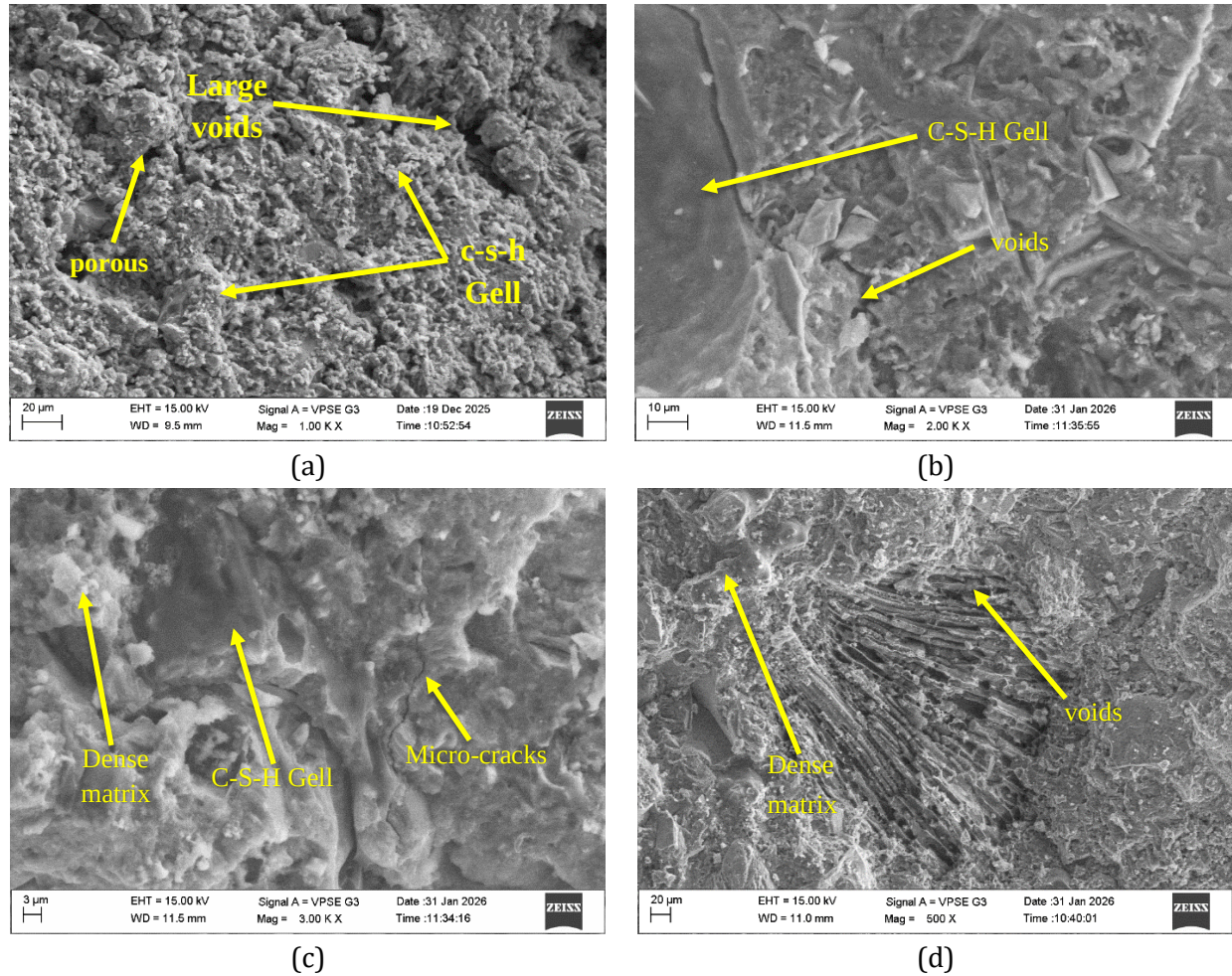


Fig. 14. SEM Analysis of (a) CM, (b) SC1, (c) SC2 and (d) SC3

5. Conclusions

From the above findings, the following key conclusion is established:

- The workability of concrete increased progressively with higher dosages of PEG-400 compared to the conventional mix, indicating improved fresh-state behaviour due to enhanced internal lubrication and moisture distribution.
- Among all the mixes investigated, the SC2 mix demonstrated the highest compressive strength, confirming that a PEG-400 dosage of 1.0% provides the most favorable performance under the conditions studied.
- Mechanical strength decreased when PEG-400 concentration was increased above the optimal level, suggesting that an overabundance of polymer dose might compromise the strength of the hardened cement matrix.
- To reduce reliance on external curing processes, PEG-400 was used as a self-curing agent. This enabled continuous strength growth under ambient curing conditions.

- The combination of PEG-400 with supplementary cementitious materials contributed to improved overall performance by facilitating better hydration continuity throughout the curing period.
- Incorporation of 30% GGBS and 10% RHA as partial replacement for cementitious material. showed improved strength performance. This approach reduces 40% of cement along with reduced water demand due to 1% PEG-400 as self-curing agent, by reducing the environmental impact and developing sustainable concrete.

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Shear behavior and failure mechanisms of reinforced concrete beams with construction joints in the shear span

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Abstract

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Interface shear transfer

This study investigates the influence of construction-joint geometry within the shear span on cracking, stiffness, deformation, energy absorption, and ultimate capacity of reinforced-concrete T-beams. Four beams were compared under three-point loading over a 1,000 mm net span (shear span, $a = 500$ mm; $a/d = 2.94$): a monolithic reference beam and beams containing vertical, inclined, and keyed construction joints in the shear span. All beams had identical reinforcement and a measured concrete cube compressive strength of 30 MPa. Relative to the monolithic reference beam, the jointed beams exhibited first-crack-load reductions ranging from 0.0% to 15.4% and ultimate-load reductions ranging from 0.9% to 4.5%. The maximum reductions in first-crack load and ultimate load were 15.4% and 4.5%, respectively, indicating that joint geometry affected cracking resistance more strongly than ultimate resistance. At 50 kN, the vertical, inclined, and keyed jointed beams exhibited midspan deflections that were 20.0%, 8.0%, and 3.0% greater than that of the reference beam, respectively. The keyed joint retained 97.1% of the reference secant stiffness at 50 kN and 99.1% of the reference ultimate load. Overall, the keyed configuration most closely reproduced monolithic behavior, plausibly because its geometry increased the effective contact area and mechanical interlock, whereas the vertical joint produced the largest stiffness and deformation penalty.

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1. Introduction

In the construction of concrete, it may be necessary to have a construction junction when casting is stopped at one point and resumed at a later period. It is important in these conditions that the deflections of the linked portions of the member are compatible, as failure of the joint can result in separation and the structural member can be effectively divided into two portions [1]. It is therefore necessary to pick the position of the construction joint and to carry it out in such a way that the structural integrity and functional performance of the member are not compromised [2]. In some circumstances the joint is produced at a site beyond the direct control of the contractor; therefore, the adequacy of the resulting structural member must be carefully reviewed. A construction joint is a potentially weak horizontal or vertical junction within an otherwise monolithic concrete element that may allow relative sliding between the adjoining surfaces [3]. The selection of an appropriate joint location is an important responsibility shared by the design engineer and the contractor [4,5].

Many reinforced-concrete structures are not cast in a single operation and so have construction joints, which are necessary because of construction sequencing, concrete supply limitations, or

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practical casting restrictions. Within the shear span, a joint cuts through the concrete matrix in an area of high shear and inclined main stress. Thus, the interface could intersect the key diagonal-crack path or disrupt the compression field between the loading point and support, resulting in an increase in deformation or premature interface-controlled damage. After the first bond fails, shear transfer across the joint is largely provided by friction, aggregate interlock, compression normal to the interface and the dowel and clamping actions of reinforcement spanning the joint. Their efficiency depends on joint position and orientation, surface preparation and interface geometry. Laitance removal, cleaning, moisture conditioning and roughening improve contact and resistance to sliding, while a keyed interface can add bearing and mechanical interlock.

The literature thus identifies four interacting control variables: the joint location, orientation, interface condition and reinforcement/detailing. Their effects are linked since each variable influences the stress state on the contact and the potential mechanisms for shear transfer [4,6–9]. After bond loss, reinforcement spanning or enclosing the joint inhibits opening, provides clamping action, and facilitates frictional transfer. Interface geometry has a role through bearing and mechanical interlock, notably in keyed joints. These mechanisms can influence serviceability and ultimate resistance differently. An interface can be sufficiently strong at failure yet still increase deformation or reduce stiffness during operating loads.

In the T-section, the web carries most of the shear while the flange carries compression and adds to the flexural rigidity. Under three-point loading, a joint in the shear span can cross the diagonal tension field and interrupt the load point-to-support force transmission path. These properties allow for the isolation of the effect of joint geometry, since the concrete strength, beam size, reinforcing and loading arrangement are kept constant. Monks and Sadgrove [10] observed that smooth construction joints did not perform well in high shear areas, whereas rough joints are more like continuous concrete. The existence of vertical joints was shown to diminish the modulus of rupture of the plain concrete, implying that the interface can be a preferential path for cracking under tension [11]. This set of results explains why the presence of a joint located inside the shear span may induce cracking or enable further sliding.

Joint location determines the stress state acting on the interface. A joint in a low-shear region may have little influence on overall behavior, while a joint between a concentrated load and the adjacent support can intersect the diagonal tensile field or the compression strut. Monks and Sadgrove [10] identified this location effect experimentally. Abdulmunaam and Farhan [8] likewise showed that construction-joint location affected the capacity, deformation, and failure mode of reinforced self-compacting concrete beams. Joint orientation controls the combination of shear and normal stress acting on the interface and its alignment with the probable diagonal-crack path. Akin and Güz [7] found that the angle of a cold joint in the shear zone affected crack development and beam capacity. Their findings also indicated that reinforcement provided near the interface reduced the adverse effect of the joint. Jabir et al. [6] reported that inclined construction joints exhibited strength losses that depended on joint inclination and the stress acting on the interface.

Reinforcement and interface geometry influence residual resistance following bond loss. Stirrups and other bars that span or confine the joint limit opening, create clamping action, and promote frictional transfer. According to Abdulmunaam and Farhan [8], increasing secondary reinforcement limited joint separation and made the response more ductile, while increasing longitudinal reinforcement without increasing interface and shear resistance could shift failure toward shear or joint separation. These findings suggest that joint performance cannot be judged in isolation of the surrounding reinforcement.

Keyed joints provide an additional load-transfer mechanism through bearing on the key surfaces and mechanical interlock at the interface. Abass [4] discovered that adequately specified keyed interfaces might approach monolithic beam behavior; however, vertical and inclined joints produced larger reductions under less favorable conditions. Horizontal construction joints showed higher deflection and different flexural response, suggesting that interface compliance might be more influential on serviceability than ultimate resistance Al-Rifaie et al., [12].

Further experimental research emphasizes the importance of the interface conditions and joint position. Aziz and Ajeel [13] examined reinforced-concrete T-beams with construction joints and

flange apertures that were simply supported. The specimen with a cold joint (defined as an interface with no specific treatment and material enhancement) showed about a 27% drop in shear strength, stressing the susceptibility of untreated joints in shear critical regions [13]. In another structural system, Bin Osman et al. [14] investigated six concrete slabs with different types of construction joints. The monolithic control had the highest overall load, stiffness, and energy response. The inclined-joint slab exhibited better load and energy capacity than the other jointed arrangements, while the Hy-Rib Mesh joint had the highest stiffness among the jointed slabs [14].

Other evidence has looked at vertical joints, specific interface materials and the positioning of joints. A study by [15] on singly reinforced beams with vertical construction joints produced charts to estimate the losses in flexural capacity in normal-strength concrete [15]. Jang et al. [16] showed that grooved UHPC interfaces with steel-fiber reinforcement may significantly improve joint shear resistance. They stressed the importance of interface geometry and material improvement [16]. Mathew and Nazeer [17] revealed that for the tested 20 and 40 MPa concrete mixes, the beams with joints in the middle third of the span had a somewhat higher load-carrying capability than beams with joints extending into the outer third [17]. These results collectively support that the structural implications of construction joints are contingent on the stress field at the joint, in addition to interface details and material properties.

Additional beam studies have addressed joint elevation, concrete type, and shear resistance. Abbas et al. [14] examined reinforced-concrete beams with longitudinal construction joints at different elevations and found that the ultimate load, first-crack load, and stiffness were affected by joint elevation. Ismael et al. [15] investigated the structural performance of self-compacting-concrete beams with construction joints; the influence on ultimate load was greater than that on first-crack load, and horizontal joints showed higher structural efficiency. Ibrahim et al. [16] investigated reinforced-concrete beams without shear reinforcement and reported that a vertical construction joint at mid-span reduced shear capacity by up to 10%, whereas an interface perpendicular to the main shear crack increased shear strength by more than 30%.

Recent studies have broadened the evidence base beyond conventional beam configurations. Abdul-Majeed et al. [18] considered the effect of the number of horizontal construction joints in reinforced-concrete beams, while Kadhum and Al-Zuhairi [19] examined construction joints in reinforced-concrete deep beams. Över et al. [20] studied the combined effects of cold-joint location and angle, and Feng et al. [21] evaluated the shear behavior of segmented cast UHPFRC beams with and without stirrups. Huang et al. [22] quantified the respective contributions of cohesion, friction, and dowel action at concrete interfaces. Together with the early shear tests of Clark and Gill [23] and the joint-detailing guidance in ACI 224.3R [17], these studies further show that interface response is governed by both joint configuration and the mechanisms available to transfer shear.

The literature review shows that the presence of construction joints can cause a reduction in the cracking resistance, stiffness, deformation capacity and ultimate capacity, but the magnitude of the reduction and the dominant failure mechanism, if any, depend on the location, orientation, surface condition, crossing reinforcement, shear-span geometry and loading scheme of the joint. It is also indicated that serviceability penalties may be observed prior to significant losses in ultimate resistance. However, direct comparison of vertical, inclined and keyed joints in the same shear span is still limited since the published studies generally vary many parameters at the same time or consider just one interface type. The present work aims to fill this gap by testing T-beams with same dimensions, material attributes, reinforcement and loading setup but with different joint configuration. The load-deflection response, secant stiffness, deformation, energy absorption and ultimate load are assessed to determine which design best mimics monolithic behavior.

2. Materials and Method

The beams were designed as reinforced-concrete T-sections with a flange width of 200 mm, a web width of 100 mm, an effective depth of 170 mm, and a flange thickness of approximately 50 mm, as shown in Figure 1. The overall beam length was 1,200 mm, with a 1,000 mm net span between supports. Under the central point load, the shear span was 500 mm and the shear-span-to-effective-depth ratio was $a/d = 2.94$. The measured concrete cube compressive strength was 30 MPa. Two

10 mm diameter longitudinal tension bars, with a total area of 157 mm^2 and a yield strength of 550 MPa tested according to [18], were provided. The shear reinforcement consisted of 8 mm diameter stirrups spaced at 100 mm center-to-center with a yield strength of 430 MPa tested according to [19].

The comparative test matrix comprised four beams: a monolithic reference beam and three beams containing vertical, inclined, and keyed construction joints in the shear span. All beams had the same geometry, reinforcement, material properties, and three-point loading arrangement. The assessment therefore isolates the effect of joint geometry on crack initiation, shear transfer, stiffness, deformation, energy absorption, and ultimate capacity. Only these four variants are included in the stated cracking, bending, and shear calculations, as well as the comparisons that follow. Concrete was produced with ordinary Portland cement. The recorded mix design, fresh-concrete slump, and $150 \times 150 \times 150 \text{ mm}$ cube-test results are summarized in Table 1. Detailed aggregate gradation was not retained in the available experimental record and is therefore not reported; the maximum aggregate size was 20 mm.

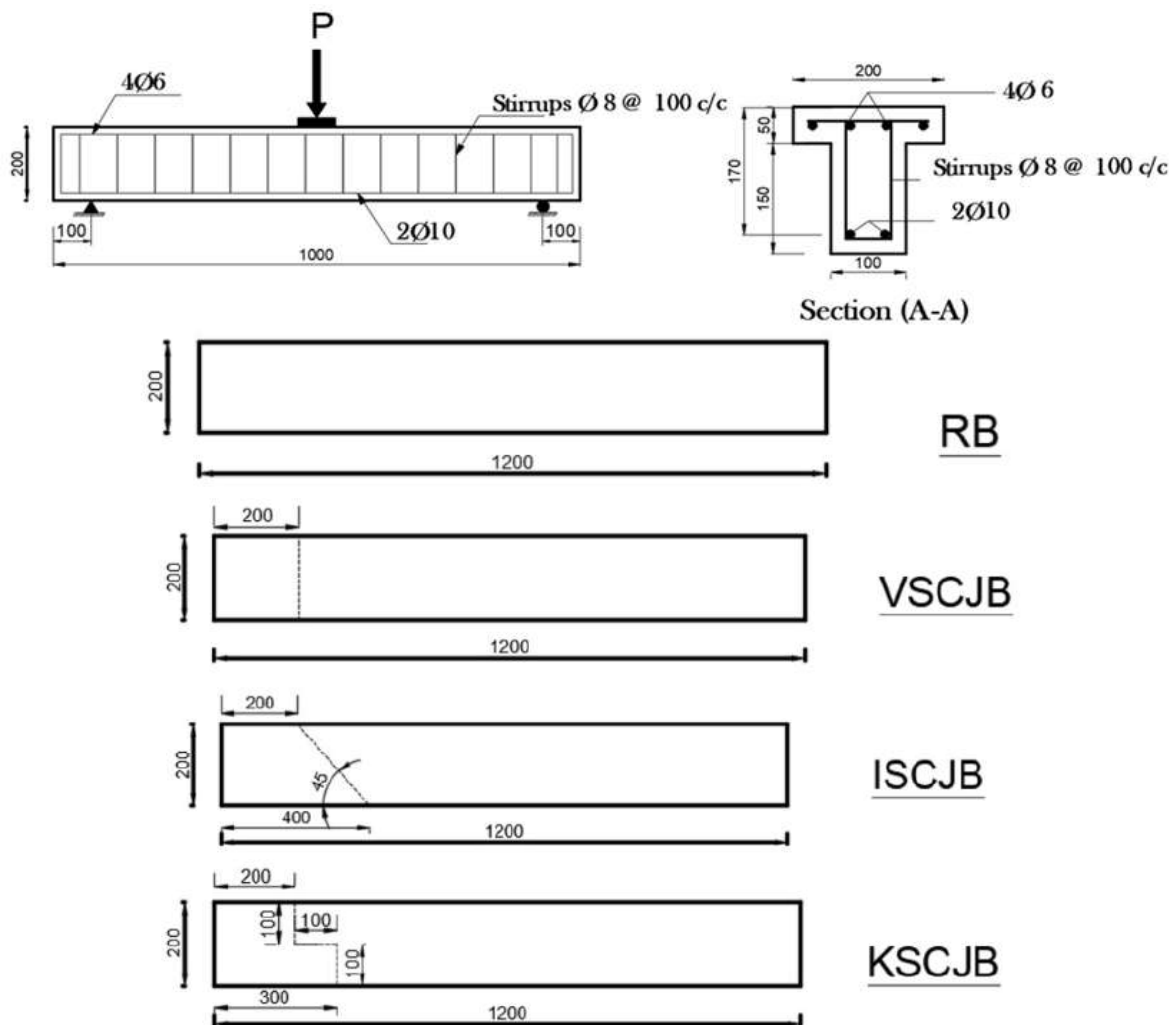


Fig. 1. Reinforced-concrete T-beam geometry, reinforcement, material parameters, joint layouts, and three-point loading; KSCJB key dimensions: height = 100 mm, width = 100 mm, recess = 100 mm



(a) Wooden formwork



(b) Reinforcement cages before concrete placement



(c) Initial concrete placement and companion cube molds



(d) Wet-burlap covering used for moist curing



(e) Specimens retained under wet burlap after casting



(f) Staged casting showing exposed reinforcement before subsequent placement

Fig. 2. Photographic record of reinforcement-cage assembly, first-stage concrete placement, wet-burlap curing, and staged casting (a-f)

Table 1. Recorded concrete mix proportions and cube-test results

Mix-proportion data (per 1 m ³)	Value	Concrete Cubes Results [20]						
Cement [21]	385 kg	Stage	Specimen	Mass (kg)	Unit wt. (kg/m ³)	Failure load (kN)	Strength (MPa)	Age (days)
Fine aggregate (sand)	820 kg	Stage 1	1	7.845	2324	465.75	20.7	7
Coarse aggregate (gravel)	965 kg	Stage 1	2	7.810	2314	474.75	21.1	7
Water	192.5 kg	Stage 1	3	7.800	2311	477.20	21.2	7
Admixture	0 L	Stage 1	4	7.820	2317	666.10	29.6	28
Total mix mass	2362.5 kg	Stage 1	5	7.790	2323	677.25	30.1	28
Water-cement ratio	0.50	Stage 1	6	7.770	2306	681.75	30.3	28
Maximum aggregate size	20 mm	Stage 2	1	7.84	2322	468.2	20.8	7
Fresh-concrete slump [22]	190 ± 20 mm	Stage 2	2	7.805	2316	472.40	21.0	7
		Stage 2	3	7.795	2312	479.10	21.3	7
		Stage 2	4	7.815	2318	664.80	29.5	28
		Stage 2	5	7.785	2321	675.60	30.0	28
		Stage 2	6	7.765	2308	684.20	30.4	28

To form the construction joints, the second concrete placement was made 48 h after the first. Before recasting, the hardened surface was cleaned with a steel wire brush to remove laitance/cement slurry and then mechanically roughened (chipped) and moistened. The specimens were wet-cured for 7 days. The inclined construction joint was formed at 45° to the beam longitudinal axis. The joint layouts are shown and documented in Figure 1 and 2.

For the comparative evaluation, secant stiffness on a given load was determined as $K = P/\delta$. The energy absorption was calculated by trapezoidal integration of the load-deflection response. The

average increase in deflection was calculated by utilizing the relative deflection increments at 15, 30, and 50 kN, employing the deformation-capacity index (δ_u/δ_{30}).

3. Results and Discussion

3.1 Load-Deflection Response

The load-deflection curves indicate that the four specimens exhibited generally comparable behavior during the initial loading phase, when differences in deformation were small. Nonlinearity began after first visible cracking, which occurred at 11.0–13.0 kN. The separation among the curves became more pronounced as the load increased beyond approximately 25–30 kN. VSCJB deflected the most, followed by ISCJB, whereas KSCJB remained close to the reference beam (Figure 3).

For the tested beams, the first visible crack was identified visually as a flexural crack in the midspan region, rather than as an interface or diagonal-shear crack at the construction joint. First-crack loads were identified by visual observation, and the corresponding midspan deflections were extracted from the original test records. Table 5 reports the principal performance metrics, including ultimate deflection, while Table 6 reports the first-visible-crack load and deflection. VSCJB reached first visible cracking at the lowest load (11.0 kN) and the greatest crack deflection (2.08 mm), whereas ISCJB reached 12.0 kN and 2.00 mm. KSCJB matched RB in first-crack load (13.0 kN) and had a crack deflection of 1.84 mm, close to the RB value of 1.75 mm. At failure, the ultimate deflections were 14.0, 15.2, 15.0, and 14.2 mm for RB, VSCJB, ISCJB, and KSCJB, respectively. Relative to RB, crack deflection increased by 18.9% for VSCJB, 14.3% for ISCJB, and 5.1% for KSCJB; the corresponding ultimate-deflection increases were 8.6%, 7.1%, and 1.4%. The larger crack and ultimate deflections of VSCJB and ISCJB indicate more compliant behavior; however, these deformation values must be interpreted with their lower first-crack and failure loads rather than as improved structural performance.

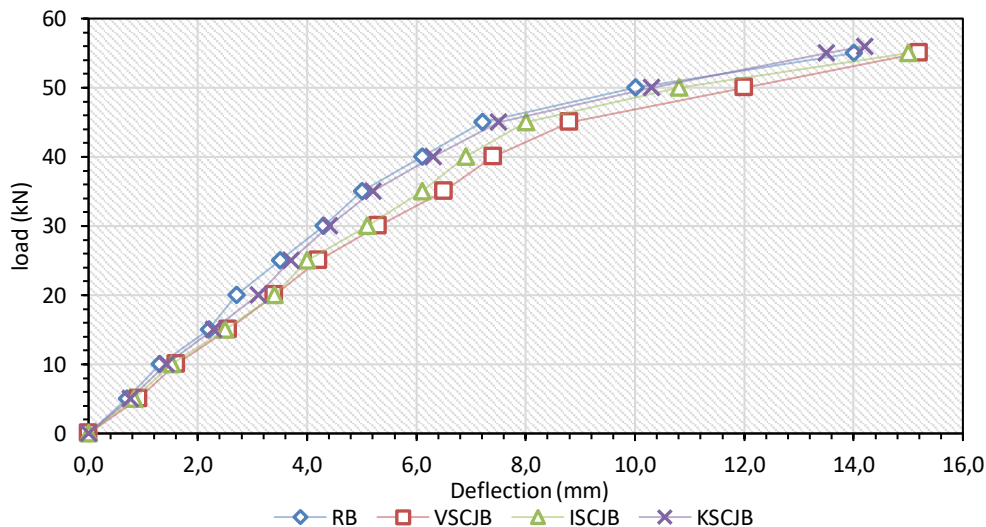


Fig. 3. Comparison of reference and construction-joint beam load-deflection responses

At 50 kN, the recorded deflections were 10.0, 12.0, 10.8, and 10.3 mm for RB, VSCJB, ISCJB, and KSCJB, respectively. The high-load stiffness penalty was greatest for VSCJB, which deflected 20.0% more than RB. VSCJB and ISCJB exhibited greater deformation, consistent with reduced composite action and increased sliding or opening along the construction-joint interface. KSCJB exhibited better stress transfer across the joint. All the curves became nonlinear after first evident cracking at 11.0–13.0 kN, and the response further steepened as the load reached 45 kN, which indicated rapid deformation of the specimen approaching failure. Corresponding ultimate deflections are described in Table 5 and first-crack deflections are provided in Table 6.

3.2 Failure-Load Comparison

The Failure load of RB was the highest (56.0 kN) as seen in the Figure 4. The failure loads of VSCJB, ISCJB and KSCJB were 53.5, 54.0 and 55.5 kN respectively, which are reduced by about 4.5%, 3.6%

and 0.9% respectively. VSCJB exhibited the largest capacity loss, indicating less effective stress transmission across the vertical interface. KSCJB maintained 99.1% of the reference capacity, showing that the keyed configuration preserved structural integrity and mechanical interlock.

The failure load of VSCJB reduced by 4.5% which is in agreement with 0-5% failure-load reduction reported by Abass [4] for vertical construction joints. The ISCJB reduction of 3.6% is slightly higher than the 1.25–2.5% range provided by [6] for favorable inclined-joint configurations, but lower than 8-20% reduction for some inclined joints with high interface requirements as suggested by [4]. These comparisons show that the joint effectiveness depends on the interaction of position, orientation, surface texture, reinforcement and crack path. Overall, the effect of construction joint geometry on failure capacity was restricted with the keyed joint performing the best.

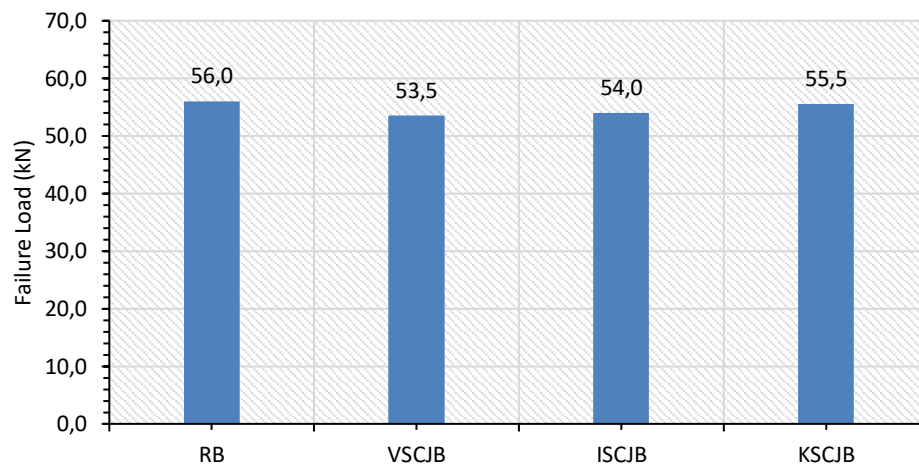


Fig. 4. Reference and construction-jointed beam failure loads

3.3 Cracking, Bending, And Shear Response

The first apparent crack in the monolithic reference beam was at 13.0 kN with an ultimate load of 56.0 kN (Table 2). The three jointed beams fractured between 11.0 and 13.0 kN, which is equivalent to first-crack-load reductions of 0.0-15.4%. Their ultimate loads were between 53.5 and 55.5 kN corresponding to lower reductions of 0.9-4.5%. These values are different indicating that for the shear-span designs evaluated, construction joints had a greater effect on crack initiation and serviceability than on ultimate load capacity.

The vertical shear-span jointed beam had the lowest first-crack load of 11.0 kN and the lowest ultimate load of 53.5 kN. Its reduction of 15.4% in first visible-crack load was greater than its reduction of 4.5% in ultimate load. Consequently, while the early apparent crack did not lead to a proportional loss of ultimate capacity, it is structurally significant as it shows earlier deterioration of serviceability. Because the crack-initiation location and specimen-to-specimen tensile-strength variability were not measured or replicated, the observed difference should not by itself be attributed solely to the interface.

The performance improved progressively from the vertical to the inclined and keyed joint types throughout the shear span. The vertical joint fractured at 11.0 kN and had a maximum failure load of 53.5 kN, whereas the inclined joint cracked at 12.0 kN and had a maximum failure load of 54.0 kN. The keyed joint was observed to crack at 13.0 kN, which was the same as the reference beam, and it failed at 55.5 kN. The near-monolithic response demonstrates that the keyed design provided an effective shear-transfer path across the interface and improved the mechanical interlock. The larger effective contact area of the keyed arrangement may possibly explain its higher capacity by providing a bigger surface for bearing, friction and interfacial shear transmission.

The moment and shear indices in Table 3 provide an equivalent interpretation. The reference and keyed shear-span beams exhibited a cracking moment of 3.25 kN.m and an average cracking web shear stress of 0.382 MPa. At failure, the keyed joint reached an ultimate moment of 13.875 kN.m

and an average web shear stress of 1.632 MPa, close to the reference values of 14.0 kN.m and 1.647 MPa. In contrast to code-based design capacity, these average shear-stress statistics are experimental indices based on the web width and effective depth.

Overall, the findings indicate that joint geometry determines the balance of fracture resistance, deformation control, and ultimate capacity. The limited ultimate-load reductions are consistent with continued shear transfer after cracking through friction, aggregate interlock, clamping by reinforcement crossing the interface, and dowel action. These mechanisms were not measured directly in the present tests; therefore, this interpretation is qualitative rather than a quantified mechanism attribution. Evaluation of construction-joint performance should compare the measured first-crack load, deflection, stiffness, and ultimate capacity rather than rely on a single derived indicator.

Table 2. Cracking and capacity metrics of the tested beams

Joint configuration	First-crack load (kN)	Ultimate load (kN)	Reduction in first-crack load (%)	Reduction in ultimate load (%)
Monolithic reference beam	13.0	56.0	0.0	0.0
Vertical joint in shear span	11.0	53.5	15.4	4.5
Inclined joint in shear span	12.0	54.0	7.7	3.6
Keyed joint in shear span	13.0	55.5	0.0	0.9

Table 3. Experimental bending and shear actions derived from the test loads

Joint configuration	Cracking moment (kN·m)	Ultimate moment (kN·m)	Cracking shear (kN)	Ultimate shear (kN)	Cracking shear stress (MPa)	Ultimate shear stress (MPa)
Monolithic reference beam	3.250	14.000	6.50	28.00	0.382	1.647
Vertical joint in shear span	2.750	13.375	5.50	26.75	0.324	1.574
Inclined joint in shear span	3.000	13.500	6.00	27.00	0.353	1.588
Keyed joint in shear span	3.250	13.875	6.50	27.75	0.382	1.632

3.4 First-Crack Response and Serviceability Implications

Table 4 isolates the first-crack response, which is the most direct experimental indicator of the service-stage effect of the construction joint. Relative to the monolithic reference, first-crack-load retention ranged from 84.6% to 100.0%. The associated cracking moment ranged from 2.750 to 3.250 kN.m, confirming that joint geometry changed the tensile continuity available before the formation of a visible crack.

The first-crack data reveal a clear ranking that is not fully visible from ultimate load alone. The keyed joint in the shear span fully retained the reference cracking load of 13.0 kN. The inclined joint retained 92.3% of the reference cracking load, whereas the vertical joint retained 84.6%, demonstrating the greatest reduction in crack resistance among the tested joint details. This pattern demonstrates that early cracking was influenced by the continuity and orientation of the interface within the shear-transfer path. A reduction from 13.0 to 11.0 kN represents a 15.4% observed decrease in first visible-crack load, even though a substantial proportion of the ultimate load remained available. First-crack response should therefore be treated as a primary serviceability criterion. The keyed shear-span configuration provided the most reliable first-crack performance because it reproduced the reference cracking load while also maintaining near-reference stiffness, deformation, and ultimate capacity.

Table 4. First-crack response and relative serviceability classification of the tested beams

Joint configuration	First-crack load (kN)	Retention vs. reference (%)	Decrease vs. reference (%)	Cracking moment (kN·m)	Relative serviceability response
Monolithic reference beam	13.0	100.0	0.0	3.250	Reference level
Vertical joint in shear span	11.0	84.6	15.4	2.750	Moderate
Inclined joint in shear span	12.0	92.3	7.7	3.000	High
Keyed joint in shear span	13.0	100.0	0.0	3.250	Reference level

3.5 Deflection, Stiffness, And Energy Assessment

Figure 5 illustrates the deflections of the examined beams under loads of 15, 30, and 50 kN. At a load of 15 kN, the deflections varied between roughly 2.2 and 2.5 mm, suggesting very minor discrepancies in the initial response.

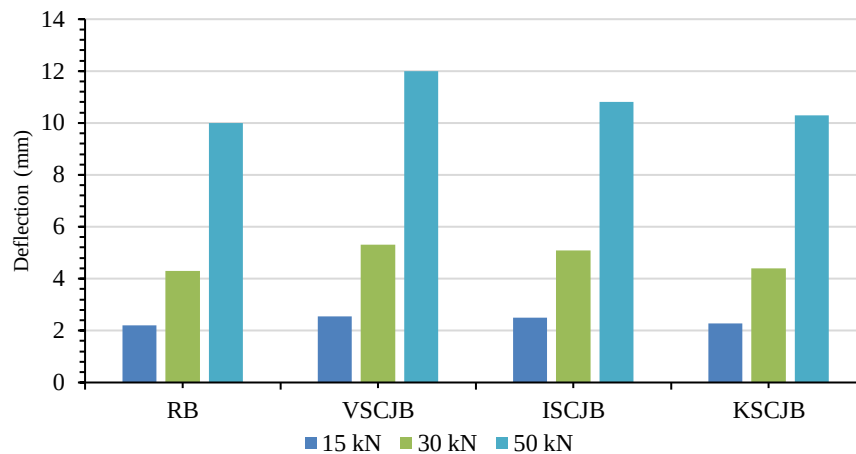


Fig. 5. Measurements of beam deflections under varying load intensities

At 30 kN, VSCJB and ISCJB recorded deflections of 5.3 and 5.1 mm, respectively, compared with 4.3 mm for RB, whereas KSCJB remained close to the reference value at 4.4 mm. Relative to RB, these values represent deflection increases of 23.3% for VSCJB, 18.6% for ISCJB, and 2.3% for KSCJB. Because 30 kN is below the ultimate-load range and closer to the service-load range examined here, these differences highlight the serviceability penalty associated with the vertical and inclined interfaces. At 50 kN, VSCJB exhibited the maximum deflection of 12.0 mm, exceeding RB by 20.0%. ISCJB and KSCJB reached 10.8 and 10.3 mm, representing increases of 8.0% and 3.0%, respectively. Consequently, the vertical joint produced the greatest serviceability penalty, whereas the keyed joint performed comparably to the monolithic beam.

Table 5. Key performance metrics, including ultimate deflection, for reference and construction-jointed beams

Specimen	δ at 30 kN (mm)	δ at 50 kN (mm)	Failure load (kN)	Ultimate deflection (mm)	K retention at 30 kN (%)	Energy to failure (J)
RB	4.30	10.00	56.0	14.0	100.0	522.0
VSCJB	5.30	12.00	53.5	15.2	81.1	532.3
ISCJB	5.10	10.80	54.0	15.0	84.3	539.7
KSCJB	4.40	10.30	55.5	14.2	97.7	522.6

The comparison results show that construction-joint geometry affected stiffness and deformation more clearly than total energy absorption. The keyed specimen (KSCJB) was closest to the

monolithic reference response. Its stiffness retention was 97.7% at 30 kN and 97.1% at 50 kN, and its average deflection increase across the selected load levels was 3.0%. Its ultimate secant stiffness was 3.91 kN/mm, compared with 4.00 kN/mm for RB.

Table 6. First-visible-crack load and deflection

Specimen	First-visible-crack load (kN)	Deflection at first visible crack (mm)
RB	13.0	1.75
VSCJB	11.0	2.08
ISCJB	12.0	2.00
KSCJB	13.0	1.84

Moreover, the deformation-capacity index of KSCJB was comparable to that of RB (3.23 vs. 3.26). The results revealed that the keyed interface yielded composite action with the relative slide limited by the mechanical interlock along the joint.

VSCJB presented the highest sanctions in terms of stiffness and serviceability. Its stiffness retention at 30 kN was 81.1% according to the deflections of 5.30 mm for VSCJB and 4.30 mm for RB. At 50 kN, the stiffness retention was 83.3% with deflections of 12.00 and 10.00mm respectively. Its ultimate deflection was 15.20 mm and its ultimate secant stiffness was 3.52 kN/mm as compared to 4.00 kN/mm for RB. Overall, these signs reveal that the vertical interface showed the worst continuity of the joint configurations considered, which allows a greater deformation of the interface after cracking and a lower diagonal transmission of shear.

ISCJB exhibited an intermediate response. The specimen retained 88.0%, 84.3%, and 92.6% of the reference stiffness at 15, 30, and 50 kN, respectively, with an average deflection increase of 13.4%. Its ultimate deflection and ultimate secant stiffness were 15.00 mm and 3.60 kN/mm, respectively. The relatively high K50/K15 ratio of 77.2% should be interpreted with caution, as it indicates that the relative stiffness reduction between the selected load levels was less pronounced after the initial joint-related penalty, but it does not mean that ISCJB was stiffer than RB or KSCJB in absolute terms.

The energy indices provide a supplementary measure but can be misleading when considered alone. The energy values for VSCJB, ISCJB, and KSCJB at 50 kN were 18.3%, 3.6%, and 1.9% greater than that of RB, respectively. The corresponding changes in energy to failure were 2.0%, 3.4%, and 0.1%. The higher energy value of VSCJB resulted from its greater displacement and the resulting larger area under the load-deflection curve, rather than from greater stiffness or shear capacity. This interpretation is supported by the lower deformation-capacity indices of VSCJB (2.87) and ISCJB (2.94), compared with 3.26 for RB. Energy absorption should therefore be evaluated together with stiffness retention, deflection, and failure load.

The trends shown here are consistent with the broader construction-joint literature. The ultimate-deflection ratios for VSCJB, ISCJB, and KSCJB were about 1.09, 1.07, and 1.01 with respect to RB, which are in or close to the range of 1.02–1.33 reported by Al-Rifaie et al. [12] for beams with horizontal construction joints. This comparison is relevant as the joints have different orientations and different failure capacity. Well-articulated keyed interfaces can appear to represent monolithic functionality, consistent with the near-reference behavior seen for KSCJB Abass [4]. In general, stiffness and serviceability are more sensitive than failure load, which is consistent with findings that joint orientation, surface condition, reinforcement crossing the interface, and the joint plane-governing crack path relationship control post-cracking deformation. Because published studies used diverse shear-span ratios, reinforcement design, joint locations, surface-preparation methods, and loading configurations, direct numerical comparison is risky.

3.6 Observed Crack Patterns

Figure 6 exhibits sample post-test pictures of the monolithic reference beam and the three construction-joint geometries in the shear span. To supplement the load, deflection and first crack results, the photographic record provides qualitative visual proof of the web cracking patterns seen during testing.

The reference beam produced a scattered field of flexural and inclined web cracks. In VSCJB, the post-test images reveal a significantly concentrated set of steep inclined web cracks in the jointed shear span, in agreement with the lower first-crack load, lower stiffness retention, and bigger deflection measured for this specimen. The ISCJB showed inclined and branching web cracks, spread over the shear-span area, which seemed to represent an intermediate behavior between VSCJB and the reference beam. KSCJB revealed several finer, branched web cracks with no single dominant crack visually following the keyed interface; this visual pattern is consistent with its near-reference first-crack load, stiffness, and ultimate capacity. The observations are limited to observable crack patterns. The photos alone do not prove interface slip, debonding or delamination, compression-strut disruption or key shearing/crushing; crack widths, interface opening or relative slip were not measured. Thus, the failure-mechanism explanation is limited to the observable qualitative cracking evidence and the measured global reaction.



(a) Monolithic reference beam (RB)



(b) vertical joint in shear span (VSCJB)



(c) inclined joint in shear span (ISCJB)



(d) Keyed joint in shear span (KSCJB)

Fig. 6. Representative post-test crack patterns (qualitative visual evidence): monolithic reference beam, vertical, inclined, and keyed shear-span joints

4. Conclusions

Across the four-beam comparative series, construction joints reduced first-crack load more strongly than ultimate load. First visible cracking occurred between 11.0 and 13.0 kN, corresponding to observed reductions of 0.0–15.4% relative to the monolithic reference beam, whereas ultimate-load reductions ranged from 0.9% to 4.5%. Construction-joint geometry reduced cracking resistance and affected serviceability while retaining a high proportion of the ultimate capacity.

Table 4 establishes a clear first-crack serviceability ranking for the three joint details in the shear span. The inclined and vertical joints retained 92.3% and 84.6% of the reference cracking load of 13.0 kN, respectively, whereas the keyed joint retained the full reference cracking load. The keyed arrangement also retained 99.1% of the reference ultimate capacity and showed the closest stiffness and deflection behavior to the monolithic beam, providing the most balanced response in terms of crack resistance and deformation control. This performance was plausibly aided by the larger effective contact area created by the key, although this mechanism was not measured directly.

The estimated bending moments, support shears and average web shear stresses also support the same ordering. The results are solely applicable for the geometry, material properties, reinforcement and loading arrangement used in the tests. A major drawback is that fracture widths, crack start locations, joint apertures, interface slip and reinforcing strains were not measured and replication specimens were not tested. Future study should include LVDTs and digital image correlation to quantify interface opening, slide and fracture development.

Where possible, avoid a vertical planar joint in a high-shear region for practical application in the tested configuration. If a construction joint is unavoidable in the shear span, the results favor a keyed, intentionally prepared interface with reinforcement crossing or confining the joint; however, these comparative findings do not replace code-based shear-friction design checks.

Notation and Abbreviations

The symbols and abbreviations used in this study are listed in the following table.

Symbol	Definition
a	Shear span
d	Effective depth of the beam
a/d	Shear-span-to-effective-depth ratio
A_s	Area of longitudinal tension reinforcement
b_f	Flange width
b_w	Web width
f_{cu}	Concrete cube compressive strength
f_y	Yield strength of longitudinal reinforcement
f_{yv}	Yield strength of transverse reinforcement
K	Secant stiffness at a specified load
L	Simply supported span
M_{max}	Maximum bending moment
P	Applied concentrated load
δ	Midspan deflection
δ_u	Ultimate midspan deflection
$KSCJB$	Keyed joint in shear span
RB	Monolithic reference beam without a joint
$VSCJB$	Vertical construction joint in the shear span
$ISCJB$	Inclined construction joint in the shear span
RC	Reinforced concrete
$UHPC$	Ultra-high-performance concrete
$LVDT$	Linear variable differential transformer
K_{50}/K_{15}	Ratio of secant stiffness at 50 kN to that at 15 kN

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Fabrication of a polymeric film supported by nanoparticles prepared by pulsed laser ablation method

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Abstract

Biocompatible polymer membranes are currently in increasing demand, especially those containing homogeneously distributed nanoparticles for medical applications. This research aims to develop a clean and effective approach using pulsed laser ablation. Silver nanoparticles (AgNPs) and gold nanoparticles (AuNPs) have been used as antibacterial agents in the consumer goods, cosmetics, and food industries. In this work, silver and gold nanoparticles were synthesized in a polyethylene glycol (PEG) medium using pulsed laser ablation. The fabricated films were characterized using Fourier transform infrared spectroscopy (FTIR) to identify functional groups, UV-Vis spectroscopy to analyze optical properties, and field emission scanning electron microscopy (FESEM) to examine surface morphology and nanoparticle distribution. In addition, the antibacterial activity was evaluated using the agar diffusion method. The results showed spherical nanoparticles with a polydisperse size distribution, indicating that the medium supports the production of small, well-dispersed nanoparticles, which was reflected in the uniform surface morphology and enhanced antibacterial performance. The results showed spherical nanoparticles with a polydisperse size distribution, indicating that the medium supports the production of small and stable silver nanoparticles, which contributed to reduced aggregation and exhibited enhanced antibacterial performance. The produced silver and gold nanoparticles have been successfully used as an antibacterial agent, as experimentally demonstrated using (*Staphylococcus aureus*), *Enterococcus faecalis*, *Escherichia coli*, and *Klebsiella pneumoniae*). The results showed that the manufactured silver and gold nanoparticles exhibited significant antibacterial activity against *Staphylococcus aureus*, with larger zones of inhibition observed compared to other tested samples, indicating an enhanced antibacterial activity based on the results of the assay.

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1. Introduction

Polymeric nanocomposite films reinforced with nanoparticles have attracted significant attention due to their enhanced optical, mechanical, and thermal properties, which make them suitable for applications in sensing, biomedical devices, and optoelectronic technologies. Among the various nanoparticle synthesis techniques, pulsed laser ablation (PLA) offers a clean and efficient method for producing high-purity nanoparticles without chemical contaminants. Therefore, this study focuses on the fabrication of a polymeric film supported by nanoparticles prepared via the pulsed laser ablation method and the investigation of its structural and functional properties [1]. Their unique optical, biological, and electrical properties make them promising materials for improving the performance of advanced functional devices and polymer-based nanocomposites [2]. Polyethylene glycol (PEG) is a hydrophilic and biocompatible semi-crystalline polymer synthesized through the polymerization of ethylene glycol monomers. In the present study, PEG was selected

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as the polymer matrix because of its low toxicity, excellent water solubility, film-forming capability, and strong compatibility with nanoparticles, which support homogeneous nanoparticle dispersion and improve the stability and functional performance of the fabricated nanocomposite film. The unique properties of polyethylene glycol (PEG), including its low toxicity, biocompatibility, and hydrophilic nature, make it a suitable polymer matrix for nanoparticle-based nanocomposite films. These characteristics promote stable nanoparticle dispersion and improve the interaction between the polymer and nanoscale fillers. In polymer nanocomposites, fillers with dimensions below 100 nm are incorporated to enhance the structural, optical, and functional properties of the resulting material [3,4]. Metal nanoparticles such as silver nanoparticles (AgNPs) and gold nanoparticles (AuNPs) have attracted considerable attention because their optical, electrical, and catalytic properties differ significantly from those of bulk materials [5]. In the present study, these nanoparticles were incorporated into the PEG matrix to enhance the functional properties of the fabricated nanocomposite film. In particular, AgNPs are well known for their strong antimicrobial activity against harmful microorganisms, while AuNPs contribute to improved optical stability and surface-related properties, making both AgNPs and resulting film potentially interesting for future advanced biomedical and functional applications, as suggested literature context [6]. Gold nanoparticles (AuNPs) possess unique optical, physicochemical, and biological properties, in addition to a large surface area and low toxicity, making them highly attractive for biomedical and biotechnological applications [7,8]. In the present study, AuNPs were incorporated into the PEG-based film to improve the stability, surface activity, and functional performance of the antibacterial nanocomposite system. Their high surface reactivity and biocompatibility also support effective interaction with the polymer matrix, which is important for enhancing the overall properties of PEG-based antibacterial films [9–12]. Gold nanoparticles (AuNPs) are widely applied in nanocomposite systems due to their strong surface plasmon resonance, high surface reactivity, and excellent biocompatibility, which make them particularly effective in modifying polymer matrices such as PEG for biomedical and antibacterial applications. In PEG-based antibacterial films, AuNPs contribute to improving surface interactions and functional stability of the composite, thereby enhancing its overall performance in biomedical environments. In this context, the integration of AuNPs into the polymer matrix is specifically motivated by their ability to enhance the structured stability and antibacterial efficacy of the resulting nanocomposite, providing a tailored material design that is frequently discussed in literature for potential biomedical applications [13]. Furthermore, this study focuses on the effectiveness of the pulsed laser ablation method in preparing these nanocomposites, specifically examining how the incorporation of AgNPs and AuNPs into the PEG matrix influences the optical and antibacterial properties of the fabricated films [11-13]. Silver nanoparticles (AgNPs) are of particular relevance in polymer-based antibacterial nanocomposites due to their strong and well-documented antimicrobial activity arising from silver ion release and their ability to disrupt bacterial cell membranes. In PEG-based composite films, AgNPs are therefore primarily employed to impart antibacterial functionality and improve the protective performance of the material for potential biomedical-related applications as highlighted in the literature [14–16]

2. Experimental Section

2.1 Materials

Polyethylene glycol (PEG), with the molecular formula $\text{HO}(\text{C}_2\text{H}_4\text{O})_n\text{H}$ and a molecular weight in the range of 3500–4000 $\text{g}\cdot\text{mol}^{-1}$, was used in this study as the polymer matrix. The PEG was utilized in solid form (pellets from LOBA CHEMIE PVT.LTD/ India). In addition, silver and gold alloys were analyzed using an energy-dispersive X-ray fluorescence (ED-XRF) spectrometer (XEPOS model/ made in Germany) to determine their elemental composition and purity. The measured purities of the silver and gold alloys were found to be 98.99% and 99%, respectively.

2.2 Synthesis of PEG/ Ag and Au Nanoparticles

Initially, high-purity gold and silver plates (99.9%) were used as ablation targets and were ultrasonically cleaned in ethanol, acetone, and deionized water (DIW) for 10 min each to remove surface contaminants. Each target was then placed at the bottom of a separate glass container

containing 3 mL of DIW. The targets were irradiated using a pulsed Nd:YAG laser operating at a fundamental wavelength of 1064 nm, pulse duration of 10 ns, repetition rate of 1 Hz, and pulse energy of 500 mJ, corresponding to a fluence of 0.5 J/cm². The laser beam (6 mm in diameter) was focused onto the target surface using a convex lens with a focal length of 10 cm. A total of 1000 pulses were applied, and the ablation process was carried out for 30 min to generate colloidal gold and silver nanoparticles dispersed in DIW. For the preparation of nanocomposite films, polyethylene glycol (PEG) solution was used as the polymer matrix. Briefly, 2 g of PEG was dissolved in 35 mL of distilled water under magnetic stirring at 70 °C for 3 h until a homogeneous, bubble-free solution was obtained. The prepared colloidal solutions containing AuNPs and AgNPs were then separately added to the PEG solution under continuous stirring to ensure uniform dispersion of nanoparticles within the polymer matrix. The resulting nanocomposite mixtures were poured into clean, leveled glass Petri dishes and allowed to dry under controlled ambient laboratory conditions (25 ± 2 °C, relative humidity ~40–50%) until complete solvent evaporation. This solution casting process resulted in uniform PEG/AuNP and PEG/AgNP composite films with an average thickness of 125 µm. The nanoparticle was incorporated by adding 3mL of the colloidal solution to the PEG matrix (2 g in 35mL) The FESEM analysis confirmed the uniform distribution of nanoparticles throughout the polymer matrix, which ensures consistent optical and antibacterial performance. Finally, the obtained films were carefully peeled from the dishes and stored in a desiccator at room temperature to prevent moisture absorption and preserve their structural stability.

2.3 Antimicrobial Activity Tests

Blood agar was used to cultivate isolates of *Klebsiella pneumoniae*, *Escherichia coli*, *Enterococcus faecalis*, and *Staphylococcus aureus*. The bacterial suspension density was standardized using a 0.5 McFarland turbidity standard to achieve a cell concentration of approximately 1.5 × 10⁸ CFU/mL. Regarding the medium preparation, 38 grams were dissolved in one liter of distilled water to create this medium, which was then autoclaved at 121 degrees Celsius and 15 pounds of pressure for 15 minutes. It was then cooled, transferred into sterile plates, and stored in the refrigerator until it was needed.

To prepare the bacterial suspension, distinct bacterial colonies were transferred via a sterile inoculation loop into tubes containing brain-heart infusion broth. The tubes were incubated at 37 °C for 18 to 24 hours. Next, a sterile swab was utilized to uniformly inoculate the bacterial suspension onto Mueller-Hinton agar plates, which were then left to dry under aseptic conditions. Wells were prepared in the culture media using a sterile cork borer. Utilizing a micropipette, a 100 µL aliquot of each tested concentration (100, 75, 50, 25, and 12.5 µg/mL) was introduced into each well. Finally, the diameters of the zones of inhibition were measured to evaluate the antimicrobial efficacy of each concentration.

2.4 Characterization Techniques

Absorption spectra and Surface Plasmon Resonance (SPR) were recorded for colloids with UV-Vis spectrophotometer (Double Beam 1800 UV, Shimadzu, Japan) manufactured in (Shimadzu, Japan) for the purpose of knowing the effective aggregates, FTIR test: using type the IR Affinity-1CE (FTIR) spectrophotometer, Shimadzu, Japan.

3. Results and Discussions

3.1. Surface Chemistry (FT-IR)

The produced Ag NPs and Au NPs may interact with the PEG molecules. In Figure 3, the FTIR spectra of both pure PEG and PEG-AuNPs are displayed. A wide absorption band at 3437 cm⁻¹ was observed, which corresponds to the O–H stretching vibration of hydroxyl groups inside the polymer structure, because PEG is generated from ethylene glycol. Furthermore, at 2873.94 cm⁻¹, the bending vibrations linked to O–C–H, C–C–H, and C–O–H groups were detected, and at 1348.24 cm⁻¹, the stretching vibrations of methylene groups were noted.

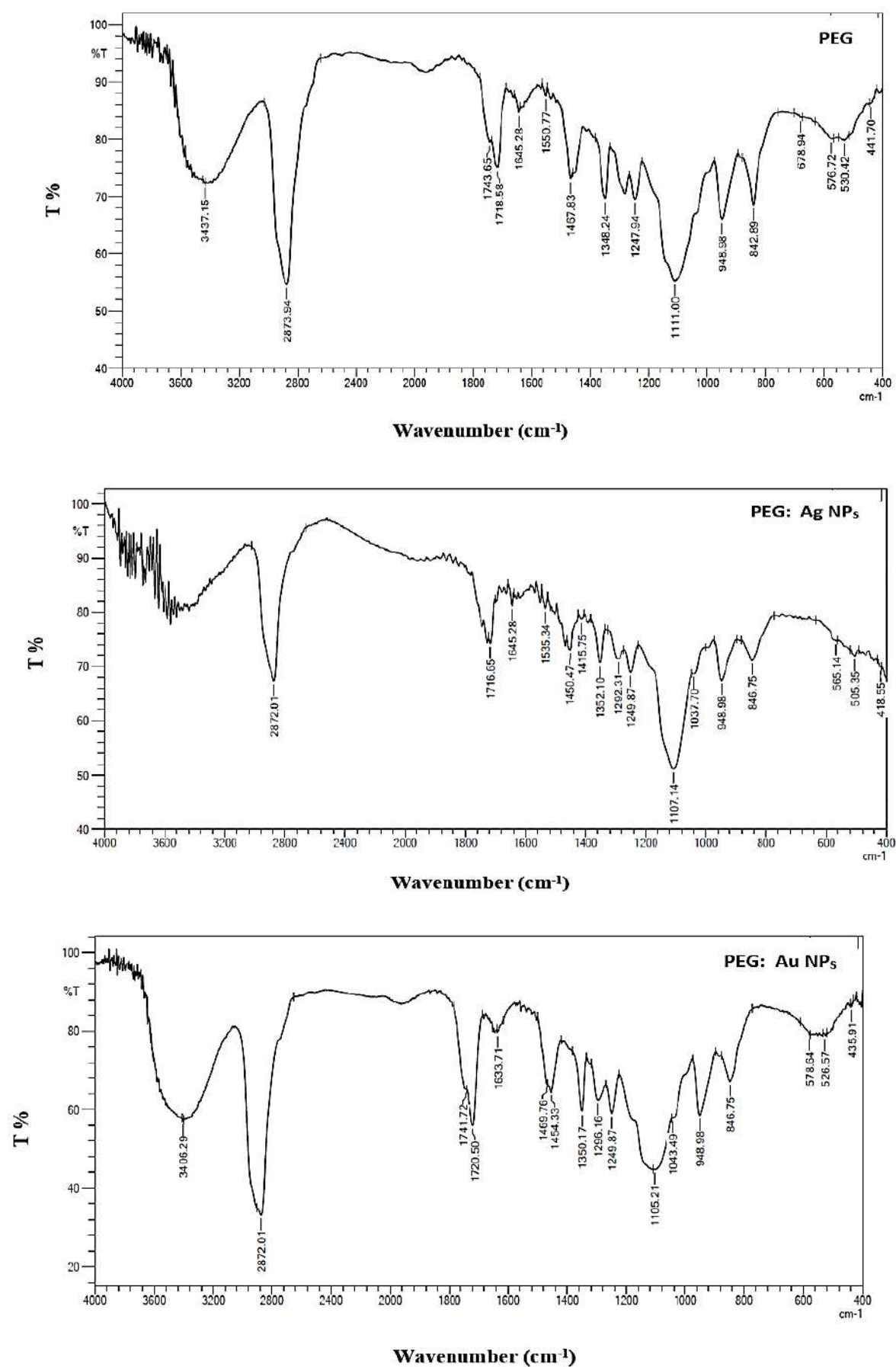


Fig. 1. FTIR spectra of (a) pure PEG, (b) PEG: Ag NP_s and (c) PEG: Au NP_s composites

Near 1111 cm^{-1} , the distinctive ether C–O–C stretching band was seen, along with –CH– out-of-plane bending vibrations at around 948.98 cm^{-1} . Characteristic spectral properties of PEG are supported by these bands. Possible changes in the chemical environment of PEG functional groups following nanoparticle production are indicated by a vibrational band at 1645.28 cm^{-1} in the PEG–AuNP spectra. In addition, the interactions between the metal nanoparticles and the oxygen-containing functional groups of the PEG chains could explain the peaks at 526.57 and 418.55 cm^{-1} . The results of this study provide credence to the idea that polyethylene glycol (PEG) could play a stabilizing and lowering role in the creation of nanoparticles through interactions with their surfaces. However, no clear proof of specific van der Waals interactions or chemical bonding mechanisms can be drawn from the FTIR results alone; further characterization techniques are needed FT-IR Surface Chemistry [17,18].

3.2 Optical Analyses

The UV–Vis spectra confirmed the formation of Ag NPs within the PEG polymeric matrix through the appearance of a distinct SPR band at 430 nm , which is characteristic of silver nanoparticles. Surface plasmon resonance arises from the collective oscillation of conduction electrons on the nanoparticle surface when interacting with incident light. The position and intensity of the SPR band depend on nanoparticle size, shape, distribution, and the surrounding dielectric environment. The PEG membrane itself exhibited low absorbance in the $100\text{--}1100\text{ nm}$ region [19], while increased absorption below 300 nm , with a maximum at 298 nm , is attributed to the intrinsic absorption of the polymer and trapped water molecules within the matrix. The emergence of the SPR peak at 430 nm therefore demonstrates the successful reduction of Ag^+ ions and stabilization of Ag nanoparticles in the PEG medium [20]. The UV-Vis spectrum of the gold nanoparticles also showed an absorption peak at 529 nm (Fig. 2.). This absorption peak at 529 nm is attributed to the SPR property of the gold nanoparticles [21]. Other researchers have observed a similar peak [23,24]. The chemical environment and preparation method play a crucial role in determining the size and shape of the nanoparticles [19].

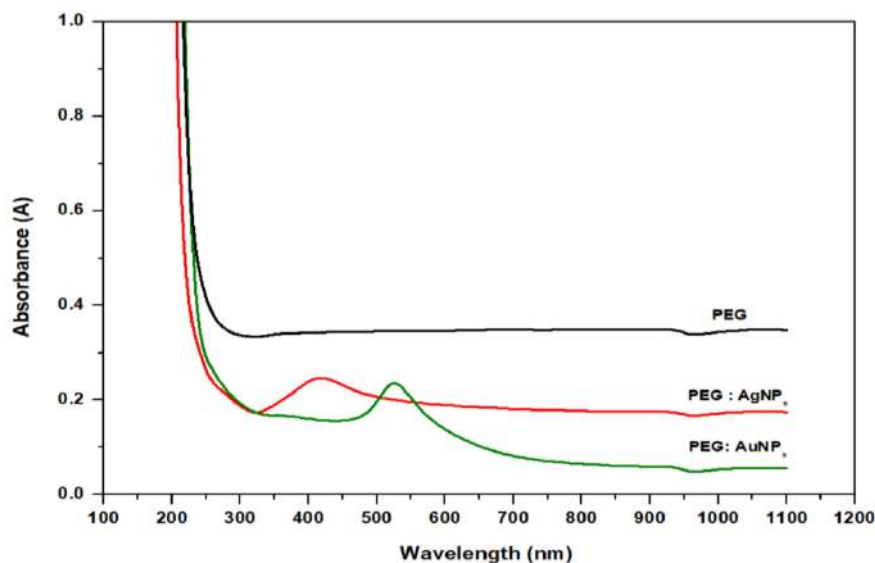


Fig. 2. UV-vis absorption spectra of PEG, PEG:AgNP_s and PEG:AuNP_s

The optical band gap energies of pure PEG and PEG films doped with Ag and Au nanoparticles were determined using the Tauc relation by extrapolating the linear portion of the $(\alpha h\nu)^{1/2}$ versus photon energy ($h\nu$) plots (Fig. 3). The intercept of this linear region with the energy axis (at $\alpha = 0$) corresponds to the optical band gap, indicating an indirect allowed electronic transition in the investigated films. This model was selected because the $(\alpha h\nu)^{1/2}$ versus ($h\nu$) plots exhibited better linearity compared to the $((\alpha h\nu)^{1/2})$ plot, which is characteristic of direct transition. This confirms that the indirect transition model provides a more accurate representation of the optical band gap for these specific nanocomposites. In indirect band gap materials, electronic transitions involve a change in momentum and therefore require phonon participation, which reduces the probability

of radiative recombination and suppresses direct photon emission. The obtained results show that the optical band gap of pure PEG (4.76 eV) decreases upon incorporation of nanoparticles, reaching 4.0 eV for Ag-doped films and 3.5 eV for Au-doped films. This reduction confirms that nanoparticle incorporation significantly modifies the electronic structure of the PEG matrix.

This behavior can be attributed to the formation of localized electronic states within the band structure, introduced by the presence of metallic nanoparticles. These states act as defect-like energy levels within the band gap, facilitating electronic transitions at lower energies and thereby reducing the effective optical band gap. Such modifications are consistent with previous reports indicating that inorganic nanoparticle fillers can strongly influence the optical and electronic properties of polymer matrices [24–26]. The observed decrease in band gap may also reflect increased structural disorder within the polymer network induced by nanoparticle dispersion [26,27].

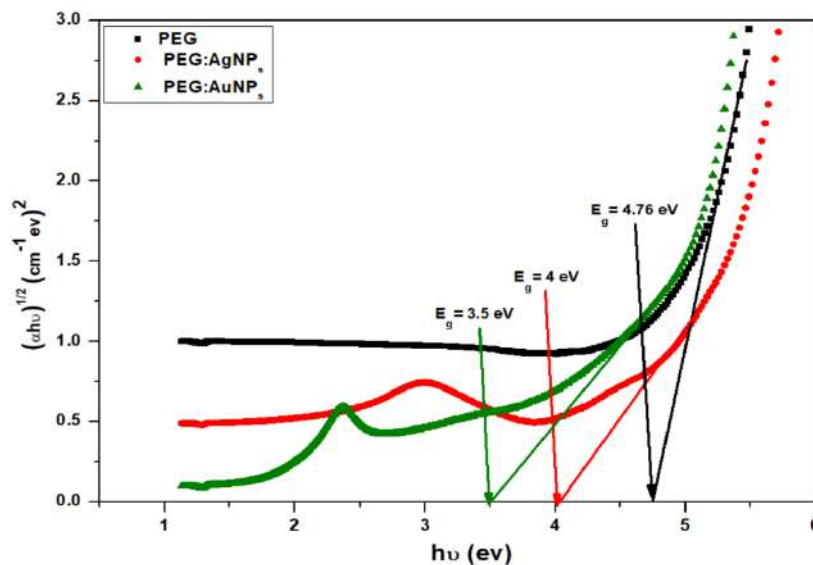
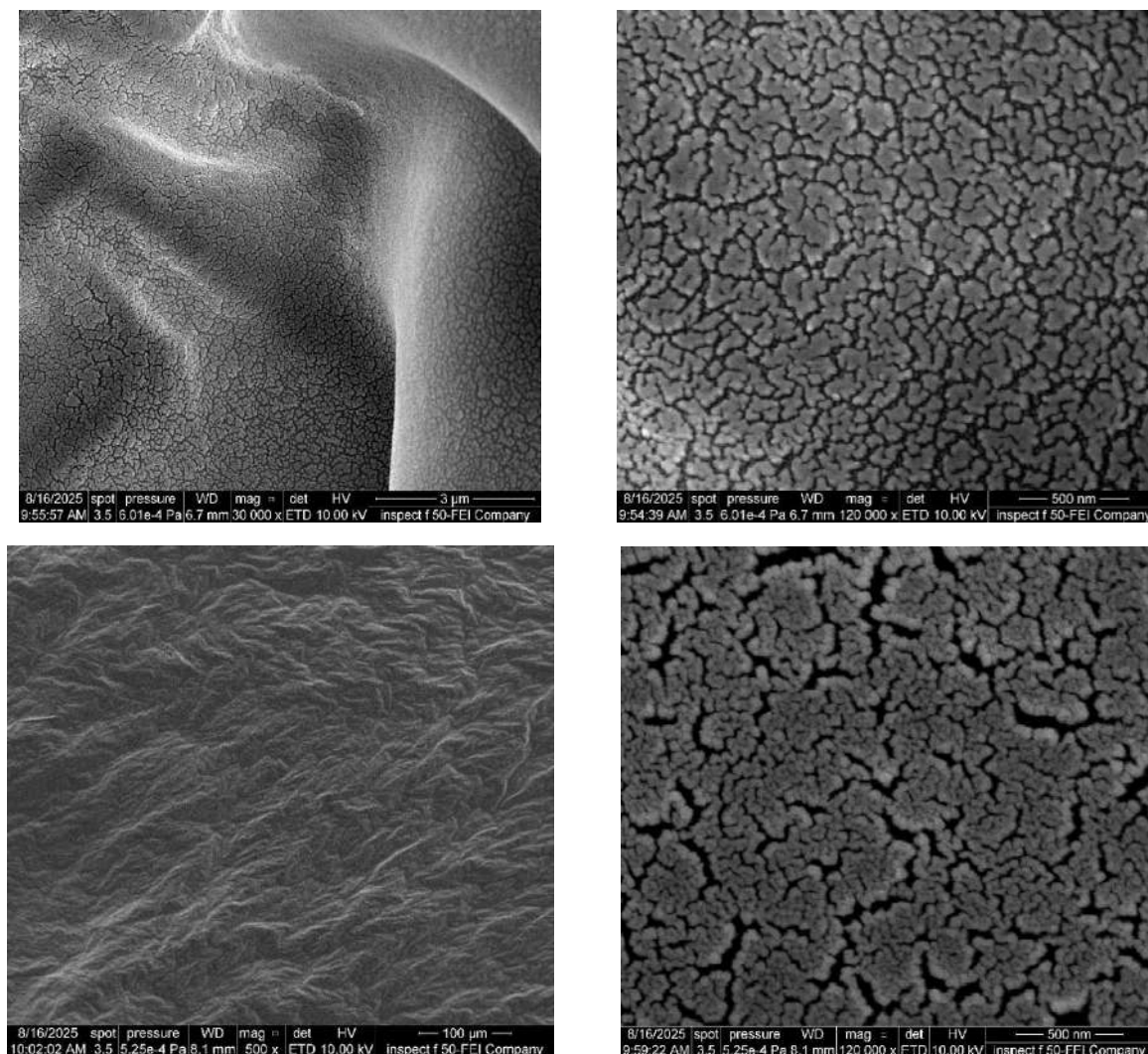


Fig. 3. $(\alpha h\nu)^{1/2}$ versus photon energy to illustrate Tauc method

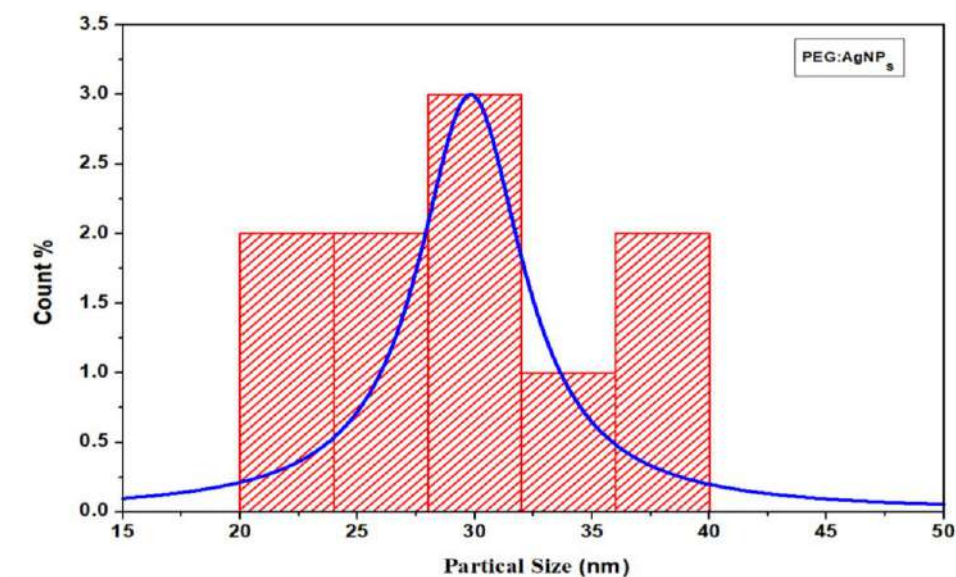
3.3 FESEM

Figures 4(a), 4(b) illustrate solution samples in which silver and gold nanoparticles are uniformly dispersed within a PEG-based polymer matrix. The PEG polymer is initially colorless, then gradually changes color to light yellow with the formation of silver nanoparticles and to light pink with the formation of gold nanoparticles. As the concentration and particle size increase over time, the solution becomes dark yellow in the presence of silver nanoparticles and dark red in the presence of gold nanoparticles, where the solutions were deposited on a glass slide of dimensions (1cm×1cm). Figures 4(a), and 4(b) show microscopic images illustrating the morphology and size distribution of the nanoparticles within the PEG-based polymer matrix. The particle size distribution was analyzed using ImageJ software. Furthermore, the surface of the biomaterial treated with silver and gold nanoparticles exhibits fewer pores compared to the untreated sample, indicating nanoparticle accumulation within the protrusion-like cavities of the surface structure. The FESEM micrographs illustrated in Figures 4(a) and 4(b) indicate the morphological modification induced by the silver and gold nanoparticle treatment. The surface morphology, particle distribution, and visual homogeneity suggest the effective role of the PEG matrix in stabilizing the nanoparticles. The surface exhibits a well-defined, network-like architecture with a highly homogeneous distribution of nanoparticles within the polymer matrix. The uniform dispersion of the particles, characterized by the absence of significant agglomeration, confirms the effective role of the PEG matrix in stabilizing the nanoparticles, the observed average particle size distribution demonstrates structural uniformity, as the nanoparticles are successfully accommodated within the surface cavities, leading to a notable reduction in surface porosity compared to the untreated control. produced surface appears more uniform, demonstrating the nearly total influence of the two chemical processes. Additionally, PEG was used to immobilize or

install the nanoparticles of silver and gold, which prevented them from aggregating. This resulted in a highly homogeneous layer of silver nanoparticles and gold nanoparticles on the surface of the polymer film [28,29].



(a)



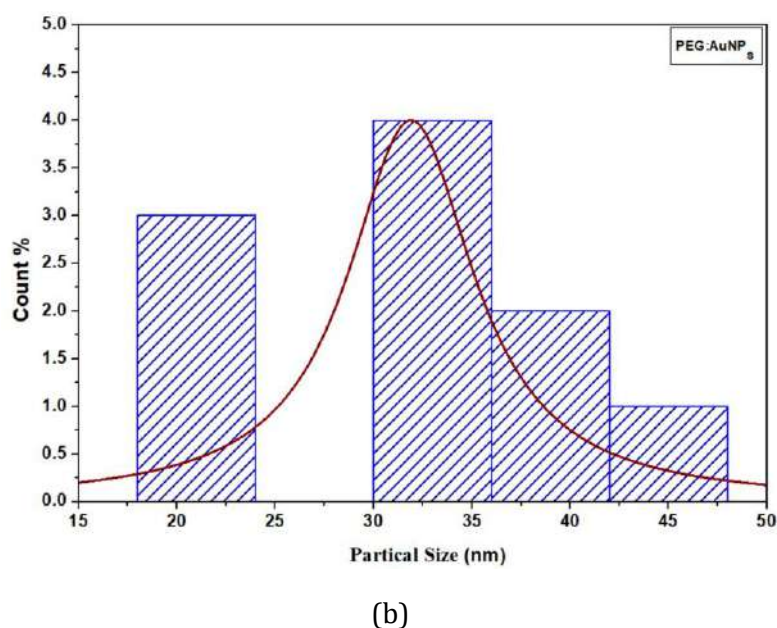


Fig. 4. (a) FESEM images and statistical distribution of PEG: AgNPs and (b)FESEM images and statistical distribution of PEG: AuNPs

3.4 Antimicrobial Activity Tests

Fig.5 illustrates the antibacterial activity of the fabricated PEG-based composite films against (*S. aureus*, *E. faecalis*, *E. coli* and *K. pneumoniae*). The inhibition zones given in Table (1) are average antimicrobial inhibition diameters under the prepared PEG/AgNPs and PEG/AuNPs films. Fig. (5), the photographs of the tested biomaterials after 48 h incubation time were shown in Fig.5. Across all panels, inhibition zones decreased progressively with concentration, indicating clear concentration-dependent activity. At 100, material PEG/AuNPs exhibited the largest zones against *S. aureus* (32 mm) and *S. faecalis* (30 mm), exceeding those of PEG: AgNPs (28 mm and 25 mm, respectively). Based on the observed inhibition zone diameters, the tested PEG-based composite films showed varying degrees of antimicrobial activity. Throughout the concentration range, the inhibition zones for PEG/AuNPs films were consistently larger than those observed for PEG/AgNPs films across the studied bacterial strains. Statistical analysis using two-way ANOVA confirmed that these differences are statistically significant ($p < 0.05$), supporting the enhanced antimicrobial efficacy of the PEG/AuNPs) formulation. These results suggest a concentration-dependent trend in the antimicrobial effectiveness of the prepared films, the statistical significance of which was confirmed by two-way ANOVA analysis. for instance, regarding *E. coli*, zones fell from 24 mm (PEG/AuNPs /100) to 20 mm (PEG/AuNPs /12.5) versus 23 → 15 mm for PEG: AgNPs; for *K. pneumoniae*, 26 → 19 mm (PEG/AuNPs) versus 25 → 21 mm (PEG: AgNPs). By organism, *S. aureus* was the most susceptible to both PEG: AgNPs and PEG/AuNPs across the concentration range, with PEG: AuNPs maintaining a ~2–4 mm advantage at each dilution and retaining a relatively large zone (20–23 mm) even at 12.5 $\mu\text{g/mL}$. *S. faecalis* showed lower zones overall than *S. aureus* but still displayed strong susceptibility to PEG: AuNPs (30 → 19 mm) compared with PEG: AgNPs (25 → 15 mm). Among Gram-negatives, *K. pneumoniae* appeared more susceptible than *E. coli* across agents and concentrations, *E. coli* exhibited the smallest zones within the PEG: AgNPs and PEG: AuNPs panels (e.g., PEG: AgNPs 23 → 15 mm, PEG: AuNPs 24 → 20 mm), consistent with relatively higher intrinsic resistance.

The antibacterial activity of PEG/Ag and PEG/Au formulations based on the mean inhibition zone diameter. All experiments were performed in three independent replicates and data are expressed as mean $\pm$ standard deviation (SD) in millimeters. As shown in table 1, the standard deviation for all samples was consistently less than 1.0 mm, indicating high experimental reproducibility. Statistical analysis confirmed that the differences across the tested concentrations are significant ($p < 0.05$).

Table 1. Antibacterial activity of PEG/Ag and PEG/Au formulations based on the mean inhibition zone diameter

Concentrations	<i>S. aureus</i>	<i>E. faecalis</i>	<i>E. coli</i>	<i>K. pneumoniae</i>
PEG/Ag / 100	28.00 ± 0.35	24.88 ± 0.29	23.28 ± 0.15	24.91 ± 0.48
PEG/Ag / 75	25.00 ± 0.17	23.29 ± 0.45	21.26 ± 0.31	24.12 ± 0.22
PEG/Ag / 50	23.09 ± 0.25	18.89 ± 0.37	18.97 ± 0.22	22.75 ± 0.39
PEG/Ag / 25	20.09 ± 0.35	16.83 ± 0.17	17.10 ± 0.34	21.97 ± 0.26
PEG/Ag / 12.5	17.83 ± 0.23	14.73 ± 0.24	15.29 ± 0.40	20.87 ± 0.31
PEG/Au / 100	31.89 ± 0.25	29.98 ± 0.30	23.81 ± 0.32	26.45 ± 0.46
PEG/Au / 75	28.19 ± 0.23	26.54 ± 0.26	22.56 ± 0.19	23.86 ± 0.50
PEG/Au / 50	25.51 ± 0.40	23.79 ± 0.27	22.17 ± 0.38	22.20 ± 0.26
PEG/Au / 25	23.01 ± 0.32	20.12 ± 0.19	21.18 ± 0.34	19.83 ± 0.36
PEG/Au / 12.5	19.98 ± 0.14	19.11 ± 0.20	19.78 ± 0.29	19.28 ± 0.39

Note: Values are presented as mean ± standard deviation of three replicates and are expressed in millimeters (mm).

Table 2. Two-way analysis of variance of the effects of nanoparticle type and concentration on inhibition zone diameter

Bacterial species	Source of variation	F (df)	p-value	Partial η^2
<i>Staphylococcus aureus</i>	Nanoparticle type (Ag vs. Au)	805.07 (1, 20)	<0.001	0.976
	Concentration	1400.06 (4, 20)	<0.001	0.996
	Nanoparticle type × concentration	8.78 (4, 20)	<0.001	0.637
<i>Enterococcus faecalis</i>	Nanoparticle type (Ag vs. Au)	1599.33 (1, 20)	<0.001	0.988
	Concentration	1397.56 (4, 20)	<0.001	0.996
	Nanoparticle type × concentration	14.02 (4, 20)	<0.001	0.737
<i>Escherichia coli</i>	Nanoparticle type (Ag vs. Au)	604.12 (1, 20)	<0.001	0.968
	Concentration	357.17 (4, 20)	<0.001	0.986
	Nanoparticle type × concentration	48.92 (4, 20)	<0.001	0.907
<i>Klebsiella pneumoniae</i>	Nanoparticle type (Ag vs. Au)	19.14 (1, 20)	<0.001	0.489
	Concentration	221.44 (4, 20)	<0.001	0.978
	Nanoparticle type × concentration	21.50 (4, 20)	<0.001	0.811

The two-way ANOVA demonstrated that the antibacterial activity of PEG/Ag and PEG/Au formulations was significantly influenced by nanoparticle type, concentration, and, in most cases, the interaction between these two factors. This indicates that the antibacterial response was not determined by the metallic nanoparticle alone, but also depended on the concentration at which each formulation was tested and on the susceptibility profile of the bacterial species.

In case of *Staphylococcus aureus*, both nanoparticle type and concentration exerted highly significant effects on inhibition zone diameter. The significant interaction term further indicates that the difference between PEG/Au and PEG/Ag changed across the tested concentrations. The PEG/Au showed greater antibacterial activity against *S. aureus*, particularly at higher concentrations. This may reflect a more effective interaction of gold-containing nanoparticles with the exposed peptidoglycan-rich cell envelope of this Gram-positive species, although the observed activity may also be influenced by particle size, surface charge, aggregation state, and dispersion within the PEG matrix.

For *Enterococcus faecalis*, the very large effect sizes for nanoparticle type and concentration suggest that both variables strongly contributed to differences in antibacterial activity. PEG/Au consistently produced larger inhibition zones than PEG/Ag, indicating a stronger activity against *E. faecalis* under the present experimental conditions. However, this interpretation should remain formulation-specific because nanoparticle activity is influenced by physicochemical characteristics, including particle morphology, surface functionalization, and the ability of the polymeric carrier to maintain nanoparticle dispersion.

Escherichia coli, the significant interaction between nanoparticle type and concentration was particularly notable. This result indicates that the relative activity of PEG/Au and PEG/Ag was not constant across all concentrations. PEG/Au appeared to retain comparatively stronger activity at lower and intermediate concentrations, whereas the difference between the two formulations was smaller at the highest concentration. Such variability may be related to the outer membrane barrier of Gram-negative bacteria, which can alter nanoparticle penetration and influence the diffusion of active components through the agar medium.

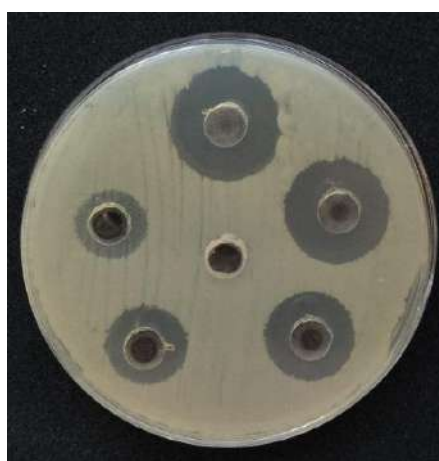
In *Klebsiella pneumoniae*, concentration remained a highly significant determinant of antibacterial activity. However, the interaction between nanoparticle type and concentration showed that the superiority of one formulation over the other was concentration-dependent. PEG/Au showed greater activity at the highest concentration, whereas PEG/Ag tended to maintain comparatively better activity at some lower concentrations. This complex response may be associated with the structural characteristics of *K. pneumoniae*, including its outer membrane, capsular material, and reduced permeability, all of which may influence the interaction between bacterial cells and metallic nanoparticles.



S. aureus



E. faecalis



E. coli



K. pneumoniae

(a)

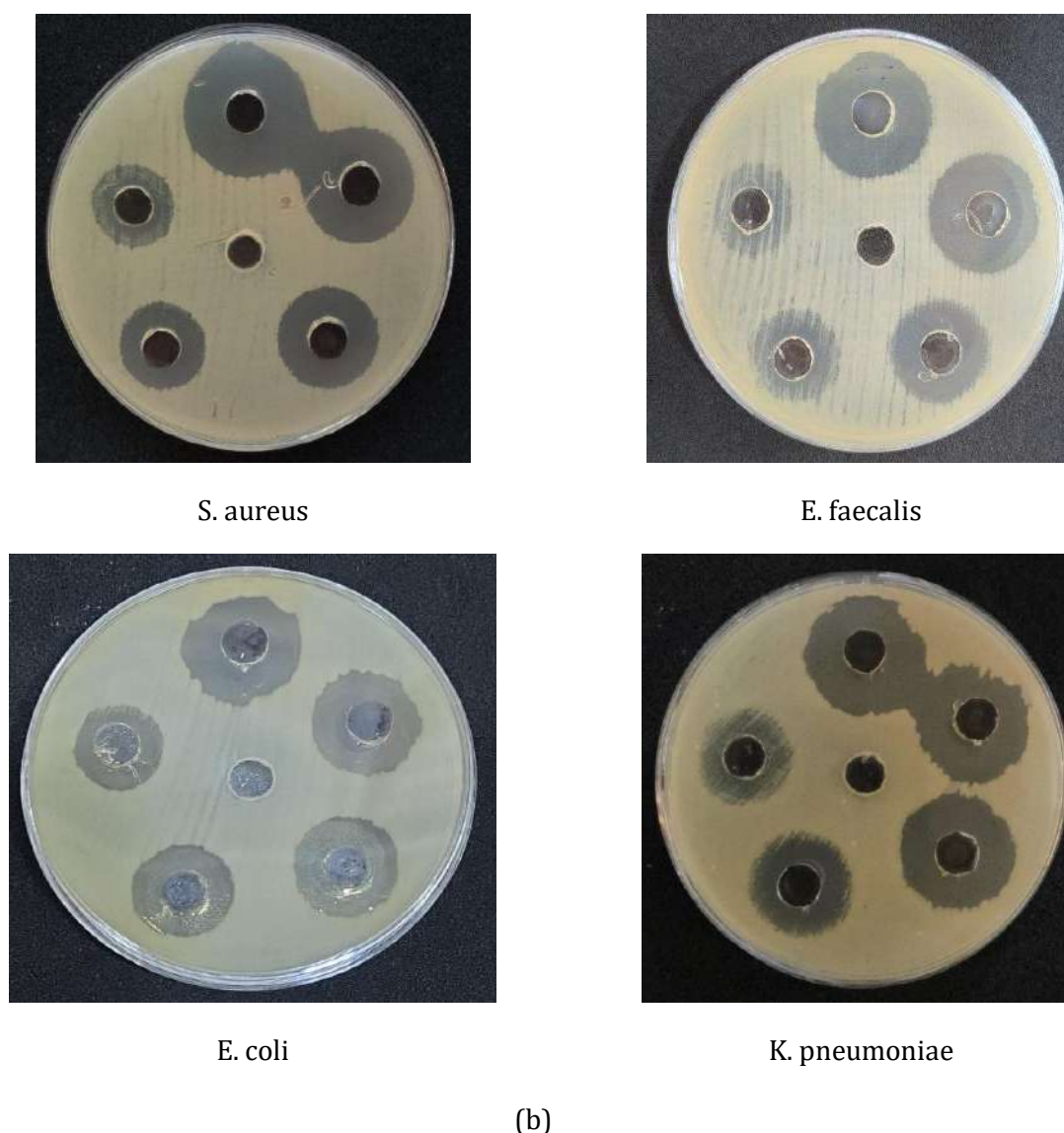


Fig. 5. (a) Antibacterial activity images of PEG: Ag NPs composite against (*S. aureus* , *E. faecalis* , *E.coli* and *K. pneumoniae*)and (b): Antibacterial activity images of PEG: Au NPs composite against (*S. aureus* , *E. faecalis*, *E.coli* and *K. pneumoniae*)

4. Conclusion

In this study, PEG/AgNPs and PEG/AuNPs nanocomposites were successfully synthesized. FESEM and spectroscopic analyses confirmed the uniform incorporation of nanoparticles within the PEG matrix. The antibacterial evaluation demonstrated that the inhibition zone diameters for the synthesized films ranged from 15 mm to 32 mm across the tested bacterial strains (*S. aureus*, *E. faecalis*, *E. coli*, and *K. pneumoniae*), with activity showing a clear positive correlation with nanoparticle concentration. While these results indicate promising antibacterial potential, the study is limited by its in vitro nature. Future research, specifically regarding cytotoxicity and long-term release kinetics, is necessary to transition these findings toward clinical application.

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Influence of petrochemical contamination on the geotechnical behavior, shear strength characteristics, and compressibility indices of sandy clay soil

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Abstract

One of the most important geo-environmental issues is the contaminated soils in the construction industry which are caused by the contamination of petrochemicals. In contrast with previous studies which evaluated individual soil properties separately, this study presents an integrated experimental evaluation of index properties, compaction behavior, shear strength, compressibility, permeability and microstructure of a samost of the existing studiedly clay soil (CL, USCS) contaminated with diesel fuel at 0%, 2%, 5% and 10% by dry weight, using standard ASTM procedures. The outcomes indicate that the liquid limit, plasticity index, MDD, OMC, cohesion and permeability decreased significantly with contamination while the compression index, settlement and failure strain increased. Carbonaceous coatings of soil particles and measurable loss of crystallinity of clay minerals were confirmed via SEM, EDS, FTIR and XRD analysis, which accounted for the observed decrease in soil strength and resulted in a change from brittle to ductile soil behavior. High regression correlations ($R^2 = 0.94\text{--}0.99$) were achieved between contamination level and all the measured parameters. The results offer site information for site assessment, foundation design, and remediation planning for petroleum-contaminated sites.

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1. Introduction

Petrochemical contamination is one of the most serious geo-environmental issues affecting the engineering performance of soils used in construction projects and infrastructure. Hydrocarbon presence (crude oil, diesel, gasoline, and petrochemical by-products) alters the physicochemical interactions between soil particles, causing substantial changes in strength, compressibility, permeability, and consistency. Past research has shown that hydrocarbon contamination adversely affects Atterberg limits, compaction characteristics, shear strength, and consolidation parameters of fine-grained and sandy soils [1]. These contaminations are commonly caused by oil spillage, pipeline and storage-tank leakage, refinery operations, and improper industrial waste disposal, distorting the inherent characteristics of soils and presenting serious environmental and geotechnical concerns [1,2]. Petroleum hydrocarbons are non-polar organic compounds that behave very differently from water, the natural pore fluid, in fine-grained soils such as sandy clay, where inter-particle bonding and water adsorption play a crucial role. This results in reduced bearing capacity, higher settlement, and unstable behavior under loading [3,4]. Reported effects include a lower liquid limit and plasticity index due to reduced particle affinity to hydrocarbons [1], and reductions in Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) caused by the lower specific gravity of hydrocarbons and their coating effect on particles [5]. Hydrocarbons also affect shear strength: the electrochemical bonding and capillary forces

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responsible for cohesion are destabilized when pore water is displaced by hydrocarbons, decreasing cohesion, while the angle of internal friction changes only slightly [6,4]. Compressibility and consolidation are also affected — contaminated soils are more compressible due to weakened inter-particle bonds, and hydrocarbons reduce permeability by blocking pore spaces, slowing consolidation [7]. These mechanisms have been studied in detail in previous research. When the soil is fine-grained, the addition of petrochemicals will reduce the Atterberg limits, maximum dry density, and shear strength parameters because the oil coats the soil particles and the bonding among particles decreases [10,11,12]. Contaminated soils exhibit increased compressibility, reduced consolidation efficiency [10,11,12] and a significant reduction in cohesion with only minor changes in friction angle [13,14] and changes in the microstructural nature of the clay that result in ductile deformation and increased settlement potential [15]. Although these are numerous now, there is no consistent definition of intermediate soils that exhibit both cohesive and frictional behavior (e.g., sandy clay), and are more complex to characterize than clean sands or highly plastic clays [1]. Furthermore, most of the existing studies analyze individual properties in isolation and not as a combination of all the properties such as index, compaction, strength, compressibility, permeability and microstructural properties which are important to establish robust relationships among the parameters [8]. The central novelty of this work is the combination, at the same time, of macro-scale mechanical tests and microstructural tests (SEM, EDS, FTIR, XRD), followed by a linkage between the observed changes in strength and compressibility and the changes that happen at the particle level, which is rarely reported for sandy clay soils in the existing literature. This is important because contaminated soils in industrial and urban areas may exhibit reduced strength, increased compressibility, excessive settlement, and instability under structural loads, leading to foundation problems, pavement deterioration, and environmental hazards. The findings will help engineers assess whether a contaminated site is suitable for construction and identify remediation needs prior to development, and will contribute a validated dataset for future predictive modelling of hydrocarbon-polluted soils. To achieve the above, specific objectives of this study are:

- To characterize the effects of contamination on the Atterberg limits and soil consistency of soil; (ii) to assess changes in the compaction characteristics;
- Quantify changes in shear strength, stress–strain, and softening behavior;
- Evaluate compressibility, time-settlement, and permeability; and
- Explain the observed macro-scale changes through microstructural and mineralogical investigation (SEM, EDS, FTIR, XRD) of soil artificially contaminated at 0%, 2%, 5%, and 10% by dry mass with diesel fuel to represent field conditions.

2. Literature Review

The presence of petrochemical in fine-grained soils has a great influence on their physicochemical and mechanical characteristics, which in turn influence the engineering behavior, once more. With an increase in the concentration of hydrocarbon, lowering in Atterberg limits, maximum dry density and shear strength parameters were reported by several researchers as a result of the coating effect of the oil on soil particles and the decrease in interparticle bonding. [10], [11] and [12] found that contaminated soils have higher compressibility and lower consolidation effectiveness. [13] and [14] also reported a considerable decrease in cohesion with minor shear angle changes, resulting in overall shear strength losses. Furthermore, [15] found that the major microstructural changes in clay matrices resulting from hydrocarbon intrusion led to ductile deformation behavior and higher settlement potential of contaminated soils.

3. Research Significance

This study is important because it helps in gaining knowledge of the negative impact of petrochemical contamination on the engineering properties of sandy clay soils frequently found in industrial and urban areas. The problem of soil contamination from petroleum products is a challenging task for geotechnical engineers as contaminated soils may have decreased strength, increased compressibility, excessive settlement and instability under structural loads. These changes can result in foundation problems, pavement deterioration and environmental hazards.

In this study, Atterberg limits, compaction behavior, shear strength parameters and consolidation characteristics of sandy clay soil have been comprehensively evaluated in the laboratory, with varying levels of petrochemical contamination. The results will help engineers determine if a contaminated site would be suitable for construction projects, as well as what types of remediation and stabilization might be necessary prior to the construction before the site is developed. Moreover, the research will help in the development of sustainable geo-environmental engineering practices by enhancing the understanding of contaminant-soil interaction mechanisms and to the foundation design in contaminated area to ensure safe foundation. The results can also be used as a useful data set for future predictive modelling and remediation studies of hydrocarbon polluted soils.

4. Methodology

4.1 Soil Sampling and Characterization

The soil in this study was a sandy clay that was sampled 1.0–1.5 m below the natural ground surface and was free from organic surface material, dried, crushed and sieved through a 4.75 mm sieve. The natural (uncontaminated) soil was completely index characterized according to ASTM standards, and the results are reported in Table 1.

Table 1. The natural sandy clay soil was measured for physical and index properties

Property	Value	Unit
Sand	48	%
Silt	22	%
Clay	30	%
Specific Gravity, G_s	2.65	–
Natural Moisture Content, ω_n	12.5	%
Organic Content, OM	1.8	%
Liquid Limit, LL	42	%
Plastic Limit, PL	22	%
Plasticity Index, PI	20	%
D10	0.002	mm
D30	0.025	mm
D60	0.15	mm
Coefficient of Uniformity, C_u	75	–
Coefficient of Curvature, $C_c(g)$	2.08	–
Property	Value	Unit
USCS Classification	CL	–

The above properties indicated that the soil was of low plasticity (CL) clay type in accordance with the Unified Soil Classification System (USCS), having a significant amount of sand that was consistent with its description of being sandy clay. The natural properties of the soil were maintained by keeping the treated soil in tight containers before testing.

4.2 Petrochemical Contaminant Properties

The contaminant was chosen to represent contamination found in the field, as a result of fuel storage and transport leakages. Table 2 shows the summary of its physical and chemical properties.

The contaminant was applied to soil samples at 2%, 5% and 10% by dry weight with even distribution into the soil matrix. Samples that were contaminated were placed in sealed containers and allowed to equilibrate for 24–72 hours to ensure that all soil particles had had sufficient interaction with the contaminant. [9]

Table 2. The physical and chemical properties of the diesel fuel contaminant

Property	Value	Unit
Density at 15 °C	850	kg/m ³
Kinematic viscosity at 40 °C	3.5	cSt
Dynamic viscosity at 40 °C	2.98	mPa·s
Flash point	≥ 60	°C
Pour point	≤ 0 (summer) / ≤ -6 (winter)	°C
Cetane number	≥ 40	–
Carbon range	C9–C28	–
Chemical composition	Paraffins (alkanes), naphthene's (cycloalkanes), aromatic hydrocarbons	–

4.3 Experimental Program

4.3.1 Index Tests

The particle size composition was found using sieve analysis and hydrometric tests (ASTM D422). Soil consistencies were determined for each level of contamination (ASTM D4318) by using the liquid limit, plastic limit, and plasticity index.

4.3.2 Compaction Tests

Multiple trial moisture contents (MTMC) were used to develop compaction curves to determine MDD and OMC at each contamination level with standard Proctor compaction tests (ASTM D698).

4.3.3 Shear Strength Tests

The samples were prepared at the corresponding MDD and OMC, and then direct shear tests were performed at normal stresses of 100, 200 and 300 kPa, according to ASTM D3080. The shearing stresses at failure were used to determine cohesion (c) and the angle of internal friction (ϕ) (Section 3.4), and the overall shearing stress–shearing strain response was recorded at each normal stress level. Triaxial testing was not an experimental component of this study, so it is not included in the methodology, all of the stress–strain characterization presented in this study is based solely on the results of the direct shear test.

4.3.4 Compressibility and Consolidation Tests

One-dimensional consolidation (oedometer) tests (ASTM D2435) were carried out under incremental loading condition. The void ratio with applied pressure was measured to get the compression index (C_c) and recompression index (C_r). Time–settlement readings were recorded at 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, and 1440 minutes to determine the rate and progression of consolidation settlement (Section 3.7).

4.3.5 Permeability Tests

Each contamination level was subjected to falling-head permeability tests (ASTM D5084) to find the coefficient of permeability (k) and the coefficient of consolidation (C_v) was determined from the oedometer test data using the square-root-of-time method.

4.3.6 Microstructural and Mineralogical Investigation

The natural and contaminated soil samples were investigated with respect to the processes that result in the macro-level changes using:

- Scanning Electron Microscopy (SEM) to compare surface morphology of particles and hydrocarbon coating;
- Electron Back-Projection (EBP) to determine 3D geometric shapes; and

- Fourier-Transform InfraRed (FTIR) to detect the presence of hydrocarbon functional groups; and
- X-ray Diffraction (XRD) to determine changes in crystallinity of the clay minerals.

4.4 Testing Conditions

To ensure consistency, all tests have been conducted under controlled laboratory conditions of temperature and humidity to maintain uniformity and uniformity. Samples were contaminated with 0% (control), 2%, 5% and 10% by dry weight. Each test was repeated three times for each contamination level and the results are presented as mean $\pm$ standard deviation, the data reliability and repeatability (CoV) were calculated (Section 3.9). When necessary, samples were sealed to minimize loss of the petrochemical due to evaporation during testing.

5. Results

5.1 Index Properties

The index properties of the sandy clay soil were greatly affected by the petrochemical contamination. In general, both liquid limit (LL) and plastic limit (PL) decreased with the increase of contamination (Table 3, Fig. 1).

Table 3. A study of how the index properties is affected by petrochemical contamination

Contamination (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	42	22	20
2	39	21	18
5	35	20	15
10	31	19	12

In the case of the untampered soil, liquid limit was comparatively greater which means that clay particles are highly water affined. Yet, at contamination of 2%, 5% and 10%, a significant decrease in LL was observed. In the same way, the plastic limit was also reduced, but at a little lesser pace. Therefore, there was a steady decrease in plasticity index (PI), which means that there was a decrease in plastic behavior of the soil. Figure 1 indicates that all the three parameters (LL, PL and PI) decrease with the increase in contamination, which proves that the effect of petrochemicals on the soil consistency characteristics is negative.

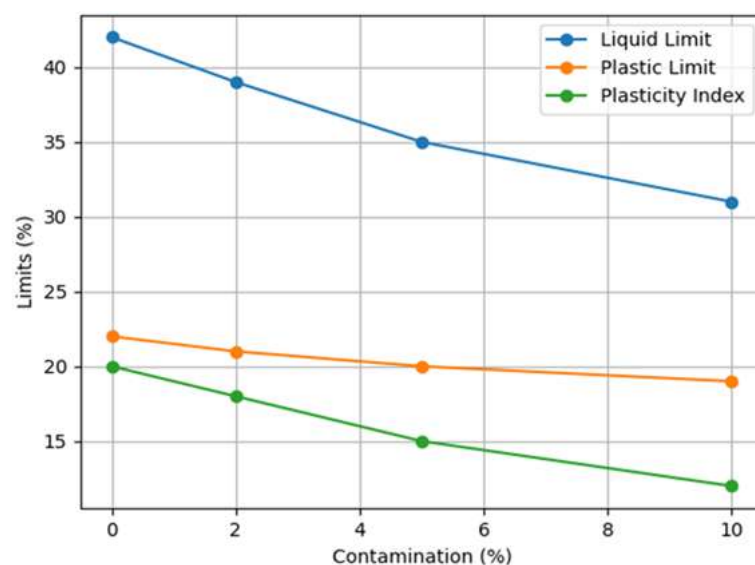


Fig. 1. The liquid limit, plastic limit and plasticity index are plotted as a function of contamination level (%)

The un-contaminated soil had the highest liquid limit which suggests that the clay particles have high affinity for water. The plasticity index (PI) was lowered significantly at 2%, 5% and 10% contamination and decreased at a slightly lower rate at PL contamination, leading to a gradual decrease in the plasticity index. This confirms that contamination leads to a progressive plastic behavior loss.

5.2 Compaction Characteristics

The compaction properties at each contamination level are summarized in Table 4 and Figure 2, and it is evident that both MDD and OMC decrease with contamination. The uncontaminated soil yielded the maximum MDD, indicating the best packing of the soil. The specific gravity of the diesel fuel and the ability of the fuel to disrupt particle rearrangement caused MDD to progressively decrease with contamination, most significantly at 10%. OMC also decreased as the degree of contamination was raised; this suggests that less water was required to obtain maximum compaction of the sediment as more hydrocarbons were introduced to the pore fluid, displacing it.

Table 4. The compaction characteristics of contaminated soil refers to the process of compressing the soil to expel air from between its particles

Contamination (%)	MDD (kN/m ³)	OMC (%)
0	18.5	14.5
2	18.0	13.8
5	17.3	12.9
10	16.5	11.8

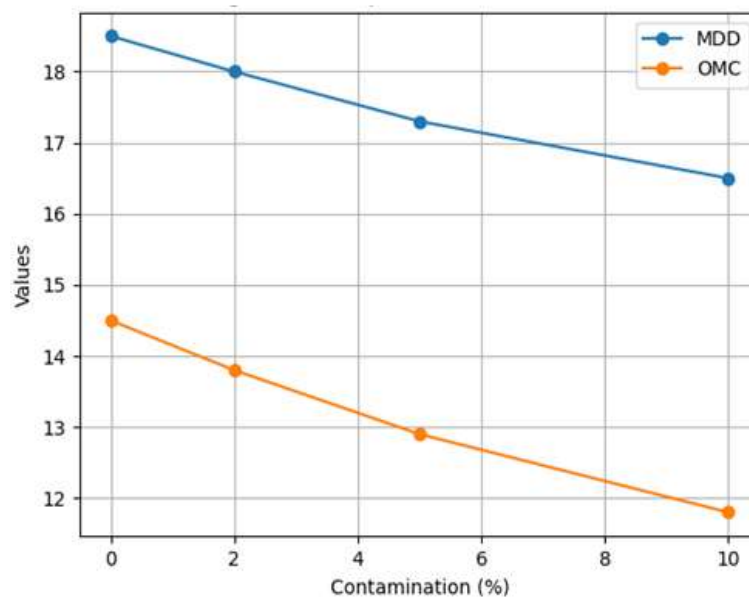


Fig. 2. The compaction curves (MDD and OMC) for the different contaminant levels are shown

5.3 Shear Strength Parameters

The results of the direct shear tests are presented in Table 5 and Fig. 3 and indicate that: 1) there is a significant decrease in cohesion (c) with contamination level, 2) the friction angle (ϕ) changes only slightly with the contamination level.

The natural soil with zero amount of contamination showed the highest value of cohesion indicating a great inter-particle bonding. Nevertheless, with contamination reaching 2%, 5% and 10%, the cohesion reduced successively. This decline is well illustrated in Figure 3 in which cohesion declines sharply.

Table 5. The shear strength and compressibility parameters of soil

Contamination (%)	Cohesion, c (kPa)	Friction Angle, ϕ (°)	Compression Index (Cc)	Recompression Index (Cr)
0	32	24	0.21	0.04
2	28	25	0.24	0.04
5	23	26	0.28	0.05
10	18	25	0.33	0.05

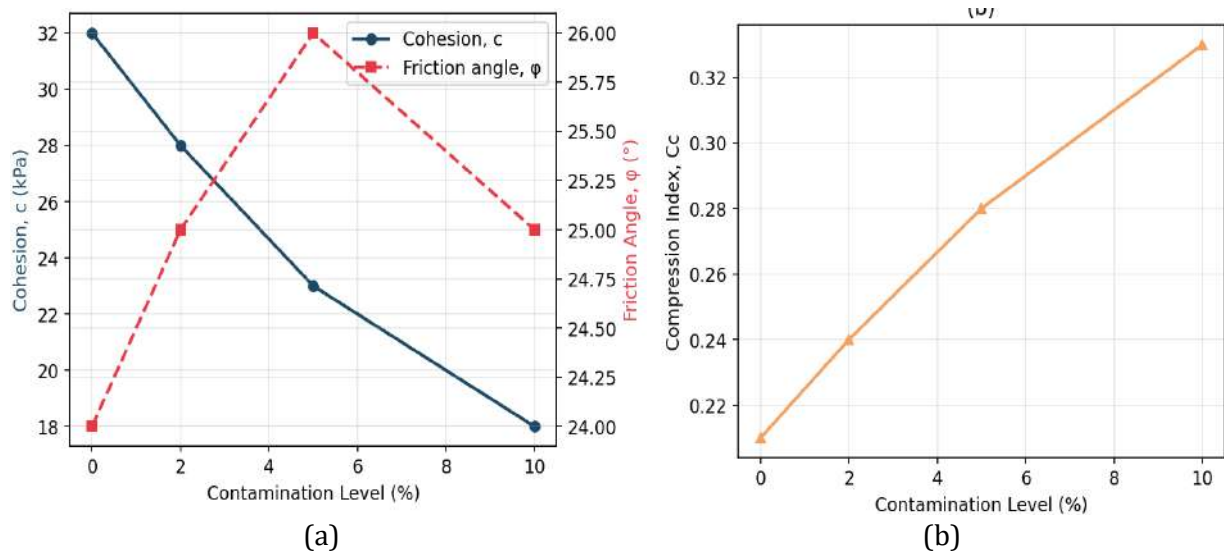


Fig. 3. The effects of contamination on cohesion, friction angle and compression index were investigated

The cohesion of the uncontaminated soil was greatest, indicating that there was strong inter-particle bonding, and that this bonding decreased progressively with contamination. The friction angle changed slightly, increasing slightly at low contaminations then becoming stable, suggesting that the overall decrease in shear strength is controlled mainly by the reduction of cohesion.

5.4 Stress-Strain Behavior

Table 6. Shear stress–strain response at normal stress of 100kPa

Shear Stress, τ (kPa)	Strain, 0% (%)	Strain, 2% (%)	Strain, 5% (%)	Strain, 10% (%)
0	0	0	0	0
5	0.20	0.25	0.30	0.38
10	0.45	0.55	0.65	0.82
15	0.80	0.95	1.15	1.45
20	1.30	1.50	1.80	2.25
25	1.95	2.20	2.60	3.20
30	2.80	3.10	3.60	4.50
32 (peak)	3.50	–	–	–
28 (post-peak)	–	4.20	–	–
23 (post-peak)	–	–	5.50	–
18 (post-peak)	–	–	–	7.00

Table 7. Shear stress–strain response at normal stress of 200 kPa

Shear Stress, τ (kPa)	Strain, 0% (%)	Strain, 2% (%)	Strain, 5% (%)	Strain, 10% (%)
0	0	0	0	0
10	0.30	0.35	0.42	0.55
20	0.65	0.78	0.95	1.20
30	1.10	1.30	1.55	1.95
40	1.70	1.95	2.30	2.85
50	2.40	2.70	3.20	3.90
55 (peak)	3.20	–	–	–
50 (post-peak)	–	3.80	–	–
42 (post-peak)	–	–	4.80	–
Shear Stress, τ (kPa)	Strain, 0% (%)	Strain, 2% (%)	Strain, 5% (%)	Strain, 10% (%)
35 (post-peak)	–	–	–	6.20

Table 8. Shear stress–strain response at normal stress of 300 kPa

Shear Stress, τ (kPa)	Strain, 0% (%)	Strain, 2% (%)	Strain, 5% (%)	Strain, 10% (%)
0	0	0	0	0
15	0.35	0.42	0.50	0.65
30	0.80	0.95	1.15	1.45
45	1.35	1.60	1.90	2.35
60	2.05	2.35	2.80	3.40
75	2.90	3.30	3.85	4.60
82 (peak)	3.80	–	–	–
75 (post-peak)	–	4.50	–	–
65 (post-peak)	–	–	5.60	–
55 (post-peak)	–	–	–	7.50

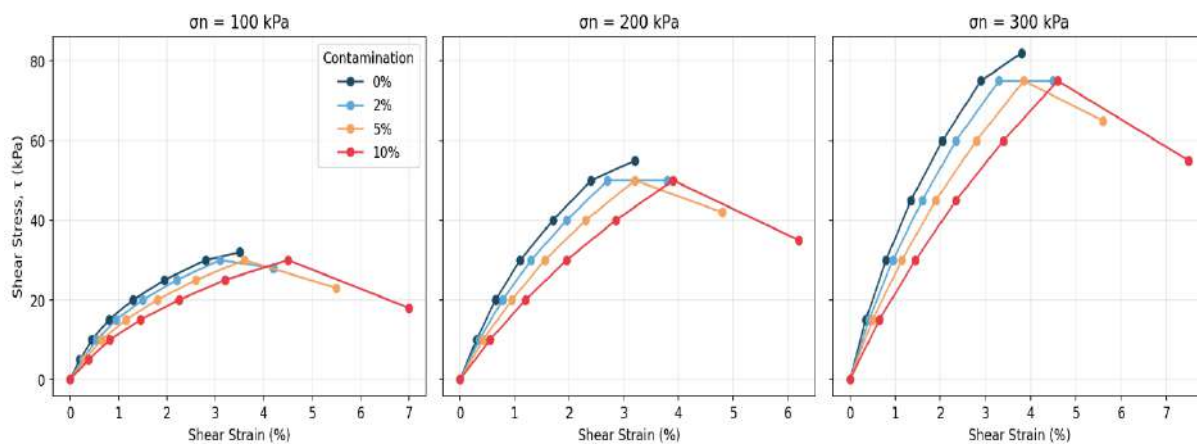


Fig. 4. The shear stress–strain curves are shown for various normal stresses of 100, 200, and 300 kPa, at different contamination levels

In the uncontaminated soil, the peak stress was sharp and well defined with a rapid drop of stress after the peak — typical brittle failure. The increase in contamination resulted in a corresponding decrease in peak stress, a sharp increase in the amount of failure strain (from 3.5% at 0% to 7.0–7.5% at 10%) and a gradual decrease in post-peak stress, as shown with increasing hydrocarbon content, from brittle to ductile failure behavior.

5.5 Softening Behavior

Table 9. The general process of softening parameters of contaminated soil

Contamination (%)	Initial Stiffness (kPa)	Post-Peak Stiffness (kPa)	Softening Ratio (%)	Strain at Failure (%)
0	18.5	8.5	54.1	3.5
2	14.0	7.5	46.4	4.2
5	10.2	6.2	39.2	5.5
10	7.0	4.8	31.4	7.0

The initial stiffness dropped by about 62 % from 0 % to 10 % contaminations, and the softening ratio dropped from 54.1 % to 31.4 % from 0 % to 10 % contaminations. This decrease and the increase in failure strain provides a quantitative confirmation of the brittle-ductile transition of Section 3.4.

5.6 Compressibility Characteristics

The compression index (C_c) increased with contamination and was relatively high for the contaminated specimens ranging from 0.21 (0%) to 0.33 (10%) as shown in Table 5, which represents the specimens being most susceptible to settlement. The recompression index (C_r) increased slightly (0.04 to 0.05), which suggests the soil structure being altered during virgin loading is relatively less altered during the unloading–reloading cycles.

5.7 Time-Settlement and Consolidation Behavior

The Final settlement (FS) (Table 10) increased from 2.80 mm at 0% contamination to 5.25 mm at 10% contamination, corresponding to an increase of 87.5%, which again shows the trend of increasing compressibility with increasing contamination.

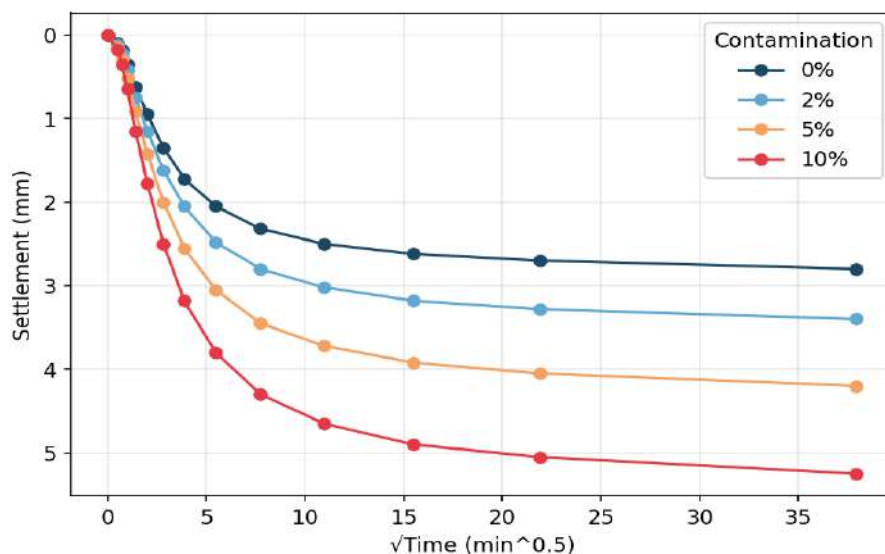


Fig. 5. The Time-Settlement Curves ($\sqrt{t}$ method) at Different Contamination Levels are shown below

Table 10. The Incremental oedometer loading is a method in which time-settlement data are collected

Time (min)	$\sqrt{t}$ (min ^{0.5})	Settlement, 0% (mm)	Settlement, 2% (mm)	Settlement, 5% (mm)	Settlement, 10% (mm)
0	0	0	0	0	0
0.25	0.50	0.10	0.12	0.15	0.18
0.5	0.71	0.18	0.22	0.28	0.35
1	1.00	0.35	0.42	0.52	0.65

Time (min)	$\sqrt{t}$ (min ^{0.5})	Settlement, 0% (mm)	Settlement, 2% (mm)	Settlement, 5% (mm)	Settlement, 10% (mm)
2	1.41	0.62	0.75	0.92	1.15
4	2.00	0.95	1.15	1.42	1.78
8	2.83	1.35	1.62	2.00	2.50
15	3.87	1.72	2.05	2.55	3.18
30	5.48	2.05	2.48	3.05	3.80
60	7.75	2.32	2.80	3.45	4.30
120	10.95	2.50	3.02	3.72	4.65
240	15.49	2.62	3.18	3.92	4.90
480	21.91	2.70	3.28	4.05	5.05
1440	37.95	2.80	3.40	4.20	5.25

5.8 Permeability

As expected, permeability values in the contaminated range (0% to 10%) decreased, by about 74%, as hydrocarbon coating gradually sealed the pores, and prevented flow through the soil, accounting for the slower, more prolonged settlement observed in Section 3.7.

Table 11. The coefficient of permeability and coefficient of consolidation are both used to describe the permeability of a soil

Contamination (%)	Permeability, k (cm/s)	Coefficient of Consolidation, Cv (cm ² /s)
0	8.5×10^{-6}	1.85×10^{-2}
2	6.2×10^{-6}	1.52×10^{-2}
5	4.1×10^{-6}	1.10×10^{-2}
10	2.2×10^{-6}	0.75×10^{-2}

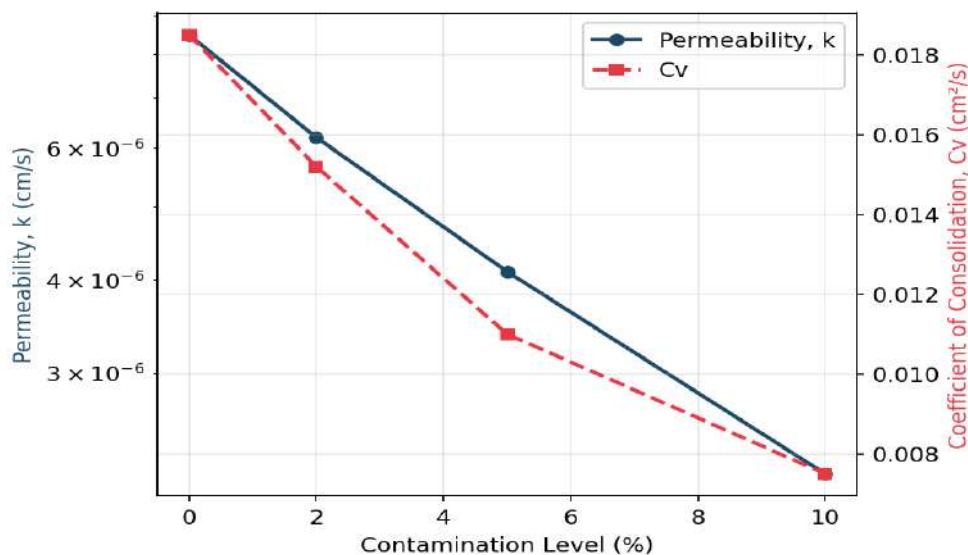


Fig. 6. The Permeability (k) and Coefficient of Consolidation (Cv) of a soil sample vary with the level of contamination

5.9 Statistical Analysis

The coefficient of variation rose steadily with contamination for each of the three parameters, particularly for cohesion (from 5.6% to 15.6%), suggesting that increasing the amount of contamination increases the mean degradation of soil behavior as well as the variation in soil behavior.

Table 12. All values shown are mean $\pm$ SD (n = 3) values

Contamination (%)	Cohesion, c (kPa)	CoV (%)	Friction Angle, ϕ (°)	CoV (%)	MDD (kg/m ³)	CoV (%)
0	32.0 $\pm$ 1.8	5.6	24.0 $\pm$ 1.2	5.0	1820 $\pm$ 25	1.4
2	28.0 $\pm$ 2.1	7.5	25.0 $\pm$ 1.3	5.2	1800 $\pm$ 30	1.7
5	23.0 $\pm$ 2.5	10.9	26.0 $\pm$ 1.5	5.8	1760 $\pm$ 35	2.0
10	18.0 $\pm$ 2.8	15.6	25.0 $\pm$ 1.8	7.2	1680 $\pm$ 40	2.4

5.10 Microstructural and Mineralogical Investigation

5.10.1 SEM Analysis. The Qualitative Morphological Observations are Summarized in Table 13.

Table 13. SEM morphological observations

Sample	Description
Natural soil (0%)	Clean particle surfaces, sharp edges, densely packed grain arrangement, low porosity, visible clay mineral crystals
2% contamination	Thin hydrocarbon film coating particle surfaces, moderate porosity, initial particle aggregation
5% contamination	Near-complete hydrocarbon film coverage, increased porosity, altered grain shape, loss of sharp edges
10% contamination	Complete particle coating, high porosity, large particle aggregates, loss of original soil fabric

5.10.2 EDS Analysis.

The changes in elemental composition were found to be significant and the results are shown in Table 14 and in Fig. 7: the carbon content increased from 2.1% (0%) to 25.5% (10%) and the oxygen, silicon and aluminum contents (main components of the clay minerals) decreased in parallel with progressive contamination, confirming progressive hydrocarbon coverage of the mineral surfaces. [26]

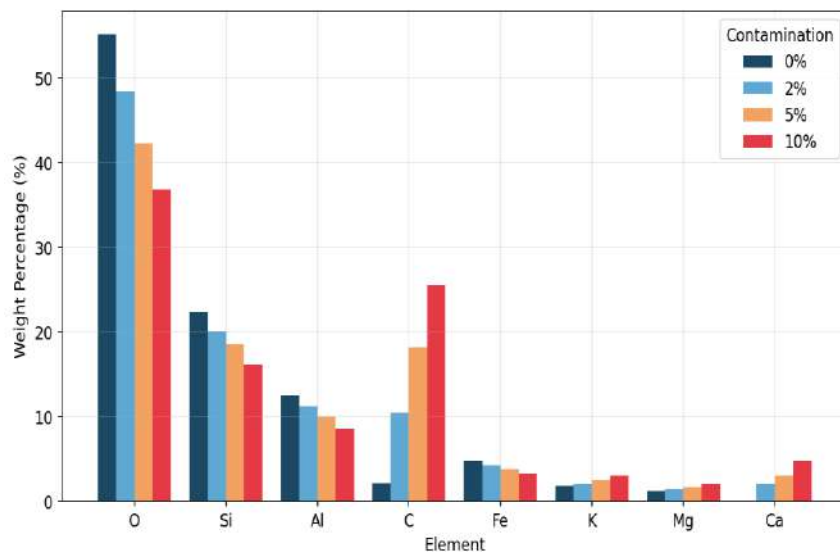


Fig. 7. The elemental composition of soil samples taken from nature and contaminated areas was analyzed using EDS

Table 14. EDS elemental composition (%)

Element	0%	2%	5%	10%
O	55.2	48.5	42.3	36.8
Si	22.4	20.1	18.5	16.2
Al	12.5	11.2	10.0	8.5
C	2.1	10.5	18.2	25.5
Fe	4.8	4.2	3.8	3.2
K	1.8	2.0	2.5	3.0
Mg	1.2	1.5	1.7	2.0
Ca	0.0	2.0	3.0	4.8

5.10.3 FTIR Analysis

The 10% contaminated sample (Table 15) exhibited new bands at 1605 cm⁻¹ (C=C stretching vibrations of the aromatic groups) and 2955, 2925 and 2855 cm⁻¹ (C-H stretching vibrations of aliphatic hydrocarbons) which were not observed in the natural soil sample. Bands corresponding to the clay mineral O-H (3620 cm⁻¹), Si-O-Si (1000–1100 cm⁻¹) and adsorbed water (1630 cm⁻¹) were used to confirm the presence of the clay mineral structure but did not allow for the progressive masking observed.

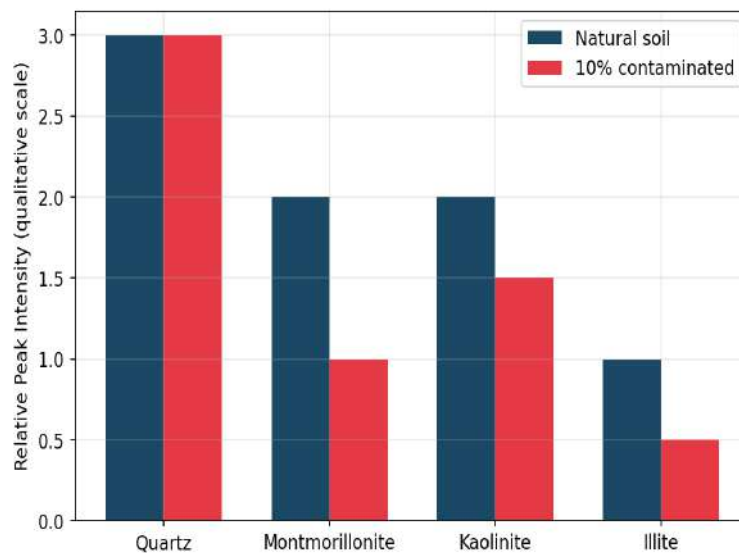


Fig. 8. The natural and 10% contaminated soil were analyzed using XRD to compare their peak intensities

Table 15. FTIR characteristic absorption bands

Peak	Natural Soil (cm ⁻¹)	10% Contaminated (cm ⁻¹)	Functional Group
1	–	2955	C–H stretch (alkanes)
2	–	2925	C–H stretch (alkanes)
3	–	2855	C–H stretch (alkanes)
4	–	1605	C=C stretch (aromatic)
5	3620	3620	O–H (clay minerals)
6	1000–1100	1000–1100	Si–O–Si (silicates)
7	1630	1630	H–O–H (adsorbed water)

Table 16. Comparisons of peak intensities from x-ray diffraction (XRD) spectra

Mineral	Natural Soil	10% Contaminated	Observed Change
Quartz (SiO ₂)	High	High	Slight change
Montmorillonite	Medium	Low	Decrease in intensity
Kaolinite	Medium	Medium-Low	Slight decrease
Mineral	Natural Soil	10% Contaminated	Observed Change
Illite	Low	Very Low	Notable decrease

5.10.4 XRD Analysis

XRD patterns (Table 16, Fig. 8) showed that quartz peak intensity was essentially constant while the peak intensity of montmorillonite and illite decreased markedly and the peak intensity of kaolinite decreased slightly with contamination. This means that the hydrocarbon intrusion is selective and occurs more in the clay mineral fraction than in the non-clay quartz fraction which is consistent with the loss of plasticity and cohesion at the macro scale.

5.10.5 Regression Correlations

The R^2 value for all correlations is ≥ 0.94 , indicating strong linear relations of the measured geotechnical parameters with the contamination level that are statistically significant.

Table 17. This is a plot of the correlations between the contamination level (C, %) and the key parameters in the regression

Relationship	Equation	R^2
PI vs. C	$PI = 20.2 - 0.82C$	0.97
LL vs. C	$LL = 42.5 - 1.15C$	0.98
MDD vs. C	$MDD = 1825 - 14.5C$	0.96
OMC vs. C	$OMC = 14.6 - 0.26C$	0.94
c vs. C	$c = 32.5 - 1.45C$	0.99
Cc vs. C	$Cc = 0.205 + 0.0125C$	0.98

6. Discussion

The results are summarized in Tables 3–17 and in Fig.3–8. The results of the tests shown in 1-8 indicate that there is significant alteration in the geotechnical behavior of the sandy clay soil when it is contaminated with petrochemicals; and the microstructural evidence explicitly accounts for the mechanisms that lead to macro-scale changes. The decrease in the Atterberg limits (Section 3.1) can be attributed to the substitution of the polar pore water by non-polar hydrocarbons, resulting in a decrease in the ability of the clay particles to form electrochemical bonds and adsorbed layers of water. This is directly confirmed by the EDS results (Section 3.10) where the carbon content is seen to increase and the oxygen content is seen to decrease at the particle surface, and by the FTIR results, which show the presence of new bands of aliphatic and aromatic hydrocarbons, which were not present in the natural soil sample. These results agree with other studies [16,17,18,19,20]. The decrease in MDD and OMC (Section 3.2) is consistent with that of [16,17,21,22] as the specific gravity of the diesel fuel is low. This mechanism is supported by direct observation from SEM (Table 13) where the progressive coverage of the hydrocarbon film and increase in porosity as the contamination level increases is visible. The marked reduction in cohesion with only minor change in friction angle (Section 3.3) is consistent with other literature [17,18,21,22] which indicates that the particles-to-particles cohesive forces are being disrupted, but that the frictional forces are only slightly affected by the presence of contamination. This is corroborated by the XRD results (Section 3.10) which indicate that the peak intensity of the clay minerals is measurably reduced during hydrocarbon intrusion, but the

intensity of quartz is actually near unchanged, and this is why the frictional (sand-dominated) component of strength is largely preserved. The conversion from brittle to ductile behavior (quantified in the Table of stress-strain data and in the softening ratios given in Sections 3.4–3.5) is similar to that reported in [19,23] and is due to the loss of stiffness and to the more gradual release of the stresses after the peak. The rise in compression index and in settlement (Sections 3.6–3.7) is consistent with [16,21] and is directly related to the measured 74% decrease in permeability (Section 3.8) which would result in slower fluid expulsion from pores during settlement and extend settlement time (consistent with [20,23]). The statistical analysis (Section 3.9) also reveals that the greater the contamination, the greater the scatter in soil behavior; a practically important conclusion that suggests that in contaminated zones the parameters should be treated with greater uncertainty than in uncontaminated zones when considering geotechnical design. Overall, the physicochemical interaction between hydrocarbons and soil particles (based on consistent evidence on macro-mechanical, permeability, statistical and microstructural/mineralogical scales) results in a coordinated ensemble of effects including decreases in plasticity and strength, increases in compressibility and settlement, and decreases in permeability. The findings are consistent with the general findings in the literature for contaminated soil mechanics [19,24,25] and demonstrates the need for consideration of contamination effects when designing contaminated ground, and for ground improvement or remediation prior to development of contaminated sites.

7. Conclusion

The present study provided a multi-scale experimental investigation of the impact of diesel fuel contamination on the geotechnical behavior of sandy clay soil (CL) for four contamination levels (0, 2, 5 and 10% by dry weight) using index, compaction, shear strength, stress-strain, softening, compressibility, time-settlement, permeability, statistical, and microstructural (SEM, EDS, FTIR, XRD) tests. The findings support the hypothesis that petrochemical contamination elicits a consistent and coherent array of response from all scales studied. The liquid limit, plastic limit, and plasticity index showed a gradual decrease with contamination and at the index level, the hydrocarbons appeared to replace the pore water in the soil, causing a reduction in plasticity and water affinity. The compaction test demonstrated similar decreases in Maximum Dry Density and Optimum Moisture Content, which are associated with poor packing of particles as a result of the lubricating and low-density properties of the diesel fuel. Shear strength testing showed that the contamination has a significant effect on the reduction of cohesion, from 32 kPa in natural soil to 18 kPa at 10% contamination; the effect on the friction angle was negligible indicating that the primary impact of the contamination is to reduce inter-particle bonding. The fully tabulated stress-strain data showed a clear, quantitative shift from brittle to ductile failure, with an increase in the strain at failure from 3.5% to 7.0%, and the softening ratio from 54.1% to 31.4% from the natural sample to the most contaminated sample. Compression index increased with increase in contamination level from 0.21 to 0.33, which confirmed the trend of increasing compressibility with increasing contamination, as seen in the time-settlement data, which indicated an increase in final settlement by 87.5% at the highest contamination level. As a result of permeability testing, it was found that this behavior is related to a coefficient of permeability decrease of 74% during the consolidation process, thus indicating that the pore spaces are progressively closed when the hydrocarbon coating is applied, thus reducing the permeability. The statistical analysis of replicate testing was conducted and the coefficient of variance for key strength parameters almost tripled from the natural to the most contaminated soil, suggesting that not only is the mean degradation of soil properties due to contamination, the variance is also increased, having direct implications for the reliability-based geotechnical design. Most significantly, for all of the above trends, the SEM imaging showed progressive hydrocarbon film coverage of soil particles, EDS showed a significant increase in surface carbon content, FTIR showed the presence of characteristic absorption bands of aliphatic and aromatic hydrocarbons, and XRD showed the visibility of a measurable decrease in the peak intensities of montmorillonite and illite, with quartz remaining largely unaffected. Finally, regression analysis revealed clear and statistically sound relationships ($R^2 = 0.94 - 0.99$) of contamination level with all measured parameters, making these parameters useful practical predictors for engineering assessments.

Together, the results of this study demonstrate that contamination by the petrochemicals reduce the physical, chemical, mechanical and hydraulic properties of soil in an interrelated manner and that a comprehensive site assessment, soil intervention and designing for contamination are necessary prior to construction on contaminated sites.

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Comparative sensitivity analysis and multi-objective optimization of CMP parameters for straight and arc edges of cemented carbide tools

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Abstract

Cemented carbide tools are widely used under demanding machining conditions such as deep-hole boring, where cutting-edge stability is essential for suppressing micro-chipping and maintaining machining quality. To improve edge stability, a three-factor three-level orthogonal experiment is constructed to investigate the effects of polishing disk rotational speed, slurry flow rate, and B-axis swing speed on the chemical mechanical polishing and passivation of cemented carbide tool edges. After passivation, the blunt radius increased by 20.50-42.00 μm for straight edges and 25.16-50.41 μm for arc edges, reaching 35.86-57.48 μm and 42.14-67.53 μm , respectively, with a stronger passivation response observed for arc edges. Increasing the polishing disk speed from 30 rpm to 90 rpm increased the average edge radius from 40.92 μm to 59.65 μm , with the response at 90 rpm being 1.46 times that at the 30-rpm baseline. The slurry flow rate had a weak effect, while increasing the B-axis swing speed enhanced passivation. A comprehensive multi-criteria evaluation integrating material removal, passivation stability, and forming consistency yielded sensitivity scores of 0.894, 0.616, and 0.318 for polishing disk speed, B-axis swing speed, and slurry flow rate, respectively. These results indicate that a polishing disk speed of 60-90 rpm, a slurry flow rate of 7 mL/min, and a B-axis swing speed of 3.6-4.8°/s provide a balanced process window for the chemical mechanical polishing passivation of cemented carbide tool edges.

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1. Introduction

Cemented carbide tools have a high proportion of applications in deep hole boring, precision turning and high-load continuous cutting. The microstructure of the cutting edge directly affects cutting force, cutting temperature, tool wear, and the quality of the machined surface [1-2]. For brittle cemented carbide substrates, Chemical Mechanical Polishing (CMP)-based edge rounding provides lower-force and more controllable material removal than brushing or sandblasting, thereby suppressing micro-chipping and producing a continuous transition surface that improves coating adhesion. For deep hole parts such as the inner hole of a hydraulic cylinder, the machining area is closed, the chip removal space is limited, and the cutting heat is not easily diffused, making the cutting edge more prone to micro-chipping, adhesive wear and local stress concentration. If the cutting edge is not stabilized and passivated, the initial micro-defects will continue to expand under the cutting load, resulting in shortened tool life and fluctuations in surface roughness [3]. Therefore, obtaining a cemented carbide cutting edge with controllable radius, continuous morphology and good surface integrity is a key step in improving the stability of deep hole machining. Abolaje A E et al. compared the performance of high-speed steel and tungsten carbide tools for turning 304 austenitic stainless steel using the response surface method. The tool wear

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was only 1.173 mm at the optimal parameters (cutting speed 1303 m/min, feed rate 0.354 mm/rev, and cutting depth 0.458 mm). This proved that tungsten carbide tools were more efficient [4]. Gangwar S et al. enhanced the cutting parameters of cemented carbide tools by combining plasma physical vapor deposition coatings, Taguchi L18 experiments, and grasshopper optimization algorithms. The coated tool performed best at a depth of 0.1 mm, a feed rate of 0.812 mm/rev, and a cutting speed of 70.45 m/min [5]. Umer U and Al-Ahmari A proposed a multi-objective optimization method based on the radial basis function response surface element model and the second-generation multi-objective genetic algorithm for tool geometry parameter optimization in hard turning. A 3D finite element model considering the radius of the cutting edge and the radius of the tool tip was established. Combined with the Usui adhesive wear model and coupled temperature-displacement explicit dynamics analysis for chip formation simulation and tool wear prediction, multi-objective optimization of tool temperature and material removal rate was achieved [6]. While geometrical optimization improves efficiency, the physical state of the cutting edge remains a bottleneck for tool reliability.

CMP achieves material removal through the synergistic effects of oxidation reaction, softening layer formation and abrasive micro-cutting, and improves the surface quality of cemented carbide tools under lower mechanical damage conditions. The chemically softened layer reduces the mechanical load required for abrasive removal, achieves low force polishing, maintains the geometric continuity of asymmetric straight arc edges, and suppresses local shape deformation [7]. For example, Deng Q et al. established a self-excited oscillation cavity model and simulated the fluid oscillation effect at different cavity lengths based on the abrasive water jet polishing method for optimizing the process parameters of carbide tool edge passivation [8]. Hung J C et al. proposed an electric CMP system integrated with a multi-axis robotic arm to address the rough surfaces, difficult and time-consuming polishing of additively manufactured titanium alloy parts. Based on offset grinding brush heads and jet electrolyte flow processes, combined with automated polishing parameter optimization, the surface roughness was reduced [9]. In response to the unclear effect of oxidant content in the polishing fluid of cemented carbide CMP, Wang B et al. analyzed the influence of hydrogen peroxide as an oxidizer on the surface corrosion behavior of cemented carbide and the polishing performance of CMP. It was found that the mass fraction of oxidizer in the range of 0-30% could accelerate the formation of soft loose oxide layer and promote mechanical removal [10].

However, existing studies have mainly focused on simple or straight cutting edges, whereas industrial inserts commonly contain asymmetric straight, arc combinations in which curvature-dependent contact can cause non-uniform material removal. Systematic comparisons of edge size, profile continuity, and surface integrity in these regions remain limited. Meanwhile, some studies mainly relied on overall processing performance or surface roughness as the evaluation basis, and did not fully reveal the specific effects of the Polishing Disk Rotational Speed (PDRS), the slurry flow rate, and the B-axis swing speed on the mechanical removal intensity, reaction product renewal, and edge contact trajectory distribution during the CMP process. Therefore, the study selects the PDRS, the slurry flow rate and the B-axis swing speed as experimental factors for orthogonal experiments. Based on edge profile detection, range comparison and multi-index comprehensive evaluation, the differences in the effects of each factor on the passivation quality are analyzed, aiming to reveal the influence law of CMP process parameters on edge passivation effect and determine the process parameter window that takes into account the controllability of the edge radius, surface integrity, and passivation stability. The novelty lies in the quantitative mapping of parameter sensitivity specifically for the transition zones between straight and arc segments, which is often overlooked in conventional polishing studies.

2. Methods and Materials

2.1 Setup and Measurement Scheme for Edge Passivation Experiments

To analyze the influence of CMP process parameters on the edge passivation quality of cemented carbide tools, a CMP edge passivation experimental platform is established, mainly consisting of a polishing unit, polishing plate, polishing disk, B-axis rotating swing fixture, polishing liquid supply

device, blade positioning fixture, and edge detection system [11]. The passivation experiment uses a modified platform of a UNIPOL-1200S automatic pressure grinding and polishing machine (Shenyang Kejing Automation Equipment Co., Ltd., China). The grinding and polishing disc diameter is 300 mm, the rotation speed range is 20-240 r/min, the minimum adjustment step is 10 r/min, and a polyurethane polishing pad is attached to the disc surface. The B-axis oscillating fixture is a self-made device with an angular velocity adjustment range of 0-10°/s and a control accuracy of $\pm 0.05^\circ/\text{s}$. The polishing fluid is delivered by a BT100-2J peristaltic pump (Baoding Lange Constant Flow Pump Co., Ltd., China), with a rotation speed adjustment resolution of 0.1 r/min. The three-dimensional profile of the cutting edge is measured using a VK-X1000 three-dimensional laser confocal profilometer (KEYENCE, Japan), with a vertical display resolution of 0.5 nm. The surface microstructure is observed using a VHX-7000 digital microscope (KEYENCE, Japan), equipped with a VH-Z100T lens, with a magnification of 100 to 1000 times and a high-precision image resolution of 6144×4608 pixels. The edge CMP passivation experimental platform and measurement process are shown in Figure 1.

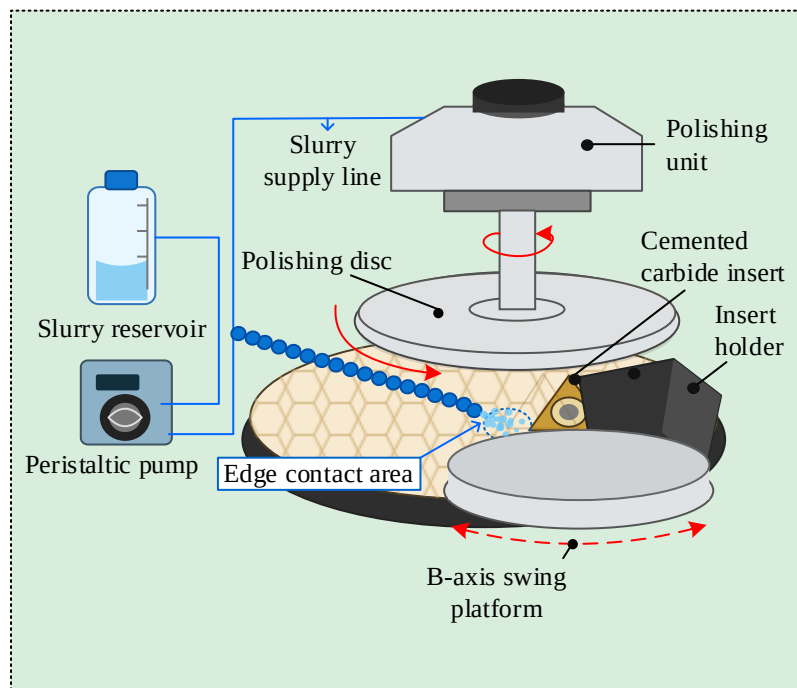


Fig. 1. Schematic of the cutting-edge CMP passivation experimental platform

In Figure 1, the study uses uncoated carbide inserts as experimental subjects. Before the experiment, the blade is placed in anhydrous ethanol and ultrasonically cleaned at 25 °C for 15 min, then rinsed thoroughly with deionized water and dried with dry air. To weaken the impact of the initial edge difference on the experimental results, the initial edge blunt-round radius and surface roughness are measured for each group of blades before passivation and used as the reference values for incremental analysis. The cutting-edge measurement area is divided into a straight cutting-edge area and a circular cutting-edge area. In the straight cutting-edge area, a 2 mm length from the center of the cutting edge is selected as the measurement range, and 5 measuring points are set at equal intervals along the cutting-edge length. In the circular cutting-edge area, 5 measuring points are set at equal intervals along an effective arc length of 2 mm. Each measuring point is measured 3 times, and the average value is taken as the blunt circle radius of the cutting edge at that position. The CMP passivation test parameters and horizontal settings are shown in Table 1.

The parameter ranges in Table 1 were determined based on the stable operating conditions of the experimental platform and preliminary single-factor experiments. Preliminary experiments showed that when the polishing disc speed is below 30 r/min, the softened reaction layer is not sufficiently removed, and the cutting-edge radius does not change significantly. When the speed

exceeds 90 r/min, the outward splashing of the polishing slurry and fluctuations in the cutting-edge contact intensify.

Table 1. CMP passivation experimental parameters, levels, and theoretical roles

Factor	Process parameter	Level 1	Level 2	Level 3	Theoretical role
A	Polishing disk speed (rpm)	30	60	90	Controls the relative sliding speed in the Preston relation, thereby regulating mechanical material removal intensity
B	Slurry flow rate (mL/min)	3	5	7	Regulates the effective Preston removal coefficient through abrasive and oxidant supply and reaction-layer renewal
C	B-axis swing speed ($^{\circ}$ /s)	3.6	4.8	6.0	Changes the local contact-pressure distribution and contact duration, thereby regulating removal uniformity between the straight-edge and arc-edge regions

Note: The B-axis performs a reciprocating oscillation within a limited angular range, rather than a continuous rotation. Therefore, its angular displacement change rate is expressed in $^{\circ}$ /s.

Therefore, 30-90 r/min is set as the investigation range, with three levels set at equal intervals. When the polishing slurry flow rate is below 3 mL/min, the supply of abrasive particles and oxidant is insufficient, while the stability improvement tends to saturate after exceeding 7 mL/min. Therefore, the flow rates are set to 3, 5, and 7 mL/min. When the B-axis oscillation speed is below 3.6° /s, the cutting-edge contact trajectory coverage is insufficient, while above 6.0° /s, the difference in forming straight and circular cutting edges increases. Therefore, the flow rates are set to 3.6, 4.8, and 6.0° /s. After each set of experiments, the blades are rinsed with deionized water, washed with anhydrous ethanol and dried. Next, the edge profile of the straight edge and the arc edge is measured and observed, respectively. The variation in the radius of the blunted edge is calculated according to Equation (1) [12].

$$\Delta R = R_p - R_0 \quad (1)$$

In Equation (1), ΔR is the increment of the blunt radius. R_p signifies the blunt radius after passivation. R_0 signifies the initial blunt radius before passivation. To improve the reliability of the measurement results, multiple measurement points are set in the straight edge and arc edge areas of the same blade, respectively. The average edge radius is calculated in Equation (2) [13].

$$\bar{R} = \frac{1}{n} \sum_{i=1}^n R_i \quad (2)$$

In Equation (2), $\bar{R}$ is the average blunted radius in a certain area. n signifies the number of measurement points in this area. R_i signifies the blunted radius of the i -th measurement point.

2.2 Chemical-Mechanical Synergistic Passivation Process Flow

CMP passivation typically begins with the reaction of the oxidizer in the polishing solution with the surface of the hard alloy to obtain a relatively softened oxide film or reaction layer on the edge surface. Next, the polishing disk and abrasive remove the softened layer under the action of relative motion and contact pressure, gradually weakening the micro-chipping defects, burrs and local peaks on the edge, and eventually forming a continuous, round and good-quality passivated edge. Therefore, the CMP passivation process flow is shown in Figure 2.

As shown in Figure 2, the study first pre-treats the carbide blade and measures the initial edge blunted radius and initial surface roughness. Secondly, the blade is fixed on the B-axis swing fixture to keep the edge to be blunted in stable contact with the polishing disk. To ensure the repeatability of the CMP passivation test, the slurry is prepared with reference to Wang et al. [10] using Al_2O_3

abrasive particles, H₂O₂ oxidant, sodium polyacrylate dispersant, and deionized water. The final mass fractions are listed in Table 2.

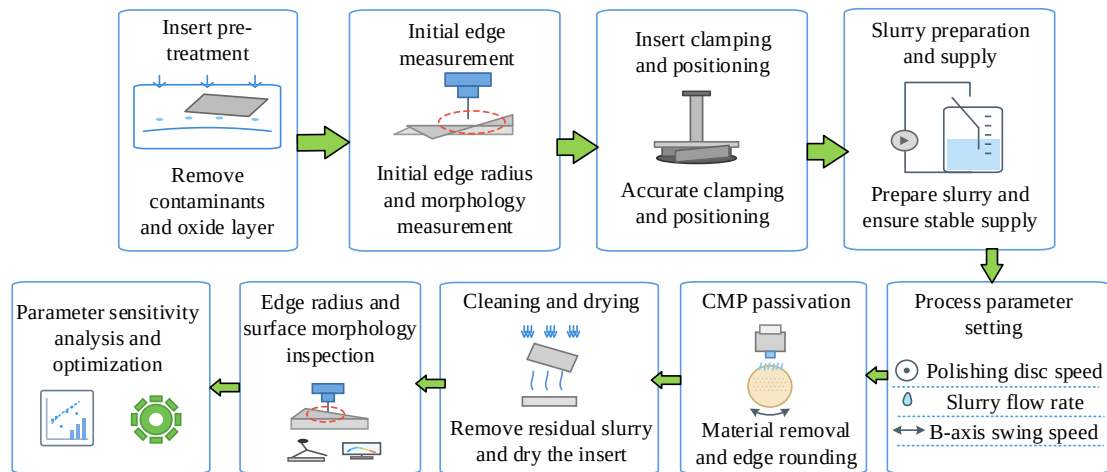


Fig. 2. CMP passivation process flowchart

Table 2. Composition and preparation parameters of the polishing slurry

Component	Material and specification	Final content	Main function
Abrasive	α -Al ₂ O ₃ , purity $\geq 99.9\%$, mean particle size of 50 nm	10.0 wt%	Micro-cutting removal of the softened reaction layer
Oxidising agent	H ₂ O ₂ , analytical grade, stock concentration of 30 wt%	5.0 wt%	Promotes surface oxidation and softened-layer formation
Dispersant	Sodium polyacrylate, analytical grade	0.5 wt%	Suppresses abrasive agglomeration and maintains suspension stability
pH regulator	Aqueous ammonia, analytical grade	As required	Adjusts the pH to 9.5 ± 0.1
Dispersion medium	Deionised water, resistivity $\geq 18.2 \text{ M}\Omega \cdot \text{cm}$	Balance to 100 wt%	Solvent and reaction medium

During preparation, sodium polyacrylate is first added to deionized water and magnetically stirred for 10 min. Al₂O₃ abrasive particles are then slowly added and ultrasonically dispersed for 20 min. Next, H₂O₂ is added, and the pH of the polishing solution is adjusted to 9.5 ± 0.1 using ammonia water. Magnetic stirring is continued for 30 min. The polishing solution is prepared fresh at 25 ± 1 °C and used immediately to minimize the impact of H₂O₂ decomposition and abrasive particle sedimentation on the experimental results. The polishing disk rotates at a set speed, and the B-axis swing platform drives the blade to swing periodically, causing the straight edge and arc edge areas to enter the effective contact area in sequence. The polishing fluid is continuously delivered through the supply system to the contact area between the edge and the polishing disk to maintain stable chemical reaction and abrasive renewal conditions. Material removal during the CMP passivation process can be expressed as the combined action of chemical softening and mechanical removal, as shown in Equation (3) [14-15].

$$M = M_c + M_m \quad (3)$$

In Equation (3), M is the total amount of material removed from the cutting edge per unit time. M_c signifies the contribution of surface softening or reaction layer formation caused by the chemical reaction. M_m is the amount of material removed by the mechanical action of abrasive grains and polishing disks. The mechanical removal is closely related to the contact pressure and the relative

motion speed. According to the Preston relation, the material removal rate per unit area is given in Equation (4) [16].

$$MRR = k_p PV \quad (4)$$

In Equation (4), MRR signifies the material removal rate. k_p signifies the Preston coefficient, reflecting the combined action characteristics of the polishing disk, abrasive grains, polishing liquid, and the material system. P signifies the average contact pressure between the cutting edge and the polishing disk. V is the relative velocity between the cutting edge and the polishing disk. The study sets the polishing disk rotation speed to mainly affect the mechanical removal intensity by changing V . The slurry flow rate mainly affects the reaction layer renewal and abrasive grain transport by changing the stability of k_p . The B-axis swing speed affects the removal uniformity of different edge areas by changing the local contact time of the edge. The linear velocity generated by the rotated polishing disk is calculated according to Equation (5) [17].

$$V_d = \frac{\pi DN}{60} \quad (5)$$

In Equation (5), V_d is the surface linear velocity of the polishing disk. D signifies the effective diameter of the polishing disk corresponding to the contact position of the cutting edge. N signifies the PDRS. Because the contact state of the arc edge area keeps changing during the swing, the contact time per unit length is different from that of the straight edge area. To characterize the intensity of local contact, the equivalent contact time is shown in Equation (6).

$$t_e = \frac{L_c}{v_s} \quad (6)$$

In Equation (6), t_e is the local equivalent contact time of the cutting edge. L_c is the effective contact path length of the cutting edge in one swing period. v_s is the equivalent movement speed of the cutting edge caused by the B-axis swing speed. To establish the relationship between the Preston material removal rate and the equivalent contact time of the asymmetric cutting edge, $j \in \{s, a\}$ represents the straight cutting-edge region and the circular cutting-edge region, respectively. The cumulative material removal amount in one effective contact process can be expressed as Equation (7).

$$\begin{aligned} Q_j(t_{e,j}) &= Q_{c,j}(t_{e,j}) + A_j \int_0^{t_{e,j}} k_p P_j(\tau) V_j(\tau) d\tau \approx Q_{c,j}(t_{e,j}) + A_j k_p \bar{P}_j \bar{V}_j t_{e,j} \\ &= Q_{c,j}(t_{e,j}) + A_j k_p \bar{P}_j \left(\frac{\pi DN}{60} \right) \left(\frac{L_{c,j}}{v_{s,j}} \right) \end{aligned} \quad (7)$$

In Equation (7), Q_j represents the cumulative material removal amount in the cutting-edge region j . $Q_{c,j}$ represents the material removal contribution corresponding to chemical softening within the equivalent contact time. A_j represents the effective contact area. $\bar{P}_j$ and $\bar{V}_j$ represent the average contact pressure and average relative velocity during a single contact process, respectively. Under the condition that the linear velocity of the polishing disc dominates, $\bar{V}_j$ is approximately taken as $\frac{\pi DN}{60}$. The local material removal amount depends not only on the polishing disc rotation speed and contact pressure, but also on the equivalent contact time formed by the B-axis oscillation. The differences between the straight cutting-edge region and the circular cutting-edge region in terms of effective contact path, local pressure, and equivalent contact time will further lead to inconsistencies in the material removal amounts between the two regions. The chemical and mechanical synergistic passivation mechanism is shown in Figure 3.

As shown in Figure 3, the key to the chemical and mechanical synergistic passivation process lies in controlling the dynamic balance between the reaction layer formation rate and the mechanical removal rate. If the chemical reaction is insufficient, the abrasive will act directly on the hard alloy substrate, which is prone to causing scratches and local chipping. If the chemical reaction is too

strong and the mechanical removal is insufficient, a reaction film may remain on the edge surface, affecting the sharpness of the edge profile and surface integrity. The PDRS, the slurry flow rate, and the B-axis swing speed regulate this balance from three aspects: mechanical energy input, polishing fluid renewal, and contact trajectory distribution.

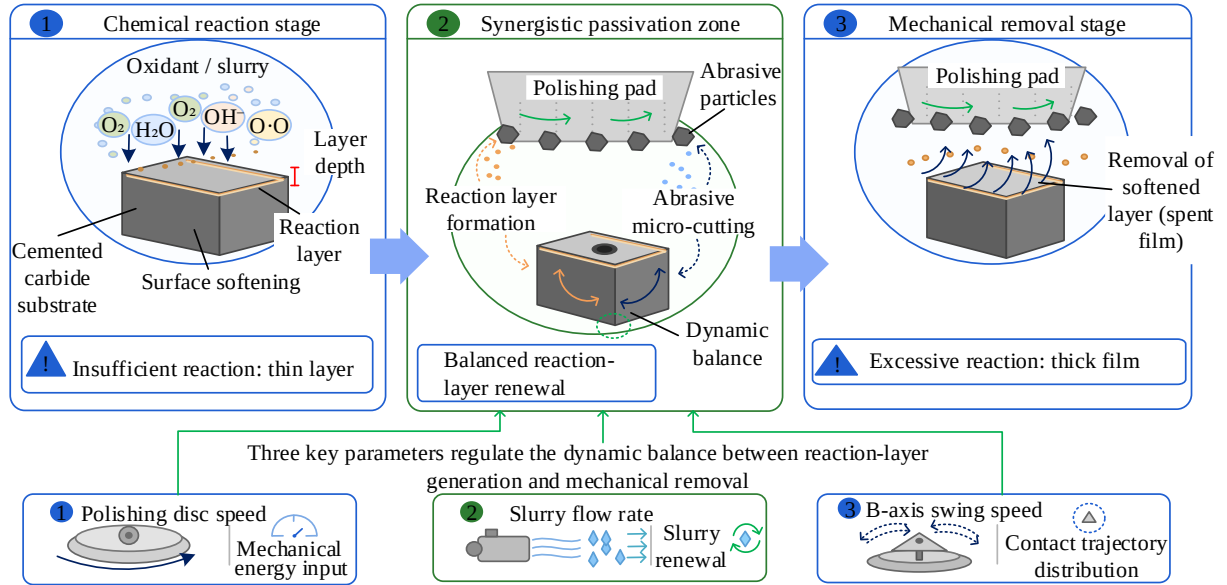


Fig. 3. Chemical and mechanical synergistic passivation mechanism schematic

2.3 Evaluation of Parameter Sensitivity and Statistical Analysis Methods

To quantitatively evaluate the effects of the PDRS, slurry flow rate, and B-axis swing speed on CMP edge passivation, range analysis, analysis of variance (ANOVA), and multi-index comprehensive evaluation are performed using the orthogonal experimental results. ANOVA is conducted at a significance level of $\alpha=0.05$. $p<0.05$ and $p<0.01$ indicate significant and highly significant differences, respectively. The parameter sensitivity evaluation process is shown in Figure 4.

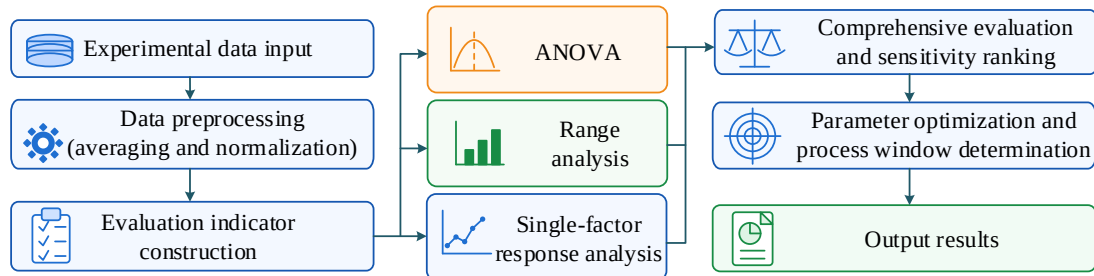


Fig. 4. Flowchart of parameter sensitivity evaluation and statistical analysis

From Figure 4, the study uses range analysis to determine the extent to which different process parameters affect the blunt circle radius of the cutting edge. For any evaluation index, the mean index of a certain factor at the l -th level can be expressed as Equation (8).

$$\bar{Y}_{ijl} = \frac{1}{m} \sum_{q=1}^m Y_{ijlq} \quad (8)$$

In Equation (8), $\bar{Y}_{ijl}$ is the mean value of the i -th evaluation index for the j -th process factor at the l -th level. Y_{ijlq} is the evaluation index value of the i -th evaluation index obtained in the q -th experimental run corresponding to that factor level. m is the number of experimental runs at the same level. The factor range is calculated according to Equation (9) [18].

$$R_{ij} = \max(\bar{Y}_{ij1}, \bar{Y}_{ij2}, \bar{Y}_{ij3}) - \min(\bar{Y}_{ij1}, \bar{Y}_{ij2}, \bar{Y}_{ij3}) \quad (9)$$

In Equation (9), R_{ij} represents the range of the i -th evaluation index for the j -th process factor. The larger the value, the more obvious the influence of this factor on the evaluation index and the higher the parameter sensitivity. Finally, to avoid the limitations of a single index evaluation, the study constructs a comprehensive sensitivity evaluation index from three aspects: material removal sensitivity, passivation stability, and forming consistency, and normalizes each index. The higher the comprehensive sensitivity score, the stronger the comprehensive influence of this parameter on the CMP passivation effect. Considering that effective material removal and edge-radius formation are the primary objectives of CMP passivation, followed by forming consistency between the straight and arc edges, while process stability serves as a supporting requirement, the weights of material removal sensitivity, stability regulation sensitivity, and forming consistency sensitivity are set to 0.50, 0.20, and 0.30, respectively.

3. Results and Analysis

3.1 Overall Response Characteristics of The Blunt Radius

To obtain the overall passivation response of carbide cutting edges under different CMP parameter combinations, a three-factor, three-level orthogonal experiment was conducted. Each parameter combination was independently polished three times with a new insert, and the blunt radius of the straight and circular cutting edges was measured. The results are expressed as the mean $\pm$ standard deviation of the three independent experiments. Table 3 presents the response results of the blunted radius of the cutting edge under different parameter combinations.

Table 3. Orthogonal experimental results of CMP passivation

No.	Polishing disk speed (rpm)	Slurry flow rate (mL/min)	B-axis swing speed ($^{\circ}$ /s)	Straight-edge radius (μm)	Straight-edge radius increment (μm)	Arc-edge radius (μm)	Arc-edge radius increment (μm)	Radius difference (μm)	Mean edge radius (μm)
1	30	7	6.0	39.08 $\pm$ 1.26	23.66	47.86 $\pm$ 1.42	30.80	8.78	43.47 $\pm$ 1.34
2	30	5	4.8	36.16 $\pm$ 1.18	20.98	44.45 $\pm$ 1.35	27.61	8.29	40.30 $\pm$ 1.27
3	30	3	3.6	35.86 $\pm$ 1.15	20.50	42.14 $\pm$ 1.28	25.16	6.28	39.00 $\pm$ 1.22
4	60	7	4.8	46.80 $\pm$ 1.41	31.53	54.27 $\pm$ 1.63	37.36	7.47	50.54 $\pm$ 1.52
5	60	5	6.0	48.69 $\pm$ 1.46	33.14	59.69 $\pm$ 1.78	42.55	11.00	54.19 $\pm$ 1.62
6	60	3	3.6	44.68 $\pm$ 1.37	29.37	51.01 $\pm$ 1.50	34.12	6.33	47.85 $\pm$ 1.44
7	90	7	6.0	57.48 $\pm$ 1.72	42.00	67.53 $\pm$ 2.05	50.41	10.04	62.50 $\pm$ 1.89
8	90	5	3.6	54.88 $\pm$ 1.65	39.64	61.30 $\pm$ 1.86	44.35	6.42	58.09 $\pm$ 1.76
9	90	3	4.8	52.74 $\pm$ 1.58	37.35	63.97 $\pm$ 1.92	46.89	11.22	58.36 $\pm$ 1.75

Note: Radius difference was calculated from the mean values of arc-edge radius and straight-edge radius.

As shown in Table 3, both the straight edge and the arc edge formed distinct blunting and rounding characteristics after CMP blunting, but there were differences in the response amplitudes of the two types of edge areas. The blunted radius of the straight edge ranged from 35.86 to 57.48 μm , and that of the arc edge ranged from 42.14 to 67.53 μm . Under the same passivation conditions, the arc-edge region exhibited a higher material removal response than the straight-edge region. According to the Preston equation, the material removal rate is proportional to the local contact pressure and relative velocity. The curved geometry of the arc edge reduces the effective contact area under a comparable normal load, thereby increasing the local contact pressure and material

removal response. From the perspective of parameter combinations, as the PDRS increased from 30 rpm to 90 rpm, the radius of the blunt circle of both the straight edge and the arc edge showed an overall increasing trend. This indicates that the PDRS has a direct impact on the removal strength of the CMP passivated material. A higher rotational speed can increase the relative sliding speed between the edge and the polishing disk, thereby enhancing the removal effect of the abrasive on the softening reaction layer. The comparison of response amplitudes of different factors indicates that the PDRS has the most prominent effect on the size evolution of the straight and arc edges, while the supply of polishing liquid has a relatively weak effect on the B-axis swing speed. The measurement results in the arc area further suggest that the combined effect of chemical reaction and mechanical friction can weaken the microscopic peaks of the edge, gradually transforming the original sharp profile into a smooth, continuous arc shape. Overall response results suggest that the PDRS is the main factor affecting the growth of the edge radius, and the arc edge area shows higher material removal sensitivity compared to the straight edge. To further analyze the overall response differences between the straight edge and the arc edge under different combinations of CMP parameters, the response index, Coefficient of Variation (CV), regional non-uniformity coefficient and form consistency index were calculated based on the measurement results of the edge blunt-round radius in Table 2, as illustrated in Figure 5.

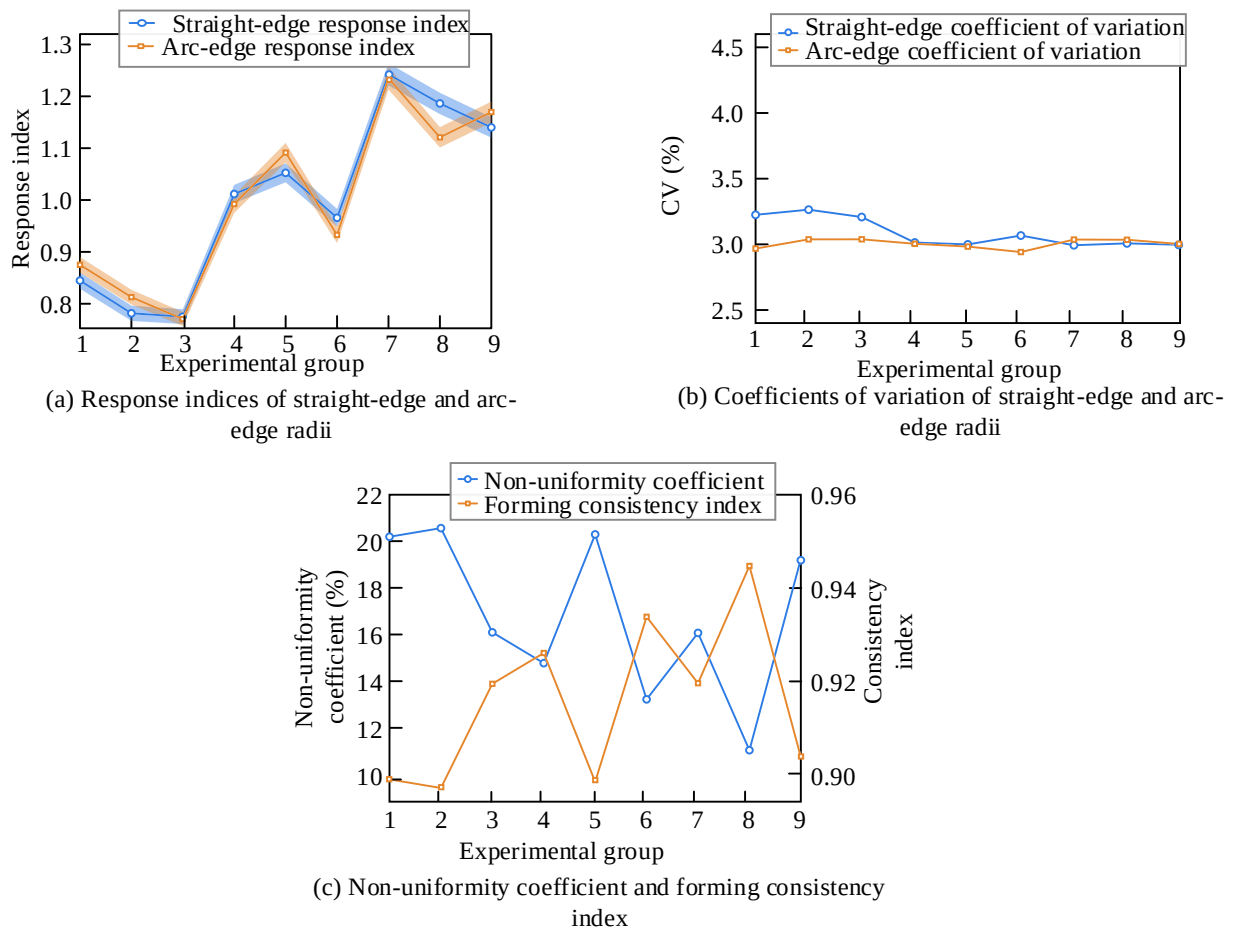


Fig. 5. Overall response characteristics of cutting-edge radii

From Figure 5(a), the variation trends of the response indices of the straight edge and the arc edge were basically the same. Both fluctuated with the combination of CMP parameters, and the response indices were distributed between 0.77 and 1.25. The high response group exceeded the low response group. The corresponding parameter combinations could enhance the edge rounding and material removal effects. The response index of the arc edge was slightly higher than that of the straight edge in some groups, indicating that it was more sensitive to CMP. In Figure 5(b), the CV of the straight edge and the arc edge was generally lower, mainly distributed in the range of 2.94%-3.26%, indicating good stability of the edge radius measurement results. The CV of the straight edge fluctuated slightly, indicating that there were differences in the local contact state and

the uniformity of abrasive action. The CV of the arc edge is relatively stable, indicating that the material removal process is more stable in the continuous curvature area. The forming consistency index was defined as one minus the absolute radius difference divided by the sum of the straight- and arc-edge radii, with a value closer to 1 indicating higher consistency. In Figure 5(c), the regional non-uniformity coefficient was inversely related to the forming consistency index. The non-uniformity coefficient was 11.05%-20.57%, corresponding to a forming consistency index of approximately 0.90-0.95. When the non-uniformity coefficient decreases, the form consistency improves, indicating a reduction in the blunting difference between the straight edge and the arc edge.

3.2 The Effect of The PDRS on Material Removal and Edge Roundness

The PDRS directly affects the relative motion speed between the edge and the polishing disk and is a crucial parameter for regulating the mechanical removal strength of CMP. To analyze its effect on material removal and edge roundness, the edge forming quality at different polishing disk speed levels was compared. The results are shown in Figure 6.

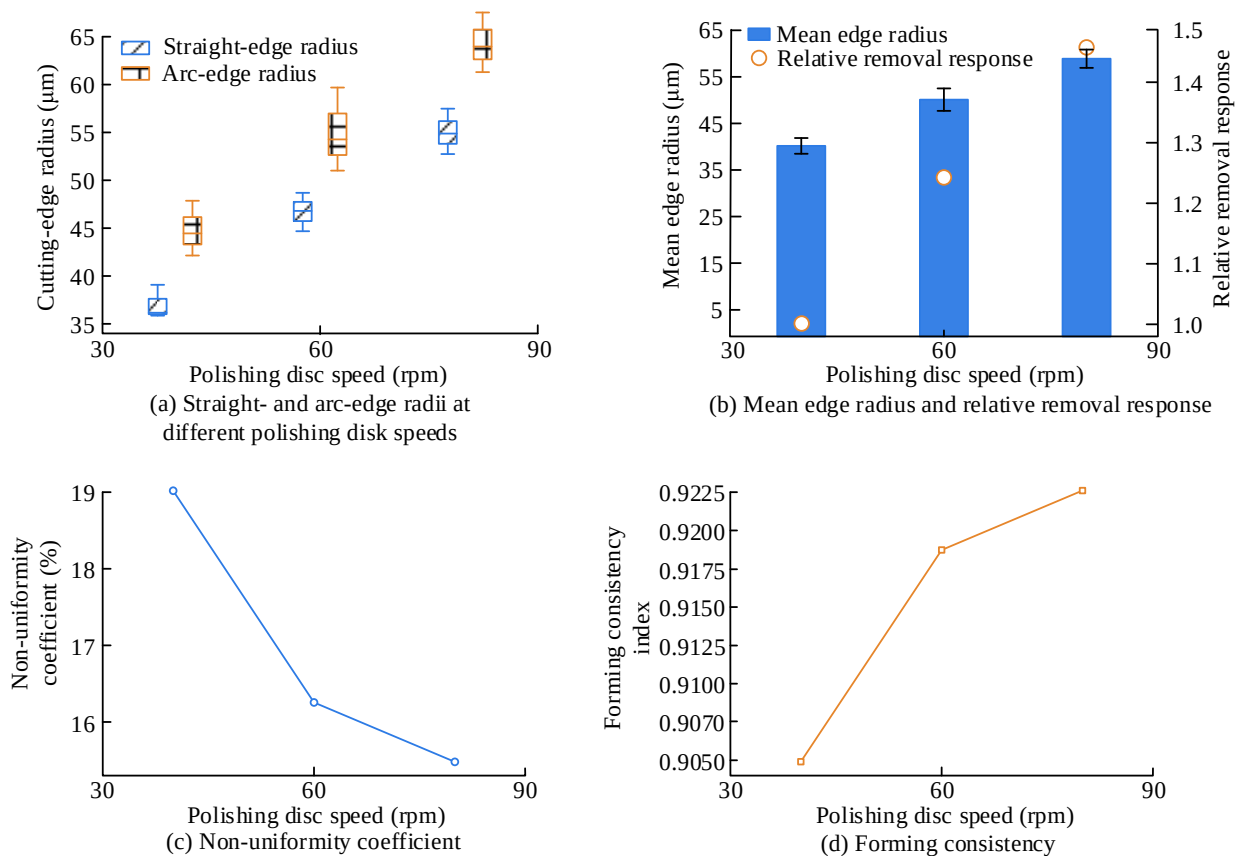


Fig. 6. Effect of polishing disk speed on material removal and edge rounding

In Figure 6(a), as the rotational speed increased from 30 rpm to 90 rpm, the average radius of the straight edge rose from 37.03 μm to 55.03 μm , and the average radius of the arc edge increased from 44.82 μm to 64.26 μm . This indicates that the increase in speed can enhance the relative sliding effect between the edge and the polishing disk, promoting more thorough removal of the surface softening reaction layer by abrasive grains. In Figure 6(b), the average edge radius increased continuously with the increase of the PDRS, and the relative material removal response increased from 1.00 to 1.46, indicating that the speed has a significant promoting effect on the material removal efficiency. When the rotational speed was low, the mechanical energy input was insufficient and the edge roundness was limited. As the speed increases, the micro-protrusions and local sharp areas are further removed, and the edge profile gradually becomes rounded. From Figures 6(c) and (d), the higher rotational speed enhances the material removal strength, and also improves the forming consistency between the straight edge and the arc edge. The rotational speed is a key factor affecting the blunt radius of the cutting edge and the degree of material removal.

However, the speed-dependent removal response should not be considered strictly linear. At excessively high speeds, centrifugal ejection may shorten the slurry residence time, while hydrodynamic lift may increase the liquid-film thickness and reduce abrasive-edge contact pressure, potentially causing removal saturation or decline. Therefore, the increasing trend observed from 30 to 90 rpm is valid only within the tested range.

To further verify the effect of the rotational speed on the roundness and surface integrity, the micromorphology of the straight edge and the arc edge was analyzed at different rotational speed levels, as shown in Figure 7.

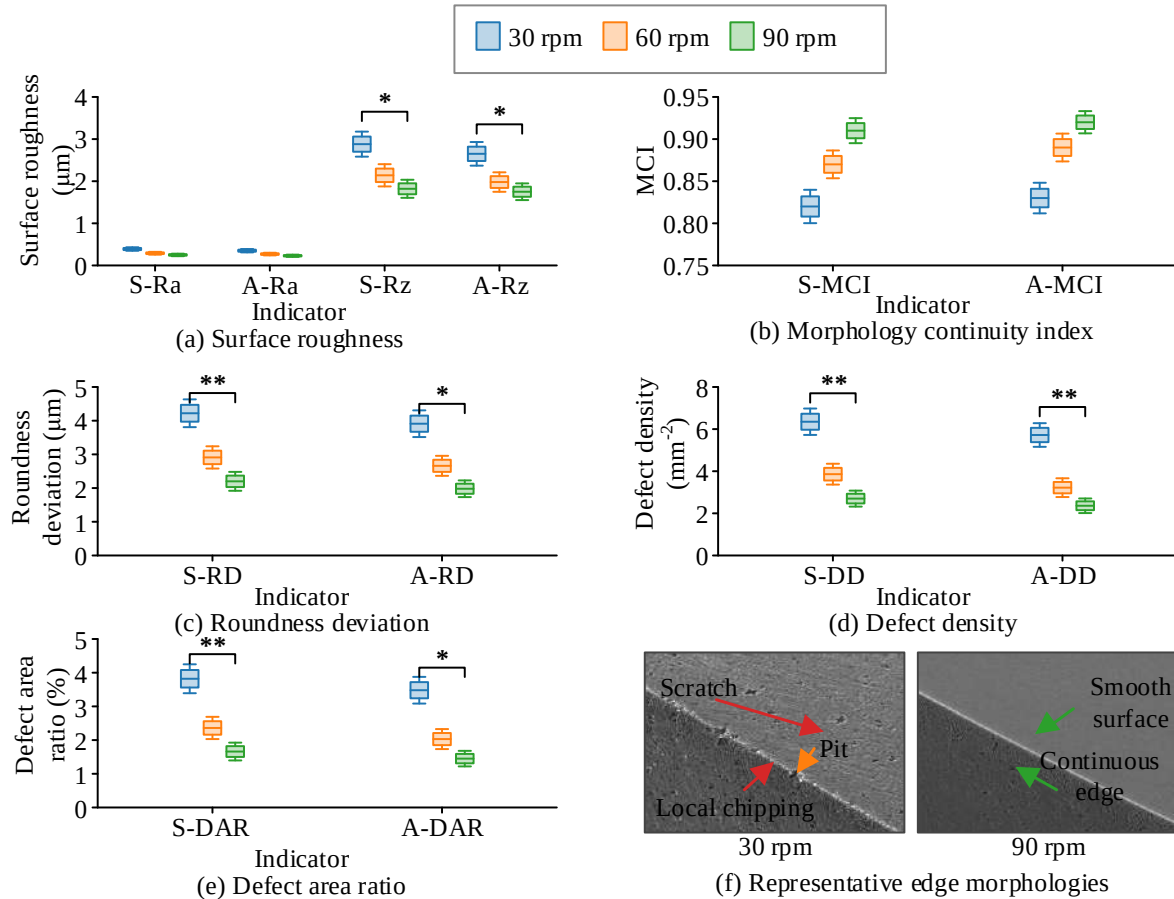


Fig. 7. Boxplots of edge morphology indicators under different speeds (Image source: Author's own photo)

Note: *S* and *A* denote the straight-edge and arc-edge regions, respectively. *Ra* and *Rz* represent arithmetic mean roughness and maximum profile height (μm), respectively. *MCI* represents the dimensionless morphology continuity index. *RD* represents roundness deviation (μm). *DD* represents defect density (mm^{-2}). *DAR* represents defect area ratio (%). * $p < 0.05$ and ** $p < 0.01$ indicate statistically significant differences between the connected groups.

In Figure 7(a), the *Ra* and *Rz* values of both straight and circular cutting edges decreased with increasing polishing disc speed, indicating that higher speeds enhance the abrasive particles' ability to remove micro peaks and soften the reaction layer. In Figure 7(b), *S-MCI* and *A-MCI* increased with increasing speed, indicating improved cutting-edge profile continuity. In Figure 7(c), *S-RD* and *A-RD* at 90 rpm were significantly lower than those at 30 rpm ($p < 0.01$ and $p < 0.05$), indicating a significant reduction in cutting edge roundness deviation. In Figures 7(d) and 7(e), *DD* and *DAR* continuously decreased with increasing speed, indicating a reduction in the number and coverage area of surface defects. In Figure 7(f), at 30 rpm, the cutting-edge surface had obvious scratches, micro-pits, and local chipping. At 90 rpm, the surface was smoother and the cutting-edge profile was more continuous. The morphological observation results were consistent with the changing trends of *MCI*, *RD*, *DD* and *DAR*, indicating that increasing the polishing disc speed within the experimental range is beneficial to improving the roundness of the cutting edge and the surface integrity.

3.3 The Effect of Slurry Flow Rate on Passivation Stability

The slurry flow rate affects the transport state of abrasive grains, oxidants and reaction products in the edge contact area and is a crucial factor in maintaining the stability of the CMP passivation process. Therefore, to analyze the effect of slurry flow rate on passivation stability, the stability of materials at different flow rate levels was compared. The results are shown in Figure 8.

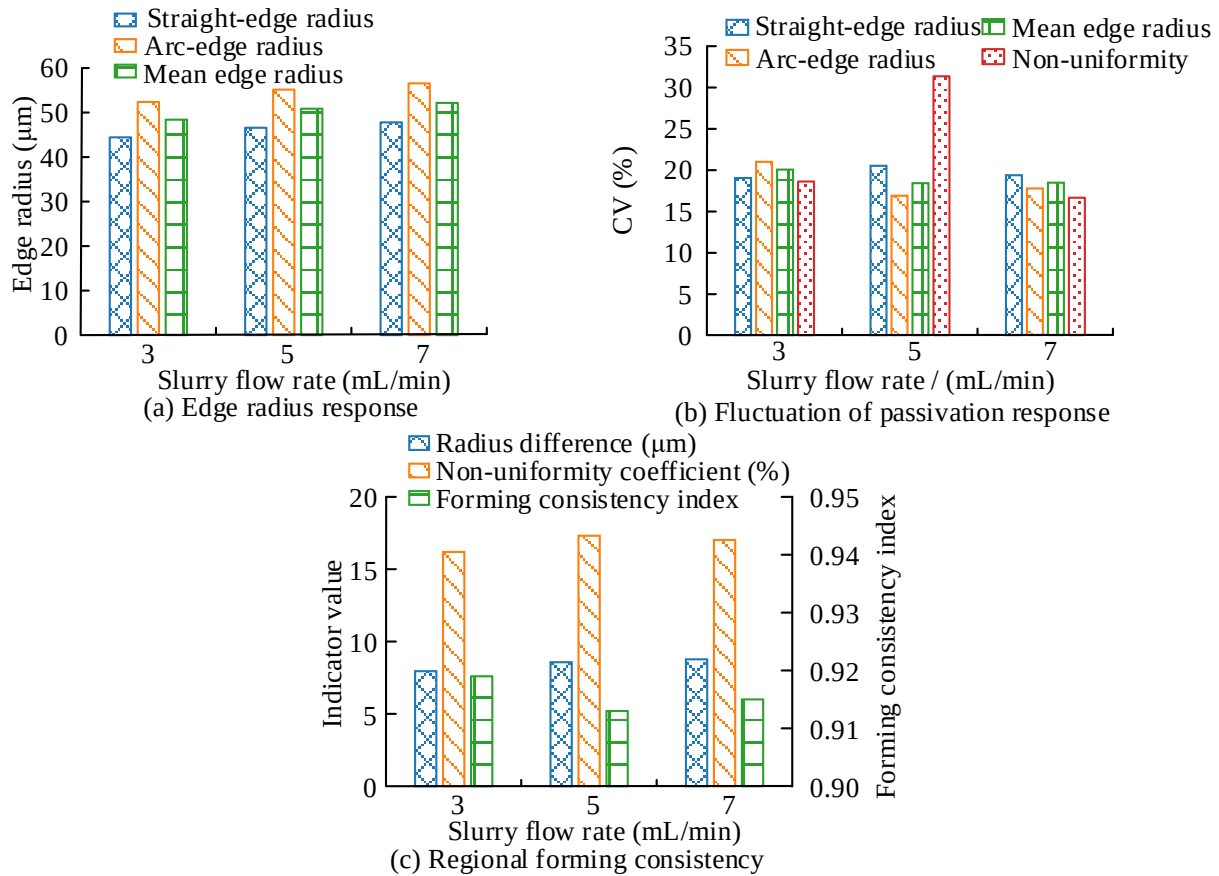


Fig. 8. Effect of slurry flow rate on passivation stability

Figure 8(a) shows that increasing the slurry flow rate from 3 to 7 mL/min raised the mean edge radius from 48.40 to 52.17 μm , an increase of 7.79%, which was substantially lower than that caused by the polishing disk speed. This limited response suggests that the tested flow-rate range was already supply-sufficient, with further increases mainly promoting slurry renewal and reaction-product discharge rather than directly enhancing material removal. From Figure 8(b), the average cutting-edge radius CV at different flow rates was mainly distributed between 18.38% and 20.02%. Within these parameters, the effect of the slurry flow rate on the overall radius fluctuation was relatively limited. A local CV peak was observed at 5 mL/min, suggesting a transitional slurry-transport state rather than a monotonic stability response. At this intermediate flow rate, intermittent transport of abrasive particles and reaction products, potentially amplified by supply pulsation, may have caused fluctuations in liquid-film thickness and local contact conditions. At 7 mL/min, more continuous slurry renewal reduced these fluctuations and improved process repeatability. In Figure 8(c), as the slurry flow rate increased, the radius difference declined from 2.84 μm to 1.29 μm . In addition, the non-uniformity coefficient remained between 16.19% and 17.31%, and the overall variation in the form consistency index was relatively small. Overall, the slurry flow rate mainly serves to stabilize the renewal of the polishing fluid and regulate the consistency of area removal. Compared with 3 mL/min and 5 mL/min, 7 mL/min can carry away reaction products and shedding particles more promptly, but the improvement in material removal is limited.

3.4 The Effect of B-Axis Swing Speed on The Forming Accuracy of Asymmetric Cutting Edges

The B-axis swing speed determines the contact trajectory distribution and local action time of the blade edge on the polishing disk and is a crucial factor affecting the forming accuracy of the asymmetric edge. To analyze its effect on the co-blunting effect of the straight edge and the arc edge, the edge forming accuracy at different B-axis swing speed levels was compared, as shown in Figure 9.

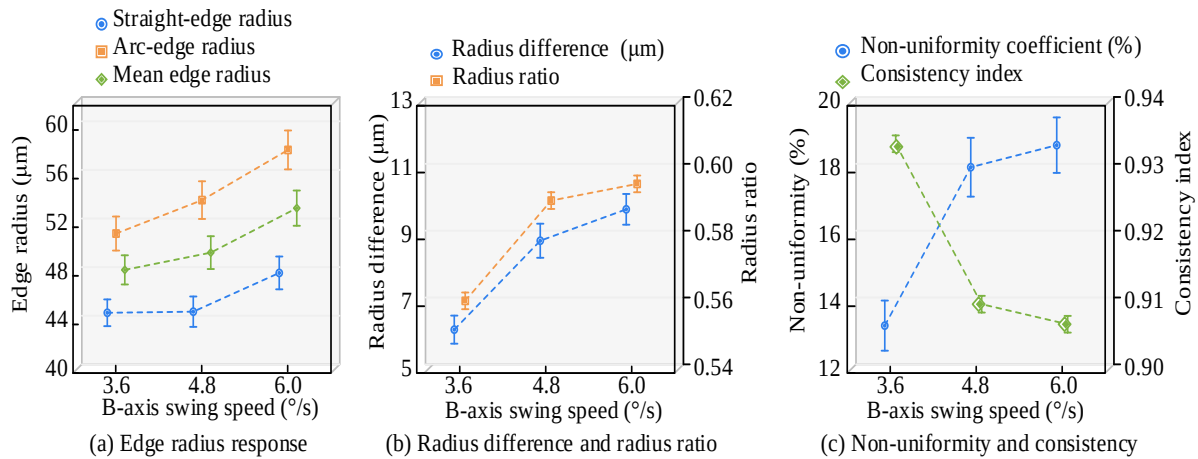


Fig. 9. Effect of B-axis swing speed on asymmetric edge forming accuracy

From Figure 9(a), as the B-axis swing speed increases from 3.60°/s to 6.00°/s, the straight cutting edge, arc cutting edge and average cutting-edge radius all show an increasing trend. The average edge radius increased from 48.31 μm to 53.39 μm. With the increase in swing speed, the edge contact trajectory coverage and local repeated contact effect were enhanced. From Figure 9(b), the radius difference increased with the B-axis swing speed. According to Equation (6), a higher swing speed shortens the equivalent contact time during a single pass but increases the frequency of repeated contact and trajectory overlap within the fixed processing period. Owing to its continuous curvature and longer effective contact path, the arc edge experiences greater cumulative contact exposure, resulting in disproportionate material removal and increased forming asymmetry. In Figure 9(c), the regional non-uniformity coefficient increased from 13.46% to 18.86%, while the form consistency index decreased from 0.933 to 0.906. It shows that an excessively high B-axis swing speed reduces the form consistency between the straight edge and the arc edge. Overall, the B-axis swing speed has a significant regulating effect on the forming accuracy of asymmetric cutting edges. A lower swing speed is beneficial for maintaining regional consistency, while a higher swing speed is beneficial for reinforcing material removal and edge rounding.

Finally, to further determine the relative influence of different CMP process parameters on the edge passivation response, a multi-dimensional sensitivity evaluation form was constructed. Material removal sensitivity was normalized based on the average edge radius, stability adjustment sensitivity was normalized based on the variation range of the coefficient of variation of average edge radius, and form consistency sensitivity was normalized based on the variation range of the form consistency index. The comprehensive sensitivity score is calculated by weighting three types of normalized indicators, with the weights of material removal sensitivity, stability adjustment sensitivity, and form consistency sensitivity being 0.50, 0.20, and 0.30, as presented in Table 4.

In Table 4, the combined sensitivity score of the rotational speed of the trajectory was the highest, at 0.894, indicating that it has the strongest influence on the CMP passivation process. The material removal sensitivity score reached 1.000. The rotational speed mainly enhances the removal effect of abrasive particles on the softening reaction layer by increasing the relative motion speed between the edge and the polishing disk, and is the dominant parameter controlling the edge roundness. The combined sensitivity score of the B-axis swing speed was 0.616, ranking second. Although its material removal sensitivity was lower than that of the polishing disk speed, it had the highest forming consistency sensitivity score. This indicates that the parameter mainly affects the

distribution of the edge contact trajectory and the forming differences in different edge areas. Therefore, the B-axis swing speed is more suitable as a key parameter for adjusting the forming accuracy and regional consistency of asymmetric cutting edges. The combined sensitivity score of the slurry flow rate is the lowest at 0.318, indicating a weaker direct impact on the amount of material removed from the cutting edge. However, its stability regulation score still reached 0.777, indicating that the flow rate mainly affects the stability of the passivation process by improving the state of polishing fluid renewal, abrasive transport and reaction product discharge. Overall, the three types of parameters have different effects: the PDRS dominates material removal, the B-axis swing speed regulates forming consistency, and the slurry flow rate mainly improves passivation stability.

Table 4. Comprehensive sensitivity evaluation of CMP process parameters

Process factor	Material removal sensitivity score	Stability regulation score	Forming consistency sensitivity score	Comprehensive sensitivity score	Sensitivity rank	Main action characteristic	Practical controllability
Polishing disk speed	1.000	1.000	0.648	0.894	1	Dominant material removal	High-direct regulation of material removal
Slurry flow rate	0.201	0.777	0.206	0.318	3	Slurry renewal and transport stability	Moderate-mainly regulates slurry transport stability
B-axis swing speed	0.271	0.900	1.000	0.616	2	Contact trajectory and regional consistency	High-regulates contact trajectory and forming consistency

Note: Sensitivity scores were normalized to 0-1 using Min-Max normalization. A higher comprehensive score indicates greater process sensitivity. Controllability represents the practical adjustability of each factor and its dominant response.

4. Discussion

PDRS produces the largest change in edge radius among the three investigated parameters. This result can be explained by the Preston equation, because increasing the disk speed directly raises the relative velocity between the polishing disk and the cutting edge. Therefore, the softened surface layer is removed more rapidly. The morphological changes in Figure 7 support this explanation. At 30 rpm, scratches, pits and local chipping are still observed, whereas the edge obtained at 90 rpm is smoother and more continuous. The decreases in Ra, Rz, RD, DD and DAR indicate that removing surface peaks and defects is more effective. These observations agree with the chemical-mechanical balance described in Figure 3. However, as the reaction layer is not directly measured, the morphological results should be regarded as indirect evidence of this mechanism.

Similar effects of chemical oxidation and abrasive removal have been reported in previous CMP studies [7,10]. Wang et al. [10] found that an oxidizer promoted the formation of a loose reaction layer on cemented carbide and facilitated its subsequent removal. Studies using abrasive water jets and magnetic-field-assisted polishing also demonstrate that abrasive transport and mechanical action strongly affect carbide-edge formation [8,13]. Most of these studies evaluated surface roughness or overall passivation performance. In contrast, the present results distinguish between straight and arc edges and show that the same process parameters do not produce equal material removal in the two regions.

The arc-edge radius is 6.28-11.22 μm larger than the straight-edge radius. This difference indicates that edge formation is not completely uniform. The curved region has a different contact area and contact path, which changes its local pressure and cumulative removal response. In addition, Equation (6) shows that the B-axis swing speed affects the equivalent contact time. When the swing

speed is too high, repeated contact and trajectory overlap become more pronounced in the arc region, increasing its removal relative to the straight region. A large radius difference may lead to uneven cutting loads and variations in local edge strength or coating thickness. Cutting and coating tests were not conducted in this study; therefore, 11.22 μm cannot yet be defined as a general acceptance limit. However, it can be used to identify parameter combinations with poor regional consistency. A low-to-moderate B-axis swing speed is more suitable when reducing the difference between the two edge regions is the main requirement.

The comprehensive sensitivity ranking is PDRS (0.894), B-axis swing speed (0.616) and slurry flow rate (0.318). This ranking also suggests a practical adjustment sequence. PDRS can first be used to bring the edge radius close to its target value. The B-axis swing speed can then be fine-tuned to reduce the forming difference between straight and arc regions. The slurry flow rate mainly maintains abrasive supply and removes reaction products, and therefore acts as a stability parameter rather than the primary means of controlling edge size. Within the tested range, a medium-to-high PDRS, a flow rate of 7 mL/min and a low-to-moderate B-axis swing speed provide a more balanced combination of material removal, surface quality and regional consistency.

5. Conclusion

This study focuses on the parameter sensitivity issues in the CMP passivation process of cemented carbide tool edges, systematically investigating the effects of PDRS, slurry flow rate, and B-axis swing speed on the edge radius response, regional removal uniformity, and forming consistency of asymmetric cutting edges (straight-edge and arc-edge regions), while quantitatively characterizing the governing laws and primary-secondary relationships among the process parameters. Orthogonal experiments showed that both the straight edge and the arc edge formed a distinct blunted round edge, and the radius of the arc edge was generally larger than that of the straight edge, with a difference of 6.28-11.22 μm . The rotational speed is the dominant factor affecting the strength of material removal. When the speed increased from 30 rpm to 90 rpm, the average edge radius range of the straight edge and the arc edge was 18.72 μm . The combined sensitivity ranking was: polishing disk speed > B-axis swing speed > slurry flow rate. The slurry flow rate has a weak direct effect on the cutting-edge radius, but it can improve the stability of abrasive transport, reaction product discharge and polishing fluid renewal. The B-axis swing speed mainly regulates the contact trajectory distribution. Although a higher swing speed can increase the passivation, it will increase the forming difference between the straight edge and the arc edge.

However, only one cemented-carbide grade is examined. Variations in cobalt content and WC grain size may alter the electrochemical interaction between the slurry and binder phase, thereby affecting reaction-layer formation and removal. Based on the main-effect results, a PDRS of 90 rpm, a slurry flow rate of 7 mL/min and a B-axis swing speed of 3.6°/s are recommended for balancing surface quality and edge uniformity. As this combination is inferred from the orthogonal results, further confirmation through cutting, coating-adhesion and tool-life tests is required.

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Enhancing open capacity in active distribution networks via GBD-based network reconfiguration considering PV integration constraints

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Abstract

As the penetration rate of distributed photovoltaic (PV) systems continues to rise, the power fluctuation, reverse power flow, and node voltage constraint challenges within active distribution networks have increased significantly. This makes traditional open capacity assessment methods—based on fixed operating conditions—less effective at accurately capturing the dynamic impact of PV integration on the network's load-bearing capacity, thereby presenting new challenges for both open capacity assessment and enhancement efforts. To address this challenge, this study proposes a proactive distribution network openable capacity model that incorporates constraints arising from distributed photovoltaic (PV) integration. The model integrates multiple components, including node power flow, voltage deviation, line capacity, reverse power flow, and N–1 security constraint. Building upon this framework, the proposed approach employs the generalized Benders decomposition method (GBD) combined with the feasible cut generation method (FCG) to solve the constructed hybrid integer nonlinear optimization model. Specifically, GBD is utilized to decompose the complex optimization problem, while FCG reduces the search space by screening infeasible regions and generating effective constraints, thereby enhancing the computational efficiency of the model. Experimental results from the IEEE 33-node active distribution network test system demonstrate that the maximum open capacity reaches 7.8 MW, with an 18.2% increase after optimization. The average and peak load rates decrease from 67.2% to 43.6% and from 89.4% to 54.8%, respectively. Under extreme load scenarios, the maximum open capacity reaches 10.7 MW with a calculation time of 10.8 seconds. The proposed method improves capacity assessment and reconfiguration performance, supporting photovoltaic integration and safe distribution network operation.

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1. Introduction

The high penetration of distributed photovoltaic (PV) systems has altered the unidirectional power flow characteristics of traditional distribution networks, gradually transforming them from unidirectional power supply networks into active distribution networks characterized by coordinated interaction among generation, grid, and load. Under conditions of high PV penetration, fluctuations in PV output exacerbate the occurrence of bidirectional power flows within distribution networks, thereby giving rise to additional challenges such as node voltage deviations,

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increased line capacity constraints, and difficulties in accurately assessing available open capacity. Open capacity, as an important indicator of the distribution network's ability to accommodate new loads or distributed power sources, is directly related to photovoltaic grid connection planning, user installation review, and distribution network expansion decisions [1-2]. The concept of Open Capacity is related to Hosting Capacity, as both describe a distribution network's ability to accommodate additional distributed energy resources under operational constraints. However, while Hosting Capacity generally refers to the maximum DG integration level, this study defines Open Capacity as the remaining margin available for new PV integration under the current topology, security constraints, and operating conditions, excluding load-bearing capacity. In high-PV-penetration scenarios, open capacity is jointly constrained by transformer capacity, line thermal limits, PV output fluctuations, voltage deviations, reverse power flow, and network topology. Increased PV output changes power flow distribution and may cause local nodes to export power, leading to voltage rise and flow reversal. As PV penetration further increases, voltage limits may become the dominant constraint, thereby restricting additional PV integration and open-capacity enhancement. Therefore, constructing an open capacity assessment and optimization reconfiguration method oriented to distributed photovoltaic access constraints is of great significance for enhancing photovoltaic consumption capacity and operational safety. For example, Othman et al. constructed a distributed power access capacity assessment model considering node voltage constraints and verified the limiting effect of voltage margin on access capacity through distribution network examples [3]. Lu et al. proposed a calculation method for photovoltaic acceptance capacity based on probabilistic power flow, introducing light randomness and load uncertainty into the capacity assessment process. This method could reduce the risk of capacity overestimation brought by the static assessment model [4]. Tian et al. used sensitivity analysis to identify key nodes for photovoltaic access and established an access capacity optimization model oriented to line capacity constraints. This method could improve the interpretability of node access schemes [5]. Liu et al. considered reverse power flow and voltage over-limit problems and proposed a multi-constraint capacity assessment framework, which demonstrated good operational safety in multiple test systems [6]. Yang et al. analyzed node voltage deviations under different photovoltaic penetration rates in combination with actual distribution network operation data. Photovoltaic access location had a significant impact on open capacity [7]. However, existing studies on open capacity assessment primarily focus on static constraints such as node voltages, line capacities, and uncertainties in photovoltaic (PV) output; these studies typically perform capacity calculations based on fixed network topologies and fail to adequately account for the role of dynamic topology adjustments within active distribution networks in enhancing capacity. Consequently, in scenarios featuring high penetration levels of distributed PV generation, relying solely on static assessment methods is insufficient to further explore the potential capacity enhancement achievable through network reconfiguration.

With the development of active distribution network operation control technology, network reconfiguration has gradually become an important means to increase open capacity and improve the operation status of distribution networks [8]. Elumalai et al. employed the particle swarm optimization algorithm to optimize the switching states of distribution networks incorporating distributed generation; the results demonstrated that this method can reduce line losses, but is prone to convergence fluctuations under complex constraints [9]. The generalized Benders decomposition (GBD) approach adopted in this paper decomposes the discrete topology decision-making and continuous power flow constraints through iterative interaction between the master problem and subproblems; it utilizes the objective value of the master problem to establish a lower bound and employs the feedback results from the subproblems to form an upper bound, thereby assessing the optimization process via the gradual convergence of these bounds, thus avoiding the convergence instability issues inherent in stochastic search from a mathematical perspective. Oni et al. proposed a distribution network reconfiguration model based on mixed integer second-order cone programming, which convexified the power flow constraints and improves the solvable model [10]. Chowdhury et al. used Generalized Benders Decomposition (GBD) to handle the coupling problem of switching variables and continuous power flow variables, and reduced the difficulty of solving the mixed-integer nonlinear model through master-slave problem interaction [11]. Khodr et al. proposed an optimal power flow (OPF)-based distribution network reconfiguration method

that incorporates Benders decomposition into the network reconfiguration solution process; by leveraging information exchange between the master problem and subproblems to handle the coupling relationship between discrete switch variables and continuous power flow variables, this approach enhances the computational efficiency of solving complex distribution network optimization problems [12]. Tavagnutti et al. combined distributed photovoltaic output prediction with topology adjustment strategies to verify the role of network reconfiguration in enhancing photovoltaic consumption capacity and reducing operational risks [13]. In summary, existing methods for open capacity assessment and active distribution network reconfiguration still exhibit certain limitations when addressing scenarios with high penetration of distributed photovoltaic (PV) generation. On one hand, some capacity assessment models only consider static loads or a single voltage boundary, making it difficult for them to simultaneously reflect the comprehensive impact of PV output fluctuations, line capacity constraints, and reverse power flow on the available open capacity; on the other hand, existing network reconfiguration methods typically prioritize optimization objectives such as reducing network losses, improving load balancing, or enhancing operational economic efficiency, focusing more on optimizing the current operating state rather than setting increasing the acceptance capability of distributed PV generation or expanding the available open capacity as their core objective. Therefore, how to achieve synergy between capacity assessment and capacity enhancement through network topology optimization remains an urgent challenge for active distribution networks with high PV penetration.

To address the aforementioned issues, this paper proposes an active distribution network-based method for evaluating openable capacity and performing optimal topology reconstruction that incorporates constraints arising from distributed photovoltaic (PV) integration. Unlike traditional capacity evaluation methods based on fixed network topologies, this approach treats the network topology as an adjustable variable, thereby extending the openable capacity evaluation process from static boundary calculations to a dynamic capacity optimization problem oriented toward active network reconstruction. At the solution level, the Generalized Benders Decomposition (GBD) method is employed to decompose the discrete switch decisions and the continuous nonlinear power flow constraints; furthermore, a Feasible Cut Generation (FCG) mechanism is introduced to linearize the voltage violation and line overloading information detected in the subproblems into feasible cuts and feed these back to the master problem, thereby actively eliminating infeasible topology regions. This framework establishes a closed-loop solution mechanism comprising "capacity evaluation – topology optimization – power flow verification – feasible region correction," which enhances the capacity for integrating new PV generation while reducing ineffective search processes. The main contributions of this paper are as follows:

- Develop an open-capacity assessment model that accounts for distributed photovoltaic (PV) output fluctuations, voltage constraints, line capacity constraints, and the impact of reverse power flow, thereby enhancing the accuracy of capacity assessment in scenarios with high PV penetration.
- Establish an active distribution network optimization and reconfiguration model oriented toward capacity enhancement, which integrates the maximization of available open capacity with the minimization of system losses within a unified optimization framework, and employs a weighting mechanism to reconcile the conflicts between these objectives and operational economic efficiency.
- This paper proposes a collaborative solution method based on Generalized Benders Decomposition (GBD) and the Feasible Cut Generation (FCG) mechanism. Within this framework, the FCG mechanism generates valid cut constraints based on infeasibility information such as voltage violations and line overloads, actively excludes invalid search regions, and shrinks the search space of the optimization problem, thereby enhancing the solution efficiency for complex mixed-integer nonlinear models.
- The effectiveness of the proposed method is verified using the IEEE 33-node active distribution network test system, and its overall performance in enhancing the integration capability of distributed photovoltaics, reducing operational losses, and ensuring system security is analyzed.

2. Materials and Methods

When distributed photovoltaic (PV) systems are integrated at a high penetration level, the power flow direction of the distribution network, the node voltage distribution, and the line load conditions will all undergo dynamic adjustments in response to changes in the network topology. The issue of increasing available capacity is not only influenced by electrical operational constraints but also exhibits significant complexity due to the diversity of possible network topologies. Therefore, it is necessary to integrate network reconfiguration strategies with capacity assessment to achieve synergy between topology optimization and the enhancement of PV integration capability.

2.1 Open Capacity Modeling for Active Distribution Networks Considering Distributed Photovoltaic Access Constraints

The large-scale access of distributed photovoltaic has gradually shifted the active distribution network from the traditional unidirectional power supply mode to the source-grid-load coordinated operation mode. Due to the randomness, volatility, and uneven spatiotemporal distribution of photovoltaic output, when a large number of photovoltaic cells are connected to the same area, it is easy to cause problems such as node voltage over-limit, line reverse power flow and local equipment overload, thereby causing large errors in the assessment results of open capacity based on static load growth [14-15]. Therefore, the study first constructs the distributed photovoltaic access structure model of the active distribution network, as illustrated in Figure 1.

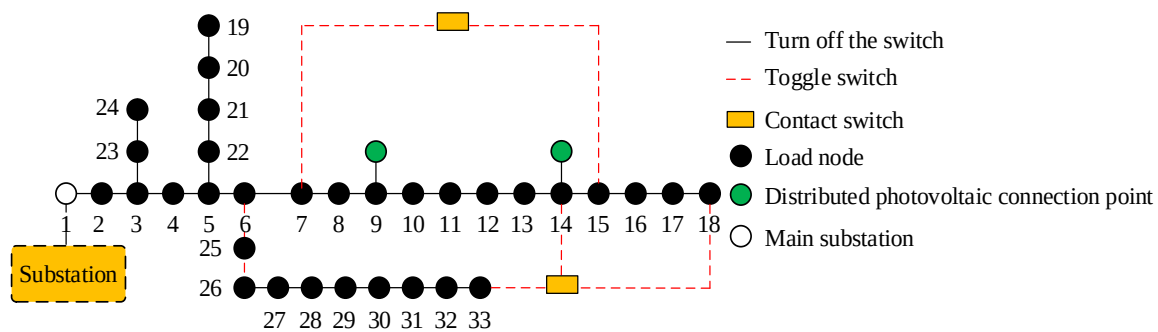


Fig. 1. Integration architecture of IEEE 33 active distribution network with distributed photovoltaic systems

As shown in Figure 1, the substation serves as the main power source supplying power to various feeder lines, with multiple photovoltaic (PV) units connected to different nodes to form a multi-source power supply structure. Given that open capacity assessment focuses on determining the maximum additional PV capacity that the system can accommodate under safety constraints, this study employs the PV peak generation and the most adverse operating conditions under low-load scenarios for analysis. When the PV output reaches its peak value, power backflow may occur in certain areas, leading to an increase in node voltage; simultaneously, under low-load conditions, the proportion of PV output further increases, making the risks of reverse power flow and voltage exceedance more pronounced. When the PV output decreases, the system still relies on the main substation to maintain power balance [16-17]. Therefore, the available open capacity is not only related to the level of load growth but is also jointly influenced by the scale of PV integration, operating conditions, and the connection location. To quantify the impact of PV integration on the node power distribution, this study defines the node net injection power as shown in Equation (1). Here, the research object is an active distribution network incorporating distributed PV resources, without considering other controllable distributed energy sources such as energy storage or batteries; consequently, the node power balance model includes only three primary power variables: the power supplied by the substation, the PV output power, and the node load power. If other distributed energy sources are present, their output power can be uniformly incorporated into the distributed energy injection term.

$$P_i^G + P_i^{PV} - P_i^L = V_i \sum_{j=1}^N V_j (G_{ij} \cos \theta_{ij} + B_{ij} \sin \theta_{ij}) \quad (1)$$

In Equation (1), P_i^G represents the active power supply provided by the substation to node i ; P_i^{PV} represents the active power injected by distributed photovoltaic systems at the node; P_i^L represents the power consumed by the node load. V_i and V_j signify node voltage amplitudes. G_{ij} and B_{ij} signify the conductance and admittance of the node admittance matrix. θ_{ij} represents the phase angle difference of the node. This analysis does not further consider other distributed energy sources such as energy storage; their role may be expanded to include the distributed power injection component based on actual requirements. The corresponding reactive power balance relationship is represented in Equation (2).

$$Q_i^G + Q_i^{PV} - Q_i^L = V_i \sum_{j=1}^N V_j (G_{ij} \sin \theta_{ij} + B_{ij} \cos \theta_{ij}) \quad (2)$$

In Equation (2), Q_i^G , Q_i^{PV} , and Q_i^L respectively represent the reactive power support power of the substation, the reactive power injection power of the photovoltaic system, and the reactive power demand of the node load. Since this paper mainly focuses on the impact of distributed photovoltaic integration on the open capacity of the active distribution network, no additional adjustable reactive power equipment such as energy storage inverters are introduced. Moreover, in order to quantify the acceptance capacity of the active distribution network for the newly added distributed photovoltaic under the safety operation constraints, the study introduces the photovoltaic integration capacity growth coefficient λ , and the objective function is expressed as Equation (3).

$$\max F = \lambda \sum_{i=1}^N P_i^{pv} \quad (3)$$

In Equation (3), F represents the maximum openable capacity of the system, which is the additional distributed photovoltaic capacity that the active distribution network can further accommodate under the constraints of node voltage, line transmission, and N-1 safety; λ represents the load growth coefficient. This paper focuses on the capacity improvement issue under the condition of distributed photovoltaic grid connection, so the increase in new load is not taken as the object for defining the openable capacity. After completing the node power flow modeling, the study constructs an open capacity assessment model considering distributed photovoltaic access constraints, whose structure is shown in Figure 2.

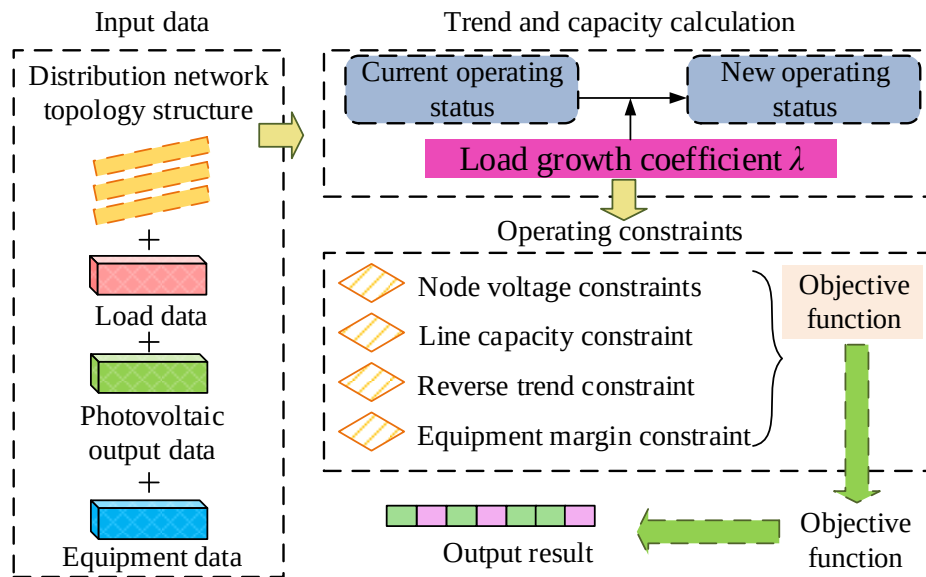


Fig. 2. Open capacity assessment model

In Figure 2, the open capacity is mainly constrained by four parts: photovoltaic access capacity, voltage regulation capacity, line transmission capacity and equipment operating margin. When the photovoltaic access capacity exceeds a certain critical value, the system will first be restricted by voltage constraints and then by line capacity constraints. To ensure the operational safety of nodes, this study investigates the development of a node voltage constraint model. Given that the scale of photovoltaic (PV) integration, network topology, and operating conditions in an active distribution network all influence the voltage response characteristics of nodes, this paper employs a sensitivity analysis method based on the Jacobian matrix to compute the relationship between node voltages and variations in PV output; furthermore, during each optimization iteration, the sensitivity matrix is updated according to the current operating conditions to enhance the adaptability of the constraint model for large-scale network scenarios. The corresponding expression is presented in Equation (4).

$$V_i^{\min} \leq V_i + \frac{\partial V_i}{\partial P_i^{PV}} \Delta P_i^{PV} \leq V_i^{\max} \quad (4)$$

In Equation (4), $\frac{\partial V_i}{\partial P_i^{PV}}$ represents the sensitivity coefficient of the node voltage to the variation of photovoltaic output. ΔP_i^{PV} represents the increment of photovoltaic output. $V_i^{\min}$ and $V_i^{\max}$ signify the upper and lower limits of voltage. Since this constraint model is based on the local linearization assumption, it introduces a certain degree of linearization error. Therefore, during the subsequent optimization and reconstruction process, the study employs nonlinear power flow calculations within the GBD subproblem to precisely validate the output results of the master problem, and utilizes FCG to generate corrected cutting constraints based on voltage limit violations, thereby gradually eliminating the infeasible region caused by the linearization error. Meanwhile, to avoid line overload, the line capacity constraint is shown in Equation (5).

$$\sqrt{(P_{ij})^2 + (Q_{ij})^2} \leq S_{ij}^{\max} \quad (5)$$

In Equation (5), P_{ij} and Q_{ij} signify the active power and reactive power of the line, respectively. $S_{ij}^{\max}$ represents the maximum allowable transmission capacity of the line. This constraint can ensure that the line operation is always within the thermal stability range. Based on the above, the voltage over-limit and reverse power flow safety constraint model is constructed, and the relationship is shown in Figure 3.

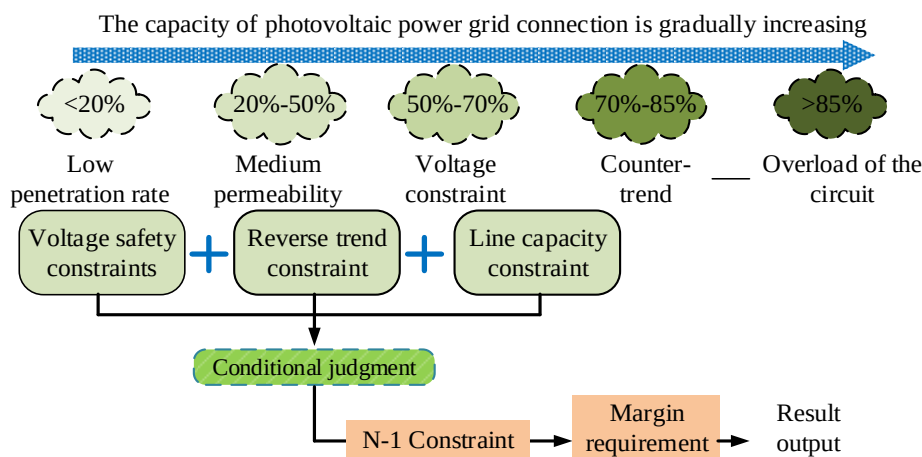


Fig. 3. Voltage and reverse power flow safety constraint model

In Figure 3, when the photovoltaic access capacity gradually increases, the system is first constrained by the node voltage. As the photovoltaic capacity continues to increase, some feeders start to show reverse power flow. If the scale of access is further increased, the line load rate and the safety margin for equipment operation will continue to decline. Therefore, considering only load growth capacity while ignoring network security constraints will result in significantly larger

capacity assessment outcomes. To ensure that the system can still operate stably in the event of a failure, the study further introduces N-1 security constraints as shown in Equation (6) [18-19]. In this study, to account for the changes in the system's operating conditions resulting from the outage of different faulty lines or equipment, normalized indices are employed to describe the power flow variations and node voltage deviations under fault conditions; specifically, the rated transmission capacity and rated voltage of each line are used as the respective normalization bases, thereby eliminating the influence arising from differences in line capacity ratings and node voltage levels.

$$\max \left(\frac{|S_{ij}^k|}{S_{ij}^{\max}} \right) \cup \max \left(\frac{|V_i^k - V_i^{\text{ref}}|}{V_i^{\text{ref}}} \right) \quad (6)$$

In Equation (6), S_{ij}^k represents the apparent power flow of line $i - j$ under the k th fault scenario; $S_{ij}^{\max}$ represents the maximum transmission capacity of line $i - j$, which is used for normalization processing of the line flow in the fault state; V_i^k represents the voltage amplitude of node i under the k th fault scenario; V_i^{ref} represents the rated voltage of node i , which is used for normalization processing of the node voltage deviation; ε represents the system safety margin threshold, which is used to limit the line load level and node voltage deviation degree in the fault state. In this paper, $\varepsilon = 0.15$ is set, indicating that in the N-1 fault situation, the comprehensive normalized index of line flow and node voltage deviation does not exceed 15%, that is, the system still retains 85% of the safety operation margin, to ensure that the active distribution network has the stable operation capability after a single component fault.

2.2 Active Distribution Network Optimization Reconfiguration Methods for Available Integration Capacity Enhancement

After completing the open capacity modeling, the study further proposes optimization and reconfiguration methods for active distribution networks to enhance the open capacity and safety margin of the system under distributed photovoltaic access conditions. Due to the large number of switch, line and node constraints in active distribution networks, the system operation is highly nonlinear, and traditional single-step optimization is difficult to simultaneously satisfy the requirements of capacity maximization and safety stability [20]. The study synergistically optimizes capacity enhancement and network security by constructing an active distribution network optimization reconfiguration model, introducing distributed photovoltaic access constraints, node voltage constraints and line load constraints. The overall process is shown in Figure 4.

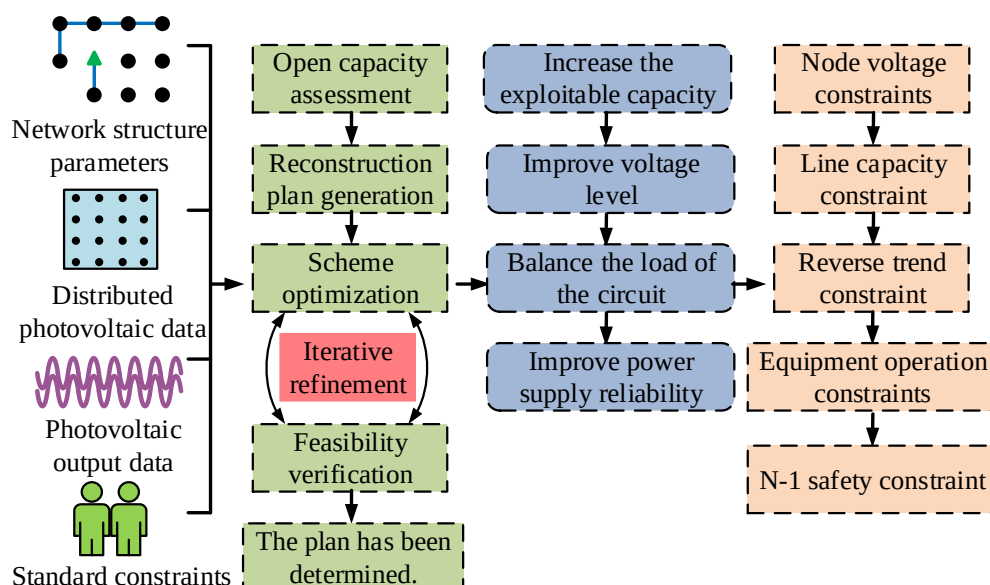


Fig. 4. Framework for active distribution network optimization and reconstruction

In Figure 4, the system first builds the initial network topology given photovoltaic access capacity and node load, then calculates the upper limit of open capacity through the optimization model, and adjusts the network switching state according to node voltage, line load and N-1 safety constraints to achieve network reconstruction. The study defines the optimization objective function as a combined objective of maximizing available integration capacity and minimizing system loss, and its calculation is shown in Equation (7). Within the GBD solution framework, Equation (7) serves as the optimization objective of the main problem, primarily responsible for determining the network switch states and topology configuration to enhance openable capacity; meanwhile, continuous constraints such as power flow calculations, voltage deviations, and line security verification are solved via subproblems, and feedback constraints are generated based on the results of these subproblems.

$$\max Z = \omega_1 \sum_{i=1}^N P_i^{PV} - \omega_2 \sum_{(i,j) \in L} R_{ij} I_{ij}^2 \quad (7)$$

In Equation (7), Z represents the comprehensive optimization objective. ω_1 and ω_2 are the capacity weight and loss weight coefficients, respectively. R_{ij} and I_{ij} are line resistance and line current, respectively. L is the collection of lines. Since capacity enhancement and loss reduction may conflict, this study adopts a normalized weighted objective. Capacity enhancement is assigned a higher weight as the primary goal, while network loss serves as an auxiliary economic objective. Sensitivity analysis is conducted for different weight combinations, and the combination with the best overall performance is selected for subsequent experiments.

To address this nonlinear mixed integer programming problem, this study introduces the GBD method, whose structure is illustrated in Figure 5 [21]. Compared with the commonly used Second-Order Cone Programming (SOCP) approach, the GBD method does not require an overall convexification of the original nonlinear power flow constraints; instead, it preserves the original physical characteristics of the voltage constraints, line power flow constraints, and N-1 security constraint in active distribution networks.

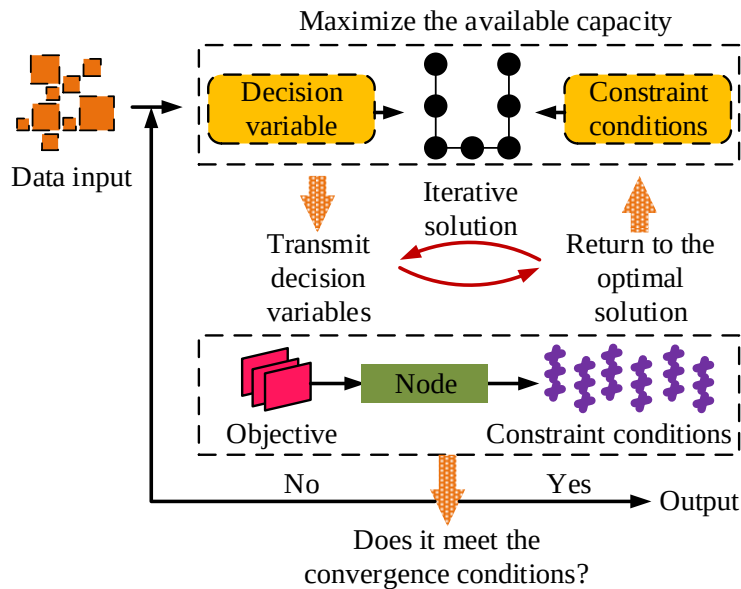


Fig. 5. Generalized benders master-subordinate coordination optimization structure

As shown in Figure 5, the main problem is responsible for optimizing the switching state of the network, generating preliminary solutions by relaxing capacity constraints and voltage constraints. The sub-problems are responsible for power flow calculation and feasibility check, calculating node voltage and line power flow based on the switch state output by the main problem, and generating feasible cut constraints to feed back to the main problem. Through the interaction and iteration of the master and sub-problems, the system can gradually increase the open capacity while meeting

the constraints. In the subproblem, the node voltage is calculated using a nonlinear power flow model, as shown in Equation (8). Given that active distribution networks typically maintain a radial power supply structure through the control of tie-switches during actual operation, this paper assumes that the optimized reconfigured network still satisfies the radial topology constraints. Under these conditions, the node voltage drop relationship can be described using a model analogous to the DistFlow model, thereby reducing the computational complexity of the power flow calculation and meeting the iterative solution requirements for the GBD subproblem.

$$V_i = V_j + \sum_{(i,j) \in L} (R_{ij}P_{ij} + X_{ij}Q_{ij}) \quad (8)$$

The line flow calculation is shown in Equation (9).

$$\theta_i - \theta_j = \arctan\left(\frac{X_{ij}P_{ij} - R_{ij}Q_{ij}}{V_i V_j}\right) \quad (9)$$

In Equation (9), θ_i and θ_j represent the phase angle of the node voltage. R_{ij} and X_{ij} represent line resistance and reactance, respectively. Since the research focus of this paper is on active distribution network reconfiguration scenarios, the network topology adjustment process must satisfy radial constraints; therefore, Equations (8) and (9) are applicable to the reconfigured radial operating structure. For mesh-type distribution systems that operate in closed-loop mode, a complete AC power flow model must be further employed for the calculations. To speed up the iterative convergence, the study introduces FCG into the main problem, and the process is shown in Figure 6.

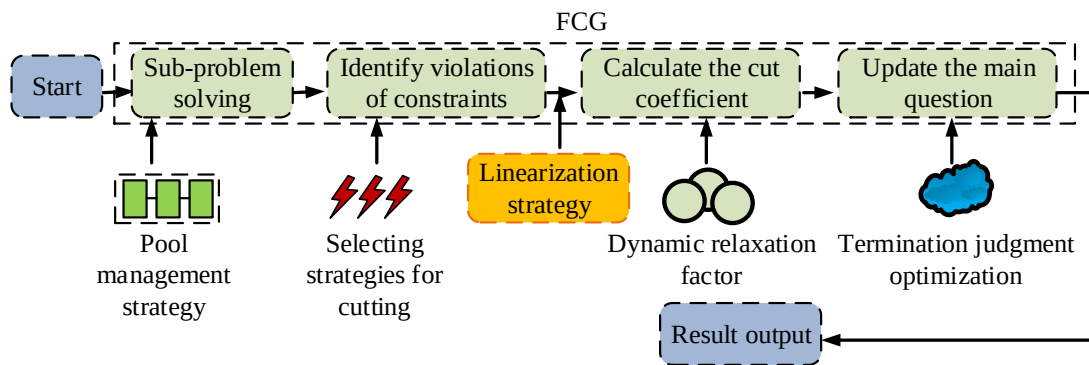


Fig. 6. Generation of feasible cuts and mechanisms for accelerated convergence

As shown in Figure 6, the feasible cut generation module checks the feasibility of the subproblem after each iteration. When constraints such as node voltage limits violation or line overloading are detected, the system linearizes the violated constraints based on the current iteration results and generates feasible cut constraints to be fed back to the GBD master problem, thereby eliminating the current infeasible search region and narrowing the subsequent optimization space. The pseudocode of Algorithm 1 is as follows:

- 1: Initialize iteration counter $k=0$;
- 2: Initialize switching variables $x_k=x_0$;
- 3: Set upper bound $UB=+\infty$ and lower bound $LB=-\infty$;
- 4: Initialize feasible cut set $\Omega=\emptyset$;
- 5: Construct the master problem:

$$\begin{aligned} &\max F(x) \\ &\text{subject to } \Omega \end{aligned}$$

```

6: while  $|UB-LB|/|UB| > \varepsilon$  and  $k < K_{max}$  do
7:   Solve the master problem and obtain the optimal
      switching state  $x_k$ ;
8:   Update the network topology according to  $x_k$ ;
9:   Calculate photovoltaic hosting capacity and
      power loss objective value;
10:  Update upper bound:
       $UB = \max\{UB, F(x_k)\}$ ;
11:  Construct the subproblem with fixed topology  $x_k$ ;
12:  Perform AC power flow calculation;
13:  Calculate node voltage  $U_i$  and line power flow  $S_{ij}$ ;
14:  if
       $U_{i,min} \leq U_i \leq U_{i,max}$ 
      and
       $S_{ij} \leq S_{ij,max}$ 
      and
      N-1 safety constraints are satisfied
      then
15:    Calculate subproblem objective value;
16:    Update lower bound:
       $LB = \max\{LB, F_{sub}(x_k)\}$ ;
17:    Record current optimal solution  $x^* = x_k$ ;
18:  else
19:    Identify violated constraints;
20:    Generate feasible cuts based on voltage
      violation and line overload:
       $g(x_k) + \nabla g(x_k)(x - x_k) \leq 0$ 
21:    Add generated cuts into master problem:
       $\Omega = \Omega \cup \{g_{cut}\}$ ;
22:  end if
23:  Update iteration counter:
       $k = k + 1$ ;

```

24: end while

25: Return x^* and Popen*;

Specifically, if in the k -th iteration the violation number of constraints detected by the sub-problem is $g(x)$, then the feasible cut constraint is expressed as shown in Equation (10).

$$g(x^k) + \nabla g(x^k)(x - x^k) \leq 0 \quad (10)$$

In Equation (10), x represents the decision variable for the network switch status in the main problem; x^k represents the switch status obtained in the k th iteration; $g(x^k)$ represents the degree of constraint violation of the sub-problem under the current iteration; $\nabla g(x^k)$ represents the first-order gradient information of the constraint function at the current iteration point. This feasible cut transforms the infeasible information such as voltage over-limit and line overload in the sub-problem into the constraints of the main problem through local linearization. This enables the GBD algorithm to gradually eliminate the infeasible topological regions and improve the search efficiency. In the final capacity enhancement phase, the study constructs an integrated open capacity optimization process, whose structure is shown in Figure 7.

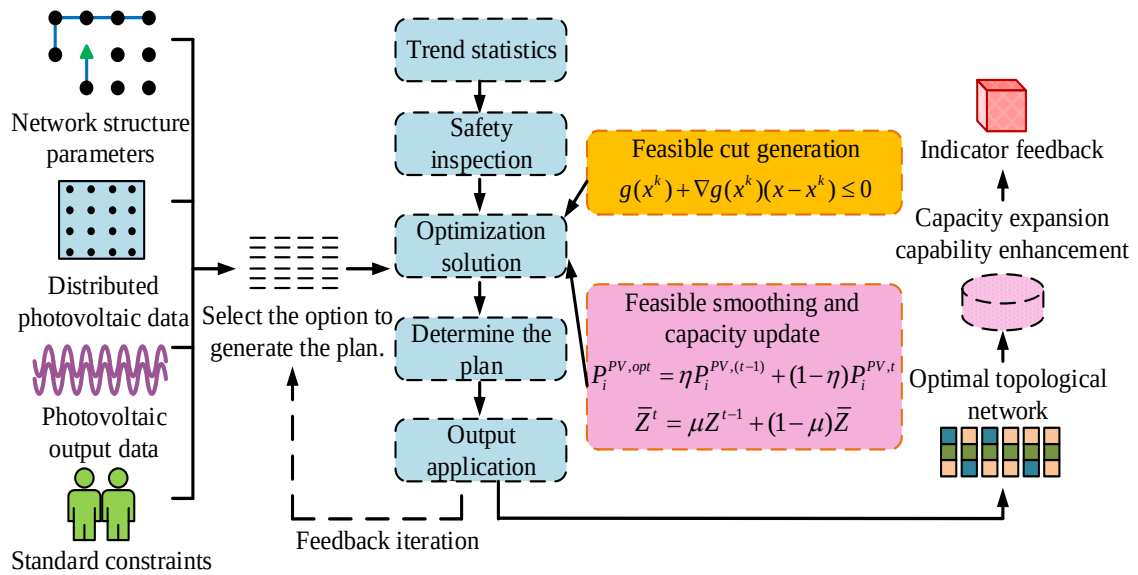


Fig. 7. Process for optimizing open capacity enhancement

In Figure 7, the system first calculates the upper limit of open capacity based on the initial network state, then iteratively adjusts the network switching state through GBD, while verifying the power flow of the sub-problem to ensure that the node voltage and line power flow meet the safety constraints. When the iteration converges, output the optimal network topology and the corresponding available integration capacity value. To mitigate significant fluctuations in capacity optimization results caused by network topology adjustments during the GBD iteration process, this study introduces a capacity sensitivity coefficient and an iterative smoothing coefficient to perform a weighted fusion of historical iteration results and the current optimization result; the corresponding calculation formula is given in Equation (11). In this formulation, the correlation coefficient remains constant throughout the optimization process, allowing the use of preset parameters to adjust the trade-off between historical information and the current search results, thereby ensuring stable convergence of the iterative process.

$$\begin{cases} P_i^{PV,opt} = \eta P_i^{PV,(t-1)} + (1 - \eta) P_i^{PV,(t)} \\ Z^{(t)} = \mu Z^{(t-1)} + (1 - \mu) Z^{(t)} \end{cases} \quad (11)$$

In Equation (11), $P_i^{PV,opt}$ represents the optimized photovoltaic access capacity. $Z^{(t)}$ represents the optimization target value for the t -th iteration. In this paper, both parameters are set as fixed values, and their appropriate values are determined during the experimental phase through a parameter sensitivity analysis. In addition, to ensure that the network reconstruction results take both capacity and security into account, the study constructs an overall constrained optimization function, as shown in Equation (12).

$$L_{opt} = \alpha L_{volt} + \beta L_{flow} + \gamma L_{loss} \quad (12)$$

In Equation (12), L_{opt} represents the comprehensive optimization loss. L_{volt} , L_{flow} , and L_{loss} represent node voltage deviation, line load deviation, and network power loss, respectively. α , β , and γ are the corresponding weight coefficients. This loss function serves as a safety evaluation indicator for the GBD subproblem, used to determine whether the topology solution generated by the main problem satisfies the operational constraints. When the subproblem detects a constraint violation, it generates a feasible cut via the FCG mechanism and feeds it back to the main problem, thereby guiding further optimization of the network topology.

3. Results and Analysis

3.1 Performance Testing of Open Capacity Modeling Methods for Active Distribution Networks

The experiment used the IEEE 33-Bus Distribution System dataset (IEEE-33) and the National Renewable Energy Laboratory Solar Dataset (NREL Solar) as test data sources. IEEE-33 is one of the most widely used standard distribution network test systems in the field of power systems, consisting of 33 nodes, 32 lines, and multiple load nodes. The NREL Solar dataset contains data on photovoltaic irradiance, meteorological parameters, and photovoltaic power generation in multiple regions, with time resolutions covering minute and hour levels. The source website for IEEE-33 is <https://matpower.org/docs/ref/matpower6.0/case33bw.html>; the source website for NREL Solar is <https://nsrdb.nrel.gov/>. All data undergoes standardization preprocessing and is divided into an optimization scenario set and a validation scenario set in a 8:2 ratio. Specifically, the optimization scenario set is used to construct typical operating scenarios under different photovoltaic penetration rates and load levels, while the validation scenario set is used to evaluate the stability of the proposed method's capacity assessment across various operating conditions. Table 1 presents the experimental environment and its parameter configuration.

Table 1. Experimental parameter settings

Parameter	Value
Operating System	Windows 11 64-bit
CPU	Intel Core i9-13900K
GPU	NVIDIA RTX 4090 24 GB
Memory	64 GB DDR5
Simulation Platform	MATLAB R2023b + MATPOWER 8.1
Test System	IEEE 33-bus Active Distribution Network
Photovoltaic Dataset	NREL Solar Dataset
Dataset Details	2022, Phoenix, Arizona, USA; Jan.–Dec.; 1 h resolution
Photovoltaic Penetration	10%–90%, step = 2%
Load Growth Coefficient	1.0–2.5, step = 0.1
Voltage Constraint Range	0.95–1.05 p.u.
Maximum Line Loading	100%
Security Constraint	N–1 Security Criterion
Security Margin Threshold	0.15
Smoothing Coefficient η	0.6
Iterative Coefficient μ	0.5
Number of Repeated Runs	20
Evaluation Method	Average of Multiple Runs
Optimization Solver	GBD-FCG
Convergence Threshold	10^{-5}
Maximum Iterations	200

To mitigate the impact of random operational errors on experimental results, all optimization methods were independently executed 20 times, with the average result used as the final evaluation metric. For the significance analysis of performance differences among different methods, this study utilized the openable capacity improvement rates obtained from these 20 independent runs to calculate the p-value using a two-sided independent sample t-test. When $p < 0.05$, it is concluded that the performance differences between the different methods are statistically significant. Based on two types of public datasets and the experimental settings, the sensitivity tests on key parameters were conducted for modeling the open capacity of active distribution networks. The results are shown in Figure 8.

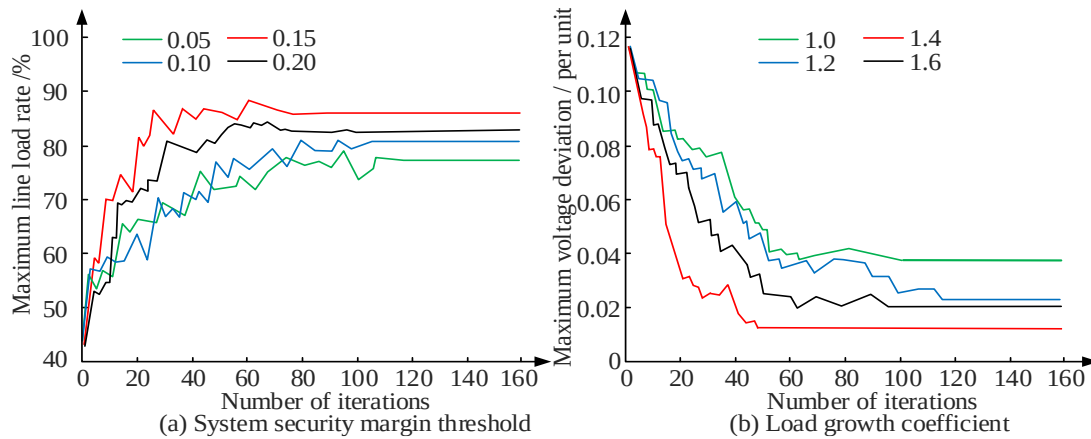


Fig. 8. Hyperparameter selection test result

Figures 8 (a) and (b) respectively show the selection test results of the system safety margin threshold ε and the load growth coefficient λ . As shown in Figure 8(a), when the system security margin threshold ε is set to 0.05, 0.10, 0.15, and 0.20 respectively, the maximum line load factor stabilizes at 77.8%, 81.6%, 86.4%, and 83.1%, respectively. It should be noted that the ε used in this paper does not represent the traditional safety margin ratio, but rather denotes the allowable range of normalized power flow deviation and voltage deviation under the N-1 fault constraint. When ε is small, the constraint conditions are more stringent, requiring the system to reserve greater operating margins, thereby limiting the available open capacity; as ε increases, the system permits a certain range of operational fluctuations, enabling the optimization model to further unleash the network's transmission capacity. However, when ε continues to increase, overly lenient security constraints may compromise the operational reliability under fault conditions; consequently, the trend of capacity enhancement gradually slows down. By comprehensively considering both the capacity enhancement capability and the safety operation requirements, this paper selects $\varepsilon = 0.15$ as the parameter for subsequent experiments. In Figure 8 (b), when the load growth coefficients were set to 1.0, 1.2, and 1.6, respectively, the maximum voltage deviation eventually stabilized at 0.038, 0.023, and 0.021 p.u. Only when the load growth factor was set at 1.4, the maximum voltage deviation reached a minimum of 0.013 p.u. and stabilized after 40 iterations, indicating that this parameter can more effectively coordinate the relationship between load growth and voltage constraints. As a result, the study ultimately set the system safety margin threshold and the load growth factor to 0.15 and 1.4, respectively.

To validate the effectiveness of the proposed method in open capacity assessment, this paper selects three typical capacity assessment methods as comparative benchmarks. Among these, the Static Capacity Method (SCM) is based on deterministic power flow analysis; it determines the maximum photovoltaic (PV) integration capacity by imposing node voltage and line capacity constraints, and thus belongs to the category of traditional deterministic capacity assessment methods; the Probabilistic Capacity Calculation Method (PCCM) accounts for the uncertainty associated with PV output and load variations; it obtains capacity assessment results at different confidence levels through probabilistic power flow analysis, thereby reflecting the impact of stochastic factors on grid integration capability [22]; and the Deterministic Voltage Margin Method (DVM) determines the capacity boundary by analyzing the relationship between node voltage

margin and PV integration level, focusing specifically on how voltage constraints limit the PV integration capacity [23]. By comparing these three methods with the proposed method, this study evaluates how the maximum openable capacity varies under different PV penetration rates. The results are shown in the Figure 9.

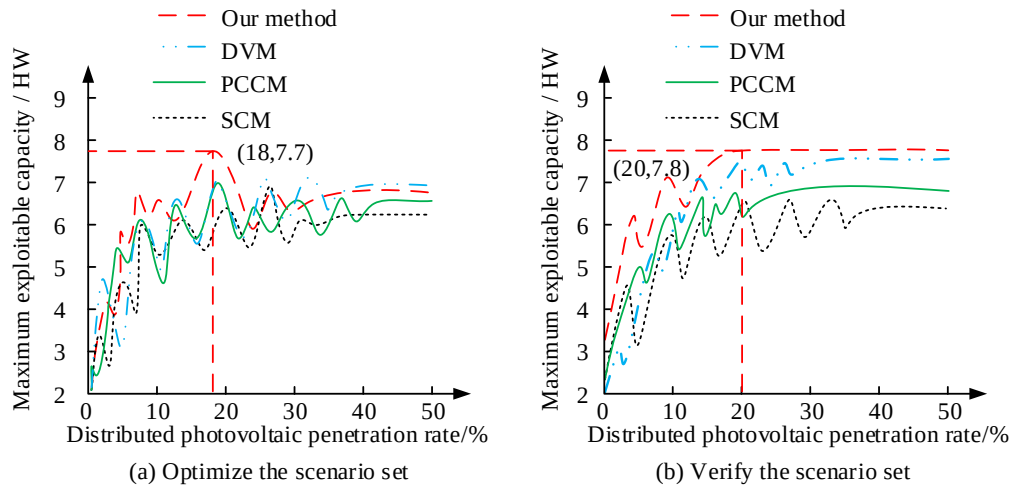


Fig. 9. Test results for the maximum developable capacity in both the training set and the test set

Figure 9(a) and Figure 9(b) respectively present the maximum available dispatchable capacity test results for each method across the optimization scenario set and the validation scenario set. As shown in Figure 9(a), the proposed method reaches a maximum open capacity of 7.7 MW at 18% PV penetration and maintains a relatively stable level thereafter. Unlike fixed-topology methods, GBD-based reconfiguration redistributes PV power among feeders, mitigating local voltage rise and reverse power flow. Figure 9(b) shows similar results for the validation scenarios, with a maximum capacity of 7.8 MW at 20% penetration, outperforming DVM, PCCM, and SCM. At higher penetration levels, capacity growth gradually saturates as line capacity and security constraints become dominant. To further analyze the impact of PV integration locations on node voltage stability, this study selects typical nodes from the IEEE 33-bus system for testing. Specifically, Node 6 serves as the near-end node, Node 18 as the central node, and Node 33 as the far-end node, representing PV integration scenarios at different electrical distances from the substation. The maximum voltage deviation, node voltage compliance rate, and number of nodes experiencing voltage violations for each method at different integration locations are presented in Table 2.

Table 2. Maximum voltage deviation results under different photovoltaic access nodes in IEEE 33-bus system

Photovoltaic access node	Method	Maximum voltage deviation / p.u.	Voltage pass rate / %	Voltage fluctuation / p.u.	Over-limit nodes
Node 6 (Near-end node)	SCM	0.043 ± 0.004	94.6 ± 1.3	0.028 ± 0.003	3
	PCCM	0.037 ± 0.003	96.2 ± 1.1	0.024 ± 0.002	2
	DVM	0.032 ± 0.003	97.4 ± 0.9	0.020 ± 0.002	1
	Our method	0.024 ± 0.002	99.1 ± 0.6	0.015 ± 0.001	0
Node 18 (Middle node)	SCM	0.051 ± 0.005	92.8 ± 1.6	0.034 ± 0.004	4
	PCCM	0.044 ± 0.004	95.3 ± 1.3	0.029 ± 0.003	3
	DVM	0.038 ± 0.003	96.8 ± 1.0	0.025 ± 0.002	2
	Our method	0.027 ± 0.002	98.7 ± 0.7	0.017 ± 0.002	0
Node 33 (Far-end node)	SCM	0.064 ± 0.006	89.5 ± 1.9	0.041 ± 0.004	6
	PCCM	0.056 ± 0.005	92.4 ± 1.6	0.036 ± 0.003	4
	DVM	0.049 ± 0.004	94.7 ± 1.3	0.031 ± 0.003	3
	Our method	0.033 ± 0.003	98.2 ± 0.8	0.021 ± 0.002	1

As shown in Table 2, for the IEEE 33-bus system, as the PV generation integration point moves from bus 6 to buses 18 and 33, the electrical distance between buses increases and the cumulative effect of line impedance intensifies, leading to an increase in voltage drop during power transmission; consequently, the maximum voltage deviation for all methods exhibits an upward trend. In the scenario where PV generation is integrated at the remote bus 33, the SCM method achieves a maximum voltage deviation of 0.064 p.u., with 6 nodes exceeding the voltage limit; in contrast, the proposed method reduces the maximum voltage deviation to 0.033 p.u. by optimizing the network topology and adjusting the power transmission path, while maintaining a voltage compliance rate of 98.2%, demonstrating that the proposed method can effectively mitigate voltage fluctuation issues induced by remote PV integration.

3.2 Verification Of Active Distribution Network Optimization and Reconfiguration Effects

The study took the IEEE 33-node active distribution network as the test object and compared the network operation status before and after reconfiguration under the distributed photovoltaic access constraints. The network topologies before and after reconfiguration were compared. The results are shown in Figure 10.

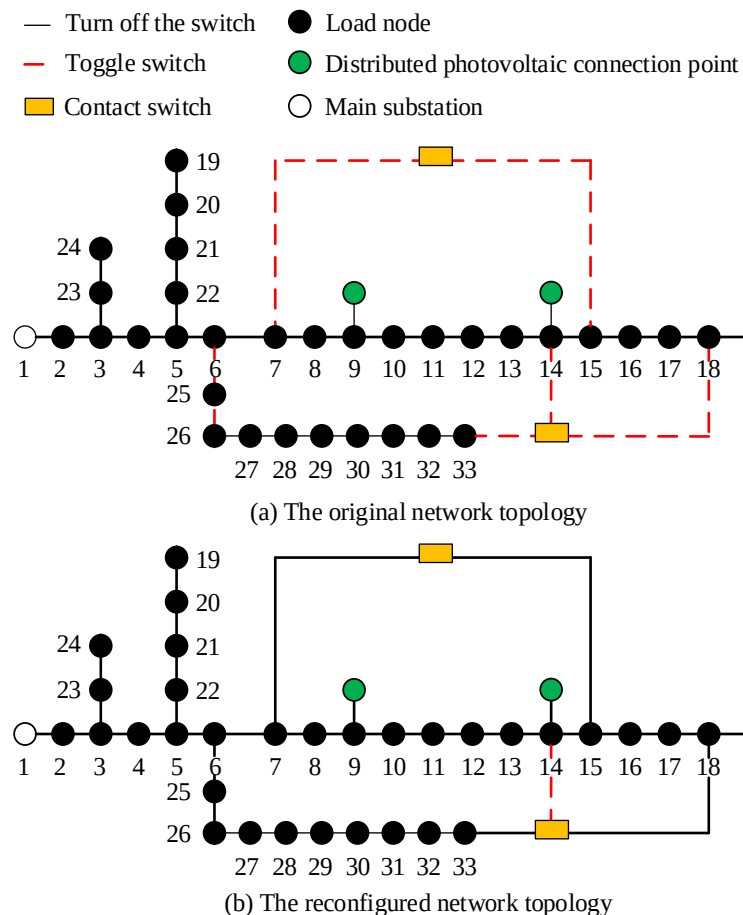


Fig. 10. Comparison results of network topology before and after reconstruction

Figures 10 (a) and (b) present the comparison results of the network topology before and after reconstruction. From Figure 10 (a), the network before reconfiguration was dominated by the original feeder structure, with the connection switch in the open state. The power transmission path between the distributed photovoltaic access point and the remote load node was long, and some branches needed to bear concentrated load transfer pressure, which easily caused problems such as uneven line load, low voltage at the end node, and local capacity limitation. As shown in Figure 10(b), the reconstructed system establishes new power flow transmission paths by closing Tie-switches 7–14 and Tie-switch 14–18, while simultaneously maintaining the radial operating

structure of the distribution network through feeder switch adjustments. Specifically, the switch adjustment strategy derived from the proposed GBD-FCG method is as follows: the set of newly closed switches is {Tie-switch 7–14, Tie-switch 14–18}, and the set of newly opened switches is {S7–8, S14–15}. Here, Tie-switches 7–14 and Tie-switch 14–18 transition from their original open state to closed state to establish new power flow transmission paths; meanwhile, S7–8 and S14–15 are adjusted from closed to open state to preserve the radial operating structure of the distribution network. Compared with the original topology, the new power supply path shortens the equivalent power supply distance for certain remote nodes, enabling the power generated at photovoltaic-connected nodes to be redistributed among multiple feeders, thereby reducing local line load pressure and improving node voltage levels. To maintain the radial operating structure of the distribution network, the algorithm synchronously adjusts the states of certain feeder switches, with the specific switch changes detailed in Table 3.

Table 3. Switch status changes during network reconfiguration in the IEEE 33-bus active distribution system

Switch No.	Switch Type	Initial State	Reconfigured State	Function of Topology Adjustment
Tie-switch 7–14	Tie switch	Open	Closed	Establishes a new power transfer path and improves power distribution among feeders
Tie-switch 14–18	Tie switch	Open	Closed	Enhances voltage support capability for downstream nodes
S7–8	Sectionalizing switch	Closed	Open	Reduces local power flow concentration and maintains radial operation
S14–15	Sectionalizing switch	Closed	Open	Optimizes feeder power distribution and alleviates branch loading pressure

This study selects the Mixed-Integer Second-Order Cone Programming (MISOCP) model, the Improved Particle Swarm Optimization (IPSO) algorithm, and GBD as comparative methods; these methods are then used in conjunction with the proposed method to compute the increase rate of the openable capacity after reconstruction compared to before reconstruction; the results are shown in Figure 11.

Figures 11 (a) - (d) present the comparison results of open capacity improvement rates under each optimization method. In Figure 11 (a), MISOCP increased by 8.4%, 9.5%, 11.2%, and 13.1% respectively in low penetration, medium penetration, high penetration, and reconfiguration scenarios, with a limited overall increase. In Figure 11 (b), IPSO reached 9.2%, 10.1%, 11.3% and 14.2% respectively in the four types of scenarios, which were higher than MISOCP, but the growth was not significant in the high penetration scenarios. As shown in Figure 11 (c), GBD had an improvement rate of 15.5% in the reconfiguration scenario, but only 7.8% and 8.2% in the low and medium penetration scenarios, indicating that it is more sensitive to the initial operating conditions. As shown in Figure 11(d), the proposed method achieves capacity improvements of 9.0%, 11.1%, 14.2%, and 18.2% across the four scenarios, respectively, with the most pronounced improvement observed in the reconstruction scenario. To validate the differences in capacity improvement results among various optimization methods, this study employs a one-way analysis of variance (one-way ANOVA) based on 20 independent replicate runs to statistically test the capacity improvement rates of different methods; the results indicate a significant difference between the proposed method and the other comparison methods ($p = 0.02$). The study continued to analyze the operational status of the distribution network before and after optimization and reconfiguration, with a focus on comparing the changes in line load distribution, as shown in Figure 12. Here, the horizontal axis represents the algorithm's runtime.

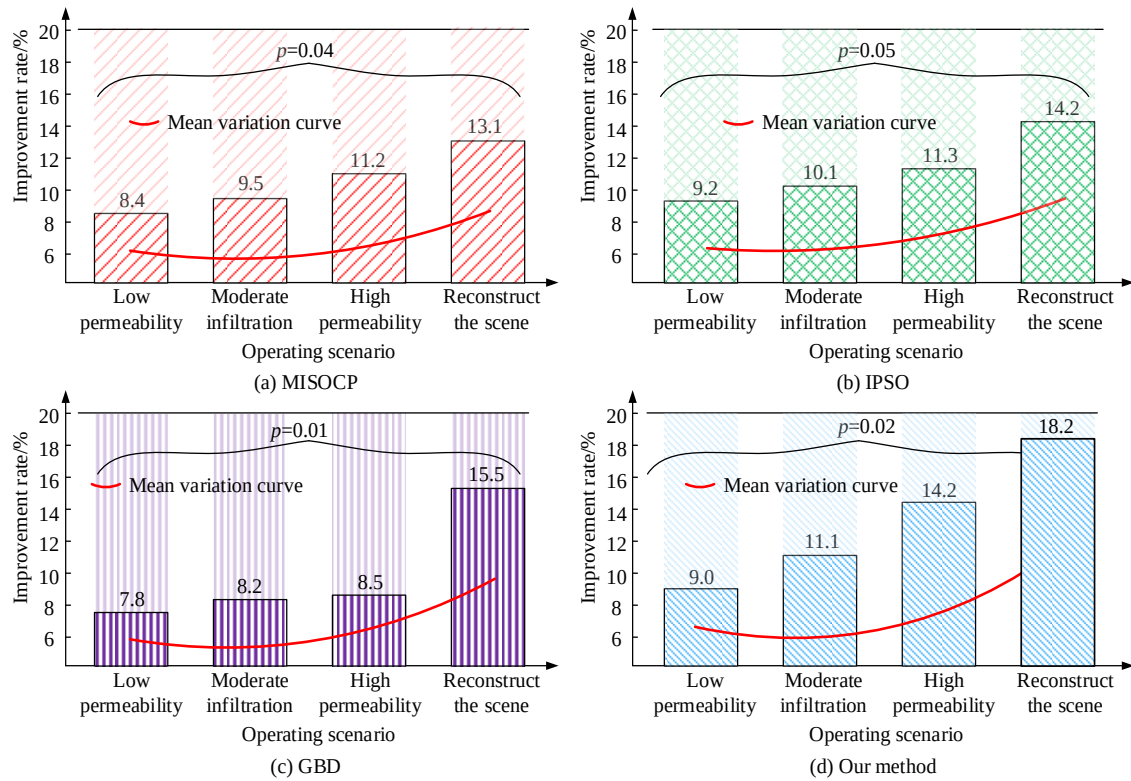


Fig. 11. Comparison of capacity improvement rates under different optimization methods

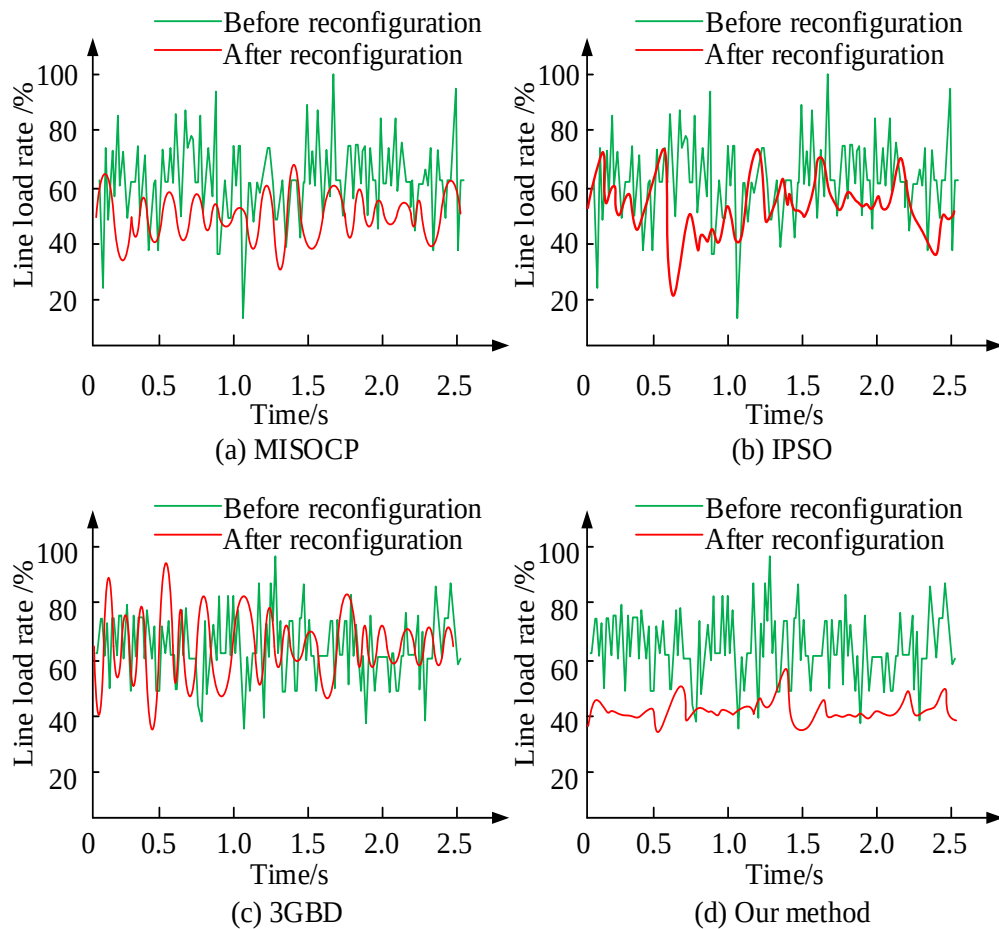


Fig. 12. Results of changes in load rates before and after the reconstruction of the route

Figures 12 (a) - (d) show the changes in line load rates for each method before and after reconstruction. In Figure 12 (a), the average line load rate after MISOCP reconstruction decreased from 64.8% to 52.6%, but the peak still reached 72.4%, indicating that it can reduce the load pressure of some lines, but the suppression effect on local heavy load is limited. In Figure 12 (b), the average line load rate after IPSO reconfiguration decreased from 66.3% to 50.7%, and the peak decreased to 68.5%. The load reduction effect was better than that of MISOCP. In Figure 12 (c), the average line load rate after GBD reconstruction decreased from 65.9% to 61.4%, and the peak still reached 83.7%, indicating its insufficient load distribution capacity under complex conditions. As shown in Figure 12 (d), the average line load rate after reconstruction by the proposed method decreased from 67.2% to 43.6%, the peak decreased from 89.4% to 54.8%, and the fluctuation range was significantly smaller than that of other methods. Overall, the proposed method can more effectively reduce the peak line load, lower the average load level, and improve the operational stability of the active distribution network. However, a reduction in line load factor does not imply a decline in grid operational efficiency. Since the reconfiguration process simultaneously incorporates both the enhancement of available open capacity and system loss constraints, the optimized topology can mitigate high-loss operating conditions resulting from local line overloads. Specifically, the redistribution of line loads leads to a more balanced current distribution and reduces line resistance losses, thereby enhancing the grid's photovoltaic integration capability while maintaining an optimal economic operational level. Consequently, the proposed method facilitates a transition from localized high-load operation to a globally balanced grid operation, rather than merely reducing line utilization rates. Finally, the study tested the convergence performance and computing time of the algorithm in different scenarios, as presented in Table 4.

Table 4. Algorithm convergence performance and computation time under different scenarios

Scenario	Method	Convergence iterations	Computation time / s	Optimal capacity / MW	Peak memory usage / GB
Low load ($\lambda=1.1$, 110% of base load)	MISOCP	43	12.3	7.8	2.84
	IPSO	36	9.8	8.1	2.31
	Standard GBD (without FCG)	29	11.5	8.6	1.96
	Our method	25	7.4	9.2	1.62
Medium load ($\lambda=1.4$, 140% of base load)	MISOCP	52	14.6	8.2	3.12
	IPSO	41	11.3	8.6	2.55
	Standard GBD (without FCG)	33	13.1	9.1	2.14
	Our method	27	8.2	9.7	1.79
High load ($\lambda=1.8$, 180% of base load)	MISOCP	61	18.3	8.7	3.56
	IPSO	47	13.8	9.1	2.88
	Standard GBD (without FCG)	38	15.7	9.5	2.43
	Our method	29	9.6	10.3	2.01
Extreme load ($\lambda=2.5$, 250% of base load)	MISOCP	70	21.5	9	4.08
	IPSO	53	16.2	9.4	3.24
	Standard GBD (without FCG)	42	17.9	9.8	2.76
	Our method	31	10.8	10.7	2.28

Note: Standard GBD denotes the conventional generalized Benders decomposition method without the feasible cut generation (FCG) mechanism, whereas Proposed GBD-FCG denotes the method developed in this study.

As shown in Table 4, under low-load scenarios, the proposed method requires only 25 iterations with a computation time of 7.4 seconds and achieves a maximum controllable power output of 9.2 MW, demonstrating superior overall performance compared to both MISOCP and IPSO. In extreme load scenarios, the proposed method takes 10.8 seconds for computation and achieves a maximum controllable power output of 10.7 MW, while maintaining a fast convergence speed. It should be

noted that this computation time is primarily allocated to the network reconfiguration decision-making process during the offline optimization phase; however, in actual active distribution network operations, day-ahead or rolling optimization strategies are typically employed. Therefore, the second-level computation time adequately meets engineering application requirements. Meanwhile, although MISOCP belongs to the class of convex optimization models, its solution process still involves addressing a complex integer combinatorial search problem due to the presence of numerous discrete switch variables and nonlinear security constraints within this problem formulation. As the load level increases, both the model size and the number of constraints expand further, leading to increased computation time. In contrast, the GBD-FCG method decomposes the master problem and subproblems and employs a feasible cut generation mechanism to reduce the ineffective search space, enabling it to achieve faster convergence speeds and higher capacity optimization results even in complex scenarios.

4. Summary

Research has demonstrated that the proposed active distribution network-based method for modeling and optimizing renewable energy integration capacity—which incorporates constraints arising from distributed photovoltaic (PV) systems—can effectively address challenges such as inaccurate capacity assessments under high PV penetration scenarios, difficulties in handling reverse power flow constraints, and suboptimal efficiency in network topology optimization. By constructing a capacity assessment model that integrates PV integration constraints, voltage security constraints, line transmission constraints, and N-1 security constraint, and by combining this model with a collaborative optimization mechanism involving GBD and FCG, this approach achieves efficient coordination between the discrete network topology decision-making process and the continuous power flow verification process.

The parameter sensitivity analysis demonstrates that an appropriate selection of the safety margin threshold and the load growth coefficient can effectively balance system security with capacity release capability. Specifically, when the safety margin threshold is set to 0.15 and the load growth coefficient is set to 1.4, the maximum line load ratio of the system reaches 86.4%, while the maximum voltage deviation is reduced to 0.013 p.u. These results indicate that the proposed GBD-FCG optimization framework can effectively coordinate voltage stability, line transmission capacity, and capacity enhancement objectives under complex active distribution network constraints; by reducing the search space for infeasible topologies, it enhances the global optimization capability of the optimization process, rather than relying solely on parameter adjustments to achieve performance improvement.

In tests conducted at various PV integration locations, the proposed method effectively mitigates the voltage fluctuation risks induced by high penetration of photovoltaic (PV) generation. Particularly in remote node integration scenarios, the maximum voltage deviation is controlled within 0.033 p.u., with a voltage compliance rate reaching 98.2%, demonstrating that the constructed model accurately captures the coupling effects among PV integration locations, changes in power flow directions, and network constraints. In the network reconfiguration experiments, the proposed method achieves an increase in available capacity of 18.2%, reduces the average line load factor from 67.2% to 43.6%, and lowers the peak load factor from 89.4% to 54.8%. These results indicate that the reconfiguration process not only enhances the curtailment capability of distributed PV systems but also reduces the risk of prolonged high-load operation on local transmission lines, thereby helping to minimize line thermal losses, alleviate equipment operational stress, and reduce long-term O&M costs for distribution networks. Furthermore, under extreme load conditions, the proposed method achieves a maximum available capacity of 10.7 MW after only 31 iterations, with a computation time of 10.8 seconds. Compared with traditional metaheuristic online reconfiguration methods, these results demonstrate that the GBD-FCG method achieves superior computational efficiency while maintaining optimal capacity optimization performance, making it particularly suitable for topology adjustment requirements in active distribution networks for day-ahead dispatching and rolling optimization scenarios.

However, current models primarily focus on optimizing scenarios involving typical photovoltaic (PV) grid integration and single fault constraints; they still exhibit certain limitations in terms of dynamic adaptability under extreme PV rapid fluctuations or situations where multiple faults occur simultaneously. When the PV output undergoes abrupt changes within a short time frame, the uncertainty in nodal power injection may lead to rapid shifts in the original feasible region, thereby increasing the information feedback frequency between the GBD master problem and its subproblems, while reducing the efficiency of FCG constraint generation; concurrently, the superposition of multiple faults may render traditional N-1 security constraints inadequate for accurately describing the system's risk boundary. Future research could further integrate energy storage systems, dynamic reactive power compensation devices, and multi-time-scale forecasting models into the existing GBD framework—by treating energy storage charging/discharging strategies and reactive power regulation processes as subproblem constraint variables—to achieve a synergistic solution for capacity optimization, dynamic regulation, and security control, thereby further enhancing the robustness of active distribution networks under complex operating conditions.

Symbol-to-Abbreviation Reference Table

i, j : Distribution network node index

l : Distribution line index

t : Time state index

N : Total number of distribution network nodes

L : Distribution line set

P_i^G : The active power supply provided by the substation to node i

P_i^{PV} : Active power injection of node-based distributed photovoltaic systems

P_i^L : Node load power consumption

V_i, V_j : Node voltage magnitude

G_{ij}, B_{ij} : The conductance and susceptance of the node admittance matrix

θ_{ij} : Node phase angle difference

Q_i^G, Q_i^{PV}, Q_i^L : Substation reactive power support, photovoltaic system reactive power injection, and node load reactive power demand

F : Maximum system openable capacity

λ : Load growth factor

$\frac{\partial V_i}{\partial P_i^{PV}}$: Sensitivity coefficient of node voltages to changes in photovoltaic power output

ΔP_i^{PV} : Incremental photovoltaic power output

$V_i^{\min}, V_i^{\max}$: Voltage upper and lower limits

P_{ij}, Q_{ij} : Line active power and reactive power

$S_{ij}^{\max}$: Maximum allowable transmission capacity of the line

S_{ij}^k : The apparent power flow of line $i - j$ in the k type of fault scenario

$S_{ij}^{\max}$: The maximum transmission capacity allowed for Line $i - j$

V_i^k : The voltage amplitude of node i in the first fault scenario k

V_i^{ref} : Node rated voltage

ε : System safety margin threshold

Z : Comprehensive optimization objective

ω_1, ω_1 : Capacity weight and loss weight coefficient

ω_1, I_{ij} : Line resistance and line current

X_{ij} : Line $i - j$ reactance

θ_i, θ_j : The voltage phase angle between node i and node j

x : Network switch state decision variable in the main problem

x^k : The switching state obtained in the k th iteration

$g(x^k)$: The first-order gradient information of the constraint function at the current iteration point

$P_i^{PV,opt}$: Optimized PV grid-connection capacity

$Z^{(t)}$: The optimization target value of the t -th iteration

L_{opt} : Comprehensive loss optimization

$L_{volt}, L_{flow}, L_{loss}$: Node voltage deviation, line load deviation, and network power loss

α, β, γ : Weight coefficient

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Adaptive multi-sensor fusion for pedestrian indoor positioning using UCR-DTW, improved CRF, and SAGE-HUSA filtering

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Abstract

Indoor positioning in smart buildings is challenging because satellite signals are obstructed, while inertial navigation methods suffer from sensor noise, magnetic interference, and cumulative errors. This study proposes a high-precision indoor positioning method integrating Pedestrian Dead Reckoning (PDR), map matching, and multi-sensor fusion for real-time pedestrian positioning. The proposed framework employs acceleration and geomagnetic features for stationary state detection, Dynamic Time Warping (DTW) for adaptive gait recognition, and a robust adaptive Kalman filter for heading estimation. Building topology constraints are incorporated through an improved Conditional Random Field (CRF), which is combined with University of California Riverside-optimized DTW (UCR-DTW) for geomagnetic sequence matching. Finally, a Sage-Husa filter fuses multi-source information to generate robust position estimates. Experiments on the IPIN 2025 Competition Track 3 (Smartphone-based) indoor positioning dataset show that the proposed method achieves a mean positioning error of 0.45 m and a trajectory integrity rate of 98.2%. The improved CRF reaches a map-matching accuracy of 96.8% after six iterations and maintains 71.6% accuracy under level-5 magnetic interference. These results demonstrate that the proposed framework effectively suppresses cumulative errors and magnetic disturbances while constraining trajectories within navigable areas, providing accurate and reliable real-time positioning for pedestrian indoor navigation. Future work will incorporate height information during multi-storey transitions to further improve cross-floor positioning performance.

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1. Introduction

Smart buildings feature enclosed spaces with dense structural elements that significantly attenuate satellite signals. Indoor positioning faces significant challenges due to severe satellite signal attenuation and blockage [1]. For pedestrian indoor positioning applications, the indoor space has dense walls and complex layouts, and satellite signals are severely blocked, making it impossible to rely on traditional navigation and positioning. Current PDR systems exhibit cumulative drift, necessitating the integration of map matching techniques for constraint correction to achieve continuous and reliable positioning within intelligent buildings [2]. However, the proposed methodology faces significant limitations: the noise from consumer-grade sensors, the frequent switching of pedestrian motion states, and the magnetic interference from household appliances, metal furniture, etc. in the indoor environment lead to inaccurate gait detection, drift in course calculation, and rapid accumulation of errors along walking distance [3]. Meanwhile, existing positioning methods often lack sufficient topological constraints from indoor maps, so filtering-based correction alone cannot explicitly distinguish passable and impassable areas. Under high-

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noise inertial and geomagnetic measurements, particle-filter or standard Kalman-filter-based methods mainly suppress estimation noise but cannot reliably enforce wall, corridor, and obstacle constraints. As a result, accumulated positioning errors may still cause trajectories to cross walls or boundaries and deviate from the actual path. Existing map matching methods exhibit limitations in adaptive gait adjustment and robust heading estimation under magnetic interference. Furthermore, most existing fusion schemes lack a collaborative design that integrates noise adaptation, dynamic weight adjustment, and global path optimization, making it difficult to meet the requirements of long-term, high-precision stable operation and precise navigation of pedestrians in signal occlusion environments [4-5]. Therefore, the study first calculates the pedestrian trajectory in real time through PDR, uses multimode sensing to identify the motion state, and adaptively optimizes gait detection and course solution. The study then combines map matching calibration with the electronic map and applies spatial constraints using Conditional Random Fields (CRFs). The UCR Suite optimization techniques are integrated into the DTW algorithm to construct the UCR-DTW framework for rapid geomagnetic matching [6-7]. A positioning process that works in tandem with adaptive filtering and map correction is built. The study aims to address error accumulation, magnetic interference, and trajectory out-of-bounds to achieve sub-meter base station-free positioning and support pedestrian navigation and tracking.

The main innovations of this study are summarized as follows. First, in the PDR segment, acceleration and geomagnetic features are jointly used for stationary-state detection, reducing false step counting caused by slight body movement or sensor disturbance. A DTW-based adaptive gait detection model is then constructed to identify different walking states and dynamically adjust peak-detection parameters, while a robust adaptive Kalman filter under least-squares constraints is employed for heading estimation. Second, in the map-matching segment, an improved CRF model is developed by incorporating geomagnetic matching features and adaptive weight updating, thereby strengthening spatial constraints on candidate trajectories under varying measurement reliability. In addition, the UCR-DTW strategy is introduced to accelerate geomagnetic sequence matching, and these components are integrated into a PDR-geomagnetic-map-matching fusion positioning framework.

The remainder of this paper is organized as follows. Section 2 reviews related studies on indoor positioning and multi-source fusion. Section 3 presents the proposed methodology, including motion-state recognition and adaptive PDR parameter estimation, improved CRF-based map matching, and geomagnetic-assisted fusion positioning. Section 4 evaluates the positioning accuracy, real-time performance, convergence, robustness, and ablation results. Finally, Section 5 summarizes the main findings and discusses the limitations and future work.

2. Related Works

Indoor positioning studies can be broadly grouped according to the information source and constraint mechanism used for position estimation. Existing research mainly includes signal- or fingerprint-based positioning, inertial and motion-assisted positioning, and map-constrained fusion approaches. These directions provide complementary advantages, but their limitations in motion adaptation, interference resistance, and spatial constraint management remain important issues for continuous pedestrian positioning.

In signal- and fingerprint-based positioning, Han Y et al. developed a crowdsourcing positioning model to improve feature extraction and the utilization of unlabeled fingerprint data. Their approach introduced a spatial fingerprint attention encoder and a signal-strength masking mechanism, resulting in improved positioning accuracy over conventional algorithms [8]. Pinto B et al. proposed an access-point selection strategy based on least-squares estimation and Bluetooth Low Energy measurements. Their method reduced the positioning error by up to 13% compared with conventional least-squares estimation and by up to 30% compared with the k-nearest-neighbor method [9]. Nguyen D K et al. combined truncated singular value decomposition with a long short-term memory model to reduce the computational burden of Wi-Fi fingerprint positioning while maintaining positioning accuracy [10]. These methods improve the exploitation

of radio-frequency fingerprints, but their performance remains dependent on the stability and distinctiveness of environmental signal features.

For motion-assisted and inertial positioning, Yang Y et al. developed an algorithm combining Brownian-motion correction and dynamic parameter optimization. Their algorithm used prior motion information to correct positioning results and achieved a maximum average positioning error of 3.31 m [11]. Assayag Y et al. combined particle swarm optimization with a signal-propagation model to reduce the dependence on pre-training in indoor ranging, reporting an average positioning error of 2.57 m [12]. Such approaches improve position estimation by introducing motion priors or optimization mechanisms, but accumulated motion errors and sensor disturbances remain difficult to suppress over long trajectories. In particular, pedestrian dead reckoning requires reliable step detection, walking-state discrimination, and heading estimation, which motivates the use of adaptive motion-state recognition and filtering rather than fixed-parameter correction.

Map-constrained positioning provides another means of limiting accumulated trajectory drift. Xu Z et al. developed a mapping method combining an improved beetle swarm algorithm with an efficient matching strategy, which reduced computation time and particle resampling while improving mapping accuracy [13]. However, optimization- or particle-based mapping alone does not explicitly guarantee that the estimated trajectory satisfies indoor topological constraints such as walls, corridors, and obstacles. Under noisy inertial or geomagnetic observations, insufficient constraint management may still result in wall crossing, boundary violations, or deviation from feasible paths. Probabilistic state-sequence modelling is therefore required to jointly consider observation information and spatial connectivity. In this study, an improved CRF is introduced for this purpose, in which geomagnetic matching information is incorporated into the state feature function and the feature weights are adaptively updated according to data reliability.

The above studies also reveal a separation between signal matching, motion estimation, and map constraints. Signal-based approaches make effective use of environmental fingerprints but can be sensitive to local signal variations; inertial approaches provide continuous positioning but suffer from cumulative drift; and existing mapping or optimization approaches do not fully coordinate observation reliability with indoor topological constraints. Moreover, the cited studies involve different positioning targets, including crowdsourced pedestrian positioning and mobile-robot mapping.

Therefore, the remaining gaps are addressed through an integrated pedestrian-positioning framework. Acceleration and geomagnetic dual features are first used for stationary-state recognition, while DTW enables adaptive gait detection. UCR-DTW is then employed for efficient geomagnetic sequence matching, and the improved CRF introduces geomagnetic features and adaptive weights to impose indoor topological constraints. Finally, Sage-Husa adaptive filtering dynamically estimates measurement uncertainty and fuses PDR and geomagnetic observations. Through this combination, the proposed method targets both poor interference resistance and insufficient utilization of heterogeneous sensing and spatial information while reducing accumulated trajectory drift.

3. Methodologies and Materials

3.1 Motion State Recognition Based on Multi-Model Features and Adaptive PDR Parameter Estimation

Smart building spaces are enclosed and satellite signals are difficult to penetrate, relying on PDR positioning. However, PDR is susceptible to noise, state changes and magnetic interference, errors accumulate rapidly, and the lack of spatial constraints leads to wall-penetrating drift [14-15]. Classical PDR usually estimates pedestrian displacement through step detection, step-length estimation, and heading update, among which acceleration-based empirical models such as the Kim model and related step-length estimators provide an important basis for subsequent adaptive PDR methods [16-17]. Existing map matching is inadequate in terms of state recognition, gait adaptation, etc., and is difficult to meet the requirements of high-precision and stable positioning.

To accurately distinguish between pedestrian motion and non-motion states and reduce the impact of static miscounting steps on positioning, a non-motion state recognition method based on acceleration and geomagnetic standard deviation is studied to achieve stable state determination through two-dimensional features. The developed approach for identifying non-motion states based on acceleration and geomagnetic standard deviation is shown in Figure 1.

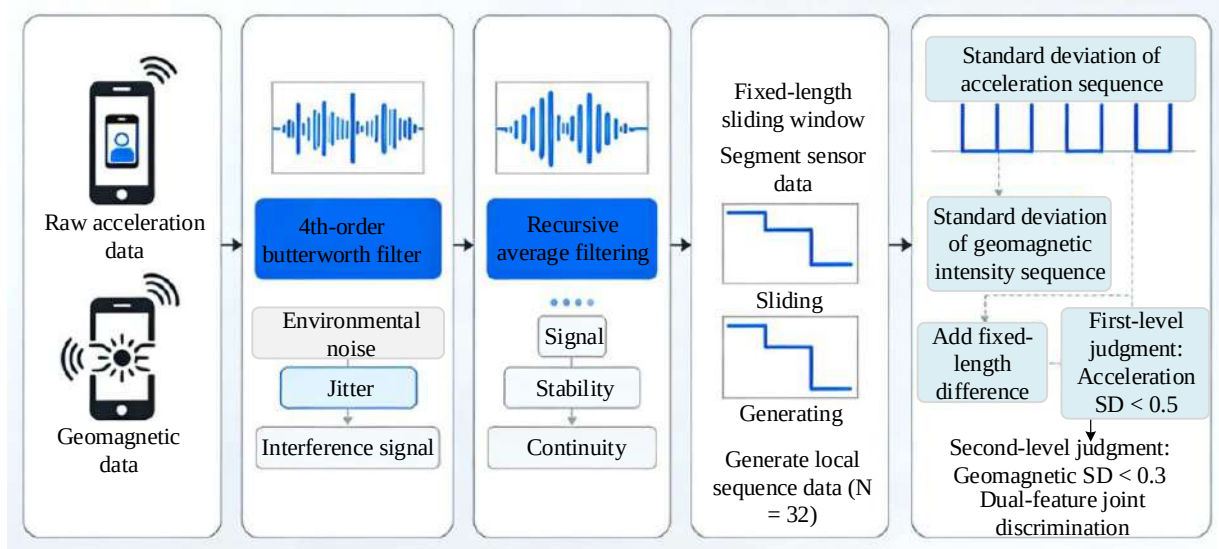


Fig. 1. Flow chart of non motion state recognition based on acceleration and geomagnetic standard deviation

According to Figure 1, the raw acceleration, gyroscope, and geomagnetic data collected by the smartphone were first synchronized according to their timestamps and uniformly resampled at 100 Hz. Subsequently, a fourth-order low-pass Butterworth filter with a cutoff frequency of 5 Hz was applied to suppress high-frequency noise and device vibration while preserving the dominant low-frequency components associated with gait and attitude changes. The data are subsequently smoothed using recursive average filtering to improve signal stability and continuity. A fixed-length sliding window is then used to segment the continuous sensor data and generate local sequences. Within each window, the standard deviations of the acceleration and geomagnetic intensity sequences are calculated, followed by the computation of feature differences between fixed-interval windows to characterize short-term variation trends. The acceleration features are then used for the first-level judgment, while the geomagnetic features are incorporated into the second-level judgment to achieve dual-feature joint recognition of motion states. In each sliding window, the standard deviation of the acceleration sequence and the standard deviation of the geomagnetic intensity sequence are calculated respectively, the standard deviation of the acceleration is shown in Equation (1).

$$\sigma_a = \sqrt{\frac{1}{N_a} \sum_{p=1}^{N_a} (a_p - \bar{a})^2} \quad (1)$$

In Equation (1), where N represents the total number of sampling points within the sliding window; a_p represents the acceleration amplitude of the sampled point p ; $\bar{a}$ represents the mean of the acceleration sequence within the window. The standard deviation of geomagnetic intensity is as shown in Equation (2).

$$\sigma_m = \sqrt{\frac{1}{N_m} \sum_{q=1}^{N_m} (m_q - \bar{m})^2} \quad (2)$$

In Equation (2), where m_q represents the total geomagnetic intensity at the sampling point q ; $\bar{m}$ represents the mean of the geomagnetic intensity within the window. This method uses the

acceleration standard deviation for the first-level state discrimination, with the threshold set to 0.5. When the acceleration feature alone cannot clearly distinguish the state, the geomagnetic standard deviation is further introduced for second-level discrimination, with a threshold of 0.3. The two thresholds were determined according to the statistical distributions shown in Figure 7. The acceleration standard deviations of the stationary and slightly wobbling states were below 0.5, whereas the geomagnetic standard deviations of the non-motion states were below 0.3 and those of the motion states were above 0.3. Because the acceleration distributions of general wobbling and walking partially overlap, geomagnetic information is used as the secondary criterion.

To enable gait detection to adapt to different walking speeds and improve the accuracy of step counting, a motion state classification and adaptive peak threshold detection method based on the DTW algorithm is proposed. The detection parameters are dynamically adjusted according to the motion state to achieve robust gait recognition in multiple scenarios. The DTW-based adaptive gait detection method for pedestrian motion state classification and adaptive peak detection threshold determination is shown in Figure 2.

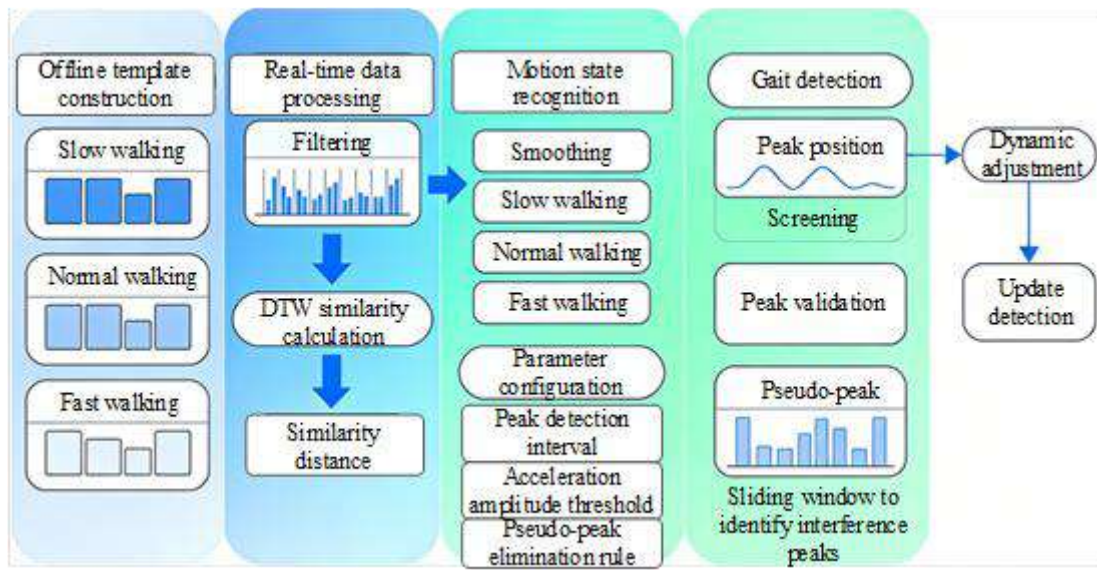


Fig. 2. Framework diagram of adaptive peak detection proposed methodology based on DTW

As shown in Figure 2, acceleration sequence templates corresponding to slow, normal, and fast walking were first constructed offline. For each motion state, 20 valid acceleration sequences were selected, with each sequence containing five consecutive complete gait cycles. After segmenting the sequences according to gait peaks, each individual gait cycle was uniformly resampled to 100 points using linear interpolation, and Z-score normalization was applied to reduce the influence of differences among individuals, device attitudes, and motion amplitudes. The normalized cycles belonging to the same motion state were then temporally aligned and averaged to obtain the three DTW reference templates. During online recognition, the real-time acceleration sequence was filtered and smoothed, and then processed using the same resampling and normalization procedures as those applied to the offline templates. The DTW algorithm was subsequently used to calculate the similarity distances between the real-time sequence and the three reference templates, and the current pedestrian walking state was determined according to the template with the minimum DTW distance. The DTW distance calculation is shown in Equation (3).

$$DTW(M, J) = \min_{\pi} \sqrt{\sum_{(r,s) \in \pi} \|m_r - j_s\|^2} \quad (3)$$

In Equation (3), $DTW(M, J)$ represents the minimum distance between two time series calculated by the DTW algorithm; M represents the acceleration sequence collected in real time; J represents an offline template sequence; π represents a regular path from sequence M to sequence J ; (r, s) represents a matching point pair on a regular path; $\|m_r - j_s\|$ represents the Euclidean distance

between points m_r and j_s . Based on the identified motion state, the system automatically configures the corresponding peak detection period threshold, acceleration amplitude threshold, and pseudo-peak elimination rule. During the gait detection phase, the processed acceleration signal is traversed point by point to search for the peak position, and effective peaks that conform to the walking pattern are screened based on the periodic constraint conditions, and false interference peaks are identified and eliminated through the sliding window. The algorithm continuously updates the detection parameters according to the changes in motion state, dynamically adjusts the constraints for peak recognition, and can perform adaptive gait detection and step count statistics in different motion modes.

To suppress the influence of indoor magnetic field anomalies on heading estimation, a robust adaptive Kalman filter under least-squares constraints is introduced. The method fuses the relative heading obtained from the gyroscope with the absolute heading estimated from the magnetometer and accelerometer, and adaptively updates the filtering parameters to obtain stable heading estimates, as shown in Figure 3.

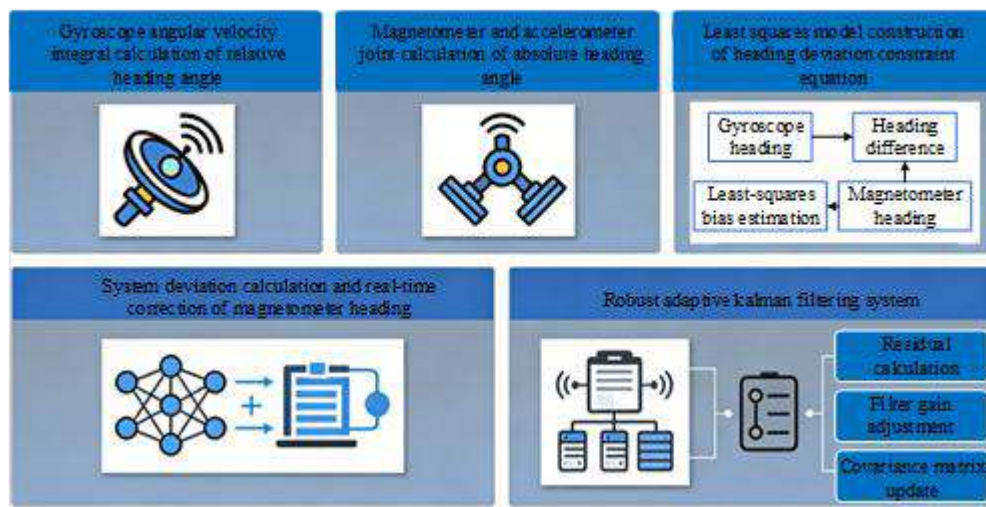


Fig. 3. Structure diagram of robust adaptive Kalman filter heading estimation

According to Figure 3, this method calculates the relative heading angle respectively by the gyroscope angular velocity product decomposition, and the absolute heading angle is calculated jointly by the magnetometer and the accelerometer. The gyroscope heading angle integral is shown in Equation (4).

$$\psi_{gyro}(t) = \psi_{gyro}(t-1) + \omega(t) \cdot \Delta t \quad (4)$$

In Equation (4), $\psi_{gyro}(t)$ represents the relative heading angle obtained by integrating the angular velocity of the gyroscope at current moment t ; $t-1$ represents the previous moment; $\omega(t)$ indicates the angular velocity measured by the gyroscope at the current moment; Δt represents the time interval between two adjacent sampling moments. The magnetometer heading angle solution is as shown in Equation (5).

$$\psi_{mag} = \arctan\left(\frac{K_y}{K_x}\right) \quad (5)$$

In Equation (5), ψ_{mag} represents the calculated absolute heading angle; K_y represents the magnetometer component in the y axial direction in the horizontal coordinate system; K_x represents the magnetometer component in the x axial direction in a horizontal coordinate system; $\arctan$ represents the arctangent function. The study substitutes two sets of independently calculated heading angle data into the least squares model, constructs the heading deviation constraint equation, calculates the systematic deviation between the two sets of heading, and uses

this deviation to correct the heading angle output by the magnetometer in real time. The least squares heading deviation estimation $\hat{b}$ is as shown in Equation (6).

$$\hat{b} = (H^T H)^{-1} H^T (\psi_{gyro} - \psi_{mag}) \quad (6)$$

In Equation (6), H is the observation matrix describing the relationship between the systematic heading bias and the synchronized heading differences obtained from the gyroscope and magnetometer. For N synchronized samples, H has a dimension of $N \times 1$, and each element is equal to 1 because the same scalar heading bias is assumed to affect all observations. Therefore, each row of H maps the common heading bias to one heading-difference observation, and the least-squares solution estimates this systematic bias from all synchronized samples. H^T represents transposition of H ; $(H^T H)^{-1}$ represent the inverse matrix of $H^T H$. The corrected heading data as observations and the gyroscope predicted heading as state predictions are both input into the anti-differential adaptive Kalman filtering system. During the filtering process, the residuals between the predicted values and the observed values are calculated in real time. The filtering gain and the observed noise weights are dynamically adjusted according to the residuals, while the system noise covariance matrix and the observed noise covariance matrix are continuously updated. The system performs recursive operations with the heading angle as the core state variable, continuously corrects the state estimation results, and completes a stable and reliable heading fusion solution.

3.2 Indoor Map Matching Proposed Methodology Based on Improved CRFs

After motion state recognition, adaptive gait detection and heading estimation, PDR positioning may still experience trajectory drift due to accumulated sensor errors, and phenomena such as wall penetration and boundary crossing are prone to occur in complex layouts of intelligent buildings [18-19]. To confine the positioning trajectory within a reasonable space, the study introduced indoor map constraints and proposed a CRF-based map matching method, using building topology and boundary information to correct the PDR results. The developed CRF-based map matching approach is shown in Figure 4.

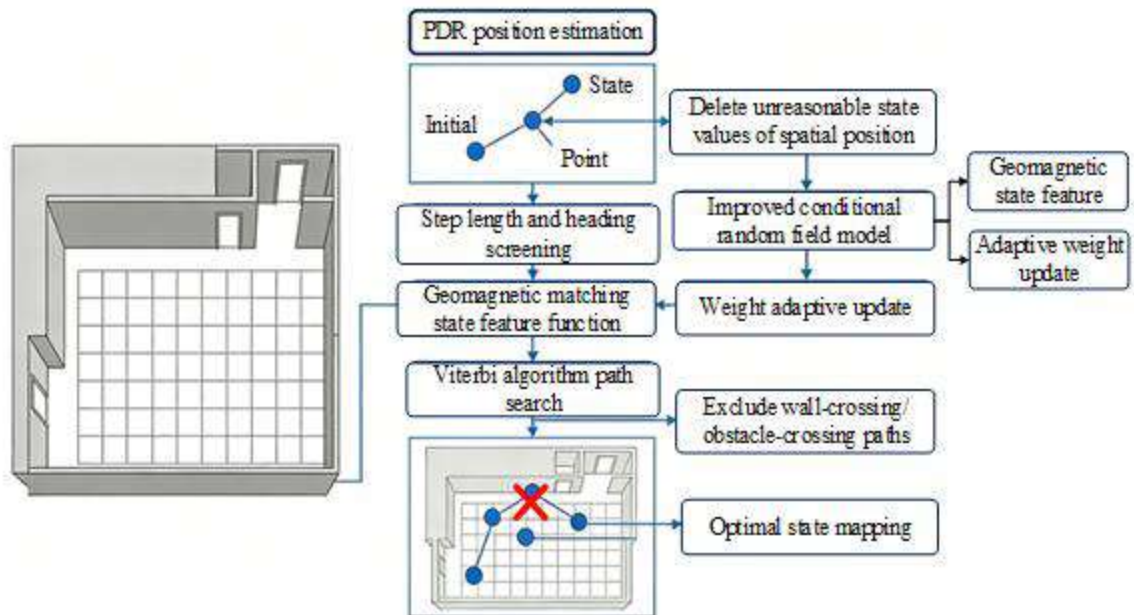


Fig. 4. Schematic diagram of map matching based on improved CRFs

According to Figure 4, the spatial boundary information of the indoor environment, including walls, corridors, corners, entrances, and exits, was first extracted. The indoor floor plan was then divided into uniform grid cells with a spatial resolution of $0.5 \text{ m} \times 0.5 \text{ m}$, and the walkable and non-walkable regions were labeled according to the building topology. This grid size provides sufficient spatial resolution for map constraints while limiting the number of CRF candidate states and avoiding the

substantial increase in computational complexity caused by excessively fine discretization. The initial candidate state points are generated based on the positions calculated by the PDR system. The candidate points are initially screened in combination with the step size information and heading information of the previous moment, and the state values with unreasonable spatial positions are deleted [20].

During candidate-state screening, the transition weights between adjacent CRF nodes are jointly determined by motion continuity and building-topology constraints. First, the feasibility of each state transition is evaluated according to the deviation between the Euclidean distance of adjacent candidate points and the PDR-estimated step length, as well as the deviation between the candidate transition direction and the current heading. Larger distance and heading deviations result in lower transition weights. Wall and obstacle information is then introduced as hard topological constraints in the edge-weight calculation. If the line segment connecting two candidate nodes crosses a wall, obstacle, or other non-walkable region, the corresponding transition is directly regarded as invalid and excluded from the subsequent Viterbi path search. For feasible transitions, the edge weights are assigned according to step-length consistency, heading consistency, and the confidence of geomagnetic matching. Through this mechanism, the CRF suppresses spatially unreasonable transitions at the state-transition stage and prevents wall-crossing, obstacle-crossing, and trajectories that significantly deviate from valid walkable regions during global path optimization. The screened valid state points are input into the CRFs model, and the geomagnetic matching results are used to construct the state feature function, and the feature function weights are adaptively updated based on the credibility of the data. The CRF state feature function is shown in Equation (7).

$$\mathfrak{R}(z_t, o_t) = \exp\left(\sum_l \theta_l f_l(z_t, o_t)\right) \quad (7)$$

In Equation (7), $\mathfrak{R}(z_t, o_t)$ represents the potential function in the CRF; z_t represents the hidden state at moment t ; o_t is the observation data at moment t ; θ_l represents the weight of the l th feature function; f_l represents the l th feature function; $\exp$ represents the exponential function. In the improved CRF, geomagnetic matching results are introduced as an additional state feature to evaluate the consistency between each candidate position and the current geomagnetic observation. A higher geomagnetic matching similarity increases the preference for the corresponding candidate state. In addition, the contribution of the geomagnetic feature is adaptively adjusted according to the reliability of the observation data. Therefore, compared with a conventional CRF mainly based on spatial transition and observation constraints, the improved CRF further incorporates geomagnetic matching information and adaptive feature weighting into the state evaluation process. The Viterbi algorithm is used for global path search of the state sequence, and the Viterbi algorithm is recursively shown in Equation (8).

$$\delta_t(j) = \max_i [\delta_{t-1}(i) \cdot \Psi(y_{t-1} = i, y_t = j, x_t)] \quad (8)$$

In Equation (8), $\delta_t(j)$ is the maximum probability among all the paths whose state is j until moment t ; j is the possible state value at the previous moment $t - 1$. In the Viterbi path search, indoor map constraints are incorporated into the state-transition evaluation. For each transition between two consecutive candidate states, the connecting segment is checked against the passable-area boundaries of the indoor map. If the transition crosses a wall or obstacle, it is treated as an infeasible transition and assigned a negligible transition score, so that it is excluded from the optimal-path competition. Transitions that remain within passable corridors and connected regions retain their normal scores. Through this constraint mechanism, the Viterbi search selects the most probable state sequence while preventing wall-crossing and obstacle-crossing trajectories.

3.3 Fusion Positioning Proposed Methodology Based on Geomagnetic Assistance and Adaptive Filtering

After the map matching constraint, the trajectory through-wall and drift problem of PDR positioning was effectively alleviated, but relying solely on inertial calculation and map correction was still difficult to deal with the cumulative deviation caused by long walking, and the position features contained in the geomagnetic signal were not fully utilized [21-22]. Geomagnetism, as a naturally stable positioning signal in the room, has the advantages of no equipment deployment and strong applicability. To fully exploit the position discrimination ability of geomagnetic sequences and improve the efficiency and accuracy of geomagnetic matching, a fast geomagnetic sequence matching method based on UCR-DTW was introduced in the study, as shown in Figure 5.

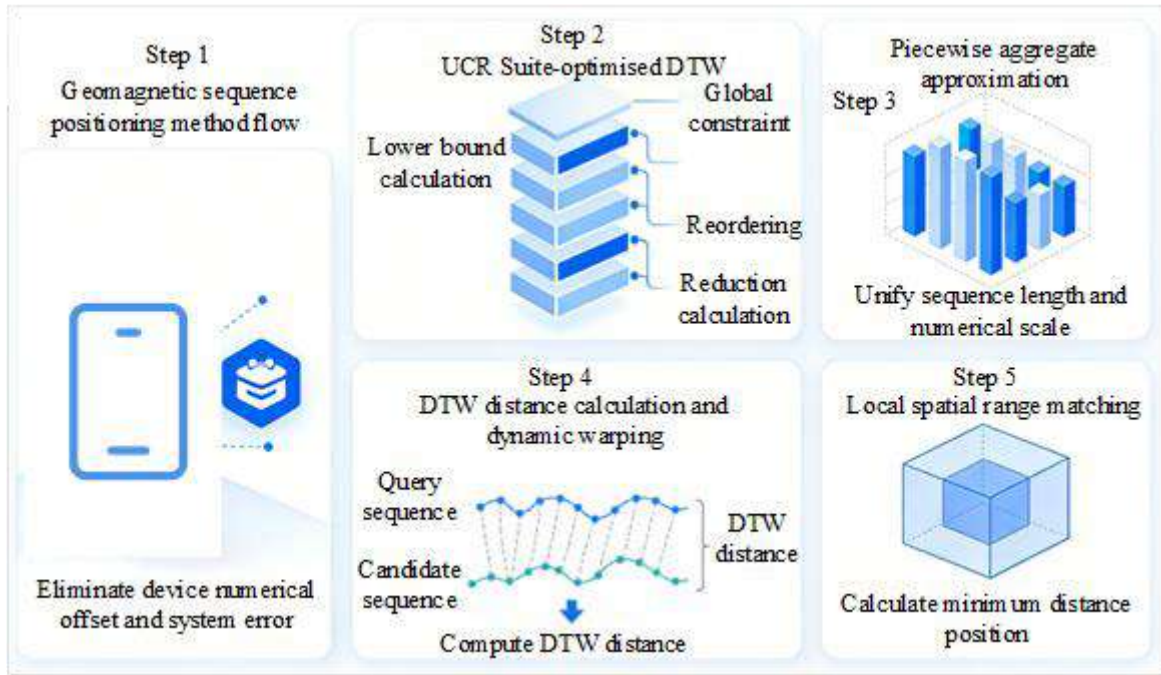


Fig. 5. Flowchart of rapid matching of geomagnetic sequences based on UCR-DTW

According to Figure 5, the geomagnetic sequences collected in real time are first subjected to standardized preprocessing to eliminate numerical offsets and system errors caused by different mobile devices. The UCR Suite optimization techniques are then integrated into the DTW algorithm to improve geomagnetic sequence matching efficiency. Global constraints are applied to restrict the warping path, and the lower-bound distance is calculated to pre-screen candidate sequences with low similarity. The UCR-DTW lower bound distance LB calculation is shown in Equation (9).

$$LB = \max(|first(M) - first(J)|, |last(M) - last(J)|) \quad (9)$$

In Equation (9), $first(M)$ represents the first element of the sequence M ; $last(M)$ represents the last element of the sequence M ; $first(J)$ represent the first element of the candidate sequence J ; $last(J)$ represents the last element of the candidate sequence J . Candidate sequences whose lower-bound values exceed the current matching criterion are discarded without performing the full DTW distance calculation. Therefore, unnecessary dynamic-programming operations can be avoided, and complete DTW matching is performed only for the retained candidates. Sequence reordering and cascading lower-bound screening are further used to improve pruning efficiency. For the remaining geomagnetic sequences, piecewise aggregation is applied to unify sequence length and scale before the full DTW distance is calculated. The matching process is limited to the local space of the map, traversing all geomagnetic fingerprint sequences in that area, calculating the regular distance between the real-time sequence and the candidate sequence, and taking the position corresponding to the minimum distance.

To fully integrate the advantages of fast geomagnetic sequence matching and PDR location sources and offset their respective errors, an integrated positioning framework combining adaptive filtering and CRFs is studied to achieve continuous, stable, and high-precision indoor location solutions. The fusion positioning proposed methodology based on the combination of Sage-Husa adaptive Kalman filtering and improved CRFs is shown in Figure 6.

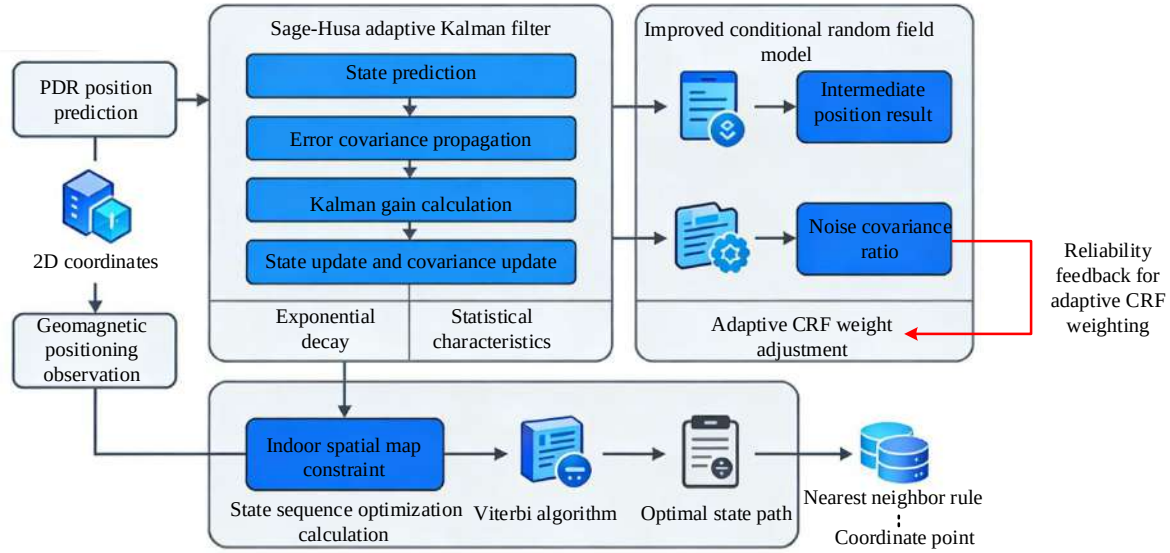


Fig. 6 Schematic diagram of PDR and geomagnetic field fusion

As shown in Figure 6, the PDR position prediction equation and the geomagnetic positioning observation equation are first established, and the two-dimensional planar coordinates are taken as the system state variables and input into the Sage-Husa adaptive Kalman filter. During filtering, state prediction, error covariance propagation, Kalman gain calculation, state update, and covariance update are performed sequentially. To adapt to the dynamic variations in PDR accumulated error and geomagnetic observation noise, the innovation sequence is used to estimate the process noise covariance matrix and the observation noise covariance matrix online. The innovation vector at time k is expressed as shown in Equation (10).

$$e_k = z_k - H_k \hat{x}_{k|k-1} \quad (10)$$

In Equation (10), where e_k denotes the innovation vector at time k , z_k denotes the position observation obtained from geomagnetic sequence matching; H_k denotes the observation matrix; and $\hat{x}_{k|k-1}$ denotes the prior predicted state calculated by PDR. The innovation vector reflects the deviation between the current geomagnetic observation position and the PDR-predicted position, and its variation can be used to characterize the dynamic behavior of observation noise and prediction error. To reduce the influence of earlier noise statistics on the current filtering result, an exponential decay factor b is introduced, and an adaptive weight varying with the iteration index is constructed. In this study, $b = 0.98$, and the corresponding adaptive weight is expressed as shown in Equation (11).

$$d_k = \frac{1 - b}{1 - b^{k+1}} \quad (11)$$

In Equation (11), d_k denotes the adaptive weight at time k ; b denotes the exponential decay factor; and k denotes the current filtering instant. As the filtering process proceeds, d_k weights the historical noise statistics according to the decay factor, thereby giving more effective emphasis to recent observations in covariance estimation. After obtaining the innovation vector and adaptive weight, the geomagnetic positioning observation noise is recursively estimated. The observation noise covariance matrix R_k is updated as shown in Equation (12).

$$R_k = (1 - d_k)R_{k-1} + d_k(e_k e_k^T - H_k P_{k|k-1} H_k^T) \quad (12)$$

In Equation (12), R_k and R_{k-1} denote the observation noise covariance matrices at the current and previous instants, respectively; $P_{k|k-1}$ denotes the state prediction error covariance matrix; and the superscript T denotes matrix transposition. By jointly correcting R_k using the innovation term and the prediction error covariance, the reliability of the geomagnetic observation in fusion positioning can be dynamically adjusted according to the geomagnetic matching quality.

Meanwhile, to characterize the variation in the PDR state prediction error during motion, the process noise covariance matrix is adaptively updated, as shown in Equation (13).

$$Q_k = (1 - d_k)Q_{k-1} + d_k[K_k e_k e_k^T K_k^T + P_{k|k} - F_k P_{k-1|k-1} F_k^T] \quad (13)$$

In Equation (13), Q_k and Q_{k-1} denote the process noise covariance matrices at the current and previous instants, respectively; K_k denotes the Kalman gain matrix; $P_{k|k}$ denotes the error covariance matrix after state updating; $P_{k-1|k-1}$ denotes the posterior error covariance matrix at the previous instant; and F_k denotes the state transition matrix. Through the synchronous recursive updating of Q_k and R_k , the filter can dynamically adjust the weights of PDR prediction and geomagnetic observation according to their respective error levels, thereby reducing the estimation bias caused by fixed noise parameters in complex indoor environments.

Subsequently, the intermediate position result and noise covariance information obtained from the Sage-Husa filter are input into the conditional random field model. The weights of the state feature functions are dynamically adjusted according to the reliability of different positioning information. Combined with indoor map constraints, the Viterbi algorithm is then used to search for the optimal state path, and the fused position is finally matched to the corresponding indoor map coordinates using the nearest-neighbor rule. The complete feedback fusion process is shown in Algorithm 1.

Algorithm 1. Feedback fusion of Sage-Husa filtering and improved CRF

Input: PDR predicted position, geomagnetic matching position, indoor map

Output: Final map-constrained position

- 1: Initialize the Sage-Husa filter, noise covariance matrices, and CRF parameters
 - 2: For each positioning instant k :
 - 3: Obtain the PDR prior predicted position
 - 4: Obtain the geomagnetic matching observation
 - 5: Calculate the innovation vector according to Equation (10)
 - 6: Calculate the adaptive weight according to Equation (11)
 - 7: Recursively update the observation noise covariance matrix according to Equation (12)
 - 8: Recursively update the process noise covariance matrix according to Equation (13)
 - 9: Perform the Sage-Husa state update and obtain the intermediate position estimate
 - 10: Evaluate the relative reliability of the PDR prediction and geomagnetic observation according to the updated covariance information
 - 11: Update the CRF candidate state set using the intermediate position estimate
 - 12: Dynamically adjust the corresponding CRF state-feature weights according to reliability
 - 13: Apply indoor map constraints and remove infeasible state transitions
 - 14: Perform Viterbi search to determine the optimal feasible state path
 - 15: Map the selected state to the corresponding indoor coordinate using the nearest-neighbor rule
 - 16: End For
-

4. Results

4.1 Motion State Recognition and Geomagnetic Matching Performance Testing

This study employed the smartphone-based indoor positioning dataset from Track 3 of the IPIN 2025 competition for algorithm validation. The dataset was collected and released by the IPIN 2025 organizing committee in a real multi-story university building in Tampere, Finland. It contains multimodal data, including WiFi, BLE, geomagnetic field, inertial sensors, barometer, light sensor, audio, and GNSS data, together with training, testing, and scoring data, indoor maps, and evaluation tools. To reduce the influence of different smartphone carrying modes, sensor attitudes, and high-frequency motion disturbances on pedestrian positioning, the accelerometer, gyroscope, and magnetometer data were first synchronized, resampled, and low-pass filtered. Features related to displacement, heading variation, and spatial geomagnetic distribution were then extracted for PDR step detection, heading estimation, and geomagnetic matching. Unified preprocessing and feature normalization were applied to improve the consistency of multi-sensor features, and the experiments focused on evaluating trajectory drift suppression, map matching, and fusion positioning accuracy. In the experiments, the sampling frequencies of the accelerometer, gyroscope, and magnetometer were uniformly set to 100 Hz. The multi-sensor data were synchronized based on timestamps and resampled using linear interpolation. Accordingly, the time interval between adjacent samples was 0.01 s, and a sliding window of 32 samples corresponded to a local sensor sequence of 0.32 s. The peak detection threshold was set to 0.6g, the lower-bound constraint radius for geomagnetic matching was set to 6, the initial covariance of the Sage-Husa filter was set to 0.01, the feature weight of the improved CRF was set to 0.7, and the maximum number of iterations was set to 6. The study set up a simulation test platform and conducted experiments on the Ubuntu 22.04 LTS operating system. The specific configuration of the experimental environment is shown in Table 1.

Table 1. Detailed information of the testing platform

-	Configure	Parameter
Hardware configuration	CPU	Intel Core i9-14900K
	Storage	1TB NVMe SSD+2TB HDD
Software configuration	Operating system	Ubuntu 22.04 LTS
	Numerical calculation library	NumPy 1.26.0
	Data processing tool	NetworkX 2.8.8
	Containerization tool	Docker 26.1.3

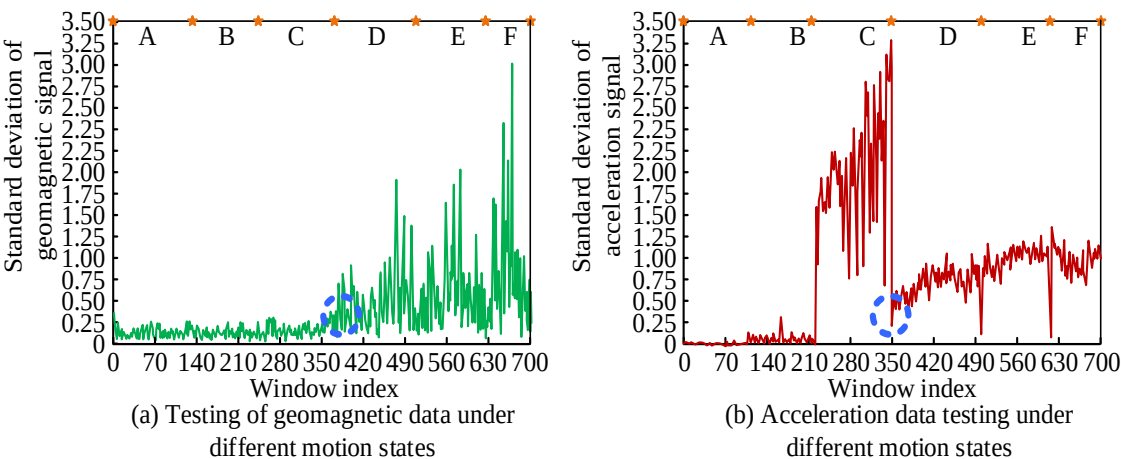


Fig. 7. Tests of geomagnetic standard deviation and acceleration standard deviation under different motion states

To ensure the reliability and statistical significance of the experimental results, all experiments were independently repeated five times, the average was taken as the final result, and the standard

deviation was calculated to measure the range of fluctuations. The experiment explored the effectiveness and limitations of the standard deviation of the geomagnetic signal and the standard deviation of the acceleration signal in differentiating the pedestrian in stationary, slightly wobbling, general wobbling, slow walking, normal walking, and fast walking (indicated by A, B, C, D, E, F, respectively) states. The experiment collected data from multiple sets of samples, calculated the standard deviation within the sliding window in each state, and compared the separation of the two types of signals between non-motion and motion states. The test results are shown in Figure 7.

As shown in Figure 7 (A), the geomagnetic standard deviations of the non-motion states (A, B, C) were all below 0.3, and the median and interquartile range of the three almost coincide; The standard deviations of the motion states (D, E, F) were all above 0.3 and showed an upward trend with the increase of step speed. The reason was that the position of the equipment was fixed or only slightly moving when it was not in motion, and the magnetic field environment was stable; When moving across different spaces and experiencing magnetic field variations, geomagnetic fluctuations increased. As shown in Figure 7(b), the standard deviations of acceleration remain relatively low under the stationary and slight-jolt states, whereas the value under the general-jolt state C increases markedly and partially overlaps with those of slow walking D and normal walking E. This overlap indicates that relying solely on acceleration features may cause local severe jolts to be misclassified as actual walking states, thereby affecting subsequent gait detection and PDR trajectory estimation. As shown in Figure 7(a), the geomagnetic standard deviation under state C remains below 0.3, while those under the walking states D, E, and F are all above 0.3. Using 0.3 as the secondary discrimination threshold, state C and the walking states are completely separated in the tested samples, corresponding to a separation rate of 100%. Therefore, when acceleration features cannot clearly distinguish the motion state, the geomagnetic standard deviation is introduced for second-level judgment, thereby reducing the ambiguity caused by overlapping acceleration features.

The experiment aimed to evaluate the positioning accuracy and stability of the geomagnetic sequence matching algorithm based on UCR-DTW in different indoor environments. Twenty test sequences of approximately 2.4 m in length were collected in each area, including Zone 1 (strong magnetic interference area), Zone 2 (conventional corridor area), and glass corridor area (weak magnetic interference area) on the third floor of the School of Environmental Measurement. Positioning was carried out using the research proposed methodology and the fastDTW algorithm, with positioning error as the indicator. The test results are shown in Figure 8.

As shown in Figure 8 (a), the probability of the research proposed methodology having a positioning error of less than 2 m in one area was approximately 45%, and the probability of less than 5 m was approximately 75%, while the fastDTW algorithm was slightly worse. Due to the presence of electronic lockers, computers, and other equipment in Zone 1, the magnetic field gradient changed sharply, and multiple similar waveforms were prone to occur within the short sequence, making it difficult for DTW matching to uniquely determine the position, resulting in an overall large positioning error. As shown in Figure 8 (b), the probability of the research proposed methodology having a positioning error of less than 2 m in Zone 2 was approximately 60%, and the probability of less than 5 m was more than 85%. The positioning performance was superior to that of Zone 1. Since Zone 2 was a common corridor, the magnetic field variation was mainly caused by the building structure, the variation pattern was monotonous, and the short sequence feature was unique. Therefore, the matching accuracy was high and the distribution of positioning errors was more concentrated. Figure 8 (c) indicates that the probability of the positioning error of the study proposed methodology being less than 1m was approximately 40%, and the probability of being less than 2 m was more than 75%. The positioning accuracy was significantly better than that of Zone 1 and Zone 2. Due to the less ferromagnetic interference in the glass corridor area, the geomagnetic signal was derived from natural variations, the noise was low and the similarity of short sequences was low, so the probability of mismatching was minimal.

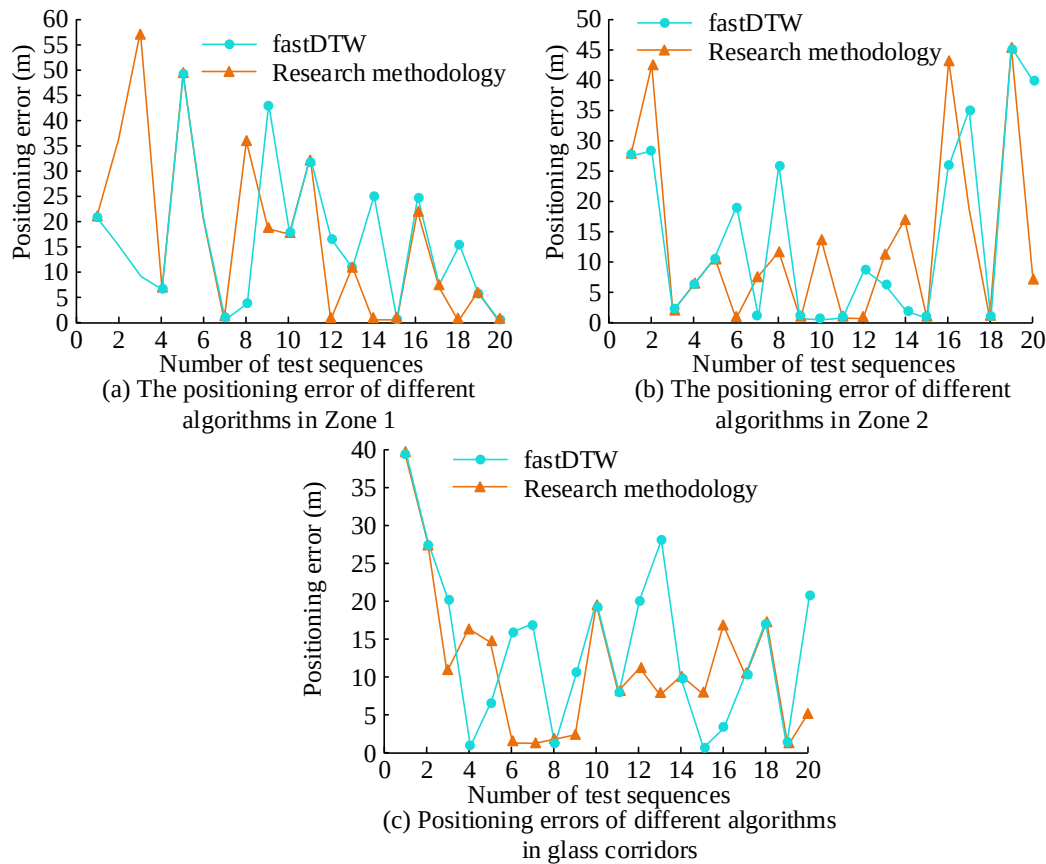


Fig. 8. Positioning error test of geomagnetic matching algorithm in different indoor areas

4.2 Verification of Accuracy and Real-Time Performance of Fusion Positioning Algorithm

The experiment was conducted to verify the advantages of the integrated fusion positioning framework combining Sage-Husa adaptive Kalman filter with improved CRFs in terms of accuracy and real-time performance by comparing two benchmarks proposed methodologies: traditional Kalman +CRFs and only improved CRFs. On the IPIN 2025 indoor positioning dataset, positioning errors were counted along eight consecutive path points, and the time spent on a single positioning was tested during the 15 s – 31 s consecutive positioning period to evaluate the accuracy and real-time performance of the algorithm. The test results are shown in Figure 9.

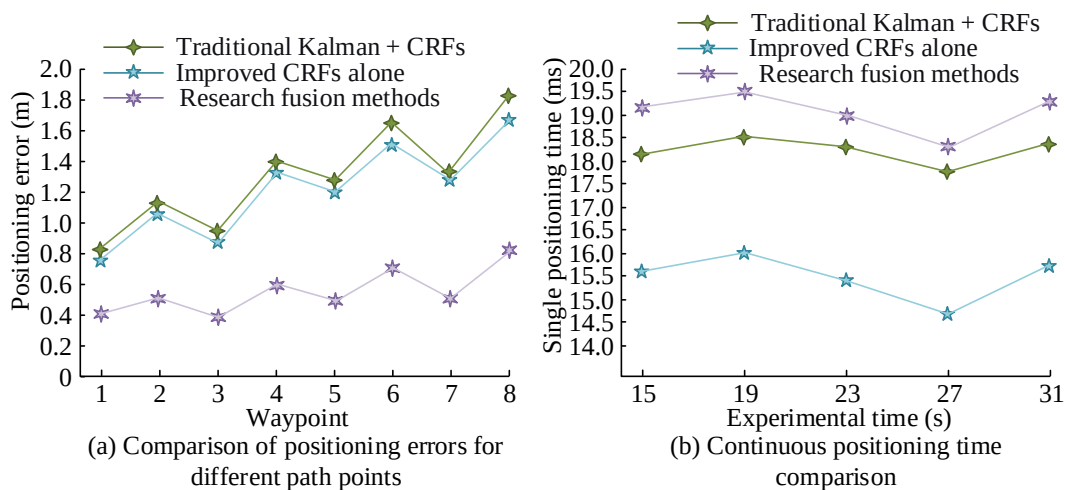


Fig. 9. Positioning accuracy and real-time test of adaptive filtering and improved CRF fusion

As shown in Figure 9 (a), with the advancement of path points, the positioning errors of all three proposed methodologies showed a slight upward trend. The error of the research fusion proposed methodology was always the lowest, with an average error of approximately 0.53 m for the 8 path points, which was about 58.2% lower than the traditional Kalman + CRFs and about 47.6% lower than the improved CRFs alone. The reason was that the Sage-Husa adaptive filter dynamically corrected noise and suppressed inertial cumulative error, while the improved CRFs provided spatial constraints on the map, and they complemented each other. The mean positioning error of 0.53 m reported here corresponds specifically to the eight consecutive path points used in Figure 9, whereas the overall mean error of 0.45 m in Table 2 is obtained from the complete fusion-positioning evaluation. As shown in Figure 9 (b), the time consumption of the three proposed methodologies remained stable in 15 s - 31 s continuous positioning. The single time consumption of the study fusion proposed methodology was between 18.3ms and 19.5 ms, slightly higher than the other two proposed methodologies, but the overall fluctuation was less than 1.2 ms, and the real-time performance met the indoor positioning requirements. The reason was that the fusion proposed methodology added adaptive filtering and multi-source fusion, but used lightweight feature extraction and Viterbi optimization, which was less costly. In addition, the state-update latency after a velocity change is mainly determined by the motion-state recognition window and the computation time of the fusion positioning process. The sampling frequency is 100 Hz and the sliding-window length is 32 samples; therefore, one complete local sequence corresponds to 0.32 s. When the motion state switches among slow, normal, and fast speeds, the DTW module updates the sliding window to re-identify the current motion state and simultaneously adjusts the peak-detection parameters. Under full-window updating, the algorithmic response time to a state change is approximately 0.32 s. With an additional fusion-positioning computation time of approximately 19.1 ms per update, the total response latency remains within approximately 0.34 s. This analysis indicates that the proposed method can complete velocity-state updating and position correction within a sub-second time scale.

The necessity and contribution of the core modules of PDR, geomagnetic matching, improved CRFs, and Sage-Husa adaptive filtering were verified through ablation experiments. Five sets of comparisons were set up, with key modules retained/added in sequence as indicators of average positioning error, maximum drift distance, positioning real-time performance, and trajectory integrity rate. The test results are shown in Table 2.

Table 2. Comprehensive test of ablation experiment

Test type	Average positioning error (m)	Maximum drift distance (m)	Positioning real-time (ms)	Trajectory integrity rate (%)
Basic PDR	2.75	5.82	12.60	62.40
PDR + Geomagnetic Matching	1.85	4.13	14.30	71.80
PDR + Improved CRFs	1.11	2.06	16.80	89.30
PDR + SAGE-Husa Filtering	0.94	1.75	15.50	86.70
Proposed Fusion proposed methodology	0.45	0.81	19.10	98.20

As shown in Table 2, performance improved step by step from the basic PDR to the full fusion proposed methodology. The average error of the base PDR was 2.75 m, the maximum drift was 5.82 m, and the trajectory completeness rate was only 62.4%. With the addition of geomagnetic matching, the average error was reduced to 1.85 m and the completeness rate was 71.8%; With the addition of the improved CRFs, the average error was 1.11 m and the integrity rate was 89.3%; The average error was 0.94 m after adding Sage-Husa filtering. The study complete proposed methodology fused all modules, with an average error of only 0.45 m, a maximum drift of 0.81 m, a trajectory completeness rate of 98.2%, and real-time performance of 19.1 ms, meeting the requirements of high-precision positioning. Geomagnetic assistance for position features, improved CRFs for spatial constraints, and Sage-Husa filtering to suppress cumulative errors - all three were indispensable.

4.3 Improved CRFs Map Matching Convergence and Anti-Interference Analysis

To verify the performance of the improved CRFs map matching proposed methodology in terms of convergence speed and anti-interference ability, the matching convergence performance was tested by different iterations, the robustness was tested under interference intensity levels 1 to 5, and the matching accuracy rate, effective constraint rate, and spatial compliance rate were statistically analyzed. Levels 1-5 corresponded to 5, 10, 15, 20, and 25 μT , respectively. The test results are shown in Figure 10.

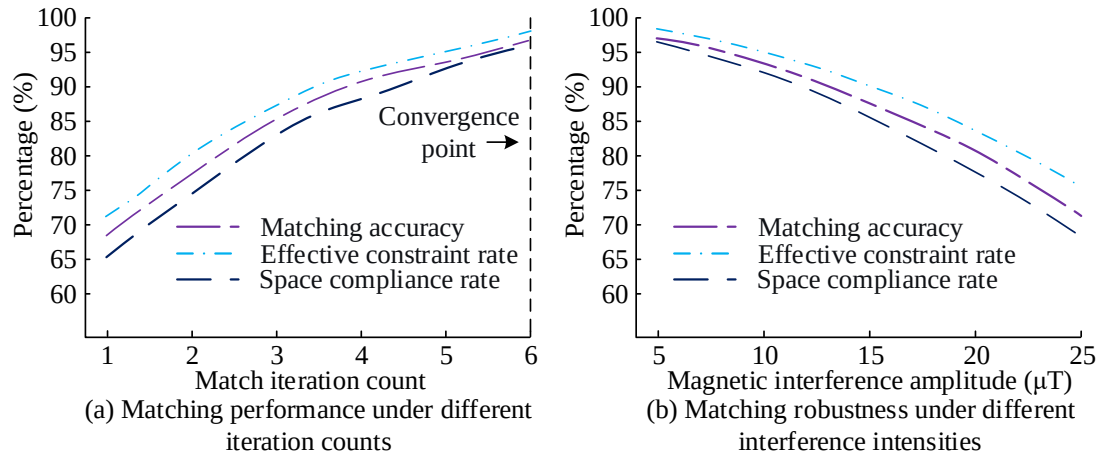


Fig. 10. Improved CRF map matching convergence and anti-interference performance test

From Figure 10 (a), with the increase of the number of iterations, the matching accuracy rate, the effective constraint rate and the spatial compliance rate continued to rise and converged rapidly, reaching 96.8%, 97.9% and 95.7% respectively after 6 iterations. The convergence curves become progressively flatter near the sixth iteration, indicating that further iterations would provide only limited accuracy improvement while increasing computational cost. Therefore, six iterations were adopted as a practical compromise between matching performance and processing efficiency. The reason was that the improved CRFs optimized the feature function and weight adaptation, quickly integrated geomagnetic and topological constraints, and eliminated invalid points one by one, allowing the map constraints to be fully exerted. As shown in Figure 10 (b), the three indicators gradually decreased as the interference level increased, but the improved CRFs decreased gently, maintaining a matching accuracy rate of 71.6% and a spatial compliance rate of 68.4% under level 5 strong interference. The reason was that the proposed methodology introduced tolerance constraints and local space limitations to reduce the influence of magnetic interference and noise; High trajectory compliance and matching accuracy are maintained in strong interference environments by weighting adaptive enhancement of reliable features and weakening of anomaly observations.

5. Conclusion

In response to the practical problems of satellite signal unavailability, complex magnetic interference, severe inertial positioning drift, and trajectories crossing walls or boundaries in smart buildings, this study focused on improving the positioning accuracy, robustness, and real-time performance of indoor pedestrian positioning and constructed an integrated positioning system combining adaptive PDR, geomagnetic matching, and map constraints. The system uses acceleration and geomagnetic dual features to identify motion states, applies DTW-based adaptive gait detection and robust adaptive Kalman filtering to improve the reliability of PDR estimation, and incorporates spatial topological constraints through an improved CRF combined with UCR-DTW for efficient geomagnetic sequence matching. The estimated trajectory is further corrected to the indoor map through the joint optimization of Sage-Husa adaptive filtering and the improved CRF. Experimental results showed that the complete fusion method achieved a mean positioning error of 0.45 m, a maximum drift of 0.81 m, a trajectory integrity rate of 98.2%, and a single-

positioning time of less than 20 ms. The improved CRF achieved a matching accuracy of 96.8% after six iterations and maintained a matching accuracy of 71.6% under strong magnetic interference of 25 μ T. These results demonstrate the practical potential of the proposed method for continuous pedestrian positioning and location-assisted services in smart buildings. However, the method still shows limited stability in areas with dense strong magnetic interference and during short-sequence geomagnetic matching, and height information has not yet been fully integrated for multi-floor transitions. Future work will further investigate multi-floor topological constraints, height information, and multimodal sensing fusion to improve robustness in complex environments and cross-floor positioning performance.

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IYOLOv7-tiny-Weld: A lightweight model for ship laser welding defect detection on complex curved surfaces

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Abstract

Laser welding is a key joining process for large complex curved ship components, where precise real-time weld defect detection is critical to hull structural safety. However, current lightweight detection models struggle to simultaneously achieve high accuracy for tiny defects and high inference speed under industrial conditions, due to challenges such as low-contrast imaging, varying surface reflections from curved geometries, and the multi-scale nature of defect features. To address these issues, this paper proposes an improved YOLOv7-tiny model named IYOLOv7-tiny-Weld (Improved YOLOv7-tiny for Weld Defect Detection). A Decoupled Self-Attention (DSA) module is embedded to enhance local defect feature extraction, and Group Shuffle Convolution (GSConv) is adopted to offset the extra computational overhead introduced by DSA. The model further incorporates a Weighted Bidirectional Feature Pyramid Network (BiFPN) and Content-Aware Reassembly of Features (CARAFE) to achieve superior multi-scale feature fusion and reconstruction. In addition, Decoupled Detection Heads (DDH) are utilized to mitigate the conflicts between classification and regression tasks. Experiments are conducted on a self-built ship laser welding dataset containing 3,200 images, as well as on the public WELD dataset. On the self-built ship laser welding dataset, the model attains a median detection confidence of 89.75% and a median IoU of 82.16%. The model achieves AP values of 92.56%, 89.14%, and 85.43% for porosity, undercut, and lack of fusion, respectively. Under low illumination and long-distance conditions, the precision drops by only 4.17% and 3.41%, outperforming other compared models. Compared with the baseline YOLOv7-tiny, the proposed model achieves a 12.62 percentage point improvement in mAP@0.5 on the public WELD dataset. The modified YOLOv7-tiny achieves simultaneous advances in accuracy, inference speed, and environmental robustness, offering an efficient lightweight solution for real-time online ship laser welding quality monitoring.

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1. Introduction

Laser welding offers concentrated energy and minimal thermal deformation, making it the mainstream advanced process for welding and forming large curved components in shipbuilding [1]. However, the welding process is susceptible to disturbances from process parameters, workpiece conditions, and environmental factors, which readily induce defects such as porosity, cracks, lack of fusion, and undercuts. These defects not only degrade the mechanical properties of welded joints but also directly affect the overall structural strength and service safety of ship components, necessitating the development of efficient and accurate defect detection methods [2]. In complex curved surface welding, curvature variations alter the laser incident angle and focal position, resulting in uneven heat input and unstable defect morphology. During image acquisition, varying reflection angles and shooting distances further introduce non-uniform illumination, scale

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variation, blur, and geometric distortion, making defect detection significantly more challenging than on flat weld surfaces. Traditional ship weld inspection primarily relies on manual visual inspection, ultrasonic testing, and radiographic testing, which generally suffer from low efficiency, strong subjectivity, and the inability to perform online real-time screening, making them difficult to adapt to the quality inspection demands of intelligent shipbuilding production lines [3]. With the continuous advancement of deep learning-based vision detection technologies, real-time online inspection requires a detector that balances speed and accuracy. The YOLO (You Only Look Once) single-stage detection algorithm, offering fast detection speed, high recognition accuracy, and strong automation capabilities, provides an effective technical solution for online intelligent inspection of ship laser welding defects [4].

In weld defect detection, scholars conducted systematic studies around acoustic feature analysis and visual feature extraction. Yusof M F et al. based on pulsed laser welding acoustic signals, used a wavelet denoising method integrating Z-score threshold. Their constructed classification model reached 95% accuracy for weld defect recognition [5]. Qiu Y et al. targeted the laser-arc hybrid welding process. They designed a hybrid pooling attention module and embedded it into a Convolutional Neural Network (CNN). The recognition accuracy reached 95.02% on a test set of four types of defects [6]. Li H et al. developed a CNN detection algorithm based on a molten pool visual sensing system. They predicted defects by analyzing the morphology of molten pool dark spots and semi-solidified regions, and the accuracy exceeded 95% [7]. Zhang M et al. pointed out that deep learning could effectively make up for the shortcomings of traditional methods. However, it still faced interdisciplinary challenges of deep integration between algorithm design and welding process [8]. These studies proved that deep learning could detect weld defects well. However, lightweight detectors such as YOLOv7-tiny still have limitations in curved-surface weld defect detection. Standard convolutions mainly extract local features with relatively fixed receptive fields, making it difficult to model long-range dependencies in irregular weld morphologies. Meanwhile, repeated down sampling and insufficient multi-scale feature fusion may weaken small defect textures and boundary details, especially for pores, cracks, and incomplete penetration with large scale variations.

Under this background, the single-stage detection algorithm represented by YOLOv7 has become the mainstream path of industrial visual detection due to its excellent balance of precision and speed. For weld defect detection, Xu X et al. improved YOLOv7. They embedded the Le-HorBlock module in the backbone network to achieve second-order spatial interaction to retain high-order features, and introduced a coordinate attention mechanism to strengthen target representation. Their mAP@0.5 reached 78.6% [9]. Gu Q et al. proposed a surface defect detection model for prefabricated steel pipes based on YOLOv7-tiny. This model integrated a squeeze-and-excitation attention mechanism and distribution-shifted convolution. The calculation amount decreased by 39.4%, and the mAP increased by 1.9% [10]. Barman J et al. modified YOLOv7-tiny for fabric defect detection. They used an enhanced residual convolutional network and a double-splicing structure to strengthen hierarchical feature extraction. Their accuracy reached 98%, and the model could identify 1 mm micro defects [11]. Huang Y C et al. designed anchor-free light-weight YOLOv7-tiny network architecture. In this work, a multi-scale attention module and adaptive spatial feature fusion bidirectional feature pyramid were developed. Its accuracy has been improved by 6.24% and the speed is above 90 FPS [12].

Similarly, Ancha V K et al. developed TRSBi-YOLO, GSS-YOLO, and CSPGhost-YOLO for PCB defect detection through module-level optimizations such as C3TR, GSConv, GhostC3, and attention mechanisms [13-15]. The researches have demonstrated the potential of YOLOv7-tiny in industrial defect detection. Nevertheless, most existing studies have been conducted on flat or nearly planar weld surfaces, where defect appearance and imaging conditions are relatively stable. In contrast, complex curved surfaces introduce geometric distortion, non-uniform reflection, and apparent scale changes of defects, which significantly increase the difficulty of feature extraction and localization. Therefore, curved-surface weld defect detection should not be treated as a simple extension of flat-surface detection.

In addition, the introduction of attention mechanisms proved effective in enhancing the feature response of the model to defect areas. Narayanasamy S A et al. combined focus super-resolution with a self-attention-based single-candidate optimization strategy to construct an adversarial network for enhancing images of laser beam welding defects. This method effectively improved the visibility of defect regions [16]. Mo H et al. introduced the concept of decoupled focused self-attention for disease detection on cotton leaf. This approach ensured the dynamic tuning of self-attention weights and improved the relationship between features [17]. Guo Y et al. suggested a decoupled and self-attention routing in the capsule network. They modeled the relationships between channel and spatial dimension separately. This approach resulted in fewer parameter calculations [18]. These studies indicated that the Decoupled Self-Attention (DSA) design could effectively reduce computational complexity and improve the relevance of feature extraction. However, its application to laser welding defect detection for large-scale shipbuilding components with complex curvatures remains unexplored.

Therefore, to break through the technical bottlenecks of difficult fine-grained feature extraction, insufficient multi-scale fusion, and limited lightweight deployment in complex curved surface laser welding defect detection, this study uses YOLOv7-tiny as the basic architecture. This study proposes the IYOLOv7-tiny-Weld (Improved YOLOv7-tiny for Weld Defect Detection) model for laser welding defect detection of large components with complex curved surfaces. By integrating DSA and multi-scale feature enhancement, the model achieves accurate recognition and lightweight deployment of weld defects. It provides efficient and reliable technical support for online inspection of laser welding quality for large ship components with complex curved surfaces. The main contributions of this study are as follows:

- A curved-surface laser welding defect dataset for large shipbuilding components is constructed, covering typical defects such as pores, cracks, undercut, and incomplete penetration under complex imaging conditions.
- Group Shuffle Convolution (GSConv) and Decoupled Self-Attention (DSA) are introduced to enhance lightweight feature extraction. GSConv reduces computational redundancy, while DSA strengthens the modeling of long-range dependencies and irregular defect morphology.
- A BiFPN-based feature fusion structure combined with Content-Aware Reassembly of Features (CARAFE) up sampling is designed to improve multi-scale representation. Their joint use helps normalize the scale variance caused by curved surfaces and preserves fine defect boundaries during feature reconstruction.
- A Decoupled Detection Head (DDH) is adopted to separate classification and localization tasks, reducing the conflict between category recognition and bounding-box regression and improving detection robustness for defects with ambiguous boundaries.

2. Defect Detection Model Based on Improved Yolov7-Tiny

2.1 Overall Architecture of Improved Yolov7-Tiny

Online industrial weld seam inspection demands strict real-time algorithm performance and tiny defect recognition capabilities. Especially in the shipbuilding field, a large number of large components with complex curved surfaces are involved. Their laser welding seams extend along three-dimensional spatial curves, which feature large curvature changes and complex spatial directions. Moreover, the significant difference in surface reflection characteristics between the weld seam and the base metal further increases the difficulty of defect feature extraction [19]. The lightweight YOLOv7-tiny has a fast inference speed and balanced feature extraction performance, which fits on-site deployment requirements. Therefore, this study adopts it as the baseline model. Meanwhile, aiming at its shortcomings in feature representation and localization, this study optimizes the backbone network, neck network, and detection heads respectively to construct the IYOLOv7-tiny-Weld model. Its structure is shown in Figure 1.

As shown in Figure 1, the input layer performs uniform preprocessing on laser welding images and crops them to a size of 640×640 pixels. The backbone network embeds DSA inside the original Efficient Layer Aggregation Network (ELAN) module to reinforce the feature extraction of fine-grained weld defects. Meanwhile, GSConv replaces part of standard convolutions to compensate for

the computational increment brought by DSA, ensuring the lightweight characteristics of the model. Furthermore, to enhance feature reconstruction, the original interpolation upsampling is replaced with the CARAFE operator.

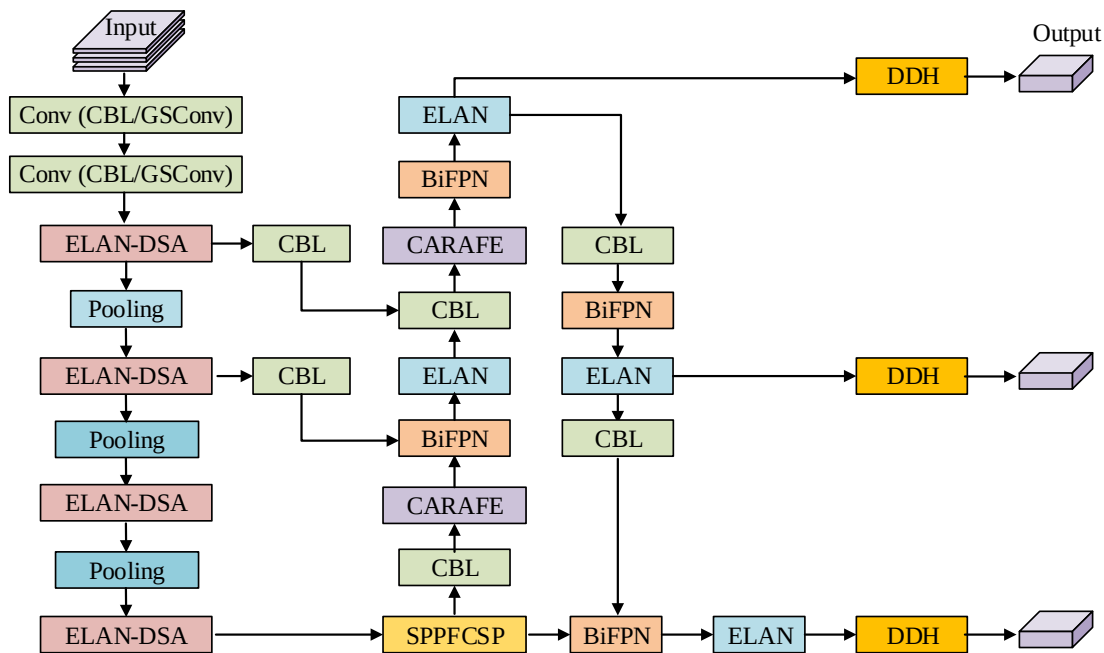


Fig. 1. Overall architecture of the IYOLOv7-tiny-Weld model

In the original coupled detection head, classification and localization share the same convolutional features. However, classification mainly relies on semantic information for category discrimination, whereas localization requires geometry-sensitive features for accurate boundary regression. Previous studies have shown that this inconsistency in feature requirements can lead to task conflict in object detection heads [20]. Therefore, DDH is introduced to separate the classification and localization branches, allowing each branch to learn task-specific representations. The structural difference between the traditional coupled detection head and the proposed DDH is illustrated in Figure 2.

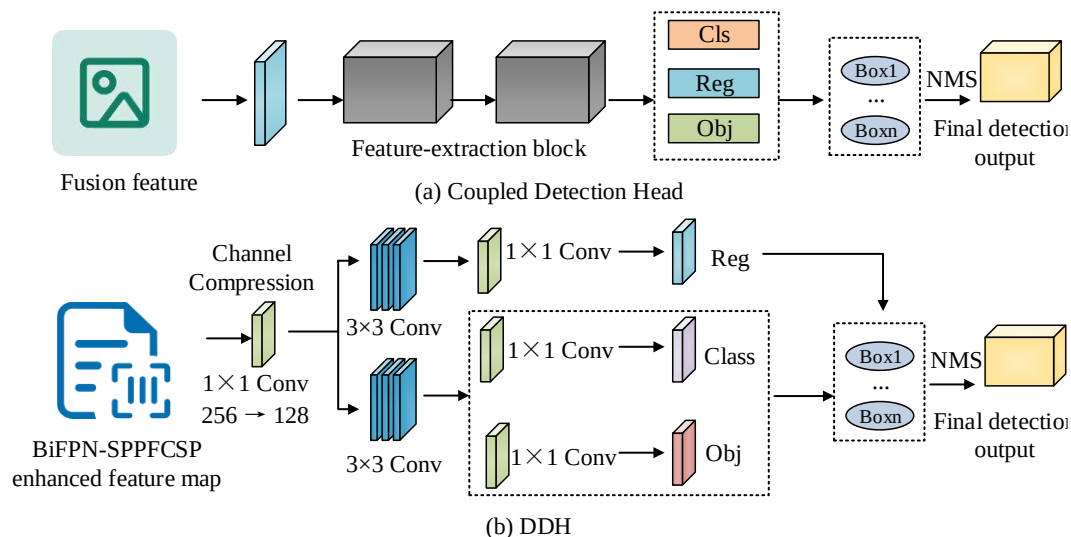


Fig. 2. Structural comparison between standard coupled detection head and DDH

As shown in Figure 2(a), the traditional coupled detection head uses shared convolutional layers to perform classification, objectness prediction, and bounding-box regression simultaneously. Since classification mainly depends on semantic features, whereas localization requires geometry-sensitive boundary features, the shared convolutional structure may cause feature conflict between the two tasks. In contrast, Figure 2(b) shows the proposed DDH structure. After channel

compression through a 1×1 convolution, the enhanced feature map is fed into independent branches for classification, localization regression, and objectness prediction. The classification branch focuses on semantic defect-category information, while the regression branch learns geometry-related features for accurate bounding-box localization. Finally, Non-Maximum Suppression (NMS) is applied to remove redundant candidate boxes and retain the optimal detection results.

2.2 Improved Backbone Network Based on DSA And Gsconv

The native ELAN module of YOLOv7-tiny only relies on standard convolutions to extract features. Limited by local receptive fields, it is difficult to establish long-distance dependencies between pixels. Laser weld defects are scattered and irregular in morphology, and local textures are closely linked with global information. Therefore, this study embeds the DSA structure inside the ELAN module to strengthen feature expression in defect areas, as shown in Figure 3.

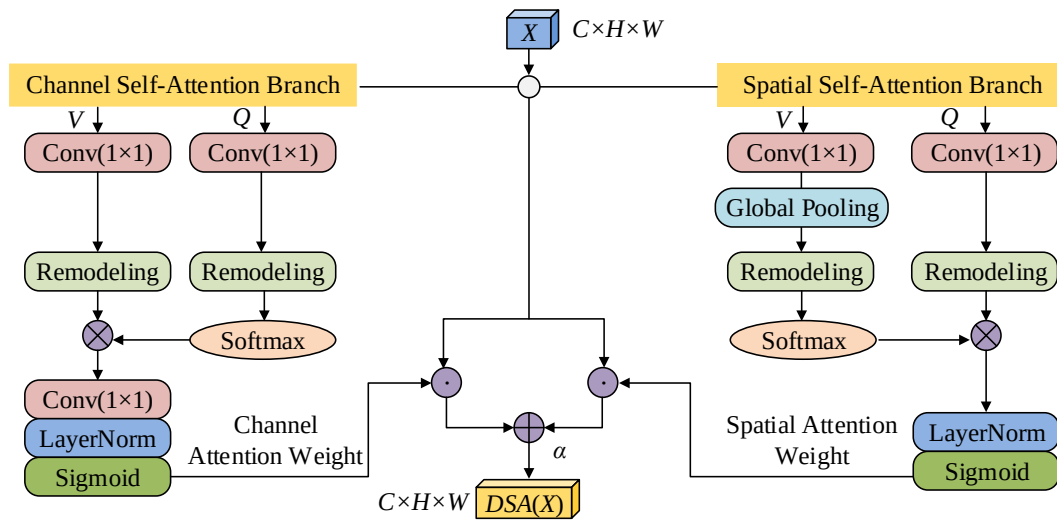


Fig. 3. Structure of the improved ELAN module with DSA

In Figure 3, DSA splits the self-attention mechanism into two independent branches: spatial attention and channel attention. They model the spatial position correlation and channel semantic correlation of feature maps respectively. Then, with the help of learnable parameters, the two outputs are adaptively fused, controlling computational overhead while strengthening feature representation capabilities. The feature tensor input to the backbone network undergoes linear mapping. The Remodeling blocks in each branch implement Reshape and Permute operations to rearrange feature tensor dimensions for attention computation. The basic scaled dot-product attention calculation method of the module is shown in Equation (1).

$$Attn(Q, K, V) = softmax\left(\frac{QK^T}{\sqrt{d_k}}\right)V \quad (1)$$

After the basic attention construction is completed, the spatial attention branch models contextual information within a single channel to extract structural features such as weld seam direction, contour morphology, and spatial distribution of defects. The channel attention branch fixes spatial coordinates and strengthens exclusive semantic features corresponding to different defects through cross-channel fusion. The operation method is shown in Equation (2).

$$\begin{cases} Attn_{sp}(X) = [Attn_{sp}(X_c)]_{c=1}^C \\ Attn_{ch}(X) = [Attn_{sp}(X_{hw})]_{h=1, w=1}^{H, W} \end{cases} \quad (2)$$

In Equation (2), $Attn_{sp}$ and $Attn_{ch}$ represent the operation results of spatial-dimension self-attention and channel-dimension self-attention, respectively. X_c is the two-dimensional space of the c -th channel, and X_{hw} is the cross-channel feature set under the same spatial position. To achieve adaptive fusion of the two types of attention features, this study introduces learnable weights and combines network layers to regulate the intensity of cross-channel interaction. Let l

be the current module layer index, and $\mathcal{M}=\{4,6,8\}$ be the set of network layers that enable cross-channel interaction. This selection is based on empirical experiments: shallow layers (1-3) primarily extract basic spatial features and are sensitive to channel attention, while deep layers (10-12) contain high-level semantics where cross-channel fusion brings limited gains. The middle layers (4,6,8) provide a balance between representational capacity and computational cost. When $l \in \mathcal{M}$, DSA simultaneously enables both branches and completes feature fusion, as shown in Equation (3).

$$DSA(X) = Proj(\alpha \cdot Attn_{ch}(X) + (1 - \alpha) \cdot Attn_{sp}(X)) \quad (3)$$

In Equation (3), α is the learnable weight parameter, initialized to 0.5, which dynamically allocates the feature proportion of spatial attention and channel attention. Proj is the feature output projection layer, which can unify the dimension specifications of fused features to guarantee the continuity of feature transmission in subsequent network layers. The introduction of the DSA module improves detection accuracy but also brings extra computational overhead. To maintain the lightweight level of the model, this study introduces the GSConv module to replace part of standard convolutions, as shown in Figure 4.

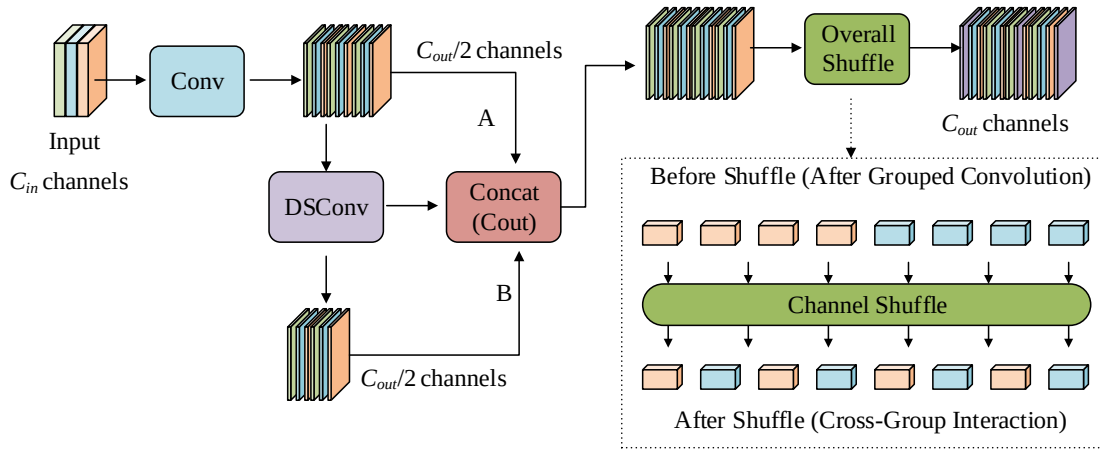


Fig. 4. Structure of the GSConv module

In Figure 4, GSConv consists of three sub-modules: standard convolution, Depthwise Separable Convolution (DSC), and channel shuffle. The Conv and DSConv branches generate $C_{out}/2$ channels, denoted as A and B, respectively. After concatenation, the feature map contains C_{out} channels, and the Overall Shuffle preserves the channel number while enabling cross-group information interaction. Its design is based on reducing the redundant computations of standard convolution through DSC, followed by restoring cross-group information flow via channel shuffle operations, thereby significantly lowering the parameter count and computational cost without sacrificing feature representation capability [21]. Among them, the parameter volume P_{SC} and computation volume F_{SC} of standard convolution is shown in Equation (4).

$$\begin{cases} P_{SC} = C_{in} \times C_{out} \times K \times K \\ F_{SC} = C_{in} \times C_{out} \times K \times K \times H' \times W' \end{cases} \quad (4)$$

In Equation (4), C_{in} and C_{out} represent the number of input channels and output channels, and K represents the side length of the convolutional kernel. When the convolutional stride is 1, $H' = H$, $W' = W$; when the stride is 2, $H' = H/2$, $W' = W/2$. DSC combines depthwise and pointwise convolutions, in which the former processes each input channel independently, while the latter performs cross-channel feature fusion using 1×1 convolution. Its calculation is shown in Equation (5).

$$\begin{cases} P_{DSC} = C_{in} \times K \times K + C_{in} \times C_{out} \\ F_{DSC} = C_{in} \times K \times K \times H' \times W' + C_{in} \times C_{out} \times H' \times W' \end{cases} \quad (5)$$

In Equation (5), P_{DSC} and F_{DSC} are the parameter volume and total floating-point operations of DSC. Thus, the computing power ratio of DSC relative to standard convolution is calculated as shown in Equation (6).

$$\frac{F_{DSC}}{F_{SC}} = \frac{1}{C_{out}} + \frac{1}{K^2} \quad (6)$$

In Equation (6), the value of C_{out} is usually large, so the $\frac{1}{C_{out}}$ term can be approximately ignored. The overall operation volume drops to about $\frac{1}{K^2}$ of standard convolution. Therefore, GSConv combines standard convolution with DSC and pairs them with channel shuffle to open up information interaction between grouped channels, which can effectively offset the extra power consumption brought by DSA. It is worth noting that GSConv is only applied to partial standard convolutions within the backbone rather than all convolutional layers. Replacing all backbone convolutions with GSConv would excessively weaken the model's feature extraction capacity for tiny, low-contrast weld defects, leading to obvious degradation of overall detection precision. This hybrid architecture achieves a favorable trade-off: GSConv offsets the extra computational overhead introduced by the DSA module, while retained standard convolutions preserve sufficient feature extraction capability for subtle micro-defect textures and edges.

2.3 Feature Fusion Network Based on Multi-Scale Feature Enhancement

In the original YOLOv7-tiny, the nearest neighbor interpolation is used in the process of up sampling. This technique can fill the features by fixed principles depending on their spatial positions but is unable to understand any semantics of weld defects. Thus, it will lead to missing some important information like small porosities and cracks' edges. This is why the CARAFE approach is used in up sampling, as illustrated in Figure 5.

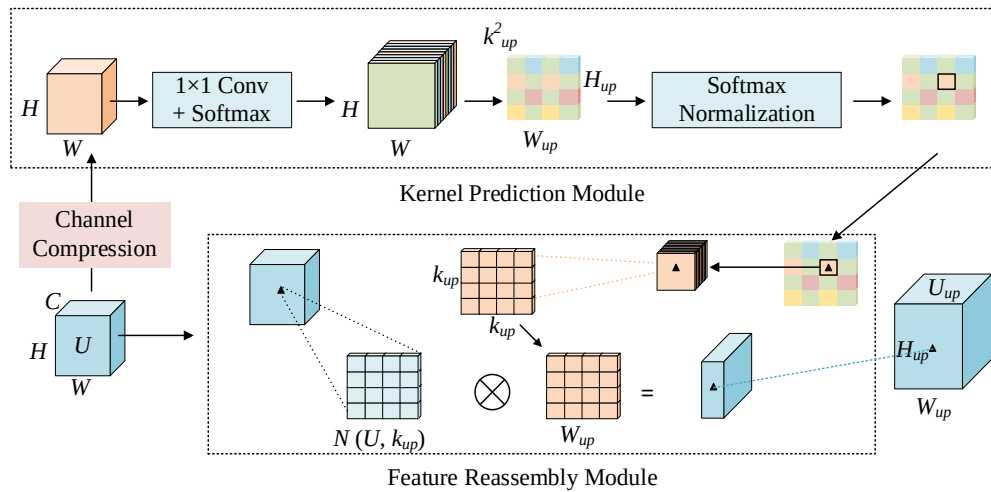


Fig. 5. Structure of the CARAFE module

Figure 5 illustrates the architecture of CARAFE, which includes an up-sampling kernel prediction module and a feature reassembly module. The former adaptively generates position-specific kernels from the input features, while the latter uses these kernels to reconstruct the feature map. Inspired by the effectiveness of feature reassembly and content-aware up sampling in steel surface defect detection and dense object detection [22-23], CARAFE is introduced in this study to better preserve boundary details in weld images. This is particularly beneficial for pores, cracks, and lack-of-fusion defects, which usually exhibit irregular shapes and weak boundaries. The upsampling kernel prediction process is shown in Equation (7).

$$\begin{cases} W_{up} = \psi(U) \\ W_{up} \in \mathbb{R}^{k_{up}^2 \times H_{up} \times W_{up}} \end{cases} \quad (7)$$

In Equation (7), $U \in \mathbb{R}^{C \times H \times W}$ represents the input feature map of the upsampling module, $\begin{cases} W_{up} = \psi(U) \\ W_{up} \in \mathbb{R}^{k_{up}^2 \times H_{up} \times W_{up}} \end{cases}$ represents the generated sampling weight kernel, k_{up}^2 represents the upsampling kernel size, and H_{up} and W_{up} are the height and width of the upsampled feature map respectively. The feature reassembly module performs weighted reassembly of input features based on W_{up} . Its operation is shown in Equation (8).

$$U_{up}(i, j) = \sum_{p=1}^{k_{up}} \sum_{q=1}^{k_{up}} W_{up}(p, q, i, j) \cdot U(i + p - \lfloor k_{up}/2 \rfloor, j + q - \lfloor k_{up}/2 \rfloor) \quad (8)$$

In Equation (8), $U_{up} \in \mathbb{R}^{C \times H \times W}$ is the upsampling output feature map. To improve multi-scale feature fusion efficiency, this study embeds BiFPN into the neck network. The high-resolution features after CARAFE up sampling are sent into the BiFPN bidirectional fusion branch to perform adaptive weighted fusion with shallow features of the same scale. The multi-layer feature fusion calculation rule is shown in Equation (9).

$$O_{OUT} = \sum_i \frac{w_i}{\varepsilon + \sum_j w_j} \cdot I_i \quad (9)$$

In Equation (9), ε is a small constant set to 0.0001 to avoid division by zero. w_i is the feature weight updated autonomously during the network training process, and O_{OUT} is the composite feature after weighted fusion. The shallow layers contain molten pool boundary details, the middle layers correspond to defect contours, and the high layers carry global semantic information. To further enlarge the receptive field and enhance the recognition capability for defects of different specifications, this study further introduces the SPPFCSPC module. Its input feature map is $F \in \mathbb{R}^{C \times H \times W}$. Based on the SPPFCSPC structure in YOLOv7, the module gradually expands the feature perception area through multi-layer max-pooling operations [24]. The three-layer serial pooling operation is shown in Equation (10).

$$\begin{cases} R_1(F) = \text{MaxPool}_{k=5}^{p=2}(F) \\ R_2(R_1) = \text{MaxPool}_{k=5}^{p=2}(R_1) \\ R_3(R_2) = \text{MaxPool}_{k=5}^{p=2}(R_2) \end{cases} \quad (10)$$

In Equation (10), R_1 represents the first pooling output feature, used to extract fine-grained local features such as molten pool edges and tiny porosity. R_2 represents the second pooling output, specifically capturing the complete spatial structure of medium-sized defects like cracks and undercuts. R_3 represents the third pooling output, fitting global context extraction for large-scale defects such as large area lack of penetration and wide cracks. Concatenating and integrating the features of three different perception ranges along the channel dimension blends local details and global structural information, yielding the enhanced feature tensor R_4 , as shown in Equation (11).

$$R_4 = R_1 \otimes R_2 \otimes R_3 \quad (11)$$

In Equation (11), $\otimes$ represents the channel-dimension concatenation operation. The SPPFCSPC module adopts a multi-branch pooling paired with a cross-stage residual fusion structure, as shown in Figure 6. As illustrated in Figure 6, the pooling branches in the three groups of SPPFCSPC module are respectively dedicated to extracting features from defects of different scales, and the residual cross-connection can make the feature reuse ability become stronger. Thus, the model not only effectively regulates the total amount of computation but also enhances the effect of multi-scale detection, which meets the lightweight demands for real-time weld seam detection of industrial production lines. The BiFPN is responsible for the weighted fusion of cross-layer features, and the SPPFCSPC is responsible for the multi-scale receptive field extension of single-layer features. The multi-scale feature maps output from the SPPFCSPC module are fed as input features to the subsequent BiFPN for further cross-layer weighted feature fusion.

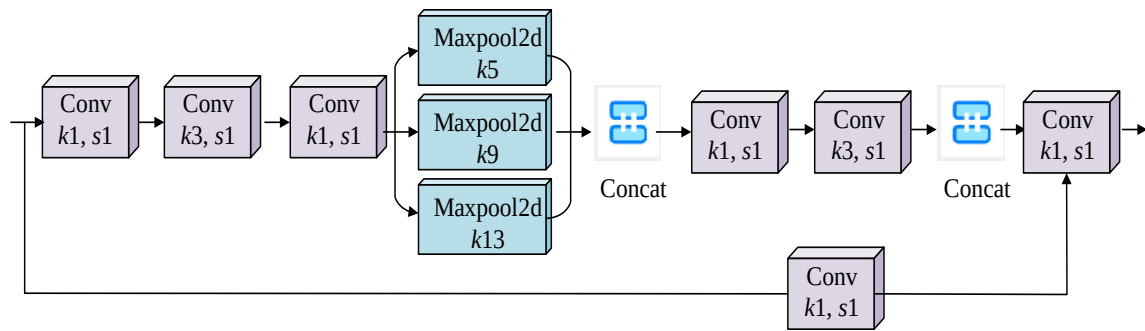


Fig. 6. Structure of the SPPFCSPC module

3. Laser Welding Defect Detection Results for Large Component with Complex Curved Surface

3.1 Performance Verification of Iyolov7-Tiny-Weld Model

Experiments were implemented using the PyTorch 1.11.0 deep learning framework with CUDA 11.3 on a platform equipped with an NVIDIA GeForce RTX 3090 GPU and an Intel Xeon Platinum 8255C CPU to evaluate the detection performance of the IYOLOv7-tiny-Weld model. During training, all input images were resized to 640×640 pixels. The model was optimized using SGD with an initial learning rate of 0.01, a momentum of 0.937, a weight decay of 0.0005, a batch size of 32, and 300 training epochs. Mosaic and MixUp data augmentation were employed to improve the model's generalization capability. Figure 7 provides schematic examples of the original weld sample, Mosaic stitching, MixUp blending, and the final network-fed training image.

The WELD dataset, publicly available on IEEE DataPort (DOI: 10.21227/k7rj-fk73), was used to evaluate the improved model. It contains high-resolution weld surface images with pixel-level annotations for defective regions, covering porosity, cracks, lack of fusion, undercuts, and pits. The dataset was randomly divided into training, validation, and test sets at a ratio of 7:1.5:1.5. The pixel-level annotations were converted into bounding-box annotations in YOLO format for object detection. To further investigate the limitations of IYOLOv7-tiny-Weld, representative failure cases from the test set were analyzed, as shown in Figure 8.

As shown in Figure 8, weld ripples easily trigger false positives, while tiny low-contrast defects often lead to missed detections. Complex weld textures also interfere with detection precision. These drawbacks reveal that micro-defect feature extraction needs further optimization. Ablation experiments were conducted to evaluate the contribution of each module. Starting from the baseline YOLOv7-tiny, the proposed modules (DSA, GSConv, CARAFE, BiFPN, SPPFCSPC, and DDH) were progressively integrated into the network. The results are shown in Table 1.

Table 1. Ablation experimental results on the public WELD dataset

No.	Modules	mAP@0.5 (%)	Params (M)	GFLOPs	Model Size (MB)	Latency (ms)	FPS
1	Baseline	76.31 ± 0.42	3.26	6.52	12.43	11.20	89.27
2	+ DSA	79.52 ± 0.38	3.41	6.98	13.01	11.53	86.74
3	+ GSConv	79.24 ± 0.35	3.28	6.37	12.51	11.31	88.51
4	+ CARAFE	81.43 ± 0.40	3.30	6.49	12.59	11.48	87.13
5	+ BiFPN	83.62 ± 0.36	3.33	6.61	12.73	11.65	85.86
6	+ SPPFCSPC	85.76 ± 0.33	3.35	6.68	12.78	11.82	84.62
7	+ DDH	88.93 ± 0.29	3.37	6.75	12.86	12.16	83.31

Note: All experiments were conducted on an NVIDIA RTX 3090 GPU with 640×640 input resolution. The mAP@0.5 values are reported as mean ± standard deviation over three independent training runs.

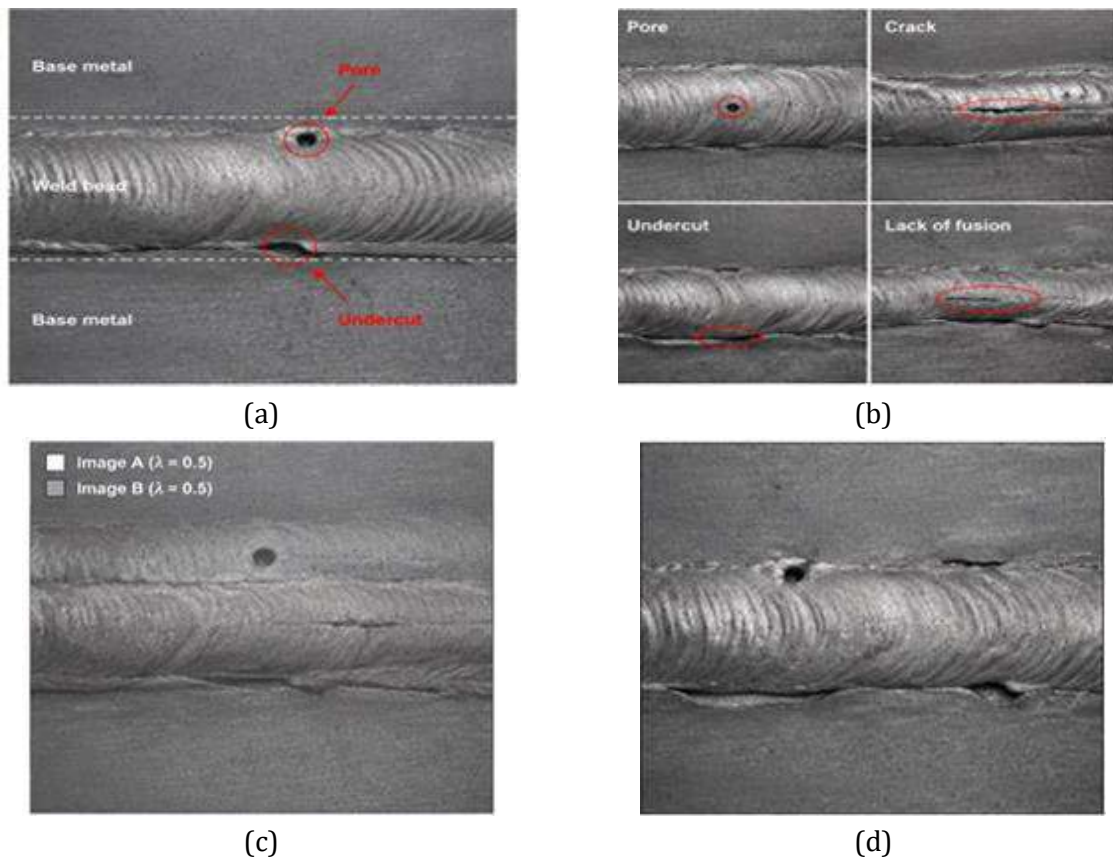


Fig. 7. Illustration of data augmentation strategies for weld defect detection (a) Original image (b) Mosaic augmentation (c) MixUp augmentation (d) Final training sample

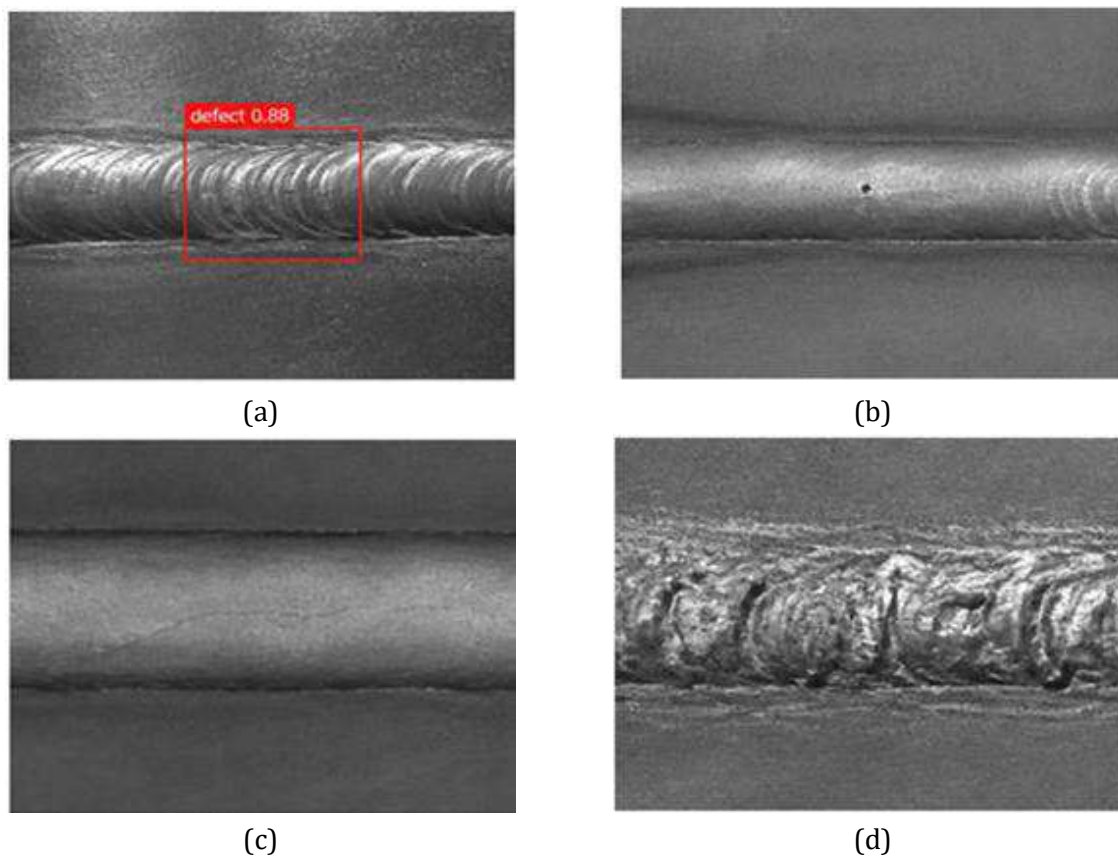


Fig. 8. Representative error and challenging detection cases (a) False alarm case (b) Missed detection case (c) Difficult sample: low-contrast micro-crack (d) Difficult sample: complex texture interference

Table 1 shows that the baseline YOLOv7-tiny achieves $76.31 \pm 0.42\%$ mAP@0.5 with 3.26 M parameters, 6.52 G FLOPs, 12.43 MB, 11.20 ms, and 89.27 FPS. Introducing DSA improves mAP by 3.21% with a slight increase in computational cost, confirming its effectiveness in enhancing fine-grained weld features. Adding GSConv reduces parameters to 3.28 M, FLOPs to 6.37 G, and model size to 12.51 MB while recovering FPS to 88.51 and latency to 11.31 ms, with only a 0.28% mAP drop, demonstrating its role in compensating for the overhead introduced by DSA. With the progressive integration of CARAFE, BiFPN, SPPFCSPC, and DDH, the model achieves 88.93% mAP@0.5 with 3.37 M parameters, 6.75 G FLOPs, 12.86 MB, 12.16 ms, and 83.31 FPS. Standalone ablation shows that DSA alone reaches 79.44% and DDH alone reaches 78.15%, confirming their independent contributions beyond the cumulative gains. To comprehensively evaluate the detection performance of IYOLOv7-tiny-Weld, this study analyzed the inference speed under different input resolutions and the changes in Average Precision (AP) under different Intersection over Union (IoU) thresholds. The proposed model was compared with several representative detectors: WDFCOS, a weld-specific detector based on FCOS; Cascade R-CNN, a classical two-stage detector; and two YOLO-series models—YOLOv7 and YOLOv8s—as strong single-stage baselines. These models were selected to cover diverse detection paradigms for a comprehensive evaluation. For fair comparison, all baseline models were retrained from scratch on the same dataset under identical experimental conditions, without using any pre-trained weights. The same input resolution, optimizer, learning rate, batch size, and training epochs were applied consistently across all models. No additional data augmentation or training strategies were employed beyond those used for the proposed model. The results are shown in Figure 9.

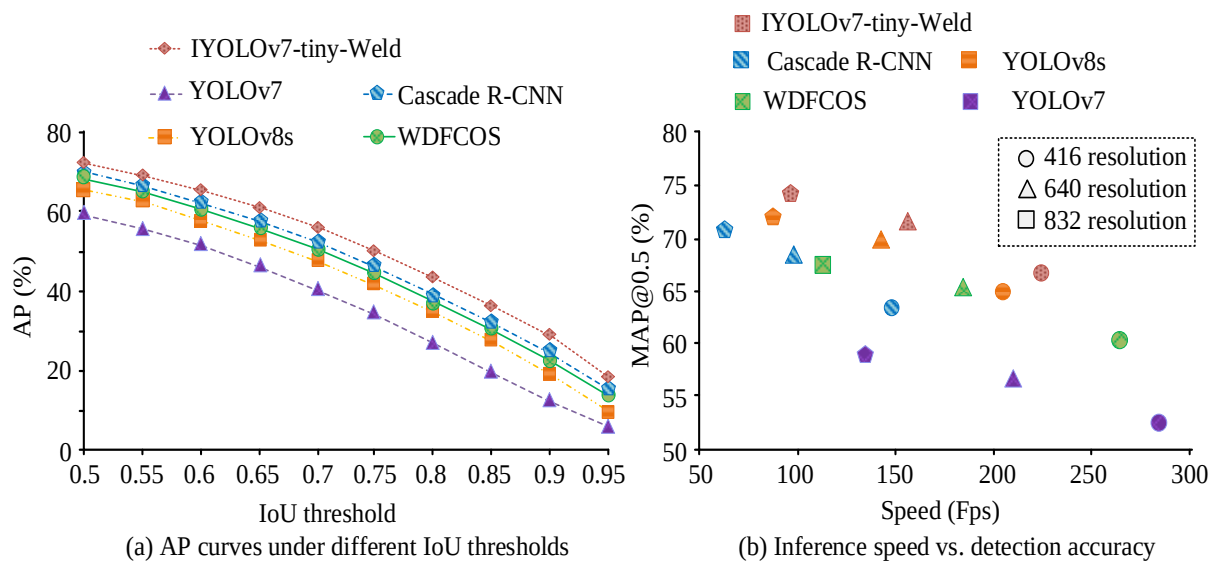


Fig. 9. Comparison of detection performance

Figure 9(a) shows that the curve of IYOLOv7-tiny-Weld had the gentlest downward trend. When the IoU thresholds were set to 0.5, 0.7, and 0.95 respectively, its AP values reached 72.86%, 61.27%, and 19.22% in sequence. This performance was clearly superior to the comparison models. In Figure 9(b), under different input scales, the inference frame rate of the IYOLOv7-tiny-Weld model reached 94.61 FPS and above, and the corresponding mAP@0.5 reached 67.58% and above, which was also better than the comparison models. This indicated that while IYOLOv7-tiny-Weld achieved high detection precision, its inference speed also increased significantly. Therefore, it is more suitable for deployment in actual online inspection scenarios of welding workpieces. To further evaluate the detection performance of IYOLOv7-tiny-Weld on different types of weld defects, this study counted the AP values and F1-Scores of each model for five categories of defects, including porosity, cracks, lack of fusion, undercut, and pits. The results are shown in Figure 10.

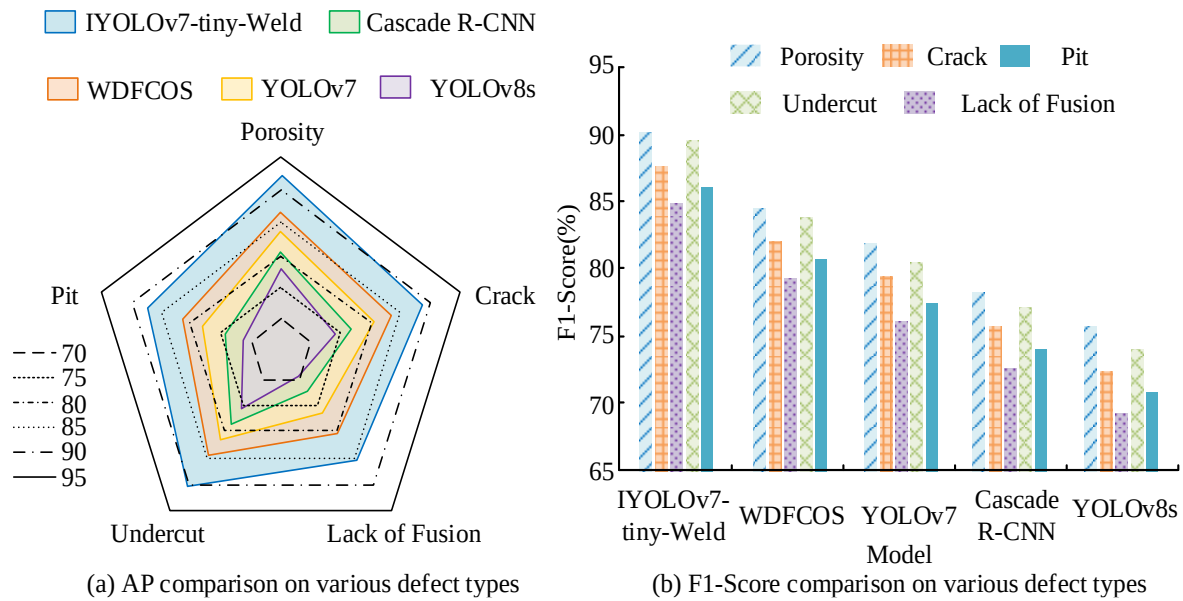


Fig. 10. Comparison of detection performance

Figure 10(a) shows that the AP of IYOLOv7-tiny-Weld on all five categories of defects was higher than that of other comparison models. Specifically, the AP of porosity and undercut reached 91.92% and 90.13% respectively, the AP of cracks and pits was 88.73% and 87.26% respectively, and the AP of lack of fusion defects was 85.44%. This advantage originated from the fact that DSA and DDH could effectively capture the differentiated features of different morphology defects. Especially, they had a stronger extraction capability for the edge semantic information of fine-grained defects. Figure 10(b) shows that the F1-Scores of IYOLOv7-tiny-Weld for the detection of porosity, cracks, lack of fusion, undercut, and pits were 90.22%, 87.63%, 84.81%, 89.55%, and 86.18% in sequence. This further verified the effectiveness of its collaborative design of multi-scale feature enhancement and DDH in complex weld defect recognition. To visually demonstrate the effectiveness of the proposed DSA module, Grad-CAM was employed to generate attention heatmaps, as shown in Figure 11.

As can be observed in Figure 11, the heatmap's high-response region precisely covers the porosity defect and overlaps the predicted bounding box (confidence 0.92). Only scattered weak activation occurs on surrounding weld ripples. This confirms that the network concentrates feature response on real defects rather than background textures, verifying the reliability and interpretability of IYOLOv7-tiny-Weld. To further verify the generalization capability of IYOLOv7-tiny-Weld, lightweight YOLO variants from YOLOv5-tiny to YOLOv12-tiny are selected for comparison on the public NEU-DET dataset, covering successive model iterations. All competing models are trained from scratch under identical configurations on the same dataset. The results are summarized in Table 2.

Table 2. Comparison with state-of-the-art lightweight detection models on the NEU-DET dataset

Model	mAP@0.5 (%)	Params (M)	GFLOPs	Model Size (MB)	Latency (ms)	FPS
YOLOv5-tiny	72.56	3.12	6.43	11.86	11.32	88.34
YOLOv7-tiny	73.84	3.26	6.52	12.43	11.20	89.27
YOLOv8-tiny	74.91	3.44	7.19	13.07	12.15	82.31
YOLOv10-tiny	75.68	3.56	7.36	13.53	12.42	80.52
YOLOv12-tiny	76.52	3.68	7.61	13.98	12.70	78.74
WDFCOS	79.35	14.86	49.72	56.47	20.41	49.00
Cascade R-CNN	82.18	68.75	198.64	261.25	125.00	8.00
IYOLOv7-tiny-Weld	85.66	3.37	6.75	12.86	12.16	83.31

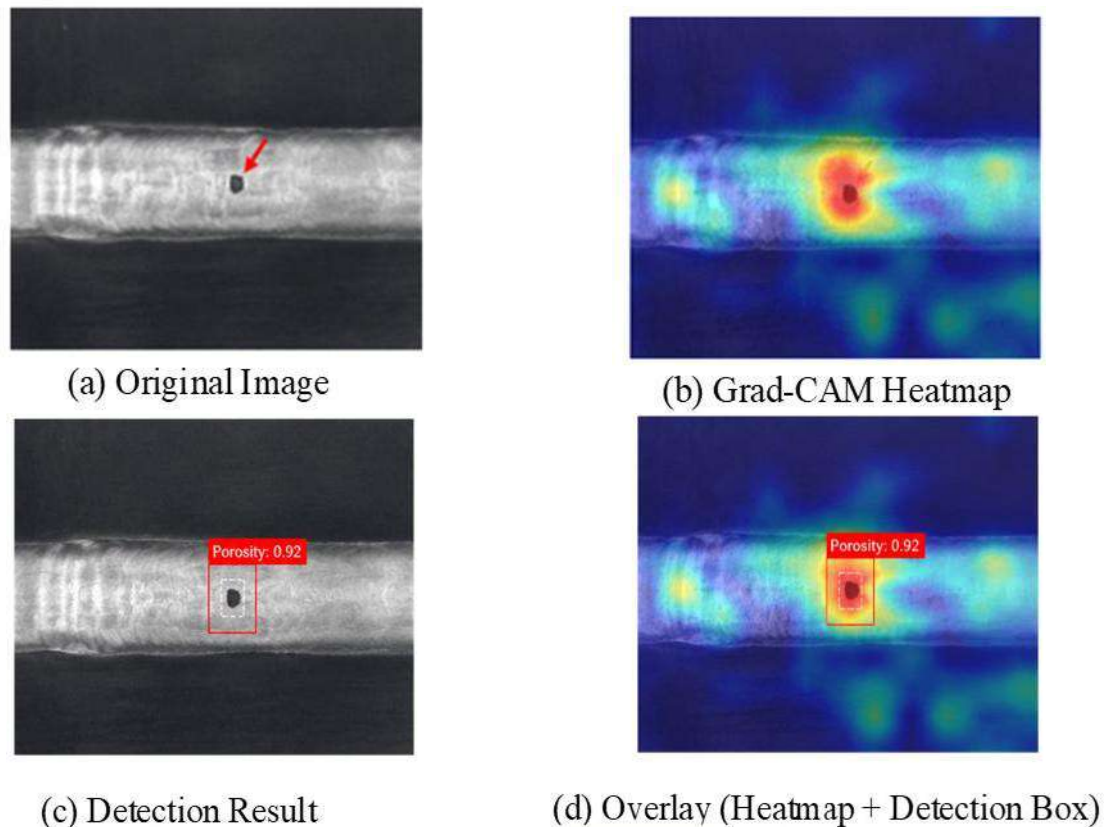


Fig. 11. Grad-CAM visualization of weld defect detection from independent test set

As shown in Table 2, YOLOv7-tiny-Weld achieves the highest mAP@0.5 (85.66%) on the NEU-DET dataset among all competitors, while maintaining only 3.37M parameters and 6.75G FLOPs, demonstrating its superior balance between detection accuracy and computational efficiency, as well as strong generalization capability across different defect detection tasks.

3.2 Application Detection Results of Ship Laser Welding Defects

To verify the applicability of YOLOv7-tiny-Weld in actual industrial scenarios, a self-built laser welding defect dataset was used in the experiment. The images came from a laser-arc hybrid welding production line of AH36 marine steel plates in a shipbuilding enterprise. A total of 3,200 weld surface images were collected, covering five categories: porosity (680), undercut (540), lack of fusion (490), spatter (510), and normal welds (980). To prevent overfitting on this relatively limited dataset, Mosaic and MixUp augmentations were applied during training. To evaluate the detection reliability and localization precision of YOLOv7-tiny-Weld, the confidence scores of predicted bounding boxes and the corresponding IoU values with the ground truth were analyzed for each method, as illustrated in Figure 12.

Figure 12(a) shows that the median detection confidence of YOLOv7-tiny-Weld reached 89.75%, and the confidence distribution was concentrated in the high confidence interval of 80%–95%. Significant differences existed when compared with other models ($p < 0.05$). This indicated that YOLOv7-tiny-Weld had a higher prediction reliability in weld defect detection. Figure 12(b) shows that the median IoU of YOLOv7-tiny-Weld reached 82.16%, and the IoU distribution was concentrated in the high precision interval of 75%–87%, which was significantly better than other comparison models ($p < 0.05$). This demonstrated that the DDH decoupled detection head effectively improved the precise localization capability of the model for weld defect boundaries by independently optimizing the regression task, reducing the risk of localization offset. To verify the detection robustness of YOLOv7-tiny-Weld, this study counted its detection precision under two working conditions: illumination changes and shooting distance changes. Illumination conditions were categorized as normal (500–800 lux), low (50–100 lux), and high (1500–2000 lux),

representing typical lighting variations on shipbuilding shop floors. Shooting distances were categorized as short (0.5 m), medium (1.0 m), and long (2.0 m). The results are shown in Figure 13.

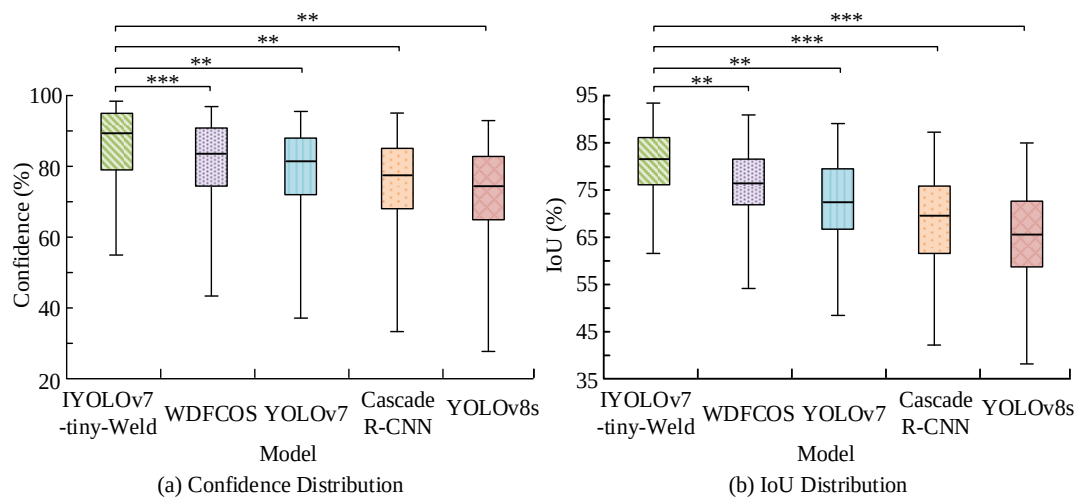


Fig. 12. Distribution of detection box confidence and IoU across different models

Note: * and ** indicate statistically significant differences at $p<0.05$ and $p<0.01$, respectively, based on independent samples t-test.

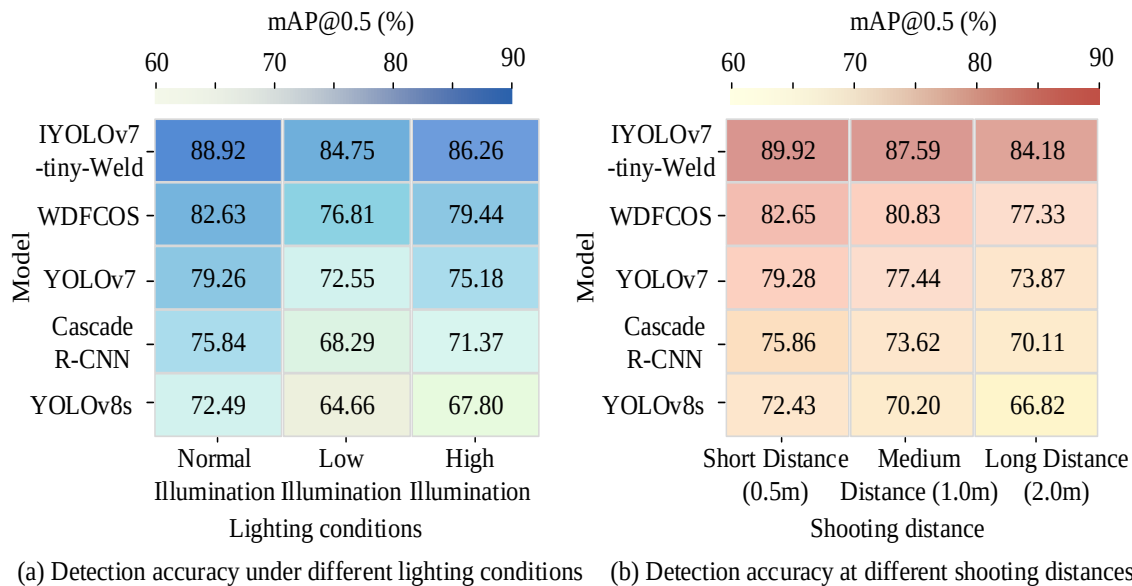


Fig. 13. Detection precision under different illumination intensities and shooting distances

Figure 13(a) shows that IYOLOv7-tiny-Weld achieved an mAP@0.5 of 88.92% under normal illumination. When the illumination was reduced, its mAP@0.5 decreased by 4.17%, representing a smaller decline than those of the other models. Under strong illumination conditions, the mAP of IYOLOv7-tiny-Weld was 86.26%, which was higher than other models. Figure 13(b) shows that the mAP of IYOLOv7-tiny-Weld at a long distance was 84.18%, which only dropped by 3.41% compared with the medium distance. This decrease was smaller than that of other comparison models. These results indicated that IYOLOv7-tiny-Weld possessed strong adaptability to actual working conditions, satisfying the weld defect detection needs in complex environments on shipbuilding sites. To compare the performance of different models across five weld defect categories, the AP and F1-score of each defect type, including porosity, cracks, lack of fusion, undercut, and pits, were analyzed. The results are shown in Figure 14.

Figure 14(a) shows that the AP of IYOLOv7-tiny-Weld on all five defect categories was higher than that of other comparison models. Among them, the AP of porosity and undercut reached 92.56% and 89.14%, respectively, and the AP of lack of fusion was 85.43%. This verifies the effectiveness of the multi-scale feature fusion module. The fine-grained features R_1 benefit small defects such as

porosity, while R_2 and R_3 provide contextual information for medium and large defects like cracks, undercut, and lack of fusion. Figure 14(b) shows that the F1-Scores of IYOLOv7-tiny-Weld on the five categories of defects were also superior to the comparison models. Its F1-Scores for porosity, cracks, lack of fusion, undercut, and pits were 90.28%, 87.55%, 84.83%, 89.51%, and 86.16% respectively. This demonstrated that the model possessed stable and balanced recognition performance for all types of weld defects. Especially, it could maintain high detection precision for easily confused defects like lack of fusion and pits, providing reliable technical support for ship laser welding quality detection.

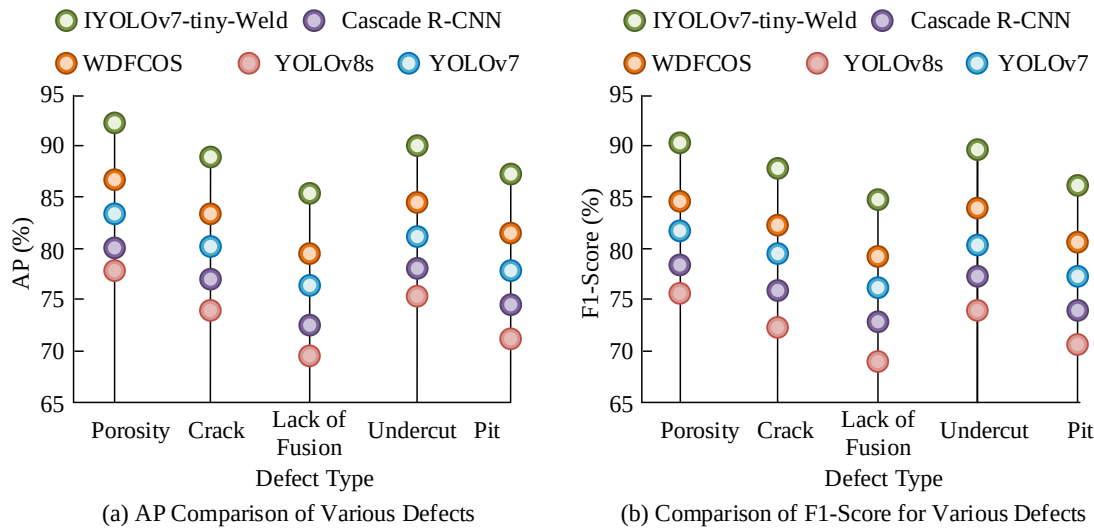


Fig. 14. Model performance comparison regarding AP values and F1-Scores on self-built ship laser welding dataset

4. Conclusions and Perspectives

Aiming at the laser welding defect detection needs for large components with complex curved surfaces, this study proposed IYOLOv7-tiny-Weld. The proposed model optimized the backbone network through DSA and GSConv, and accomplished multi-scale feature enhancement relying on BiFPN, CARAFE, and SPPFCSPC. By incorporating DSA to capture long-range dependencies and BiFPN with CARAFE to normalize multi-scale features, the model effectively handles geometric distortion and varying surface reflections caused by curved geometries. Meanwhile, it separated detection tasks through DDH, which effectively improved defect recognition and localization precision. Ablation experiments on the public WELD dataset reveal that the full proposed model achieves a 12.62% increase in mAP@0.5 compared with vanilla YOLOv7-tiny, while increasing the parameter count by only 0.11 M and maintaining an FPS of 83.31. Compared with mainstream detection models, its AP curve on the public WELD dataset was the best throughout the entire process, demonstrating a significant comprehensive advantage in inference speed and detection precision. In actual ship laser welding defect detection, the median detection confidence of the model reached 89.75%, and the median IoU was 82.16%. Its AP values and F1-Scores for multiple categories of defects, such as porosity, undercut, and lack of fusion, were all superior to the comparison models. These results demonstrate that IYOLOv7-tiny-Weld has a good adaptability to different types of defects and complex working conditions. It provides an effective technical solution for the online inspection of laser welding quality in industrial scenarios like shipbuilding. However, its detection precision under low illumination (50–100 lux) and long-distance (2.0 m) conditions remains a key challenge that will be addressed in future work, where the performance drops by 4.17% and 3.41%, respectively. Future work will enhance robustness under extreme conditions via data augmentation and domain adaptation. Additionally, edge deployment will be optimized through pruning and quantization, with evaluation on devices such as NVIDIA Jetson Nano to validate its practicality on real-world shipbuilding production lines.

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Coupling effects of laser cladding parameters on the morphology and phase evolution of FeCoNiCrCu high-entropy alloy coatings

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Abstract

Laser cladding of high-entropy alloys is an effective surface-strengthening technology, yet the coating formation mechanism still requires further optimization. In this work, FeCoNiCrCu high-entropy alloy powder and a 45-steel substrate were adopted to investigate laser-cladding behavior under four representative process states: under melting, baseline, high-energy over melting, and high powder-feeding rate. The powder was designed for equiatomic fraction; the nominal mass fraction of each element was 20 wt%, which deviates from strict equiatomic ratio. The results demonstrate that the under melting condition triggers incomplete powder melting and coating defects with a porosity of 4.23 %. Excessive heat input under the high-energy condition yields a high substrate dilution rate of 37.26 % and enlarges the heat-affected zone. A high powder-feeding rate induces elemental segregation. Under the baseline condition, the molten pool spreads uniformly, defects are suppressed, and elemental diffusion is sufficient, achieving the minimum porosity of 0.32 % and the optimal interfacial shear strength of 426.24 MPa. The high-energy over melting specimen (Condition 3) reaches an average hardness of 339.62 HV, though this process brings obvious substrate over-melting risk. The as-deposited coating consists of FCC and BCC solid-solution phases, whereas laves and σ phases precipitate after annealing. This study reveals the coupling effects of process parameters on coating morphology and phase evolution and provides guidance for process optimization of such quinary high-entropy-alloy laser-cladding systems.

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1. Introduction

Mechanical components frequently undergo prolonged exposure to severe friction and cyclic alternating loads, making them highly susceptible to surface degradation, wear, and subsequent delamination (peeling) failures. Laser cladding High-Entropy Alloy (HEA) coatings become a mainstream technology to improve the comprehensive performance of substrate surfaces at a low cost due to multi-principal element solid solution strengthening characteristics [1-2]. The quinary Iron-cobalt-nickel-chromium-copper (FeCoNiCrCu) HEA has a uniform and adjustable composition. It balances hardness and plasticity, showing outstanding application potential in the field of wear-resistant protection. However, the internal relationship among molten pool evolution, element diffusion, and phase precipitation under the coupling of multiple process parameters still lacks complete quantitative analysis. This knowledge gap restricts the standardized industrial adoption of the process [3-4]. To address these challenges, several research groups have investigated the cladding performance of various high-entropy alloy systems. Wen X et al. fabricated FeCoNiCrAl HEA composite coatings reinforced with varying contents of SiC ceramic particles via laser cladding and systematically analyzed their reinforcement mechanisms. They

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systematically analyzed the reinforcement mechanism of ceramic phases on coating microstructures. The hard phase can significantly improve the wear resistance of coatings. This work provides ideas for the design of high-entropy composite coatings [5]. Wu B et al. explored the interdiffusion behavior between multi-layer films of multi-element HEAs and aluminum substrates by means of atomistic simulation. They explained the causes of interfacial bonding quality from the perspective of microscopic atomic migration [6]. Li S et al. reviewed a large amount of existing research. They comprehensively summarized the corrosion evolution laws of HEA coatings prepared by laser cladding. The regulation logic of element segregation and phase structure on corrosion resistance was summarized [7]. Xiao D et al. adjusted the doping ratio of Niobium (Nb) to prepare Chromium-Iron-Cobalt-Nickel (CrFeCoNi) cladding coatings. They clarified that changes in alloy element content directly induce differential transitions in dendrite morphology and mechanical properties [8]. Kaviyarasan K et al. used gas tungsten arc welding to prepare HEA filled joints. They compared the effects of different entropy fillers on the joint microstructure and corrosion resistance [9]. Samoilova O V et al. explored the effects of silver doping on the micro-morphology, hardness, and surface oxidation behavior of AlCoCrCuFeNi HEA. They confirmed that trace alloy elements can change the phase precipitation tendency [10]. Xu K X et al. carried out electrochemical tests to compare the passivation and corrosion characteristics of CoCrFeNi HEA and 304 stainless steels in salt solution. The anticorrosion advantages of multi-element solid solution structures were quantified [11]. Fals H D C et al. improved cavitation and erosion resistance by adding Nb to Inconel coatings. They verified that alloying is an effective way to improve the service life of thermal spray coatings [12]. However, existing research mostly focuses on single element modification or single performance testing. Few studies simultaneously explore the comprehensive effects of multi-parameter coupling, such as laser power and powder feeding rate, on FeCoNiCrCu coating formation, thermophysical properties, mechanics, and phases. Therefore, this study sets four categories of gradient cladding conditions. Combined with characterization methods such as metallography, the changing rules of forming defects, element distribution, and mechanical indicators are quantified. The present work elucidates the synergistic regulation mechanisms the process coupling regulation mechanism to provide experimental support for process optimization of this type of quinary HEA laser cladding. The primary objective is to clarify the evolution of cladding cross-sectional morphology and molten pool spreading status under the synergistic action of laser power, powder feeding rate, and scanning speed. The differences in coating forming quality and mechanical properties under different process matchings are quantified. The innovation lies in coupling multi-indicator analysis. The regulation mechanism of multi-process synergy on the microstructure and defects of HEA cladding layers is fully revealed. This work provides systematic experimental support for the optimization of the laser cladding process of this type of quinary HEA.

2. Materials and Methods

2.1 Experimental Materials and Instruments

This study selects a common structural steel as the substrate and employs the equiatomic FeCoNiCrCu quinary HEA powder to perform laser cladding process coupling experiments. The detailed parameters of the raw materials, reagents, and shielding gas utilized in the experiment are listed in Table 1. The models, manufacturers, and specific applications of the primary laboratory instruments used in this work are listed in Table 2.

Table 1. Materials list

Material Name	Specification & Model	Manufacturer
45 steel (AISI 1045, Chinese GB standard)	100 mm×100 mm×10 mm	Ansteel Co., Ltd.
FeCoNiCrCu quinary high-entropy-alloy (HEA) powder	Particle size 150-300 mesh, sphericity ≥90%, equal-weight mixing ratio, each metallic element accounts for 20 wt%	Beijing General Research Institute of Nonferrous Metals
High-Purity Argon	Purity ≥99.999%	Hangzhou New Century Gas Co., Ltd.

Note: The powder was initially designed for equiatomic composition; the as-supplied powder adopts 20 wt% for each constituent element. Owing to differences in elemental atomic mass, 20 wt% does not yield equal atomic fractions. Calculated atomic fractions: Cr 22.1 at.%, Fe 20.4 at.%, Co 20.2 at.%, Ni 19.2 at.%, Cu 18.1 at.%.

Table 2. Instrument list

Equipment	Model	Manufacturer	Country
3D Laser Profilometer	VR-6000	Keyence Corporation	Japan
Metallographic Specimen Preparation Equipment	Tegramin-20	Struers A/S	Denmark
X-ray diffractometer	D8 ADVANCE	Bruker Corporation	Germany
Vickers microhardness tester	HVS-1000Z	Laizhou Huayin Testing Instrument Co., Ltd.	China
Scanning Electron Microscope (SEM)	Sigma 300	Carl Zeiss AG	Germany
Optical Microscope	Axio Scope A1	Carl Zeiss AG	Germany
Vacuum heat-treatment furnace	SK-V-1200	Shanghai Jvjing Electric Furnace Co., Ltd.	China

2.2 Laser Cladding Experimental Equipment and Process Plan

Surface pretreatments are required for both the substrate and the alloy powder before the cladding experiments. The substrate steel plate is progressively ground with various grits of abrasive papers to eliminate surface oxide scales, rust, and oil contaminants, followed by ultrasonic cleaning in anhydrous ethanol and drying [13-14]. The FeCoNiCrCu HEA powder is dried in a vacuum drying oven at a constant temperature to thoroughly eliminate internal adsorbed moisture. After the pretreatment steps, the FeCoNiCrCu HEA coupling process experiments are conducted on a coaxial powder-feeding fiber laser cladding experimental platform.

The system configuration and working principle are shown in Figure 1.

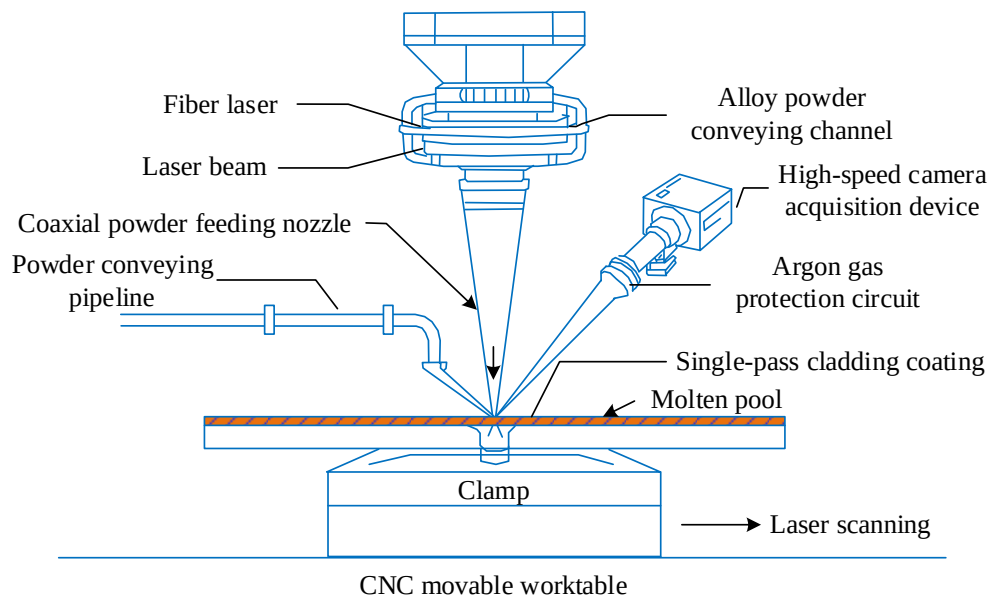


Fig. 1. Laser cladding experimental platform with coaxial powder feeding

In Figure 1, the laser beam emitted from the laser generator converges downward through a focusing nozzle, merging with the FeCoNiCrCu HEA powder delivered via the pipeline to create a molten pool on the substrate surface under continuous Argon shielding. The laser beam adopted a

Gaussian energy distribution with a fixed focal spot diameter of 2.0 mm on the substrate surface. During the process, the substrate workpiece is clamped onto a CNC moving stage. The laser scanning speed is regulated by adjusting the feed speed of the stage, completing the fabrication of cladding samples under diverse process parameters. All the steps in experiments are shown in Figure 2.

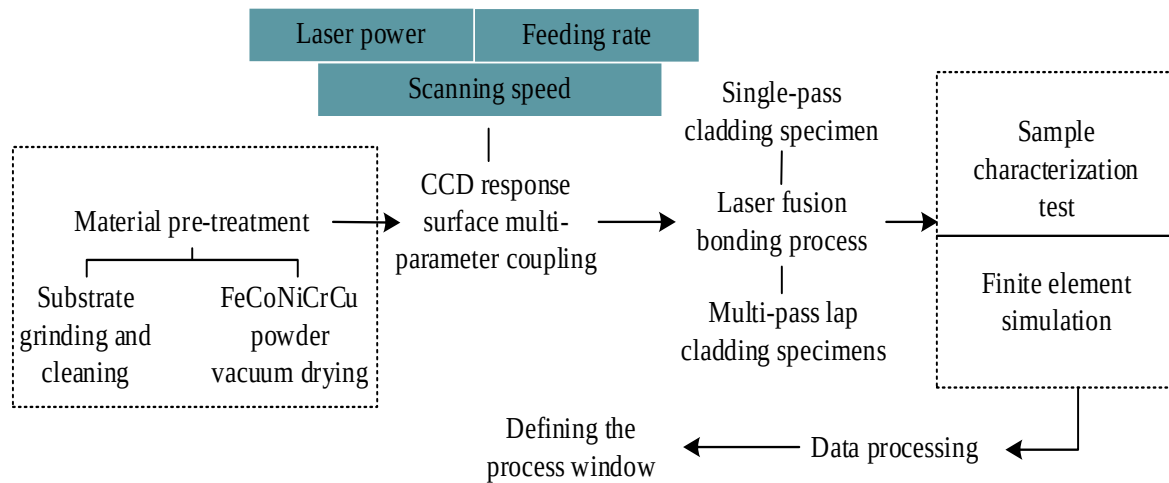


Fig. 2. Experimental technical route

As shown in Figure 2, the substrate undergoes grinding and decontamination in the early stage, while the FeCoNiCrCu HEA powder is dried in a vacuum environment to remove adsorbed moisture. The defocus distance was maintained as a constant value of 0 mm throughout all laser cladding tests. Single-track and multi-track overlapping specimens are fabricated on the laser cladding platform. In the experimental stage, the surface morphology, internal microstructure, and element distribution of the cladding layer are checked and analyzed via a three-dimensional profilometer, an optical microscope, and a scanning electron microscope equipped with energy dispersive spectroscopy. Three key parameters including laser power, scanning speed and powder feeding rate were selected. Four representative coupled process conditions were designed for comparative investigation. The detailed process parameters for the four representative conditions are presented below:

- Condition 1 (under melting): laser power = 1600 W, powder-feeding rate = 8 g/min, scanning speed = 8 mm/s
- Condition 2 (baseline): laser power = 2400 W, powder-feeding rate = 12 g/min, scanning speed = 6 mm/s
- Condition 3 (high-energy over melting): laser power = 3200 W, powder-feeding rate = 12 g/min, scanning speed = 6 mm/s
- Condition 4 (high powder-feeding rate): laser power = 2400 W, powder-feeding rate = 18 g/min, scanning speed = 6 mm/s

The defocus distance was kept constant at 0 mm throughout all laser cladding experiments.

2.3 Sample Preparation Process

After the laser cladding experiments, the fabricated specimens undergo a series of preprocessing steps to facilitate subsequent macro-morphology examination and microstructure characterization. The complete process flow for metallographic sample preparation is shown in Figure 3. In Figure 3, standard-sized specimens are first sectioned from the formed workpieces using wire electrical discharge machining equipment. After hot mounting, the specimens are subjected to coarse and fine grinding using different grits of abrasive papers, followed by mechanical polishing to achieve a mirror-like surface. After grinding and polishing, the sample surface was cleaned with anhydrous ethanol. The metallographic surface was etched by nitric-acid-based corrosive agent for microstructure observation.

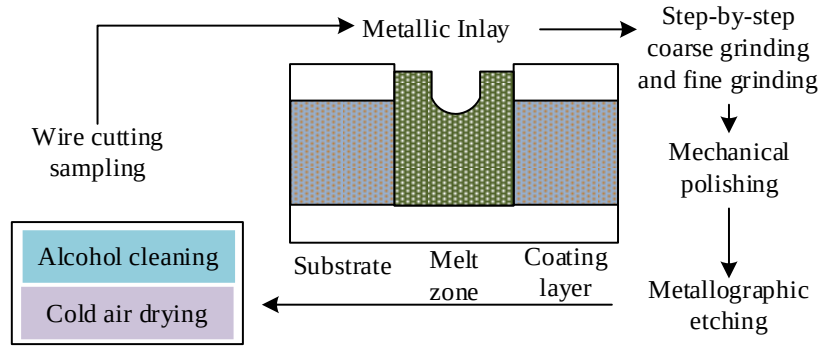


Fig. 3. Manufacturing process flow of metallographic specimens

Diamond lapping paste was adopted for fine polishing during specimen preparation. An appropriate etchant is selected to corrode the cross-section of the specimen to reveal the microstructure of the cladding layer. Finally, the specimens are rinsed with anhydrous ethanol and dried with cold air to complete the preparation. The calculation equation for the cladding Energy Utilization Rate (EUR) is shown in Equation (1) [15].

$$EUR = \frac{m_{\text{powder}} \cdot \Delta H_{\text{alloy}}}{P \cdot t} \times 100\% \quad (1)$$

In Equation (1), m_{powder} denotes the mass of the alloy powder consumed in a single track, m_{powder} is calculated as the product of powder feeding mass rate and single-track scanning duration. ΔH_{alloy} indicates the latent heat of fusion of the FeCoNiCrCu HEA, while P and t represent the actual output power of the laser and the total scanning time of a single track, respectively. The expression for the Specific Energy Consumption (SEC) is shown in Equation (2) [16].

$$SEC = \frac{P \cdot t_s}{m_{\text{powder}}} \quad (2)$$

In Equation (2), t_s is the total scanning time of single-track cladding (s). The calculation for the dilution rate of the cladding layer is shown in Equation (3) [17].

$$\eta = \frac{S_{\text{sub}}}{S_{\text{sub}} + S_{\text{coat}}} \times 100\% \quad (3)$$

In Equation (3), η is the dilution rate, while S_{sub} and S_{coat} represent the cross-sectional area of the melted substrate and the cross-sectional area of the cladded alloy layer within the molten pool, respectively. Part of the single-track laser-cladded specimens were subjected to vacuum annealing treatment. The annealing temperature was set to 800 °C with a holding time of 2 h. The heating rate was controlled at 5 °C/min. After heat preservation, the samples were cooled down to room temperature inside the furnace at a cooling rate of 3 °C/min. The whole heat-treatment process was carried out under a vacuum atmosphere of 5×10^{-3} Pa to prevent oxidation of the coating surface. Two-step chemical etching was adopted to reveal the microstructure of the substrate and the high-entropy alloy coating separately. The 45-steel substrate was etched with 4% nital solution (4% HNO_3 + 96% ethanol). The FeCoNiCrCu high-entropy alloy coating was etched with modified aqua regia solution ($\text{HCl}:\text{HNO}_3:\text{H}_2\text{O} = 3:1:2$) to clearly display dendritic boundaries and phase precipitation structures.

2.4 Specimen Testing and Characterization Methods

Four typical coupled process conditions were selected for comparison. Condition 1 is an under melting condition characterized by low laser energy and the minimum powder feeding rate. Condition 2 is a baseline matching condition featuring a balanced ratio of laser power to powder feeding rate. Condition 3 represents a high-energy over melting condition where the laser output power is increased while maintaining a moderate powder feeding rate. Condition 4 is a high

powder feeding condition where the powder feeding rate reaches the upper experimental limit and the laser power remains at a medium level. The comparative analysis is carried out on all these cladding specimens in a unified manner. The separate test methodologies are listed below:

Macro-morphology observation of single-track cross-sections: Cross-sectional specimens of the coating are cut via CNC wire electrical discharge machining. After mounting, grinding, and polishing, the cross-sectional profiles are observed using an Olympus vertical optical microscope. The cladding width, height, and depth dimensions are measured via ImageJ software according to standard metallographic preparation procedures [18-19].

Dendritic microstructure, surface micro-morphology, and elemental mapping of the coating were observed after a two-step etching procedure. The 45-steel substrate was etched with 4% nital solution, while the FeCoNiCrCu high-entropy alloy coating was etched with modified aqua regia solution (HCl: HNO₃: H₂O = 3:1:2). A Zeiss field emission scanning electron microscope was utilized to capture secondary electron morphologies of the coating surface. The use of an Oxford energy dispersive spectrometer which is attached is done to map the elemental distribution of Cr, Fe, Ni, and Si. Cr, Fe, Ni, Si, Co and Cu were selected for elemental distribution mapping to evaluate the chemical homogeneity and diffusion behavior across the cladding zone. Elemental detection and microstructure observations comply with the operational standards of scanning electron microscopy [20].

Thermophysical property testing of the substrate and HEA coatings: The laser flash method is used to collect thermophysical data at various temperatures, with the heating interval covering room temperature to 2000 K. The testing of thermophysical parameters refers to relevant standards for determining high-temperature thermophysical properties of metallic materials. **Process parameter testing of EUR, SEC, and aspect ratio of the track:** The measurement of the dimensional characteristics of the single-track cladding is done using a laser profilometer of Keyence, which is a three-dimensional laser profiler. The laser power and the mass flow of the powder are measured at the same time. The profile dimension measurement accuracy follows the operational specifications of three-dimensional morphology inspection equipment [21].

Microhardness testing along the layer depth: A Vickers microhardness tester is utilized with a load of 0.1 kgf to perform step-by-step indentation testing from the coating surface toward the substrate. The hardness data collection interval is 0.05 mm, and the Vickers hardness testing of metallic materials strictly complies with the GB/T 4340.1-2018 standard. **Phase analysis of as-deposited and annealed coatings:** The coating surface is ground and polished before undergoing X-ray diffraction scanning, with a scanning angle range of 20° to 80°. The Face-Centered Cubic (FCC) and Body-Centered Cubic (BCC) solid solution phases, as well as intermetallic compounds, are identified according to general phase identification methods for metallic multi-component alloys.

Additional quantitative parameter test method to define quality: Calculation of wetting angle and porosity of the melt pool according to binary statistics of metallographic photographs. Surface roughness is measured using a white light interferometric three-dimensional profiler. The interfacial shear strength tensile test was performed with reference to GB/T 7124-2008. Adapted specimen geometry was adopted for metallurgically bonded coating-substrate samples rather than adhesive-bonded assemblies to measure coating-substrate interfacial shear performance. Thermal cracking susceptibility was evaluated by counting crack length and crack density based on metallographic cross-section micrographs [22].

3. Results and Discussion

3.1 Macro-Morphological Characteristics of Cladding Layers

Figure 4 presents the cross-sectional morphology of the single-track laser cladding coatings under four sets of process parameters. The regulatory effects of process parameters on the molten pool morphology are visual through three dimensional parameters: cladding width *W*, cladding height *H*, and cladding depth *D*.

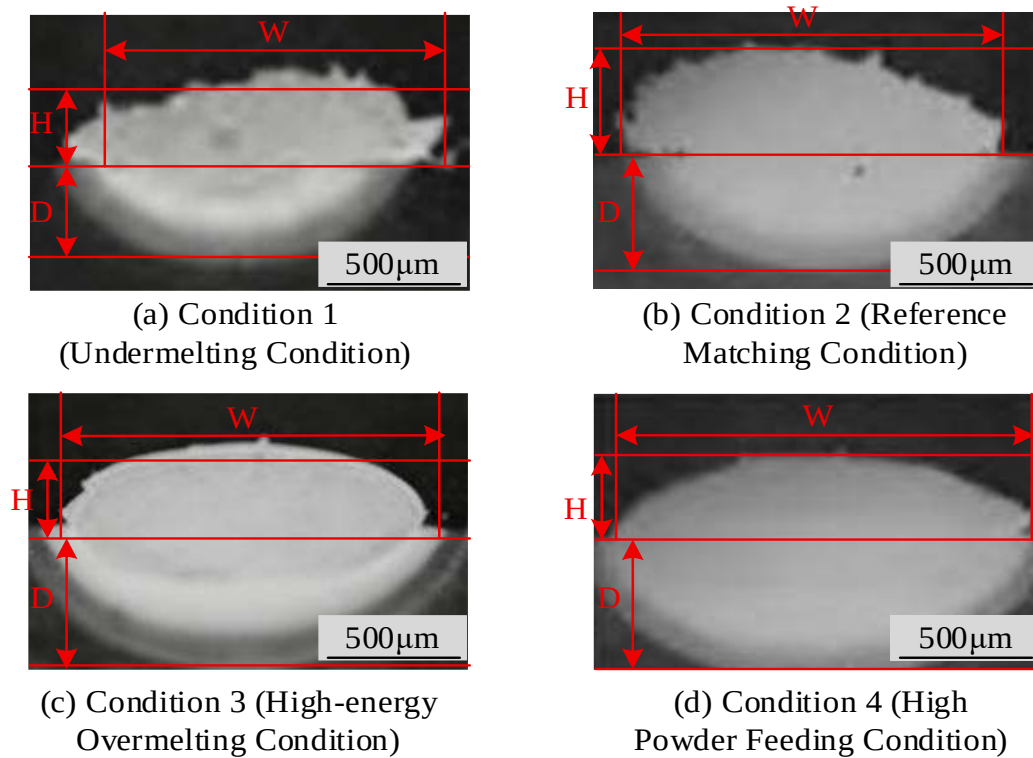


Fig. 4. Cross-sectional morphology of single-track laser cladding coatings

(Condition 1: 1600 W, 8 g/min, 8 mm/s; Condition 2: 2400 W, 12 g/min, 6 mm/s; Condition 3: 3200 W, 12 g/min, 6 mm/s; Condition 4: 2400 W, 18 g/min, 6 mm/s)

As shown in Figure 4(a), under the under melting condition, low laser heat input caused insufficient powder melting. The overall width and height of the cladding layer stayed relatively small, and the molten pool profile remained shallow. Slight irregular fluctuations occurred at the coating edges, which easily induced lack-of-fusion defects and led to poor metallurgical bonding quality. Figure 4(b) illustrates that the baseline condition offered a well-matched energy level. The molten pool exhibited a regular and symmetrical morphology with balanced width, height, and depth dimensions. The coating edges appeared flat and smooth, without obvious defects like pores or inclusions. The continuous and stable bonding interface between the substrate and the coating indicated the best forming quality. In Figure 4(c), excessive laser energy under the high-energy over melting condition significantly increased substrate melting, causing a pronounced enhancement in cladding depth. Although the powder melted fully and the coating formed uniformly, such excessive heat input under high-energy conditions is likely to induce substrate grain coarsening and deterioration of base-metal mechanical properties according to laser thermal-cycling characteristics. Direct microstructural verification of this phenomenon will be conducted in future work. In Figure 4(d), the high powder feeding rate required more laser energy per unit area to melt the powder. The cladding layer height and width increased further, whereas the cladding depth decreased slightly, yet the molten pool spread well. However, excessive powder feeding easily induced uneven powder deposition, leading to forming defects like surface agglomeration. Overall, the cladding layer achieved the highest forming integrity under the baseline condition, representing a relatively reasonable process parameter range.

3.2 Microstructure and Elemental Distribution Patterns of The Coating

Figure 5 displays the surface SEM morphology of the coatings and the EDS elemental mapping results of four major elements (Cr, Si, Fe, and Ni) under the four process conditions. Combining the observations from Figure 5, the coating surface under Condition 1 appeared relatively flat, and the overall element distribution was quite uniform. Si exhibited only scattered dot-like distribution without obvious element-rich zones. Low heat input restricted the degree of elemental diffusion, creating slight local composition segregation. In Condition 2, as the laser energy increased, the coating microstructure became finer and denser. The distribution uniformity of Cr, Fe, and Ni

improved, and Si began to aggregate in small areas, which indicated that elemental diffusion was enhanced. Condition 3 exhibited the most extensive elemental diffusion within the molten pool. The coating surface structure was dense and uniform, and the four elements dispersed globally. Although small Si enrichment zones existed, severe segregation did not occur. Sufficient element mixing was achieved in the molten pool. However, the excessive heat input of Condition 3 also induced undesirable substrate dilution and high cracking susceptibility.

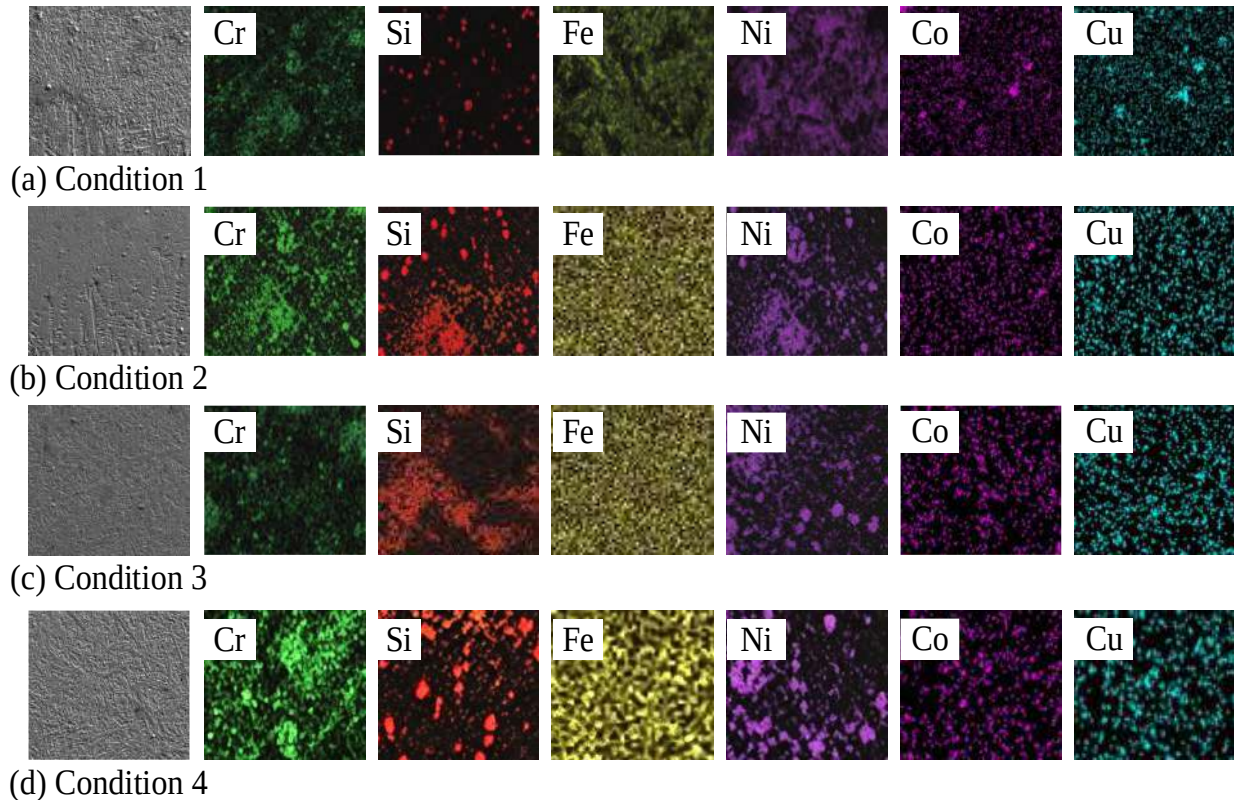


Fig. 5. SEM morphology and representative elemental EDS-mapping results of coatings under four different process conditions

Under Condition 4, the high powder feeding rate caused a notable increase in Cr and Ni enrichment regions and aggravated Si agglomeration, which raised local composition segregation. There was a considerable quantity of powder that melted within a relatively short period of time, but some materials had quickly solidified before there was adequate diffusion. In conclusion, Condition 3 gave the most sufficient diffusion of elements, while Condition 2 delivered balanced comprehensive coating performance with low dilution and cracking risk. Silicon is not an initial constituent element of FeCoNiCrCu powder. The observed Si enrichment originates from trace-element diffusion from the 45-steel substrate into the molten pool during laser melting. Cu element presents scattered local enrichment regions across the coating. This segregation behavior originates from the positive mixing enthalpy between Cu and other constituent metallic elements, which tends to drive Cu atoms to accumulate within interdendritic areas during rapid laser-cladding solidification. No severe segregation was observed for Co element within the detection range of this test.

3.3 Thermophysical Property Differences

Figure 6 demonstrates reference temperature-dependent variations in specific heat capacity and thermal conductivity for both the FeCoNiCrCu HEA coating and 45-steel substrate. Within this temperature range, the variations in specific heat capacity and thermal conductivity between the two materials directly determined the cooling efficiency and high-temperature duration of the molten pool, serving as the core physical foundation for the differential evolution of cladding morphology under diverse processes.

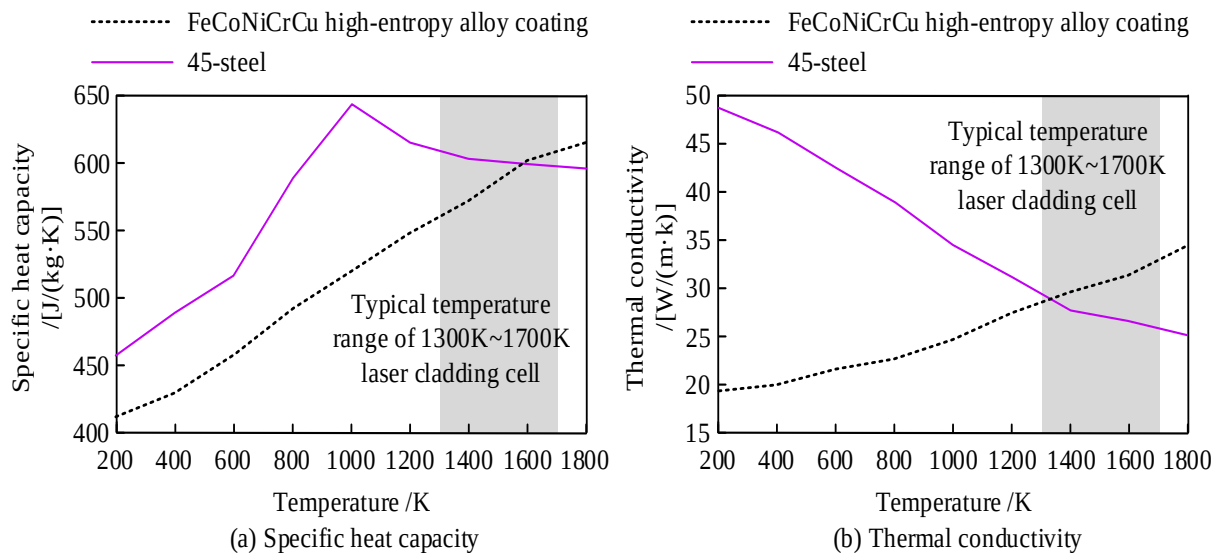


Fig. 6. Reference thermophysical property-temperature curves of FeCoNiCrCu HEA coating and 45-steel substrate

Figure 6(a) depicts the specific heat capacity curves of the two materials. In the low-temperature stage, the 45-steel substrate possessed a significantly higher specific heat capacity than the HEA. As temperature rose, the substrate specific heat capacity increased rapidly first, reaching a peak around 1100 K before dropping slowly. The specific heat capacity of the HEA steadily enlarged with increasing temperature. Within the working temperature range of the molten pool, its specific heat capacity gradually surpassed that of the substrate material. This phenomenon meant that the alloy coating could store more heat during heating, which helped stabilize the temperature field of the molten pool. Figure 6(b) shows the trend of thermal conductivity variation. While the thermal conductivity of the 45-steel substrate kept decreasing continuously with the increase in temperature, that of the HEA kept increasing with the increase in temperature. The point of intersection of the thermal conductivities was about 1300 K. In the temperature range of the cladding, the thermal conductivity of the HEA was higher than that of the substrate. This feature accelerated the outward transmission of excess heat from the molten pool, reduced the internal temperature gradient of the coating, relieved thermal stress concentration, and suppressed defects like cracks and deformation, thereby enhancing coating forming stability.

3.4 Influence of Process Parameters on Cladding Energy Consumption and Track Geometry

Figure 7 reveals the influence of three process parameters (laser power, powder feeding speed, and scanning speed) across four levels on three evaluation indicators: EUR, SEC, and track aspect ratio. The factor levels correspond to actual experimental values as follows: Laser power: Level 1 = 1600 W, Level 2 = 2400 W, Level 3 = 2800 W, Level 4 = 3200 W. Powder-feeding rate: Level 1 = 8 g/min, Level 2 = 12 g/min, Level 3 = 15 g/min, Level 4 = 18 g/min. Scanning speed: Level 1 = 4 mm/s, Level 2 = 6 mm/s, Level 3 = 7 mm/s, Level 4 = 8 mm/s. The symbol α in the figure represents the orbital cross-sectional ratio, which is defined as the ratio of the cladding width to the cladding height ($\alpha = W/H$).

In Figure 7(a), the EUR increased continuously with the increase of laser power and powder feeding speed. When the laser power rose from Level 1 to Level 4, the EUR increased from 10.65% to 13.28%. The powder feeding speed brought an increase in the utilization rate from 10.23% to 14.17%. Increasing the scanning speed only changed the utilization rate slightly from 12.13% to 12.32%. Higher power and powder volume allowed the laser energy to melt the powder more efficiently, whereas excessive scanning speed shortened the thermal interaction time and caused energy loss. As shown in Figure 7(b), the SEC decreased significantly with the increase of powder feeding speed, dropping from 31200 J/g at Level 1 to 15300 J/g at Level 4.

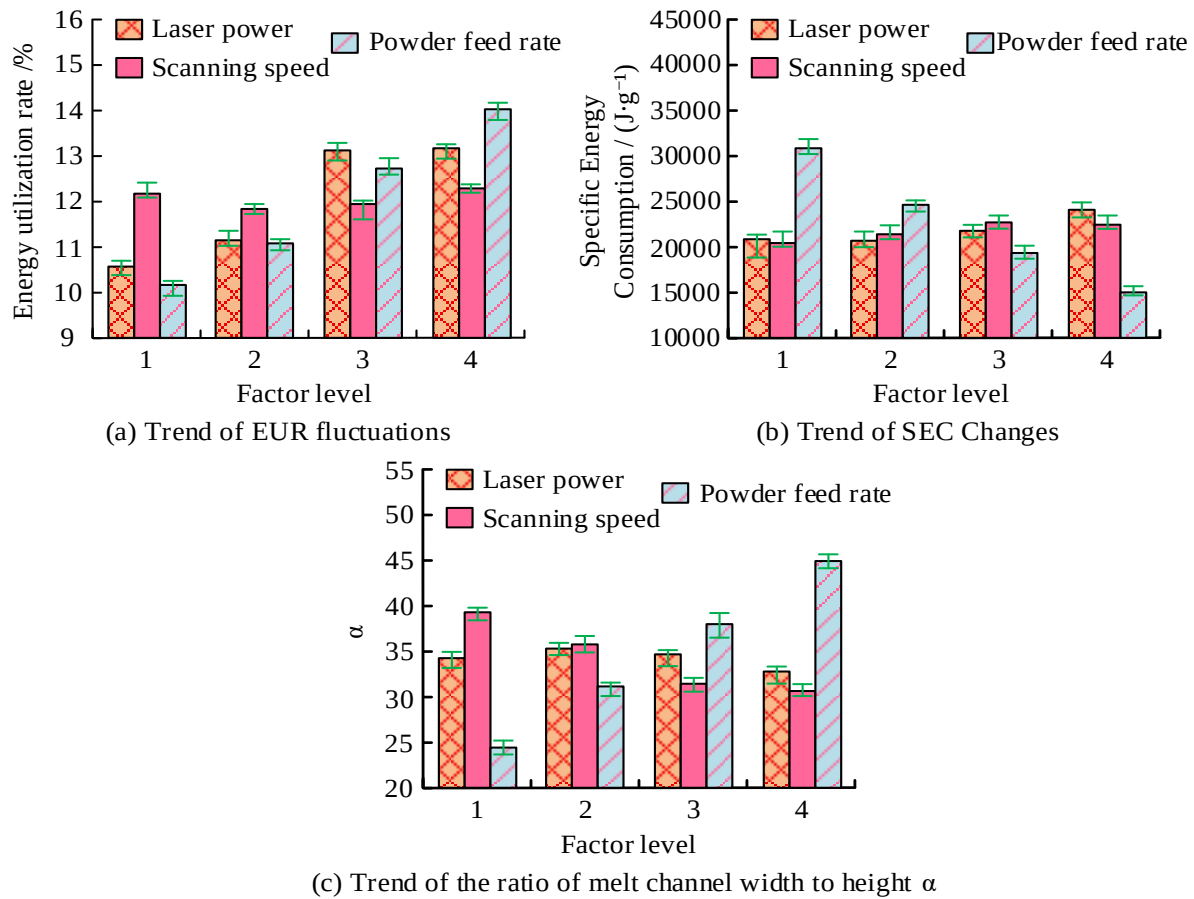


Fig. 7. Influence trends of process factors on cladding evaluation indicators

Increasing the laser power slightly raised the SEC, while the scanning speed caused minor fluctuations in energy consumption. The increase in powder delivery enhanced material utilization efficiency, which fully utilized the laser energy consumed per unit of powder. The results in Figure 7(c) demonstrate that the powder feeding rate had the greatest influence on the track aspect ratio. When the powder feeding rate was increased from Level 1 to Level 4, the aspect ratio increased from 24.83 to 44.98. The laser power increased the aspect ratio, whereas the scanning speed decreased the aspect ratio. Higher powder feeding increased the cladding accumulation height, whereas accelerating the scan promoted lateral spreading of the molten pool, which ultimately reduced the track aspect ratio.

3.5 Microhardness Distribution of Coatings Along the Layer Depth Under Different Conditions

At each depth interval, five independent Vickers indentations were performed, and the average hardness value was calculated ($n=5$). Small error bars in the bar charts indicate good repeatability of hardness tests and high process stability. Figure 8 presents the microhardness test results of the HEA cladding coatings under four sets of processes. The influence of different processes on the mechanical properties of the coatings was analyzed through the hardness distribution curves along the depth and the bar charts of average hardness.

As shown in Figure 8(a), within the coating region of 0 to 0.6 mm, the microhardness of the four specimens remained stable overall. The values followed the order of Condition 3 > Condition 2 > Condition 4 > Condition 1. The small hardness fluctuation inside the coatings indicated good microstructure uniformity across all groups. Upon entering the heat-affected zone of 0.6 to 1.0 mm, the hardness of all four specimens dropped noticeably, which was mainly because heating induced tempering softening of the substrate. When the depth exceeded 1.0 mm and entered the substrate region, the four curves converged gradually, and the hardness stabilized near 150 HV0.1, matching the hardness of the base metal.

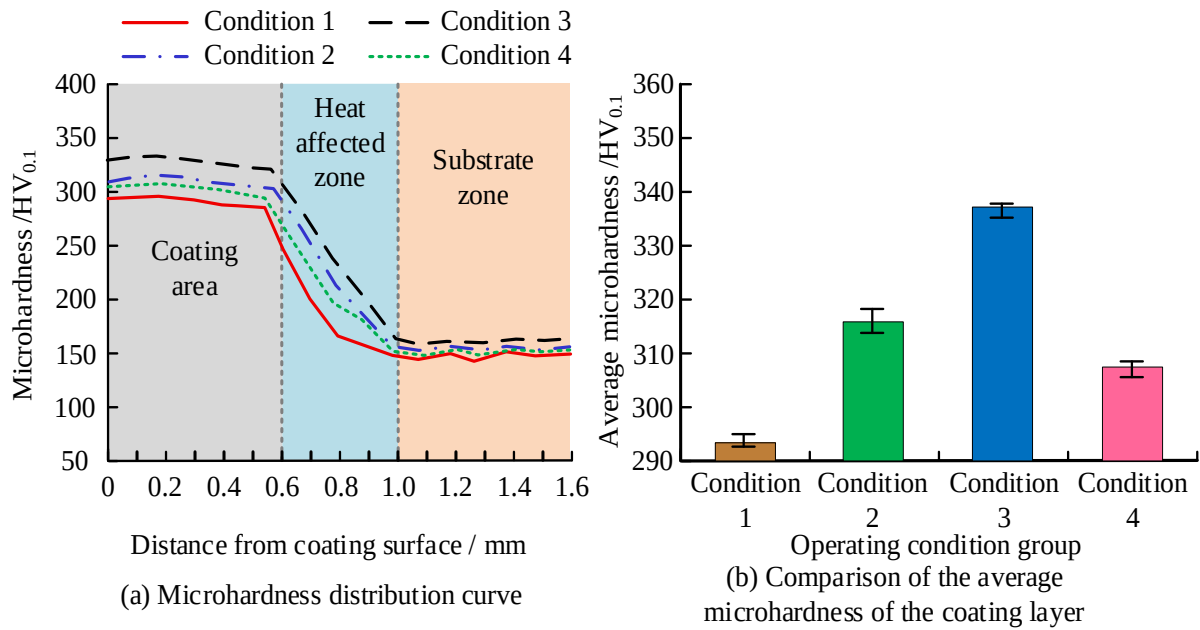


Fig. 8. Microhardness test results

Considering the average hardness statistics in Figure 8(b), Condition 3 yielded the highest average coating hardness of 339.62 HV_{0.1}. Here, fine-grain strengthening and solid-solution strengthening showed the most prominent effects. Condition 1 lacked sufficient heat input, creating unmelted defects inside the coating, so the hardness was the lowest. The small error bars in the bar charts indicated good repeatability of the hardness tests and high process stability. Comparative analysis demonstrated that the parameter combination of Condition 3 fabricated cladding coatings with optimal hardness performance.

3.6 Phase Evolution Analysis of As-Deposited and Annealed Coatings

Figure 9 shows the X-Ray Diffraction (XRD) phase patterns of the FeCoNiCrCu HEA coating in both the as-deposited and annealed heat-treated states. This analysis was utilized to compare the evolution of crystal structures and phase constitutions before and after heat treatment.

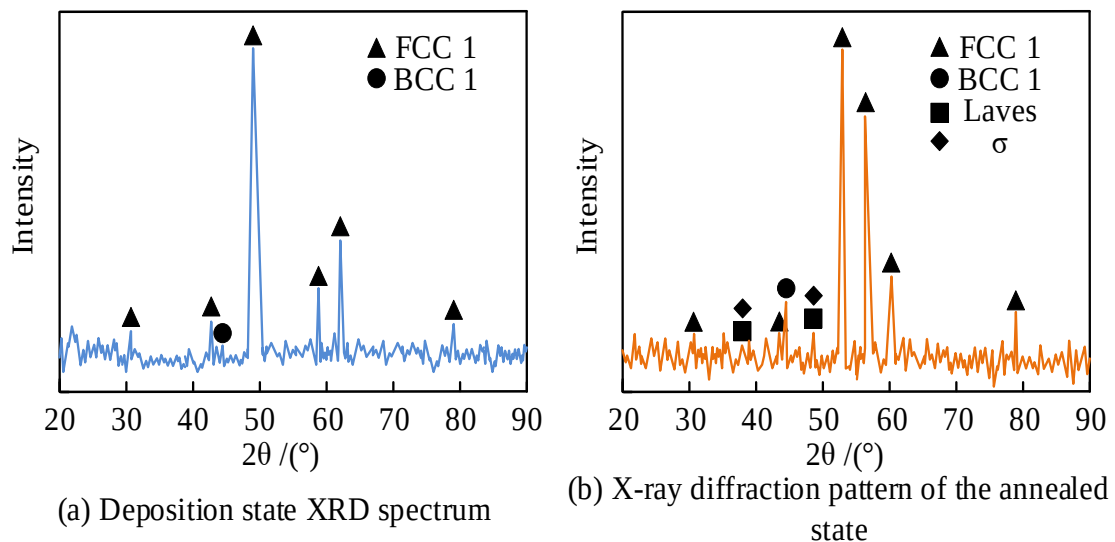


Fig. 9. XRD patterns of laser cladded FeCoNiCrCu HEA coating

The diffraction pattern of the as-coated layer in Figure 9(a) showed the presence of just two solid solutions, that is, FCC 1 and BCC 1. Diffraction peaks corresponding to intermetallic were absent. This was attributed to the very fast cooling process in the molten pool during laser cladding. Consequently, different alloying elements did not get sufficient time for long-range diffusion and segregation. Meanwhile, the broad diffraction peaks confirmed the characteristics of grain

refinement under rapid cooling conditions. At this stage, solid solution strengthening served as the primary mechanism to enhance the mechanical properties of the coating. Figure 9(b) indicates that after annealing heat treatment, the coating still retained the two matrix solid solution phases, FCC 1 and BCC 1. The main peak positions did not shift obviously, but the diffraction peaks became sharper, which suggested that the grains grew to some extent during high-temperature holding. Furthermore, a large number of new diffraction peaks appeared in the range of 35° to 48° , which were indexed as two types of intermetallic compounds: Laves phase and σ phase. The FCC solid-solution phase was identified according to ICDD-PDF #04-0850, the BCC solid-solution phase was matched with ICDD-PDF #06-0696. The Laves phase and σ phase were indexed with ICDD-PDF #15-0687 and ICDD-PDF #47-1423, respectively. Annealing provided sufficient energy for atomic diffusion. Elements segregated locally to precipitate secondary phases, which can further improve coating performance such as hardness and wear resistance through secondary phase strengthening. The Laves phase and the σ phase both belong to typical brittle intermetallic compounds. Although both can enhance the microhardness, their precipitation may increase the brittleness of the coating and the risk of crack initiation. This study only conducted a crack sensitivity assessment on the deposited-state coating and did not conduct crack behavior characterization on the annealed samples containing brittle precipitated phases. Therefore, before the practical application of this annealing process, further experimental evaluations are needed to assess the embrittlement risk brought by these intermetallic phases.

3.7 Quantitative Comparison of Cladding Forming Quality Under Multiple Conditions

To further quantify the comprehensive influences of different process parameters on cladding layer forming quality, interfacial bonding, and defect susceptibility, supplementary tests including molten pool wetting state, surface roughness, and dilution rate were performed. These tests quantitatively characterized the forming morphology variations and internal defect levels under the four conditions, and the results are listed in Table 3.

Table 3 Supplementary testing results of forming morphology and quality

Test indicators	Condition 1 (Under melting Condition)	Condition 2 (Baseline Condition)	Condition 3 (High- energy Overmelting Condition)	Condition 4 (High Powder Feeding Rate Condition)
Molten pool wetting angle ($^{\circ}$)	128.67 $\pm$ 3.24	76.32 $\pm$ 1.47	58.75 $\pm$ 2.08	109.43 $\pm$ 2.69
Coating surface roughness (μm)	12.73 $\pm$ 1.78	3.21 $\pm$ 0.39	4.54 $\pm$ 0.57	8.62 $\pm$ 1.09
Dilution rate of the cladding layer (%)	8.34 $\pm$ 0.87	21.52 $\pm$ 1.18	37.26 $\pm$ 2.47	12.63 $\pm$ 0.98
Coating porosity (%)	4.23 $\pm$ 0.68	0.32 $\pm$ 0.09	0.84 $\pm$ 0.17	2.13 $\pm$ 0.47
Crack sensitivity coefficient	0.78 $\pm$ 0.06	0.12 $\pm$ 0.03	0.65 $\pm$ 0.05	0.36 $\pm$ 0.04
Interfacial shear strength (MPa)	218.37 $\pm$ 11.86	426.24 $\pm$ 17.93	352.18 $\pm$ 14.87	287.42 $\pm$ 13.79
Coefficient of variation of cladding track dimensions (%)	11.34 $\pm$ 1.17	2.62 $\pm$ 0.38	3.15 $\pm$ 0.47	6.83 $\pm$ 0.88
Width of the heat affected zone (μm)	126.35 $\pm$ 7.89	245.27 $\pm$ 11.92	487.46 $\pm$ 17.83	189.32 $\pm$ 9.78

Note: Condition 1: Laser power=1200 W, Scanning speed=8 mm/s, Powder-feeding rate=12 g/min; Condition 2: Laser power=1400 W, Scanning speed=10 mm/s, Powder-feeding rate=15 g/min; Condition 3: Laser power=1600 W, Scanning speed=10 mm/s, Powder-feeding rate=15 g/min; Condition 4: Laser power=1400 W, Scanning speed=10 mm/s, Powder-feeding rate=20 g/min.

The data in Table 3 indicate that Condition 1 lacked sufficient heat input, yielding a molten pool wetting angle of 128.67° . Incomplete powder melting led to a porosity of 4.23%, and the crack sensitivity coefficient was relatively high at 0.78, while the interfacial shear strength reached only 218.37 MPa. This result meant that forming defects were numerous and bonding strength was poor. Condition 2 provided the best matching baseline parameters, where the wetting angle decreased to 76.32° , the surface roughness was only $3.21\text{ }\mu\text{m}$, and the porosity dropped to 0.32%. The crack tendency was weak, and the interfacial shear strength reached 426.24 MPa. The variation coefficient of track size was only 2.62%, indicating uniform and stable forming. In condition 3, the amount of heat input was too high, resulting in a very high dilution rate of 37.26%, and the width of the heat affected zone increased to $487.46\text{ }\mu\text{m}$. This resulted in excessive thermal softening of the substrate, and the susceptibility to cracking increased again. Condition 4 involved an excessive powder feeding rate, which degraded molten pool spreading, causing the wetting angle to rebound to 109.43° . Its porosity and dimensional fluctuations both exceeded those of the baseline group, showing reduced forming stability. Considering all quantitative data, the baseline condition effectively balanced molten pool wetting, substrate dilution, and internal defects, making it the optimal process window that satisfied both forming appearance and interfacial mechanical properties.

4. Conclusions and Perspectives

Based on the experimental characterization results of laser-cladded FeCoNiCrCu high-entropy alloy coatings under four representative process conditions, the main findings can be summarized as follows. Under the under melting condition (Condition 1), insufficient laser heat input limited elemental diffusion. The cladding width and depth remained relatively small. Low energy input suppressed metallurgical bonding at the coating-substrate interface. Excessively high laser energy adopted in Condition 3 drove the 45-steel substrate dilution rate to 37.26% and substantially widened the heat-affected zone. According to laser thermal-cycling characteristics, such high heat input is likely to trigger undesirable substrate grain coarsening, which needs further microstructural verification in future studies. While the high-energy over melting condition (Condition 3) yielded the highest average microhardness of 339.62 HV0.1, the baseline process (Condition 2) represented the most balanced and optimal process window, effectively optimizing molten pool wetting, minimizing substrate dilution, and maximizing interfacial shear strength. Combined with the XRD results, FCC and BCC solid-solution phases formed in the as-deposited coating. Additional Laves phase and σ -phase precipitates were generated after annealing treatment.

Combining all quantitative indicators, the baseline process effectively balanced the melting state, 45-steel substrate dilution, and interfacial bonding. Notably, obvious Cu-rich segregation can be observed within the cladding coatings from EDS elemental mapping results, which is consistent with the metallurgical characteristics of FeCoNiCrCu high-entropy alloy systems. Although post-annealing treatment promotes the precipitation of hard Laves and σ -phases to improve coating hardness, such brittle intermetallic precipitates may also bring potential embrittlement risks to the coating. However, the present study is limited to a narrow window of laser processing parameters. Furthermore, in-service performance characteristics—such as wear and corrosion resistance—and multi-physics coupled numerical simulations were not evaluated. Further research will focus on expanding the experimental design matrix to investigate multi-track overlapping effects, conducting high-temperature wear and electrochemical corrosion testing to assess environmental durability, and refining the finite element models to accurately predict residual stresses and phase precipitation during post-cladding annealing.

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Design, steady-state optimization, and cascade dynamic control of a continuous four-columns vacuum distillation process for coal tar-derived mixed phenols

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Total annual cost (TAC)

Abstract

Mixed phenolic homologues derived from coal tar, characterized by close boiling points and high thermal sensitivity, exhibit poor separation efficiency and thermal degradation in traditional atmospheric distillation processes. This study designs a continuous four-columns vacuum distillation process. The NRTL-HOC thermodynamic model is selected to rigorously describe the liquid-phase non-ideality and vapor-phase association of polar phenolic molecules. A separation sequence comprising a phenol column, an o-cresol column, an m-/p-cresol mixed-fraction column, and a xylenols column is constructed. The theoretical stages, reflux ratios, and feed locations are optimized by steady-state simulation to minimize energy consumption and total annual cost while satisfying purity and recovery constraints. Considering the strong coupling and large time delays, a temperature-concentration cascade control system with feedforward compensation is designed. Phenol, o-cresol, the m-/p-cresol mixed fraction, and xylenols achieved purities of 99.8%, 99.5%, 99.2%, and 99.0%, with recoveries of 99.1%, 98.7%, 98.2%, and 97.5%, respectively. The total reboiler duty of the four-columns system was 5686.7 kW. Under feed-flow disturbances of $\pm 10\%$ and feed-composition disturbances of $\pm 5\%$, product purity was restored within 80 min, with a maximum deviation of no more than 0.15%. At 18.0 kPa, the optimized four-columns process achieved a maximum annual net economic margin of USD 1.585 million.

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1. Introduction

Coal tar, as an important byproduct of the coal chemical industry, contains extremely rich basic chemical raw materials. Phenolic compounds, including phenol, o-cresol, m-/p-cresol, and xylenols, are high-value fine-chemical intermediates widely used in resins, pharmaceuticals, pesticides, and flame retardants [1-2]. How to convert the extremely complex coal tar mixed phenols into high-purity monomer products is not only the core link to improve the overall economic benefits of the coal chemical industry chain, but also the only way to improve the efficiency of resource recycling [3]. However, the efficient separation of coal tar mixed phenols faces great technical challenges in chemical separation engineering. The boiling points of phenolic homologues are close, and the relative volatility of m-cresol to p-cresol is approximately 1.02, making conventional distillation difficult. There are strong polar interactions and highly non-ideal vapor-liquid phase equilibrium characteristics in the system. Meanwhile, phenolic substances are typical heat-sensitive materials, which are prone to oxidation, discoloration, and condensation reactions in high-temperature environments [4-5]. Therefore, the key issue of this system is not simply increasing the number of separation stages, but simultaneously balancing phase-equilibrium prediction accuracy, bottom heat duty, product purity, and thermal degradation risk under vacuum conditions. This feature

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requires the following focus on the logic of “polar close-boiling mixed phenols-vacuum distillation-dynamic stability” rather than on general distillation methods only. Because of the importance and complexity of phenol separation in coal tar, its efficient separation and refining has become a research hotspot in the modern coal chemical industry, and has economic and environmental significance. However, traditional solvent extraction and chemical washing suffer from high solvent consumption, substantial wastewater generation, and limited selectivity. Lv Q et al. conducted a comprehensive review of the extraction of phenols from coal tar. By deeply analyzing its component distribution and separation mechanism, exploring new green separation processes has become an inevitable trend to realize the high-value utilization of coal tar [6]. To overcome the pollution and energy consumption problems in traditional processes, Guo J et al. proposed an environmentally friendly conceptual design process. The process simultaneously and efficiently extracted phenols and naphthalene compounds in tar, significantly improving the overall separation efficiency of the system [7]. Yang Z X et al. used choline chloride-urea deep eutectic solvent to extract phenolic compounds from coal tar, and the experimental results showed that the method achieved effective separation of phenol and cresol components based on liquid-liquid phase equilibrium and hydrogen-bond interaction [8]. This provides a reference for understanding the phase behavior and separation characteristics of polar phenolic systems.

For coal tar mixed phenols with strong polarity and close-boiling characteristics, the property method directly affects column-sequence judgment, theoretical-stage number, reflux ratio, and heat-duty calculation. Therefore, the thermodynamic model is the starting point of the steady-state process design in this work. Fonseca F G et al. explored the Vapor-Liquid Equilibrium (VLE) of pyrolysis bio-oil (rich in phenols). The activity coefficient model such as Non-random Two-liquid (NRTL) had extremely high prediction accuracy when dealing with highly polar and non-ideal mixture systems [9]. For steady-state distillation architecture design, the separation sequence directly determines the energy consumption and equipment investment. Tang W T et al. compared various separation alternatives for industrial multi-component mixtures. By optimizing the complex column sequence and topology, the total annual cost of the entire process could be effectively reduced [10]. Although the pressure-swing heat-integrated distillation obtained by Yin et al. concerns an azeotropic system, its principles of pressure selection, heat integration, and parameter optimization provide design guidance for the four-columns vacuum process [11]. These process design theories provide some guidance for the continuous four-columns vacuum distillation separation of mixed phenols.

Steady-state optimization can only determine the column structure and operating parameters under nominal conditions. However, the composition of coal tar crude phenols is strongly affected by upstream feed fluctuations. Changes in feed flow rate or composition can propagate through the four columns and amplify the coupled fluctuations of column temperature, reflux flow, and product purity. Therefore, dynamic control is not an additional topic, but a necessary part to ensure the operability of continuous processes. As a typical multiple-input multiple-output (MIMO) system, a multi-component distillation column exhibits strong coupling and large time-delay characteristics. The development of its dynamic control strategy is the core of ensuring product quality. Flores-Gómez L R et al. evaluated fractional-order controllers in an extractive distillation column, and the results showed that the optimized controller improved the dynamic response and control stability of the column compared with conventional control strategies [12]. This provides a reference for the dynamic control design of strongly coupled distillation systems. For control under specific operating modes, Qu Q et al. constructed an optimal control framework for semi-continuous distillation. The system exhibited greater adaptive adjustment capabilities when dealing with feed fluctuations [13]. To improve the response speed and robustness of the control system, advanced control algorithms have been largely introduced into the field of distillation. Qian X et al. applied a model predictive control method based on dynamic mode decomposition. This algorithm more accurately predicted multi-variable complex dynamic behaviors and effectively shortened the adjustment time [14]. Guo et al. showed that feedforward-feedback cascade control using sensitive-tray temperature as the controlled variable can suppress product-quality deviations caused by feed disturbances [15].

In summary, many experts have explored the purification and steady-state separation processes of phenolic compounds in coal tar, as well as the thermodynamic modeling and dynamic control algorithms of complex distillation systems. Existing studies have addressed crude phenol separation processes, property methods, and distillation control, but these three aspects are often discussed separately. Steady-state process studies rarely further examine product-quality recovery under disturbances, while control studies are mostly based on general distillation systems and do not sufficiently consider the thermal sensitivity, close-boiling separation difficulty, and cumulative disturbance characteristics of series-connected mixed-phenol columns. Minor feed disturbances first alter the T101 bottoms stream and are then transmitted successively as the feed to downstream columns, accumulating material- and energy-balance deviations, amplifying T104 purity fluctuations, and prolonging recovery. Therefore, the study proposes a steady-state process design for continuous four-columns vacuum distillation of coal tar mixed phenol and a temperature-concentration cascade dynamic control strategy based on feedforward compensation. The research innovations are as follows: Based on the highly polar thermodynamic characteristics of the phenolic system, taking into account product purity and material thermal constraints, the continuous four-columns pressure reduction separation sequence is reconstructed and optimized from the steady-state level. Breaking the limitation of isolated steady-state investigations, a temperature-concentration cascade control system with feedforward compensation is developed based on the coupling characteristics of the four-columns process. A dynamic mathematical model is constructed to reveal the transient energy compensation mechanism, providing a feasible path for mixed phenol refining.

2. Design of Continuous Four-Columns Vacuum Distillation Process and Control Strategy

2.1 Raw Material Characteristics and Thermodynamic Physical Property Method Selection

The coal tar mixed phenol system has an extremely complex composition, mainly including phenol, o-cresol, m-cresol, p-cresol and xlenol and other homologues [16]. The boiling point differences between the components of this system are non-uniformly distributed, especially the boiling points of m-cresol and p-cresol are close, and their relative volatility is close to 1, which is a typical heat-sensitive system that is difficult to separate [17]. To accurately carry out process simulation and steady-state design, the feed composition of the mixed phenol raw materials and their basic thermodynamic parameters are first clarified. The specific parameters are detailed in Table 1.

Table 1. Feed composition and basic thermodynamic properties of coal tar mixed phenols

Component	Mass Fraction (%)	Molar Flow Rate (kmol/h)	Normal Boiling Point (°C)	Critical Temperature (K)	Critical Pressure (bar)
Phenol	35.2	45.1	181.7	694.2	61.3
o-Cresol	20.5	22.8	191	697.5	50.1
m-Cresol	25.1	27.9	202.3	705.8	45.6
p-Cresol	12.4	13.8	201.9	704.5	51.5
Xylenols	6.8	6.7	211–225	730	43.5

Table 1 represents the phenolic design feed after water settling, dewatering, and light-end removal, with the composition renormalized on an anhydrous phenolic basis. Before entering T101, the industrial feed should pass through an upstream dewatering and light-end removal unit, reducing water and light neutral oils to no more than 0.05 wt% and 0.10 wt%, respectively. The baseline feed rate was 116.3 kmol/h, corresponding to approximately 12.04 t/h. Before entering stage 28 of T101, the feed was preheated to 140 °C and maintained at 20 kPa. The flash calculation identified the feed as a saturated liquid. Both the liquid fraction and the feed thermal-state parameter used in the Underwood calculation were therefore set to 1.00. Critical properties and normal boiling points were retrieved from the Aspen Physical Property System used in this study. Xylenols were treated as a lumped isomeric pseudo component. Therefore, their boiling point is reported as a

range. After determining the basic properties of the raw materials, the rational selection of thermodynamic property methods is the core link to ensure the simulation accuracy of the vacuum distillation process. Since the coal tar mixed phenol system contains a large number of polar molecules and exhibits extremely significant non-ideal vapor-liquid equilibrium characteristics under negative pressure conditions, conventional equations of state cannot accurately describe its phase behavior. Therefore, this study uses a NRTL activity coefficient model to calculate the liquid phase non-ideality of the multi-component system. This model can accurately characterize the complex interactions between polar compounds. To ensure the traceability and reproducibility of property calculations, steady-state simulations are performed using the rigorous RadFrac module in Aspen Plus with the NRTL-HOC property method. NRTL describes liquid-phase non-ideality, whereas HOC corrects vapour-phase association. Binary interaction parameters are preferentially taken from the Aspen Plus database. Missing or incomplete parameters are regressed against literature VLE data. The activity coefficients required for simultaneous solution in this physical process are shown in Equation (1).

$$\begin{cases} \ln \gamma_i = \frac{\sum_j x_j \tau_{ji} G_{ji}}{\sum_j x_j G_{ji}} + \sum_j \frac{x_j G_{ij}}{\sum_k x_k G_{kj}} \left(\tau_{ij} - \frac{\sum_k x_k \tau_{kj} G_{kj}}{\sum_k x_k G_{kj}} \right) \\ \tau_{ij} = a_{ij} + \frac{b_{ij}}{T}, \quad G_{ij} = \exp(-\alpha_{ij} \tau_{ij}), \quad \alpha_{ij} = \alpha_{ji} = 0.30 \end{cases} \quad (1)$$

In Equation (1), γ_i represents the activity coefficient of the component i in the solution. x_j , x_k , and x_m represent the mole fractions of components j , k , and m in the liquid phase, respectively. T is the absolute temperature of the system, measured by Kelvin. n represents the total number of components in the system. τ_{ji} and τ_{ij} are dimensionless interaction energy parameters between components. G_{ji} and G_{ij} are local composition parameters related to non-randomness. α_{ji} is a non-randomness parameter characterizing the non-randomness of the mixture and satisfies the constant symmetry condition. a_{ji} and b_{ji} are the fitting constants for the temperature dependence of the binary interaction parameters. The parameter form is defined as $\tau_{ij} = a_{ij} + b_{ij}/T$, and the non-randomness parameter is uniformly set as $\alpha_{ij} = 0.30$. The non-randomness parameter is initialized at 0.30 and jointly regressed with the temperature-dependent binary interaction parameters against literature VLE data by minimizing deviations in vapor-phase composition and equilibrium temperature. The value of 0.30 is retained only when freeing this parameter produced no significant improvement. The final Aspen Plus input is checked: the non-randomness field is set to 0.30 for every binary pair, and the first value listed for each pair is entered in the forward temperature-independent binary interaction-parameter field represented by the lowercase Latin letter a. The main binary parameters used in the simulation are as follows: for phenol-o-cresol, $a_{ij}=0.214$, $b_{ij}=-86.5$, $a_{ji}=-0.137$, and $b_{ji}=128.4$. For phenol-m-cresol, $a_{ij}=0.236$, $b_{ij}=-92.8$, $a_{ji}=-0.151$, and $b_{ji}=137.6$. For phenol-p-cresol, $a_{ij}=0.228$, $b_{ij}=-89.3$, $a_{ji}=-0.146$, and $b_{ji}=132.1$. For phenol-xylenols, $a_{ij}=0.318$, $b_{ij}=-121.7$, $a_{ji}=-0.204$, and $b_{ji}=168.9$. For o-cresol-m-cresol, $a_{ij}=0.082$, $b_{ij}=-34.6$, $a_{ji}=-0.061$, and $b_{ji}=42.3$. For o-cresol-p-cresol, $a_{ij}=0.076$, $b_{ij}=-31.8$, $a_{ji}=-0.054$, and $b_{ji}=39.7$. For m-cresol-p-cresol, $a_{ij}=0.028$, $b_{ij}=-12.5$, $a_{ji}=-0.021$, and $b_{ji}=15.4$. For cresol isomers-xylenols, a_{ij} ranges from 0.115 to 0.156, b_{ij} ranges from -52.4 to -68.7, a_{ji} ranges from -0.083 to -0.112, and b_{ji} ranges from 63.5 to 84.2. These parameters are used to describe the liquid-phase non-ideality among phenol, cresol isomers, and xylenols, and served as the unified thermodynamic basis for the subsequent calculations of theoretical stages, reflux ratio, heat duty, and dynamic control. The activity coefficients calculated by the above simultaneous equations can provide extremely reliable thermodynamic data support for the subsequent global material balance and energy balance calculations of the continuous four-columns vacuum distillation process.

2.2 Architecture and Parameter Design of Steady-State Four-Columns Vacuum Distillation Process

After establishing thermodynamic property methods, a global architecture for steady-state four-columns vacuum distillation is constructed to achieve high-purity extraction of phenol, o-cresol,

m/p-cresol, and xylenol. This process employs a direct separation sequence, with the material flowing sequentially through four vacuum distillation columns. This arrangement effectively reduces the heat load at the bottom of each column according to the increasing boiling point of the components, thereby protecting the heat-sensitive phenolic components, as shown in Figure 1.

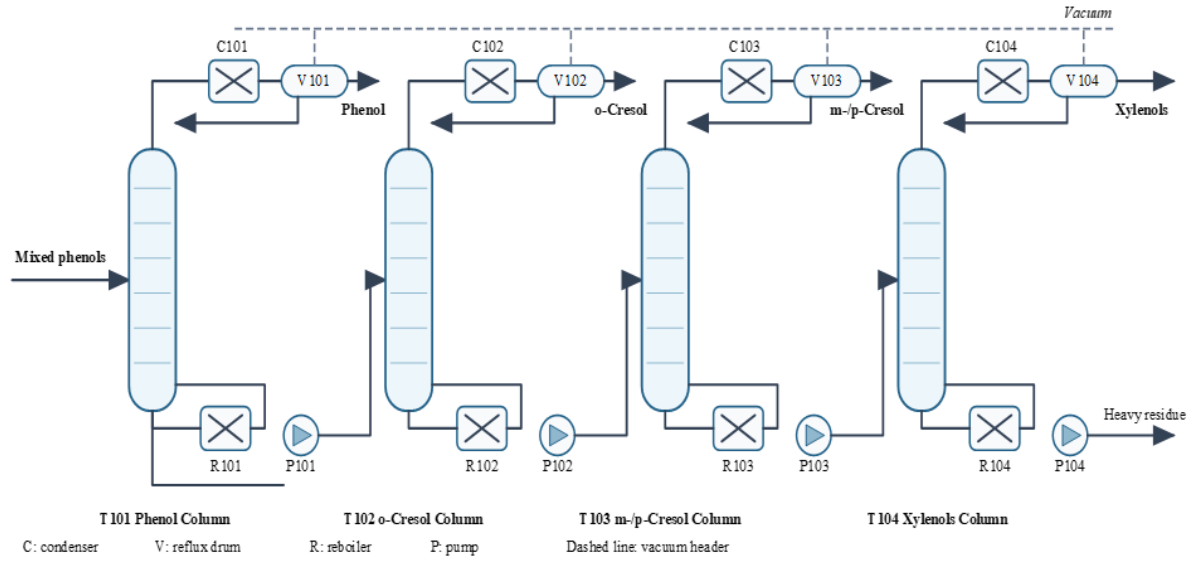


Fig. 1. Flow chart of continuous four-columns vacuum distillation process for coal tar mixed phenols

Figure 1 presents the continuous four-columns vacuum distillation process of coal tar mixed phenol (PFD). The diagram details how the material is separated in cascades between the phenol column (T101), o-cresol column (T102), m/p-cresol column (T103), and xylene column (T104) after the feed is fed. The condensers, reflux drums, reboilers, and pumps of T101-T104 are consistently labelled, and the T104 overhead product stream is identified as xylenols. The T103 column is not designed to separate m-cresol and p-cresol into individual high-purity products, but to obtain an m-/p-cresol mixed fraction. Because the boiling points of m-cresol and p-cresol are close and their relative volatility is close to unity, their economical separation by conventional vacuum distillation is difficult. Therefore, the product purity of T103 is defined as the total mass fraction of m-cresol and p-cresol in the mixed fraction, namely $w(\text{m-cresol} + \text{p-cresol})$, rather than the individual purity of m-cresol and p-cresol both exceeding 99%. If high-purity single m-cresol or p-cresol products are required, additional separation methods such as crystallization, extraction, adsorption, or other specialized processes should be introduced, which is beyond the present four-columns vacuum distillation process. The four-columns steady-state flowsheet is established under unified boundary conditions. The feed composition is taken from Table 1, and the operating pressure is set at 18.0 kPa with internal column pressure drop considered. A total condenser and a kettle reboiler are used for each column. Because the freezing points of high-purity phenol and o-cresol are 40.8 °C and 30.9 °C, respectively, the T101 and T102 overhead condensers use a closed hot-water loop operated at 50-60 °C. Heat tracing and low-temperature alarms and interlocks are provided to keep the heat-transfer surfaces and reflux lines above the corresponding freezing points. Circulating cooling water is used for T103 and T104 after solidification-risk screening. The number of stages, feed location, and reflux ratio are first estimated using the Fenske-Underwood method and then iteratively optimized in the RadFrac module with product purity, recovery and heat duty as the target indicators. To determine the optimal structural parameters of the distillation column, the preliminary values of the theoretical plate number and reflux ratio are obtained by a simple calculation method. The minimum theoretical plate number is estimated by the Fenske equation, as shown in Equation (2) [18].

$$N_{min} = \frac{\ln\left[\left(\frac{x_L}{x_H}\right)_D \left(\frac{x_H}{x_L}\right)_B\right]}{\ln \alpha_{avg}} \quad (2)$$

In Equation (2), N_{min} represents the minimum number of theoretical plates required for distillation to achieve the specified separation requirements. x_L and x_H represent the mole fractions of the light key component and the heavy key component in the overhead distillate D and bottom residue B , respectively. α_{avg} represents the average relative volatility of the system under the average operating pressure. Meanwhile, the minimum reflux ratio depends on the Underwood equation, as shown in Equation (3).

$$1 - q = \sum_{i=1}^n \frac{\alpha_i x_{Fi}}{\alpha_i - \theta} \quad (3)$$

In Equation (3), q is a parameter characterizing the thermal state of the feed. The value used in the shortcut calculation was 1.00, corresponding to the saturated-liquid feed specified in Section 2.1. α_i is the relative volatility of the component i relative to the reference component. x_{Fi} is the mole fraction of the component i in the feed liquid. θ is the root of the equation, whose value is usually limited to the relative volatility values of the two key components. n is the total number of components involved in the calculation. After the Underwood root is obtained from Equation (3), the minimum reflux ratio is determined by dividing, for each distillate component, the product of relative volatility and distillate mole fraction by the difference between relative volatility and the root, summing the resulting terms, and subtracting unity. The minimum theoretical stages for T101-T104 are 32, 37, 42, and 26, and the corresponding minimum reflux ratios are 2.13, 2.73, 3.67, and 1.87. The RadFrac-optimised stage numbers and reflux ratios are 55/3.2, 62/4.1, 70/5.5, and 45/2.8, respectively. After calculating macroscopic parameters, the physical design of the column internals needs to take into account both the requirements of low pressure drop and high efficiency. All four columns use Mellapak 250Y structured packing to minimize pressure drop and high-temperature residence time under vacuum. An HETP of 0.45 m is used only to convert the optimized equilibrium-stage counts into preliminary packed heights. The RadFrac separation calculation remains stage-based. Allowances for liquid distributors and installation are added separately during engineering sizing.

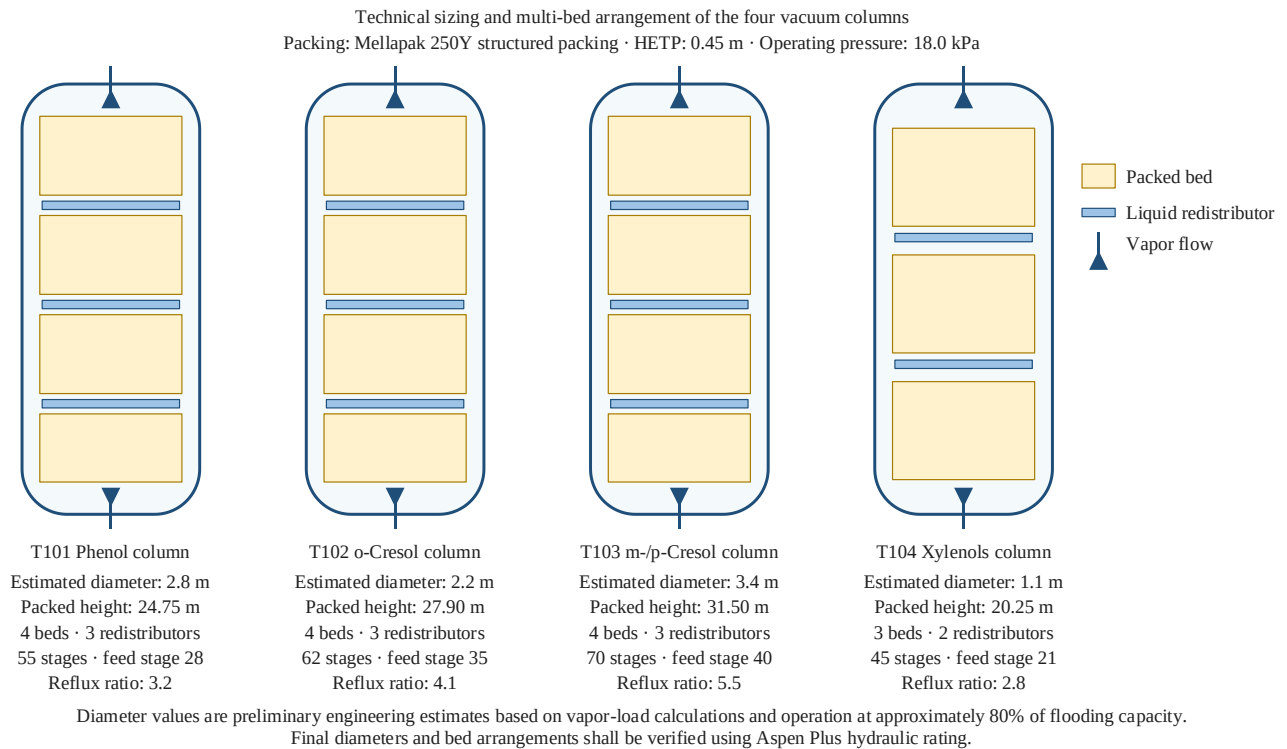


Fig. 2. Process-specific sizing and multi-bed hydraulic arrangement of columns T101-T104

Figure 2 presents the preliminary engineering dimensions and multi-bed arrangements of the four columns. T101-T104 employ Mellapak 250Y structured packing, with preliminary diameters of 2.8, 2.2, 3.4, and 1.1 m and packed heights of 24.75, 27.90, 31.50, and 20.25 m, respectively. The first

three columns contain four packed beds and three intermediate liquid redistributors, whereas T104 contains three beds and two redistributors. The diameters are preliminarily estimated from vapor loads at 18.0 kPa using approximately 80% of flooding capacity and should be confirmed by Aspen hydraulic rating.

2.3 Mathematical Model and Tuning Algorithm of Dynamic Control Strategy

Establishing a dynamic mathematical model for multi-component vacuum distillation is the basis for designing control strategies. Its core lies in accurately describing the mass and energy conservation process within each equivalent theoretical stage of the packed beds. The dynamic model is established by importing the converged Aspen Plus steady-state flowsheet into Aspen Dynamics, while retaining the state variables of sump holdup, reflux-drum holdup, pressure, temperature, and composition. The control structure includes pressure control, level control, sensitive-tray temperature control, and product-composition soft-sensor control. The composition feedback is not assumed to be an ideal instantaneous concentration signal, but is estimated from sensitive-tray temperature, column pressure, reflux flow rate, feed flow rate, and reboiler duty. A pressure-driven rigorous unsteady-state column model is adopted rather than applying empirical disturbances to steady-state results only. Dynamic holdups in the condenser, reflux drum, reboiler, and column liquid phase are retained for all four columns. The reboiler holdups of T101-T104 are set as 8.5, 7.8, 9.2, and 6.5 kmol, respectively, while the reflux drum holdups are set as 3.0, 2.8, 3.5, and 2.4 kmol. The liquid holdups are determined from the product of the nominal liquid flow rate and residence time. For T101-T104, the nominal reboiler liquid flow rates are 85, 78, 92, and 65 kmol·h⁻¹ with a 6 min residence time. The reflux-drum values are 36.0, 33.6, 42.0, and 28.8 kmol·h⁻¹ with a 5 min residence time. The packed beds were discretized into equivalent theoretical stages using an HETP of 0.45 m. A first-order liquid-hydraulic lag was adopted in Aspen Dynamics, with a nominal lag time of 2.0 min for each packed bed, while the dynamic liquid holdup varied with the 0.50 power of the liquid load. Based on this model, a $\pm 20\%$ change in holdup altered the settling time of T101 from 45 min to 43-47 min and the maximum purity deviation from 0.08% to 0.07%-0.09%. The liquid-level set point is fixed at 50% of the effective equipment volume. The top pressure of each column is set at 18.0 kPa, with an allowable fluctuation range of ± 0.5 kPa, and is regulated jointly by condenser duty and the vacuum system. Sensitive trays are selected according to the maximum steady-state temperature-composition sensitivity while avoiding the feed stage. Therefore, stages 34, 42, 48, and 30 are selected as the temperature-control trays for T101-T104, respectively. When ignoring the amount of gas phase retention and assuming complete mixing of the liquid phase, the total mass conservation differential equation of the j -layer tray is shown in Equation (4) [19].

$$\frac{dM_j}{dt} = L_{j-1} + V_{j+1} + F_j - L_j - V_j \quad (4)$$

In Equation (4), M_j represents the liquid phase molar retention on the j -th tray. t represents the independent variable time of the dynamic response. L_{j-1} represents the molar flow rate of the liquid phase flowing from the upper layer, that is, the $j - 1$ -th tray. V_{j+1} represents the molar flow rate of the gas phase rising from the next layer, the $j + 1$ -th tray. F_j is the feed molar flow rate of the tray. L_j and V_j respectively represent the molar flow rates of the liquid phase and gas phase flowing out of the tray. Based on the overall mass conservation, the component mass conservation equation for a specific component i in the system needs to be further calculated, as shown in Equation (5) [20].

$$\frac{d(M_j x_{i,j})}{dt} = L_{j-1} x_{i,j-1} + V_{j+1} y_{i,j+1} + F_j z_{i,j} - L_j x_{i,j} - V_j y_{i,j} \quad (5)$$

In Equation (5), $x_{i,j}$ and $x_{i,j-1}$ represent the mole fractions of the component i in the liquid phase of the j and $j - 1$ trays, respectively. $y_{i,j+1}$ and $y_{i,j}$ represent the mole fractions of the component i in the gas phase of the $j + 1$ and j trays, respectively. $z_{i,j}$ is the mole fraction of the component i in the feed. Based on the above rigorous mechanistic model, a global dynamic architecture

including a basic loop of liquid level and pressure and a higher-order regulation loop is constructed, as shown in Figure 3.

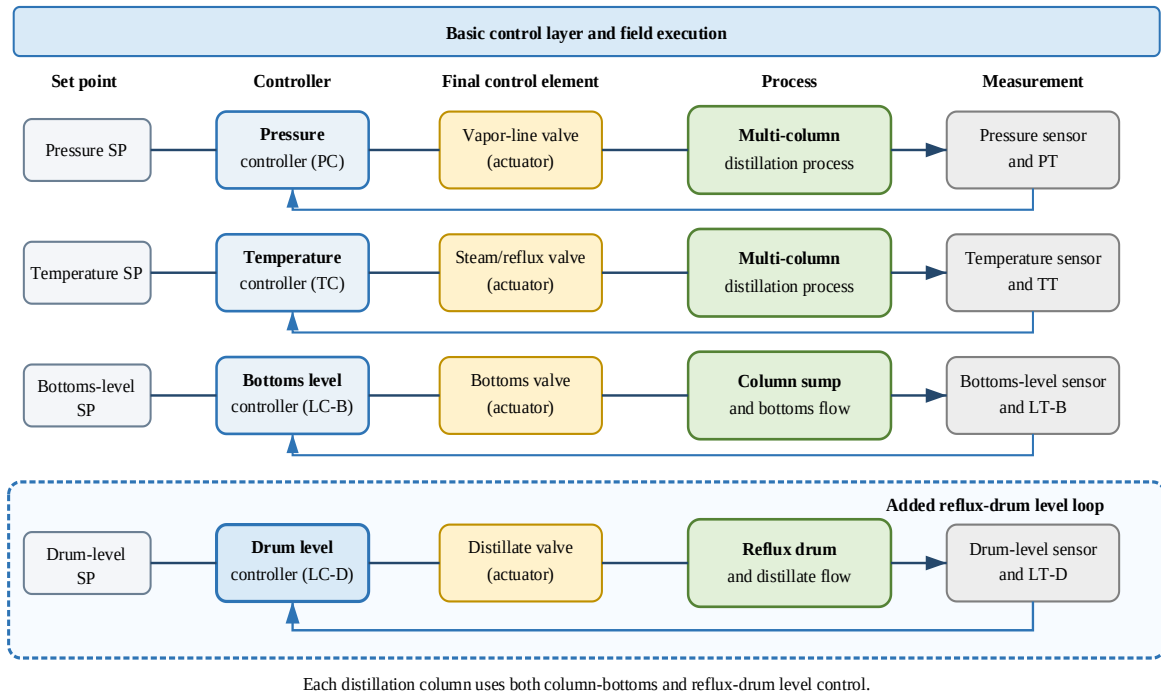


Fig. 3. Architecture and signal transmission of the global dynamic control system for multi-column distillation

Figure 3 illustrates the overall topology from the basic control loops for liquid level and pressure to the temperature control loop, using a standard system architecture logic block diagram. It clearly shows the closed-loop logic relationship between sensor measurement, transmitter signal transmission, and actuator valve opening adjustment. Because the vacuum distillation process exhibits strong nonlinearity and large hysteresis, single temperature control is insufficient to guarantee product purity during feed fluctuations. Therefore, a temperature-concentration cascade control algorithm with feedforward compensation is developed, as shown in Equation (6).

$$u(t) = u_0 + K_c \left[e(t) + \frac{1}{T_i} \int_0^t e(\tau) d\tau + T_d \frac{de(t)}{dt} \right] \quad (6)$$

In Equation (6), $u(t)$ is the adjustment signal output by the controller to the actuator at time t . K_c is the proportional gain constant of the controller, which determines the initial strength of the system's response to error. $e(t)$ represents the real-time deviation between the set value and the actual measured value. T_i is the integral time constant, used to eliminate the steady-state residual of the system. τ is the time integration variable in the integral operation process. T_d is the derivative time constant, used to provide advance adjustment based on the rate of change of deviation. The controller bias is the nominal valve position at zero error. The feedforward path superposes feed-flow and key-component mass-fraction disturbances. Each branch uses the negative ratio of the disturbance-path model to the manipulated-path model, implemented as a lead-lag element. The total command is the sum of the bias, PID feedback, and feedforward compensation and remains subject to the stated valve-position, rate, and dead-band constraints, as shown in Equation (7).

$$\begin{cases} \Delta u_{ff}(s) = G_{ff,F}(s)\Delta F(s) + G_{ff,z}(s)\Delta z(s) \\ G_{ff,j}(s) = -\frac{G_{d,j}(s)}{G_u(s)} = K_{ff,j} \frac{(\tau_{lead,j}s + 1)}{(\tau_{lag,j}s + 1)}, j \in \{F, z\} \end{cases} \quad (7)$$

In the actual tuning process, the initial value is obtained by the relay feedback method and the Tyreus-Luyben tuning rule. The optimal anti-disturbance performance is achieved through dynamic optimization, as shown in Figure 4.

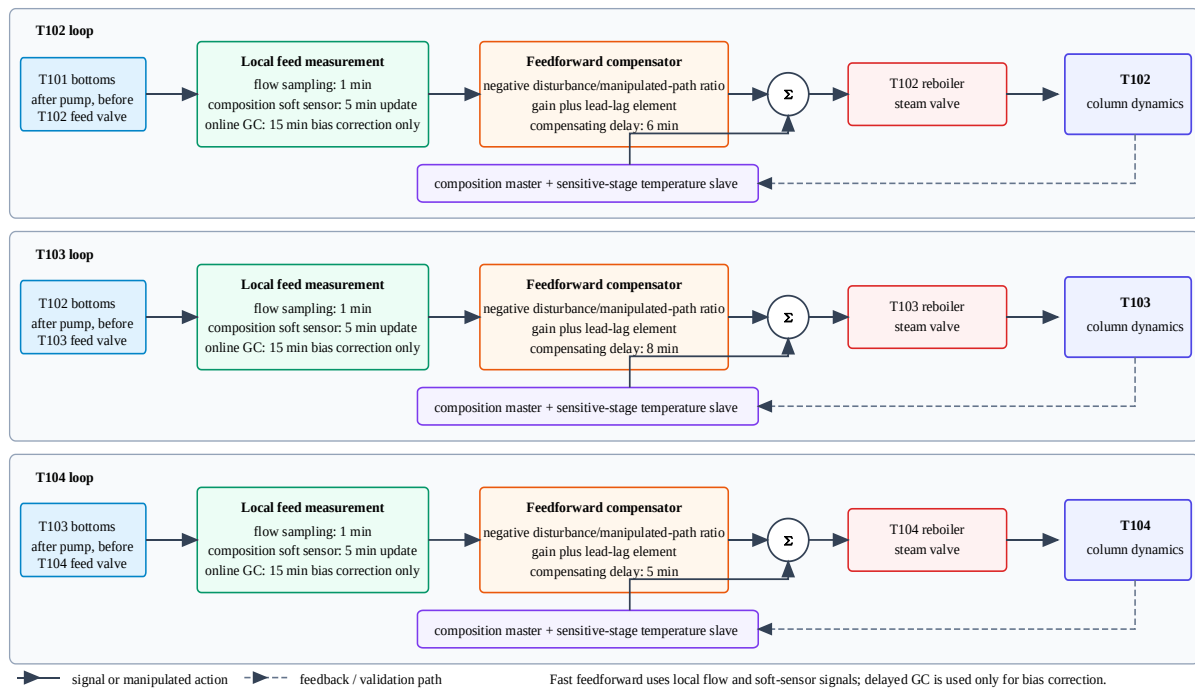


Fig. 4. Feedforward-feedback temperature-concentration cascade control strategy logic

Figure 4 is the algorithm control logic flow chart. It demonstrates in detail the logical judgment conditions and action execution steps of the main controller, that is, the concentration controller, and the secondary controller, that is, the sensitive plate temperature controller, to collaboratively calculate and adjust the heat load of the reboiler when the feed flow or concentration is disturbed. To avoid simplifying composition control as ideal concentration feedback, a soft-sensor-based concentration feedback scheme is adopted in the dynamic control system. The top or bottom product composition is not assumed to be directly and instantaneously measurable. Instead, it is estimated from online variables, including sensitive-tray temperature, column pressure, reflux flow rate, feed flow rate, and reboiler duty, combined with the NRTL thermodynamic model and dynamic material balance. Online gas chromatography can be used for steady-state validation and parameter correction, but it is not taken as the only input for fast closed-loop control. The concentration controller corrects the sensitive-tray temperature set point according to the estimated composition deviation, and the secondary temperature loop further adjusts the reboiler steam valve opening. The feedforward path pre-corrects the heat duty according to feed flow and composition disturbances, thereby reducing the influence of composition-analysis delay on control performance. The dynamic simulation is established by importing the steady-state Aspen Plus model into Aspen Dynamics, and the model file, control modules, and disturbance scripts are provided as Supplementary File S1. The sensitive tray is selected according to the maximum steady-state temperature-composition sensitivity of each column while avoiding the feed tray, namely the tray with the peak value of $|\partial x_D/B/\partial T_j|$. The sampling periods of temperature, pressure, and liquid level are set to 1.0, 0.5, and 1.0 min, respectively. The composition soft sensor is updated every 5 min, with a 15 min delay assigned to online chromatographic correction. Equal-percentage steam and reflux valves are used as actuators, with an opening range of 0-100%, a maximum rate of change of $10\% \cdot \text{min}^{-1}$, and a dead band of 1%. The primary controlled variable is the soft-sensed product composition, the secondary controlled variable is the sensitive-tray temperature, and the manipulated variables are reboiler duty and reflux flow rate. The feedforward path pre-compensates the reboiler duty according to changes in feed flow rate and key-component mass fraction. The feedforward signals for T102, T103, and T104 are obtained from the flow rate and soft-sensed key-component fraction of the T101, T102, and T103 bottoms streams, respectively,

measured downstream of the bottoms pump and upstream of the downstream-column feed valve. The fresh-feed signal of T101 is not used directly. Consistent with Equation (7), gain and lead-lag compensation are applied, with initial compensating delays of 6, 8, and 5 min, respectively. Flow signals are updated every 1 min and soft-sensor estimates every 5 min. The online GC delay of 15 min is used only for bias correction and is excluded from the fast feedforward loop.

3. Results and Discussion

3.1 Steady-State Design Calculation Results and Overall Column Distribution Characteristics of The Four-Columns Process

Based on the aforementioned rigorous thermodynamic model and process architecture design, the global steady-state calculation of the continuous four-columns vacuum distillation process was completed in chemical simulation platforms such as Aspen Plus, and the high-purity separation of each phenolic component was successfully achieved. Table 2 summarizes in detail the core data of the steady-state process optimization parameters and energy consumption distribution of the four-columns vacuum continuous distillation.

Table 2. Optimization parameters and energy consumption distribution of steady-state process for continuous four-columns vacuum distillation

Column ID	Theoretical Stages	Feed Stage	Reflux Ratio	Top Pressure (kPa)	Top/Bottom Temperature (°C)	Condenser Duty (kW)	Reboiler Duty (kW)	Product Purity (wt%)	Recovery (%)
T101 (Phenol)	55	28	3.2	18	112.5/142.0	1420.5	1580.2	99.8	99.1
T102 (o-Cresol)	62	35	4.1	18	132.4/158.5	1150.8	1295.4	99.5	98.7
T103 (m-/p-Cresol)	70	40	5.5	18	148.6/175.0	1680.3	1850.6	99.2 ^a	98.2
T104 (Xylenol)	45	21	2.8	18	165.2/192.1	890.1	960.5	99	97.5

Table 2 lists the optimized process parameters and energy consumption distribution of the four continuous vacuum distillation columns (T101, T102, T103, and T104) under steady-state conditions. T103 has 70 theoretical stages, a reflux ratio of 5.5, a condenser duty of 1680.3 kW, and a reboiler duty of 1850.6 kW, all of which are the highest among the four columns. T104 has 45 theoretical stages, a reflux ratio of 2.8, a condenser duty of 890.1 kW, and a reboiler duty of 960.5 kW. Product purity is at least 99% for all columns, with T101 reaching 99.8%, while recovery is at least 97.5%, with T101 reaching 99.1%. The feed-stage location and total number of theoretical stages were coordinated for each column to ensure effective separation and reasonable energy consumption. Feed-temperature sensitivity was estimated at 20 kPa. At 130 °C, the feed became slightly subcooled, the thermal-state parameter increased to approximately 1.05, and the T101 reboiler duty increased to 1654 kW. At 150 °C, the parameter decreased to approximately 0.95 and the duty decreased to 1507 kW. Both deviations were approximately 4.7% relative to the baseline duty of 1580.2 kW. To further reveal the thermodynamic and mass transfer microstates within each column under optimal conditions, Figure 5 illustrates the temperature, concentration, and phase distribution characteristics of the entire column during steady-state operation.

Figure 5(a) shows the temperature distribution across the entire column, with the top and bottom temperatures of T101 to T104 exhibiting a gradient increase from 112.5 to 142.0°C, 132.4 to 158.5°C, 148.6 to 175.0°C, and 165.2 to 192.1°C, respectively. The maximum temperature changes in T101, T102, T103, and T104 occur at relative stage positions of 0.51, 0.56, 0.57, and 0.47, respectively, consistent with the actual feed stages in Table 2. Figure 5(b) presents continuous liquid-phase mole-fraction profiles of phenol, o-cresol, m-cresol, p-cresol, and xylenols over all

theoretical stages of T101. Stage efficiency is calculated as the ratio of the actual key-component composition change in the outgoing vapor to the change required for vapor-liquid equilibrium and is then averaged using vapor load as the weighting basis. Low-load operation tends toward weeping, whereas high-load operation tends toward entrainment and flooding, reducing the efficiency from 72.8% to a minimum of 45.2%.

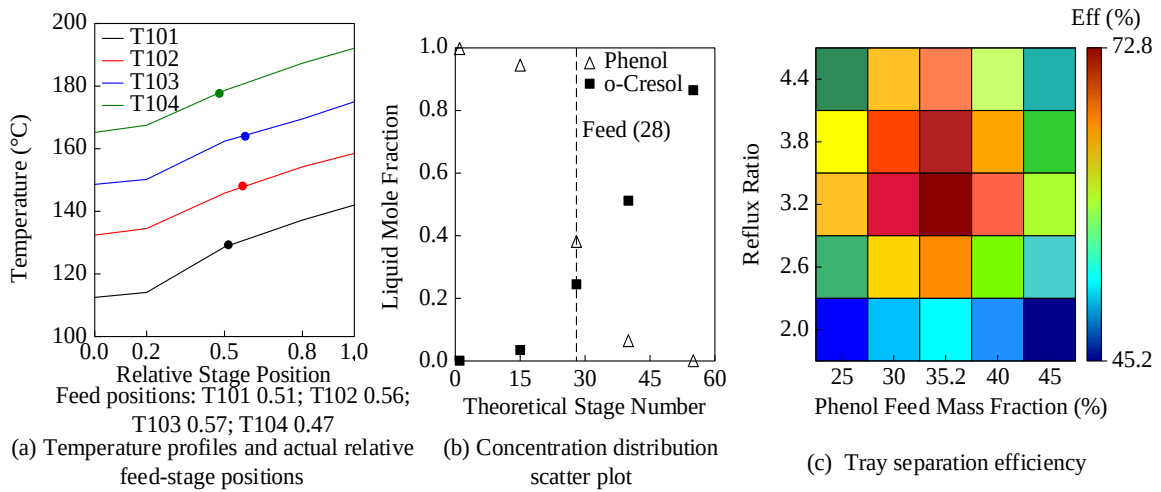


Fig. 5. Changes in temperature, concentration, and phase distribution during steady-state operation

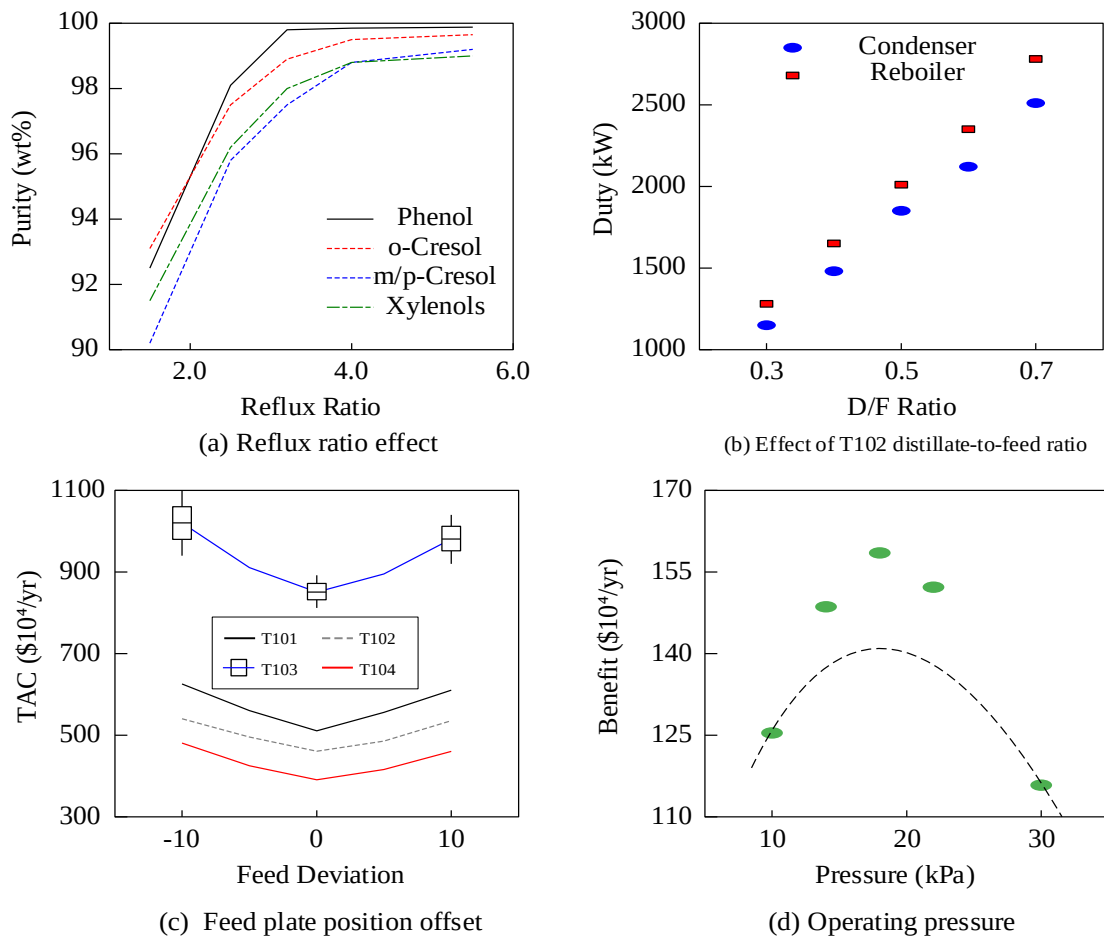


Fig. 6. Comprehensive impact of process operating parameters on product purity and heat load

The comprehensive separation efficiency reaches a maximum of 72.8%. For each stage, the efficiency is calculated as the ratio of the actual change in the key-component composition of the outgoing vapor to the change required to reach vapor-liquid equilibrium, and the column-average value is obtained by weighting the stage values by vapor load rather than imposing a constant input.

Away from the nominal condition, low-load operation tends toward weeping, whereas high-load operation tends toward entrainment and flooding, reducing effective vapor-liquid contact and lowering the efficiency to 45.2%. Figure 6 presents the comprehensive impact of process operating parameters on product purity and heat load.

Figure 6(a) presents the effect of reflux ratio. When the reflux ratio increased from 1.5 to 4.0, the purity of phenol and m/p-cresol increased from 92.50% and 90.20%, and eventually stabilized at nonlinear limit thresholds of 99.85% and 98.80%. Based on T102, Figure 6(b) shows the effect of the distillate-to-feed ratio on heat duties. As this ratio increased from 0.3 to 0.7, the reboiler duty increased from 1280.5 to 2780.2 kW. To avoid presenting the economic evaluation only as numerical results, the total annual cost and comprehensive economic benefit were calculated under unified boundary conditions. The equipment investment included four distillation columns, condensers, reboilers, vacuum systems, reflux pumps, and auxiliary instruments, and was estimated from the simulated column diameter, column height, heat-transfer area, and vacuum load using conventional chemical equipment cost correlations. With hot-water supply and return temperatures of 50 and 60 °C, the circulation rates for T101 and T102 were estimated as 122.3 and 99.1 t/h, respectively. Assuming an overall heat-transfer coefficient of 0.60 kW/(m²·K) and a 15% area allowance, the preliminary condenser areas were estimated as 47.5 and 28.5 m², respectively. The annualization factor was calculated with a depreciation period of 10 years and an annual interest rate of 8%. The operating cost included heating steam, hot-water circulation for T101 and T102, indirect cooling water, circulation pumping, electricity, and vacuum maintenance. Based on a loop pressure drop of 200 kPa, a pump efficiency of 70%, and a makeup rate of 1%, the annual hot-water-system operating cost was estimated as USD 0.1323 million, representing an increase of USD 0.0437 million per year over the direct-cooling-water case. Product revenue was calculated from the flow rate, purity, and average market price of phenol, o-cresol, m-/p-cresol mixed fraction, and xylene products, while the raw-material cost was deducted according to the mixed-phenol feed rate. The total annual cost was calculated as “annualized equipment investment + annual operating cost”, and the comprehensive economic benefit was calculated as “annual product sales revenue - raw-material cost - annual operating cost - annualized equipment investment”. Therefore, the values of USD 8.505 million, USD 10.204 million, and USD 1.585 million/year in Figures 6(c) and 6(d) were obtained from the above unified economic boundary conditions rather than directly inferred from heat duty alone. Figure 6(c) presents the feed plate position offset line and box plot. The solid black, dashed gray, and solid red curves represent T101, T102, and T104, respectively, whereas the box plot represents T103. A deviation of $\pm 10\%$ from the optimal position caused the annual total cost of column T103 to increase from the baseline of US\$8.505 million to US\$10.204 million. The cost variance and investment risk were greatest in this deep offset range. Figure 6(d) shows the sensitivity of the annual net economic margin to operating pressure. The economic margin was defined as the product sales revenue minus raw-material cost and the total annual cost of T101-T104, rather than the TAC of a single column. The maximum annual net economic margin was US\$1.585 million/year at 18.0 kPa. This value corresponds to the original direct-cooling-water baseline; after including the additional hot-water-system cost, the corrected maximum annual net economic margin was approximately USD 1.541 million per year. Although the TAC of T103 alone reached US\$8.505 million/year under the optimal feed-stage condition, the positive margin was obtained only after the sales revenue of phenol, o-cresol, m-/p-cresol mixed fraction, and xylenols was included. Therefore, Figure 6(c) reflects the cost sensitivity of T103, whereas Figure 6(d) reflects the net economic margin of the whole four-columns process. The product purity, recovery, and heat duty in Table 2 were obtained from the unified property model and rigorous RadFrac calculation described above. To verify the stability of the results, local perturbation calculations were performed for reflux ratio, feed stage, and operating pressure. When the key parameters changed slightly around their optimal values, the variation trends of product purity and heat duty were consistent with the results shown in Figures 5 and 6, indicating that the data in Table 2 were not single-point empirical estimates.

3.2 Dynamic Anti-Interference Response Analysis Under Feed Flow Disturbance

After completing the overall steady-state baseline design, the robustness of the control system to external operating condition fluctuations is verified. This study sets a step disturbance of $\pm 10\%$ to

examine the designed temperature-concentration cascade control system's ability to reconstruct the system's dynamic equilibrium. Table 3 details the PID tuning parameters of each control loop after optimization in dynamic simulation and provides key quantitative evaluation indicators for measuring disturbance rejection performance, including settling time and maximum dynamic deviation. The total dynamic simulation time was set as 120 min, the disturbance was introduced at 10 min, and the integration step was 0.05 min. The settling time was defined as the time required for product purity to return to and remain within $\pm 0.15\%$ of the set point. The maximum deviation was defined as the maximum absolute deviation of product purity from the steady-state set point after disturbance. The dynamic disturbance test used the optimal steady-state condition as the initial state. A $\pm 10\%$ step disturbance in feed flow rate and a $\pm 5\%$ disturbance in key component composition were introduced. The settling time was defined as the time required for product purity to return to and remain within $\pm 0.15\%$ of the set value, while the maximum deviation was defined as the maximum absolute deviation of product purity from the steady-state set value after disturbance. The IAE values in Table 3 were calculated from the absolute deviation of the primary-loop product mass fraction from its set point, rather than from the sensitive-stage temperature error. Integration covered 0-120 min, with the disturbance introduced at 10 min and zero error before the disturbance. The absolute deviations were accumulated using a 0.05 min integration step. Therefore, the IAE unit is $\text{wt}\% \cdot \text{min}$. Values of 1.25, 1.84, 2.10, and 0.95 represent the accumulated product-purity deviations over the complete response period.

Table 3. Dynamic controller tuning parameters and performance evaluation indicators

Control Loop	Controller Role	Controller Gain (Kc)	Integral Time (Ti, min)	Derivative Time (Td, min)	Settling Time (min)	Max Deviation (%)	IAE Index
T101 Distillate Purity	Master (Purity)	12.5	8.2	0.6 ^b	45	0.08	1.25
T102 Distillate Purity	Master (Purity)	15.8	10.5	0.8 ^b	52	0.12	1.84
T103 Distillate Purity	Master (Purity)	18.2	12	0.9 ^b	65	0.15	2.1
T104 Distillate Purity	Master (Purity)	10.4	7.5	0.5 ^b	40	0.09	0.95

Table 3 lists the tuning parameters and performance evaluation indicators of the dynamic controllers, covering four control loops. The listed parameters correspond to the primary product-purity controllers. The secondary controllers regulate the sensitive-stage temperatures at stages 34, 42, 48, and 30 of T101-T104, respectively. ^bDerivative times were preliminarily inferred from the stated Tyreus-Luyben tuning rule and should be verified against the actual Aspen Dynamics controller settings. The T103 controller had the highest gain (18.2), integral time (12 min), settling time (65 min), maximum deviation (0.15%), and IAE index (2.1). The T104 controller had the best performance (10.4), integral time (7.5 min), settling time (40 min), and IAE index (0.95), with a maximum deviation of 0.09%. The dynamic response characteristics of the system under a $+10\%$ step disturbance in feed flow rate are shown in Figure 7.

Figure 7(a) shows the purity response line graph. After the disturbance occurred at the 10th minute, the purity of m/p-cresol showed the maximum absolute deviation of 0.15% and dropped to 99.05%. Following controller action, the purities of the four products steadily returned to their respective set points within 40-65 min after the disturbance. Figure 7(b) presents sensitive-stage temperature deviations from their respective set points. T103 exhibited the largest increase of 1.7 °C. feedforward compensation limited the response to fewer than two oscillations and restored steady state by 70 min.

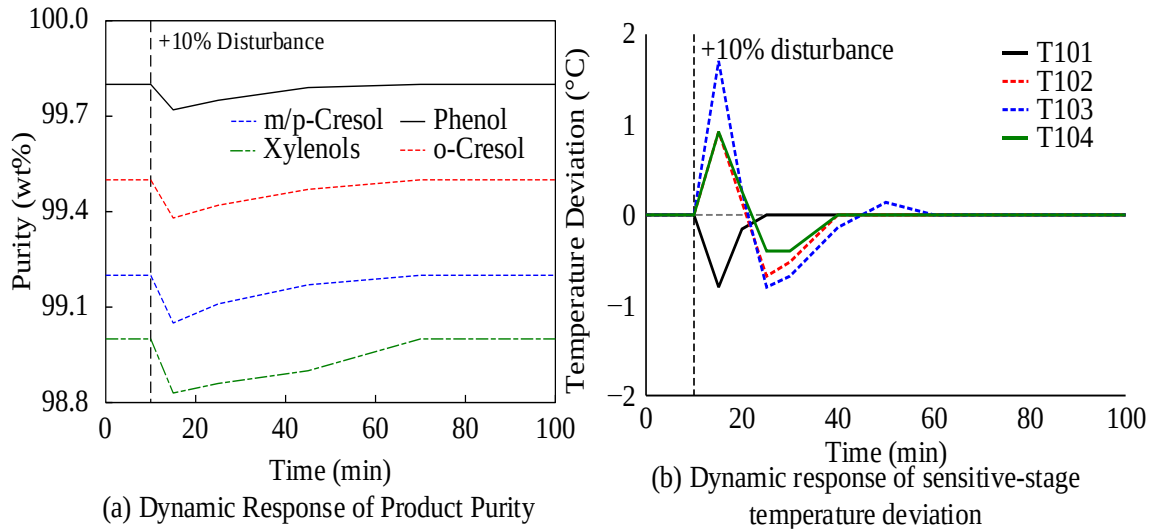


Fig. 7. Dynamic response characteristics of the system under feed flow +10% step disturbance

3.3 Dynamic Adaptive Adjustment Analysis Under Changes in Feed Composition

In actual chemical production operations, fluctuations in upstream processes often lead to significant transient or steady-state abrupt changes in the feed composition of distillation columns. This places high demands on the dynamic maintenance of system energy balance and the baseline assurance of product purity. When the content of light or heavy components in mixed phenol feed undergoes a large abrupt change, the original vapor-liquid phase equilibrium and concentration gradient within the column are rapidly disrupted. The study specifically introduced harsh working conditions in which the content of key components such as phenol suddenly changes by $\pm 5\%$ in the dynamic simulation to deeply examine the dynamic compensation mechanism and adaptive adjustment performance of the reboiler and condenser in the feedforward-feedback cascade control system. To provide precise quantitative support, Table 4 summarizes the steady-state reconstruction data of key energy input nodes of the system before and after the feed composition disturbance and the final deviation magnitude.

Table 4. Steady-state reconstruction data and deviation index under feed composition disturbance

Column ID	Initial Feed Fraction (wt%)	Disturbed Feed Fraction (wt%)	Initial Reboiler Duty (kW)	Compensated Reboiler Duty (kW)	Final Purity Deviation (%)
T101	35.2	30.2 (-5 percentage points)	1580.2	1465.5	-0.05
T102	20.5	25.5 (+5 percentage points)	1295.4	1410.2	-0.08
T103	25.1	20.1 (-5 percentage points)	1850.6	1735.8	-0.12
T104	6.8	11.8 (+5 percentage points)	960.5	1085.4	-0.04

Table 4 lists the steady-state reconstruction data and deviation indices under feed composition disturbances. The disturbance reported in Table 4 denotes an absolute change of five percentage points in mass fraction rather than a five-percent relative change. The feed fractions of all four-columns were disturbed by $\pm 5\%$, with T101 decreasing from 35.2% to 30.2%, T103 from 25.1% to 20.1%, T102 increasing from 20.5% to 25.5%, and T104 increasing from 6.8% to 11.8%. Correspondingly, the reboiler load was compensated, and the final purity deviation ranged from -0.04% to -0.12%, with T103 showing the largest deviation and T104 the smallest. The adaptive

control performance of the system under significant feed composition disturbances is shown in Figure 8.

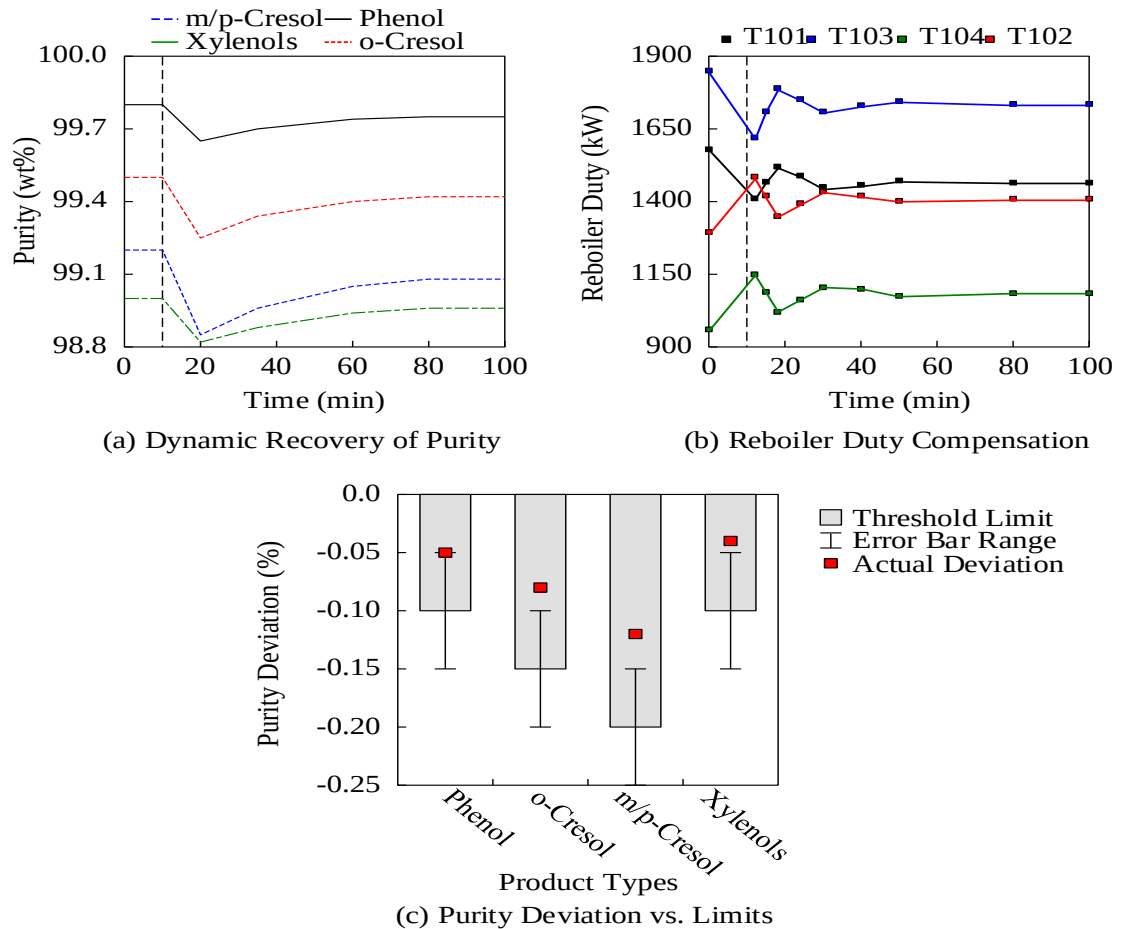


Fig. 8. Control adaptive adjustment performance of the system under large disturbances in feed components

Figure 8(a) shows the purity recovery line graph. After the component mutation at the 10th minute, the purity of m/p-cresol once bottomed out to 98.85%, but all four purity curves were forcibly pulled up within 80 minutes and aligned with the new steady-state benchmark (99.08%). Figure 8(b) presents continuous reboiler-duty compensation trajectories. The T101 reboiler duty converged from 1580.2 to 1465.5 kW after transient oscillations. Figure 8(c) presents signed steady-state purity deviations and allowable limits. The largest deviation was -0.12% for m/p-cresol and remained within the corresponding limit. The actual absolute purity deviation of each product (maximum of -0.12% for m/p-cresol) was strictly limited within the corresponding bar chart safety tolerance boundary (e.g., -0.20%) and the allowable range of the error bar, eliminating the risk of long-term concentration back mixing caused by extreme fluctuations.

4. Conclusion

Traditional separation processes are insufficient to address the low product purity, high energy consumption, and poor stability arising from the close boiling points and high thermosensitivity of coal-tar-derived mixed phenolic homologues. Therefore, an advanced continuous four-columns vacuum distillation process based on the NRTL model was developed. A continuous four-columns vacuum distillation steady-state process based on a non-random dual-liquid model was developed. A separation sequence consisting of phenol, o-cresol, m-/p-cresol mixed fraction, and xylenols columns was constructed. Considering the system's strong multivariate coupling and large time-delay characteristics, a temperature-concentration cascade dynamic control framework with feedforward compensation was designed. Steady-state optimization results showed that when the comprehensive operating pressure was maintained at 18.0 kPa, high-temperature condensation

reactions could be suppressed, achieving a maximum annual net economic margin of US\$1.585 million after deducting raw-material cost and the total annual cost of the four-columns process. The total annual cost of the m-/p-cresol column (T103) was reduced to USD 8.505 million, while the average tray separation efficiency of the phenol column (T101) reached 72.8%. Phenol, o-cresol, the m-/p-cresol mixed fraction, and xylenols achieved purities of 99.8%, 99.5%, 99.2%, and 99.0%, with recoveries of 99.1%, 98.7%, 98.2%, and 97.5%, respectively. Dynamic tests showed that the maximum product-purity deviation remained within 0.15%. Steady-state recovery required 40-65 min under positive or negative 10% step disturbances in feed flow and up to 80 min under an absolute five-percentage-point change in feed composition. The research has limitations such as insufficient pilot test verification and failure to consider the impact of trace impurities. For an anhydrous feed rate of approximately 12.0 t/h, a water content of 0.5-2.0 wt% would introduce an additional vaporization duty of approximately 40-160 kW in T101 and increase the molar load on the vacuum system by approximately 3%-12%. Therefore, the reported four-columns simulation results apply only to feed after dewatering and light-end removal. Future work will establish a pilot-scale calibration model, investigate heat-integration options, and integrate the feedforward cascade-control strategy into an industrial distributed control system for real-time validation. The research results can help increase the added value of coal chemical by-products and promote green development.

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Multi objective geometry optimization of deep-hole trepanning tools via a reinforcement learning enhanced Grey Wolf optimizer

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Article Info	Abstract
<p>Article History:</p> <p>Received 17 July 2026</p> <p>Accepted 14 Aug 2026</p> <p>Keywords:</p> <p>Deep-hole trepanning; SORD-RL-MOGWO; Pareto-optimal design; Machinability enhancement; Chip evacuation stability</p>	<p>Deep-hole trepanning is challenged by enclosed cutting conditions and limited chip evacuation, leading to increased cutting loads, chip blockage, and reduced tool reliability. These challenges are particularly severe in trepanning because the retained core material confines the annular cutting zone and narrows the chip evacuation path. To optimize the geometric parameters of deep-hole trepanning tools, a constrained multi-objective optimization model is developed to simultaneously minimize axial force, cutting torque, and chip blockage risk, with rake angle, clearance angle, point angle, edge radius, chip-breaker width, and chip-groove guide angle as design variables. To solve this model, a Sobol-Optimization Initialization, Reference-Vector Dual-Archive Management, and Reinforcement Learning-Regulated Multi-Objective Grey Wolf Optimizer (SORD-RL-MOGWO) is developed. Sobol-opposition initialization enhances initial population diversity and feasible-space coverage, reference-vector dual-archive management balances convergence and distribution uniformity, and reinforcement learning adaptively regulates exploration and exploitation during the search. Under validation conditions of a cutting speed of 20 m/min and a feed rate of 0.060 mm/r, the mechanical performance model achieved coefficients of determination of 0.98 and 0.99 for axial force and cutting torque, with average relative errors of 1.38% and 1.04%, respectively. SORD-RL-MOGWO achieved a hypervolume of 0.78 ± 0.01 and a feasible solution ratio of $95.93 \pm 1.72\%$, outperforming MOGWO, C-TAEA2, CMOEA-TSRA, and DRLOS-EMCMO. Across feed rates of 0.040–0.100 mm/r at 20 m/min, the optimized tool consistently reduced axial force, cutting torque, and chip blockage. At 0.100 mm/r, axial force and cutting torque were reduced by 14.23% and 14.00%, respectively, while the chip blockage rate decreased from 55.0% to 20.0%. These results demonstrate that the proposed method provides a practical approach to reducing cutting loads and chip-evacuation failure while maintaining structural feasibility, thereby improving tool reliability and process stability in the industrial deep-hole trepanning of difficult-to-machine alloys such as GH4169.</p>

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1. Introduction

Deep-hole components made from difficult-to-machine alloys are widely used in aero-engine shafts, landing-gear cylinders, nuclear-power equipment, and other safety-critical systems, where machining accuracy, material utilization, and process stability are strictly required. Deep-hole trepanning has considerable application value because it enables hole formation while retaining the central core material. However, the enclosed cutting zone, narrow chip-evacuation path, and strong coupling among tool parameters can increase cutting loads, promote chip blockage, and reduce cutting-edge reliability [1-3]. Therefore, coordinated optimization of deep-hole trepanning

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tool geometry is necessary to improve machining performance and adaptability under demanding operating conditions. Foundational studies have shown that BTA deep-hole trepanning involves coupled cutting-force generation, chip formation, and guide-pad support. The asymmetric cutting structure governs axial and radial loads, while the guide pads provide self-guiding and burnishing effects that influence bore accuracy and process stability [4-5].

In terms of deep-hole machining mechanism and tool structure, Strodict, S., et al. studied the problem of subsurface control in the deep-hole drilling process of the Boring and Trepanning Association (BTA), analyzed the influence of deep-hole machining conditions on the surface performance and service performance of the workpiece. Moreover, they demonstrated that deep-hole tool structure and machining state not only affected the hole formation process, but also related to the functional reliability of the machined surface [6]. Wegert R et al. further proposed a sensor integrated tool approach for single-edge deep-hole drilling, studying the relationship between process state and surface performance through near-cutting area information acquisition, providing a technical basis for load monitoring and condition regulation in deep-hole machining [7]. The above studies strengthened process-state perception and machining-quality evaluation but mainly focused on surface-performance regulation and process monitoring. Therefore, systematic co-optimization of the primary trepanning-tool variables, including rake angle, clearance angle, point angle, edge radius, chip-breaker width, and chip-groove guide angle, remains insufficiently investigated.

In terms of tool structure influence and drilling performance evaluation, Atif M et al. studied the effect of different tool designs on drilling performance using ultrasonic-assisted drilling of fiber-metal laminas as the object, indicating a close connection between tool geometry and machining load and hole machining quality [8]. Despite the differences in the processing objects and deep-hole trepanning conditions, the study showed that it was feasible to improve the performance of the drilling process through structured design. For deep-hole trepanning, annular cutting structures, closed chip removal channels, and retained mandrel features made the coupling between tool parameters more prominent [9]. This coupling arises because the rake and point angles simultaneously redistribute cutting deformation, axial load, and chip-flow direction, whereas the edge radius, chip-breaker width, and chip-groove guide angle affect both edge strength and chip curling, with excessive strengthening or flow restriction potentially increasing friction, torque, and chip congestion. Therefore, relying solely on empirical selection or single-objective optimization makes it difficult to obtain a parameter scheme that simultaneously ensures low cutting loads, low chip-blockage risk, and structural safety.

Constrained multi-objective evolutionary algorithms provide an effective approach to complex mechanical parameter optimization. Qiao K et al. emphasized the balance among convergence, solution distribution, and constraint satisfaction [10], which is consistent with the coupled constraints of edge thickness, structural safety, and chip-evacuation area in deep-hole trepanning tool design. Ming F et al. introduced deep reinforcement learning for adaptive operator selection under complex constraints [11], while Zheng J incorporated Q-learning into Multi-objective Grey Wolf Optimizer (MOGWO) to dynamically switch search strategies and improve multi-objective optimization performance [12]. However, general-purpose MOGWO methods are not well suited to deep-hole trepanning geometry optimization because the coupled structural and chip-evacuation constraints create a narrow and discontinuous feasible region. Conventional population initialization and leader selection may generate excessive infeasible solutions, concentrate individuals around a few dominant solutions, and cause premature convergence and diversity loss, thereby limiting the coverage of feasible Pareto-optimal designs. In deep-hole trepanning, the retained central core narrows the annular chip-evacuation space, increases the confinement of chip curling and transport, and strengthens the coupling between chip-control geometry, cutting load, and blockage risk. However, this core-induced constraint has not been adequately considered in existing tool-geometry optimization studies.

In summary, previous studies have not provided a unified framework for the constrained multi-objective geometry optimization of deep-hole trepanning tools. To address this gap, a coupled evaluation model of axial force, cutting torque, and chip-evacuation risk is established, and a Sobol-

Opposition Initialization, Reference-Vector Dual-Archive Management, and Reinforcement Learning-Regulated Multi-Objective Grey Wolf Optimizer (SORD-RL-MOGWO) is developed. The MOGWO provides the Pareto-search framework for balancing conflicting performance objectives. Sobol-opposition initialization improves population coverage within the narrow feasible region, while reference-vector dual-archive management prevents solution aggregation and maintains convergence and diversity. Reinforcement learning dynamically selects search strategies according to the population state, thereby reducing premature convergence and balancing exploration and exploitation. The resulting framework generates structurally feasible Pareto-optimal geometries under multiple feed conditions.

2. Methods and Materials

2.1 Research Framework

During the deep-hole trepanning process, the enclosed cutting zone and restricted chip evacuation make tool performance highly sensitive to geometric parameters, potentially increasing axial force and torque, destabilizing chip breaking, and weakening edge load capacity [13–14]. Therefore, a parameter-optimization method combining mechanical-performance evaluation with an improved MOGWO is developed to coordinate cutting load, chip evacuation, and structural reliability. The research framework for the multi-objective geometric optimization of deep-hole trepanning tools is shown in Figure 1.

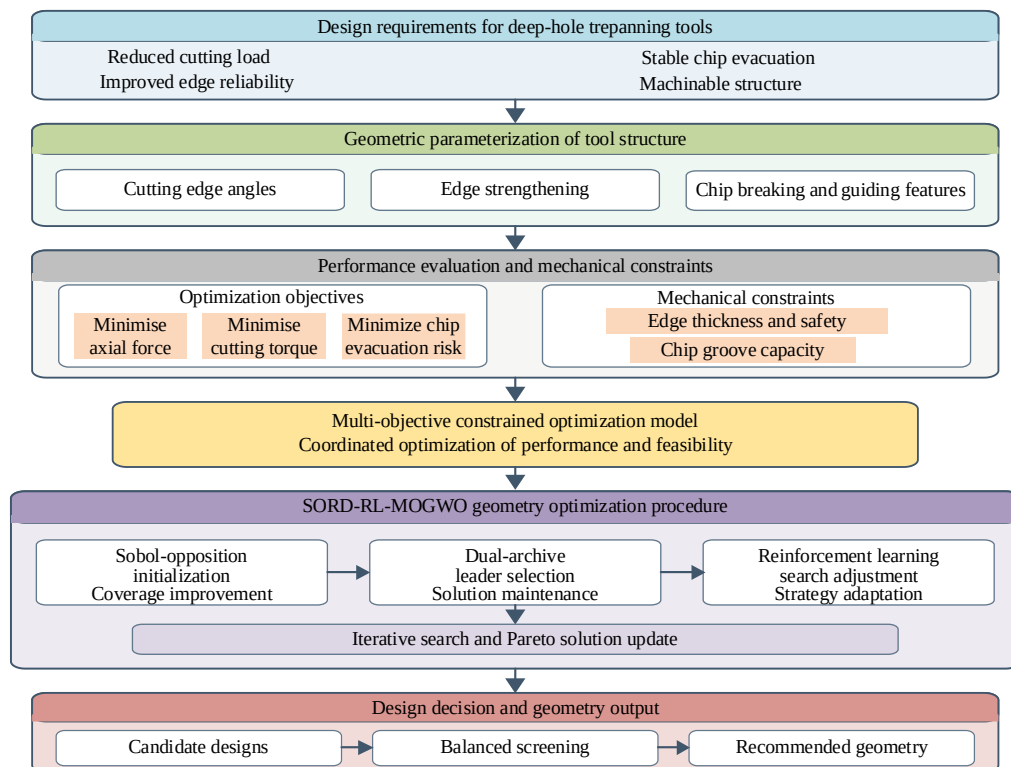


Fig. 1. Research technology roadmap

Figure 1 illustrates the overall research framework. First, the key geometric parameters affecting cutting load, edge reliability, and chip-evacuation stability are identified and defined as bounded design variables. Under consistent workpiece, tool-material, and cutting conditions, axial force, cutting torque, and chip-evacuation risk are jointly evaluated to establish a constrained multi-objective optimization model that also considers structural safety and chip-channel capacity. During optimization, Sobol-opposition initialization improves design-space coverage, the reference-vector dual-archive mechanism balances convergence and solution distribution, and reinforcement learning dynamically selects the grey wolf search strategy. Finally, Pareto-optimal parameter combinations satisfying low cutting load, low chip-evacuation risk, and structural-safety requirements are screened to obtain the optimized deep-hole trepanning tool design.

In Figure 1, the design requirements represent system-level performance targets rather than direct constraint equations. Reduced cutting load is expressed through the axial-force and cutting-torque objectives, while stable chip evacuation is represented by the chip-evacuation-risk objective and the minimum chip-groove area. Edge reliability and structural manufacturability are ensured through the minimum effective edge thickness, structural safety margin, and variable bounds. Therefore, the mechanical constraints define the feasible design region within which the performance objectives are optimized.

2.2 Materials, Tool Configuration, and Experimental Setup

Physical cutting tests were designed to evaluate the applicability of SORD-RL-MOGWO to the geometric optimization of deep-hole trepanning tools. The workpiece material was aged GH4169 bar stock, and an indexable carbide trepanning tool with an outer diameter of 32 mm, a retained-core diameter of 18 mm, and an effective machining depth of 100 mm was used. Because GH4169 retains high strength at elevated temperatures and has low thermal diffusivity, heat tends to accumulate near the tool–chip interface during enclosed deep-hole machining, thereby changing the material flow stress, friction state, contact length, axial force, and cutting torque. High-pressure internal coolant also generates additional axial thrust and a smaller torque component. These thermal and hydraulic effects were considered to avoid incorrectly attributing condition-dependent loads to tool geometry. Coolant pressure, flow rate, and cutting speed were therefore kept constant in all tests. Axial force and cutting torque were synchronously collected at a sampling frequency of 10 kHz. The stable cutting signals were low-pass filtered, and their mean values were used for subsequent analysis. The main material properties and experimental conditions are listed in Table 1.

Table 1. Experimental and platform, processing conditions settings

Category	Item	Setting
Workpiece Conditions	Verification material	GH4169 aged bar
	Material hardness	42.6 ± 0.7 HRC
	Blank dimensions	Diameter 52 mm × Length 160 mm
Tool Conditions	Tool structure	Indexable deep-hole trepanning tool
	Insert material and coating	Carbide inserts, TiAlN coating
	Trepanning outer diameter / Core diameter	32 mm / 18 mm
Cutting Conditions	Cutting speed	20 m/min
	Feed rate	0.040, 0.060, 0.080, 0.100 mm/rev
	Machining depth	100 mm
	Cooling method	High-pressure internal coolant
	Coolant pressure	4.0 MPa
	Coolant flow rate	18 L/min
	Axial force range	0–5 kN
Measurement Conditions	Axial force measurement accuracy	±0.2% full scale, ±10 N
	Torque range	0–50 N·m
	Torque measurement accuracy	±0.2% full scale, ±0.10 N·m
	Sampling frequency	10 kHz
Repetition Settings	Number of load test repetitions	5 times per condition
	Number of chip clogging observation trials	20 times per condition

The coolant pressure of 4.0 MPa and flow rate of 18 L/min were selected based on preliminary cutting tests and the stable operating range of the cooling system. Lower settings resulted in intermittent chip retention at a machining depth of 100 mm, whereas further increases provided no clear improvement in chip evacuation and increased hydraulic disturbance. Therefore, these values were adopted to balance chip-evacuation stability and load-measurement reliability. The 10

kHz sampling rate was used to capture the original force and torque signals and identify unstable cutting segments. However, the optimization focused on quasi-steady cutting performance; therefore, tool-entry transients and chatter-related fluctuations were excluded, and the low-pass-filtered mean values from the stable cutting stage were used as the evaluation responses. Dynamic stability indicators were not included as independent optimization objectives.

Chip blockage was evaluated through 20 independent cutting trials for each tool geometry and feed-rate condition. A trial was classified as blocked when accumulated chips visibly obstructed the annular evacuation channel and required feed interruption or manual chip removal. Transient torque fluctuations without visible channel obstruction were not counted as blockage. The chip-blockage rate was calculated as the number of blocked trials divided by 20.

2.3 Geometric Parametric Design of Deep-Hole Trepanning Tools and Multi-Objective Optimization Model

Recent research on BTA deep-hole machining of GH4169 has shown that tool structure and material selection significantly affect cutting performance and hole quality [15]. Therefore, the cutting region and chip-guiding region were parameterized to provide the design variables for subsequent multi-objective optimization. The structure diagram of the key geometric parameters of the deep-hole trepanning tool is shown in Figure 2.

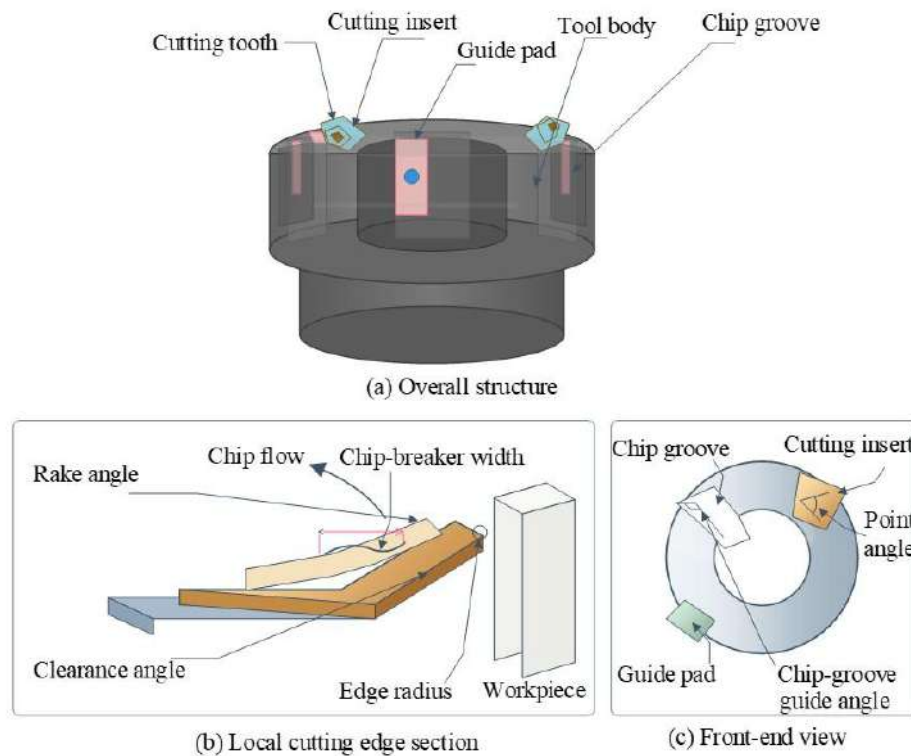


Fig. 2. Schematic diagram of the key geometric parameters and structure of the deep-hole trepanning tool

In Figure 2, the rake angle regulates material flow along the rake face and cutting resistance, while the clearance angle affects flank-workpiece contact and cutting-edge support. The point angle redistributes the cutting load, and the edge radius balances cutting sharpness and resistance to chipping. The width of the chip breaking table regulates the bending and breaking state of the chips. The chip-groove guide angle defines the preferred chip-flow direction and alters the effective chip-evacuation channel area. This design function is consistent with established findings that chip-channel geometry affects chip transport, coolant flow, and tool stiffness in deep-hole drilling [16-17]. This system of geometric parameters can systematically characterize the combined effect of tool structure on cutting performance and chip removal efficiency. Thus, the study defines the geometric design variables of deep-hole trepanning tools as shown in Equation (1).

$$\mathbf{x} = [\gamma, \alpha, \varphi, r_e, b_c, \theta_g]^T, \quad \mathbf{x}^L \leq \mathbf{x} \leq \mathbf{x}^U \quad (1)$$

In Equation (1), $\mathbf{x}$ represents the tool geometric parameter vector. γ represents the rake angle. α represents the clearance angle. φ represents the point angle. r_e indicates the radius of the edge arc. b_c indicates the width of the chip breaking table. θ_g represents the chip-groove guide angle. $\mathbf{x}^L$ and $\mathbf{x}^U$ represent the lower limit vector and upper limit vector of the allowable values for each design variable. Superscript T indicates matrix transposition. The physical limits used in the optimization are listed in Table 2.

Table 2. Physical limits of the geometric design variables

Variable	Physical meaning	Unit	Lower limit	Upper limit
γ	Rake angle	°	6.0	18.0
α	Clearance angle	°	5.0	12.0
φ	Point angle	°	110.0	125.0
r_e	Edge radius	mm	0.1	0.3
b_c	Chip-breaker width	mm	0.8	1.6
θ_g	Chip-groove guide angle	°	15.0	35.0

Accordingly, the lower- and upper-bound vectors were defined as $\mathbf{x}^L = [6.0, 5.0, 110.0, 0.10, 0.80, 15.0]^T$ and $\mathbf{x}^U = [18.0, 12.0, 125.0, 0.30, 1.60, 35.0]^T$, respectively. These bounds restrict the search to geometrically feasible and manufacturable tool configurations. Tool geometry affects machining performance through coupled mechanisms: cutting-edge angles govern load generation, the edge radius influences structural capacity, and chip-breaker and chip-groove parameters control chip flow and blockage risk [18]. Their relationships with the multi-objective responses are shown in Figure 3.

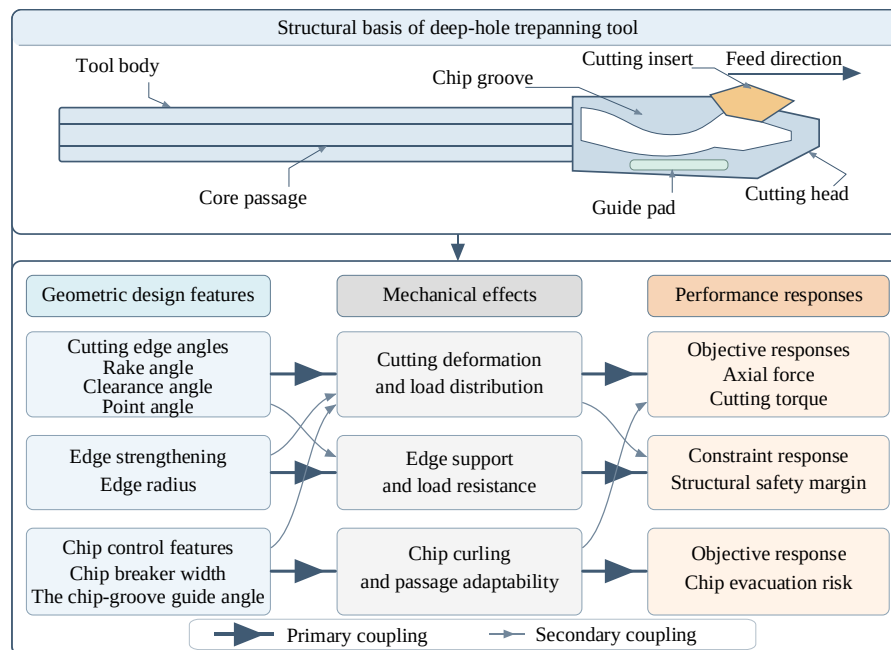


Fig. 3. Relationship diagram between tool geometric parameters and multi-objective performance responses

Under the conditions of fixed cutting speed, feed rate and processing material, the study divides the annular cutting edge of the trepanning tool into N micro-element cutting edge segments and calculates the axial force and tangential force contributions generated by each micro-element cutting edge segment based on the mechanical cutting coefficient. Unlike a conventional drill, the

indexable trepanning tool has an annular cutting edge constrained by the retained core. Along this edge, the radial position, local cutting speed, undeformed cutting area, effective engaged length, and torque arm vary continuously. Therefore, the cutting edge is divided into N micro-element segments, within each of which the local geometry and cutting coefficients are assumed to be approximately uniform. The axial-force and tangential-force contributions of all segments are then summed, while the torque contribution is weighted by the corresponding radial distance, as expressed in Equation (2).

$$\begin{cases} F_z(\mathbf{x}) = \sum_{j=1}^N [K_{zc,j}(\mathbf{x})A_j + K_{ze,j}(\mathbf{x})l_j] \\ M_t(\mathbf{x}) = \sum_{j=1}^N r_j [K_{tc,j}(\mathbf{x})A_j + K_{te,j}(\mathbf{x})l_j] \end{cases} \quad (2)$$

In Equation (2), $F_z(\mathbf{x})$ represents the evaluation value of the overall axial force of the tool. $M_t(\mathbf{x})$ represents the overall cutting torque evaluation of the tool. j represents the micro-element number of the cutting edge. N represents the total number of micro-elements. A_j represents the j th undeformed cutting area corresponding to the cutting-edge segment. l_j represents the j th effective cutting-edge length of the cutting-edge segment. r_j indicates the radial distance from the cutting edge to the center of rotation of the tool. $K_{zc,j}$, $K_{ze,j}$ and $K_{tc,j}$ and $K_{te,j}$ represent the j th axial shear load coefficient and the axial edge load coefficient of the cutting-edge section. $K_{tc,j}$ and $K_{te,j}$ represent the tangential shear load coefficient and the tangential edge load coefficient.

Under the fixed workpiece, feed, spindle-speed, and coolant conditions, the cutting coefficients were assumed to remain constant along the insert. Although the local cutting speed increases with radial position, its variation within the narrow annular cutting range was limited. The resulting torque difference was represented through the local undeformed cutting area, effective cutting-edge length, and radial torque arm of each micro-element. Any residual influence of cutting-speed variation was implicitly included in the experimentally calibrated coefficients. Therefore, the model is applicable to the tested cutting-speed range, whereas a wider speed range would require speed-dependent coefficient calibration.

GH4169 work hardening increases local flow stress, particularly at smaller edge radii. Since the micro-elements represent simultaneously engaged edge segments, they do not successively cut the same hardened layer. The average work-hardening effect was therefore incorporated into the experimentally calibrated cutting coefficients. The cutting coefficients were identified experimentally using 18 tool-geometry schemes. The geometric terms and stable-stage measured loads were fitted by least squares to determine the axial and tangential shear and edge coefficients. All calibration tests were conducted on GH4169 at 20 m/min and 0.060 mm/r under fixed coolant conditions, and the identified coefficients were kept unchanged during validation and optimization.

Considering the characteristics of long chip removal paths and limited channel space in deep-hole machining, the study further constructs the chip removal risk assessment function based on the chip curling state and the matching relationship of the chip removal channel, as shown in Equation (3).

$$R_c(\mathbf{x}) = \lambda_1 \frac{R_\chi(\mathbf{x})}{R_{lim}} + \lambda_2 \frac{A_\chi(\mathbf{x})}{A_g(\mathbf{x})} + \lambda_3 \frac{L_\chi(\mathbf{x})}{L_{lim}}, \quad \lambda_1 + \lambda_2 + \lambda_3 = 1 \quad (3)$$

In Equation (3), $R_c(\mathbf{x})$ represents the risk assessment value for chip removal. $R_\chi(\mathbf{x})$ represents the chip curling radius determined by the chip breaker structure. R_{lim} represents the maximum allowable chip curling radius. $A_\chi(\mathbf{x})$ represents the equivalent occupied area of chips in the direction of chip discharge. $A_g(\mathbf{x})$ represents the effective channel area of the chip removal groove. $L_\chi(\mathbf{x})$ represents the evaluation value of chip continuous length. L_{lim} represents the maximum allowed continuous chip length. λ_1 , λ_2 and λ_3 represent the weight coefficients of the three chip removal evaluation items, all of which are non-negative. The three terms were normalized before aggregation. Equal weights were adopted, with $\lambda_1 = \lambda_2 = \lambda_3 = 1/3$, to avoid subjective preference among chip curling radius, occupied channel area, and continuous chip length. These weights were

kept fixed throughout the optimization. The maximum allowable continuous chip length was defined as a condition-specific empirical threshold rather than a universal material constant. It was determined from preliminary chip-morphology observations by identifying the transition from regularly broken chips to long continuous chips associated with unstable evacuation. Because the work-hardening behavior of GH4169 affects chip stiffness and fracture, this threshold is applicable only within the tested tool-geometry, feed-rate, and coolant ranges and should be recalibrated under substantially different machining conditions.

Equation (3) is a reduced-order geometric risk function established under fixed coolant conditions. The coolant pressure and flow rate were maintained at 4.0 MPa and 18 L/min for all calibration, validation, and optimization tests. Therefore, the hydraulic effects on chip curling and transport were implicitly incorporated into the experimentally observed chip behavior, but were not independently identified as model variables. The chip-groove guide angle may alter local flow separation, coolant return velocity, and turbulence intensity, thereby affecting the chip curling radius and evacuation stability. Because coolant velocity and pressure distributions were not measured, introducing an explicit hydraulic term would lack sufficient experimental support. Consequently, the current chip-risk model is valid within the specified coolant conditions and geometric ranges, but its predictive accuracy may decrease when coolant parameters or the chip-groove guide angle vary substantially. Future work will couple coolant-flow measurements or computational fluid dynamics analysis with the chip-risk model to quantify these hydraulic effects.

The retained core further narrows the annular coolant-return and chip-evacuation passage, producing a Venturi-like flow acceleration and an associated pressure gradient. Local contraction, chip accumulation, and flow losses may generate back pressure and additional fluctuations in the measured axial force. Under the fixed retained-core diameter, coolant pressure, and flow rate used in this study, these hydraulic effects were implicitly included in the experimentally calibrated axial-force response. However, the optimization model uses the stable-stage mean axial force and does not treat pressure-induced force fluctuations as an independent objective. Therefore, the current model is applicable to the specified core geometry and coolant conditions, while substantial changes in the annular clearance may require coupled fluid-structure modelling and dynamic pressure measurements.

Based on this, the study aims to minimize axial force, cutting torque, and chip removal risk as optimization objectives. Then, the study uses effective blade thickness, structural safety margin, and chip removal channel area as mechanical design constraints to establish a multi-objective optimization model for the geometric parameters of deep-hole trepanning tools, as shown in Equation (4). Accordingly, the design requirements are converted into performance objectives, whereas the mechanical constraints are imposed as mandatory feasibility conditions for every candidate geometry.

$$\begin{cases} \min_{\mathbf{x}} \mathbf{F}(\mathbf{x}) = [F_z(\mathbf{x}), M_t(\mathbf{x}), R_c(\mathbf{x})]^T \\ \mathbf{x}^L \leq \mathbf{x} \leq \mathbf{x}^U \\ t_e(\mathbf{x}) \geq t_{min} \\ S_s(\mathbf{x}) \geq S_{min} \\ A_g(\mathbf{x}) \geq A_{min} \end{cases} \quad (4)$$

In Equation (4), $\min_{\mathbf{x}} \mathbf{F}(\mathbf{x})$ represents the multi-objective function vector. $t_e(\mathbf{x})$ represents the effective blade thickness. t_{min} represents the minimum allowable effective blade thickness. $S_s(\mathbf{x})$ represents the safety margin of the tool structure. S_{min} represents the minimum safety margin requirement. A_{min} represents the minimum chip removal channel area that ensures chip passage.

The constraint values were calculated from the parameterized tool model. The effective edge thickness was defined as the minimum distance between the rounded cutting edge and the insert-support boundary, and was jointly determined by the rake angle, clearance angle, point angle, and edge radius. The structural safety margin was calculated from the allowable stress and the maximum equivalent stress produced by the predicted axial force and cutting torque. The effective chip-groove area was defined as the minimum groove cross-sectional area projected normal to the chip-flow direction, and was mainly controlled by the chip-breaker width and chip-groove guide

angle. These quantities were automatically updated for each candidate geometry before feasibility screening.

A separate thermal objective or constraint was not introduced because the present study focuses on initial tool-geometry optimization under fixed thermal boundary conditions. The cutting speed, coolant pressure, and coolant flow rate were maintained at 20 m/min, 4.0 MPa, and 18 L/min, respectively, so that the thermal state remained comparable among candidate geometries. The influence of heat accumulation on material flow stress, friction, and retained-core clearance was therefore indirectly reflected in the experimentally calibrated axial-force and cutting-torque responses. However, temperature fields and thermally induced deformation were not measured independently, and introducing an explicit thermal constraint without such data would lack experimental support. Future work will incorporate temperature sensing and thermo-mechanical modelling to evaluate thermal expansion and tool-core clearance under higher-load conditions.

2.4 Geometric Parameter Optimization Algorithm Design Based on SORD-RL-MOGWO

In response to the problems of insufficient initial population coverage, easy aggregation of non-dominated solutions and weak adaptability of search strategies in multi-objective optimization of deep-hole trepanning tools, the SORD-RL-MOGWO algorithm is proposed based on the traditional multi-objective grey wolf optimization algorithm. The Sobol-oppositional learning initialization, the leader wolf selection mechanism based on the reference vector double archive, and the reinforcement learning search adjustment mechanism are integrated. To improve initial population coverage, Sobol low-discrepancy sequences are mapped onto the bounded geometric design space [19], while opposition-based learning generates complementary candidates around the center of each parameter interval [20]. The combined initialization mechanism is shown in Figure 4. In Figure 4, the algorithm first maps the Sobol sequence to the actual parameter space based on the upper and lower limits of each geometric parameter of the tool, and then expands the initial candidate schemes through opposition learning. The initialization process is shown in Equation (5).

$$\begin{cases} \mathbf{x}_i^0 = \mathbf{x}^L + \mathbf{s}^i \odot (\mathbf{x}^U - \mathbf{x}^L) \\ \mathbf{x}_i^{op} = \mathbf{x}^L + \mathbf{x}^U - \mathbf{x}_i^0 \end{cases} \quad (5)$$

In equation (5), $\mathbf{x}_i^0$ represents the i th Sobol initial individual. $\mathbf{s}^i$ represents the i th Sobol low difference sampling vector, with each component value located in the [0,1] interval. $\odot$ denotes the Hadamard product, in which each component of the Sobol vector is multiplied by the corresponding component of the tool-parameter range vector. It is an element-wise multiplication operator rather than an additional model parameter. $\mathbf{x}_i^{op}$ represents the opposing solution of the i th individual. The physical outcome of the opposition mapping is a complementary distribution of the original and opposition candidates around the midpoint of each parameter interval. After feasibility screening, the retained candidates provide broader coverage of the allowable geometric space, particularly near parameter boundaries and sparsely sampled regions. After merging two types of individuals, the algorithm performs initial population screening based on the degree of constraint satisfaction and non-dominated level, thereby enhancing the search coverage ability of different tool structure regions in the initial stage.

To balance the convergence quality and distribution uniformity of the Pareto-optimal tool designs, a reference-vector-based dual-archive leader wolf selection mechanism is introduced. Non-dominated candidate solutions are first associated with different reference-vector regions. The convergence archive preferentially retains solutions close to the ideal objective region, whereas the diversity archive preserves representative solutions from sparsely populated reference directions. During leader selection, the primary leader wolf is selected from the convergence archive, while the other two leader wolves are selected from different sparse regions of the diversity archive [21–22]. This mechanism coordinates convergence and solution-space coverage while preventing excessive concentration of candidate geometries, as shown in Figure 5.

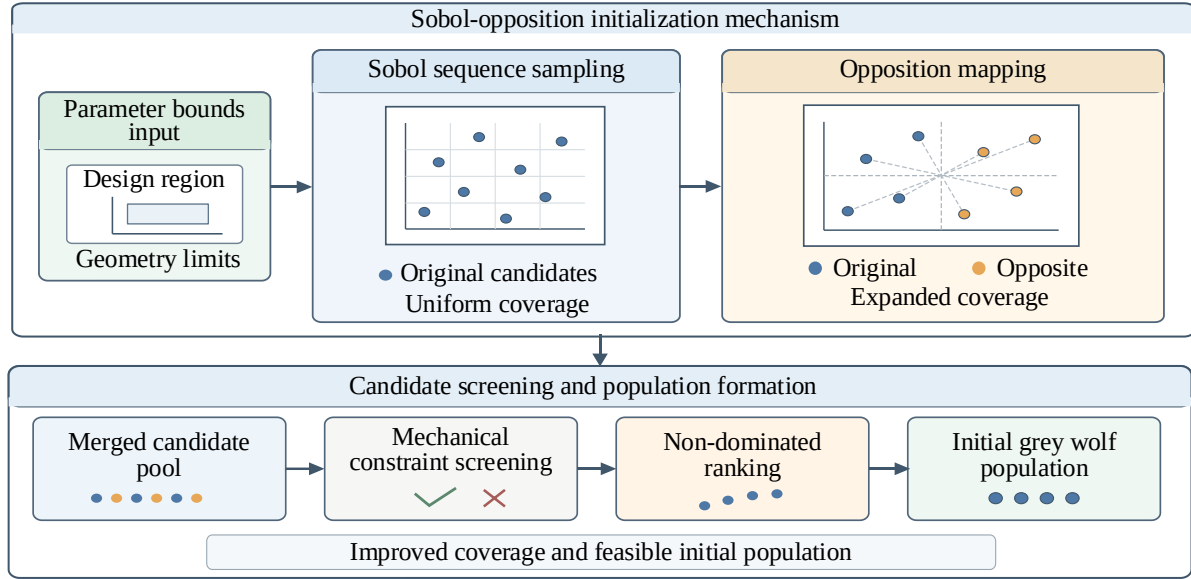


Fig. 4. Sobol-opposition learning initialization mechanism diagram

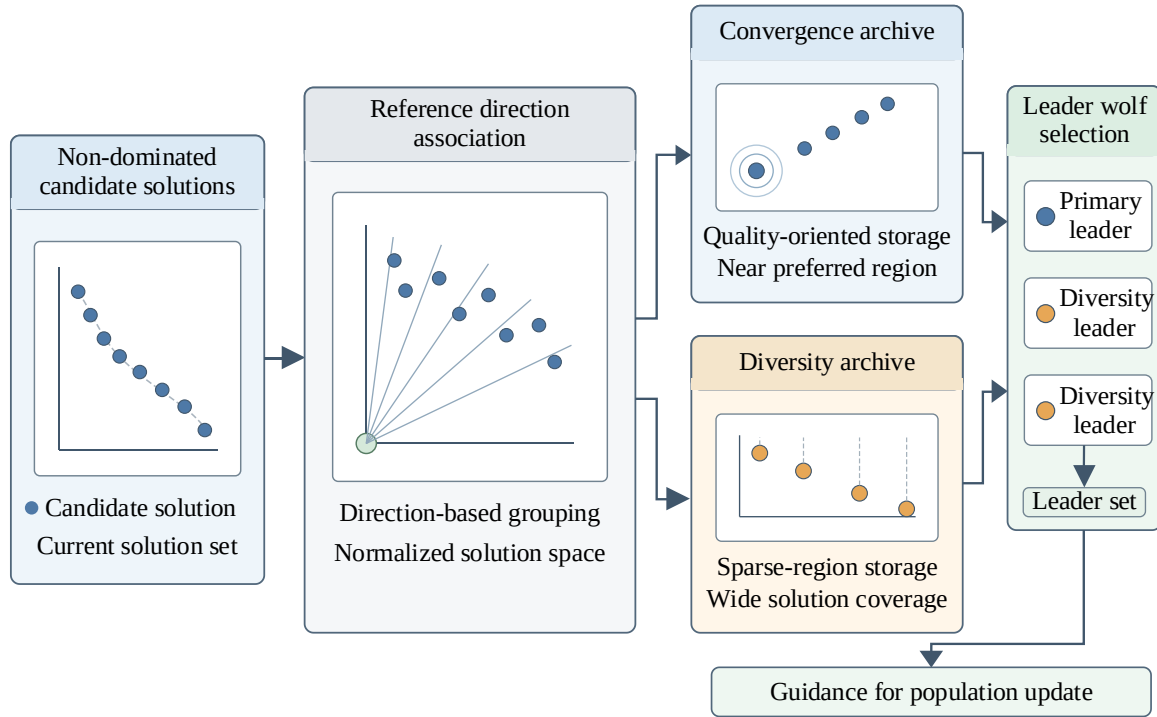


Fig. 5. Reference-vector dual-archive leader-selection mechanism

The probability of the q th reference area being selected as the diversity leadership area is defined as shown in equation (6).

$$p_q = \frac{(n_q + \varepsilon)^{-1}}{\sum_{m=1}^{N_r} (n_m + \varepsilon)^{-1}} \quad (6)$$

In equation (6), p_q represents the probability of selecting the q th reference region. n_q represents the number of schemes associated with the q th reference vector in the diversity archive. N_r represents total number of reference vectors. m represents the reference vector region index. ε is a small positive constant set to 10^{-12} to prevent division by zero when a reference-vector region contains no associated solutions. It was fixed throughout all optimization experiments and has a negligible influence on the selection probabilities of non-empty regions. Regions with fewer options have a higher probability of selection, which can avoid excessive concentration of the solution set. When selecting the leader wolf, α wolf selects from the convergence profile, while β

wolf and δ wolf select from different reference areas in the diversity profile to jointly guide the search of the gray wolf population. The preference for sparsely populated reference-vector regions does not imply unstable preservation of extreme solutions. Candidate solutions are associated with reference vectors in the normalized three-objective space, and boundary sectors corresponding to low axial force, low cutting torque, or low chip-evacuation risk generally remain sparsely occupied. These sectors therefore receive higher selection probabilities from the diversity archive, whereas the convergence archive prevents poorly converged boundary solutions from dominating leader selection. Selecting leader wolves from different reference-vector regions consequently preserves extreme solutions while reducing excessive concentration around a single compromise region.

Based on this, the study uses Q-learning to adapt the grey wolf search strategy, dividing the search process into three states: exploration, balance, and exploitation, and configuring corresponding position update actions. The exploratory action focuses on expanding the range of tool parameter search, the balanced action takes into account scheme expansion and target approximation, and the exploitation-oriented action strengthens the guidance of the superior leader wolf to the adjacent area. The search state was updated at the beginning of each iteration according to the normalized iteration progress. The exploration state was assigned during the first 30% of iterations, the balance state during 30%–70%, and the exploitation state during the final 30%. If the hypervolume did not improve for five consecutive iterations, the state was temporarily reset to exploration to avoid search stagnation. These transition criteria were fixed for all optimization runs. The SORD-RL-MOGWO algorithm process is shown in Figure 6.

In Figure 6, SORD-RL-MOGWO generates initial populations through Sobol-opposition learning, calculates axial force, cutting torque, and chip removal risk for each tool scheme, and screens out infeasible individuals based on mechanical constraints. During the iteration, the reference vector double file mechanism is used to update the non-dominated scheme and select the leader wolf, while the reinforcement learning module adaptively selects the position to update the action based on the search state. To measure the contribution of different actions to the optimization process, the study constructs a reward function by combining the degree of improvement in the Pareto solution set, the number of new feasible non-dominated schemes, and the degree of constraint violation. Moreover, the study uses Q-learning to update the action value, as shown in Equation (7).

$$\begin{cases} r_t = \eta_1 \Delta HV_t + \eta_2 \Delta N_t - \eta_3 V_t \\ Q_{t+1}(s_t, a_t) = Q_t(a_t, a_t) + \rho \left[r_t + t \max_{a \in A} Q_t(s_{t+1}, a) - Q_t(s_t, a_t) \right] \end{cases} \quad (7)$$

In equation (7), r_t represents the reward value obtained in the t th iteration. ΔHV_t represents the increment of the Pareto solution set hypervolume in this iteration. ΔN_t represents the number of feasible non dominated tool options added. V_t represents the degree of violation of population constraints. η_1, η_2 and η_3 represent reward evaluation weights. $Q_t(s_t, a_t)$ represents the value of selecting action a_t in state s_t during the t th iteration. ρ represents the learning rate. t represents the discount factor. A represents the set of search actions. a represents candidate actions in the action set. s_{t+1} represents the next iteration state. All reward terms were normalized before aggregation. The weights η_1, η_2 and η_3 were set to 0.4, 0.3, and 0.3, respectively, through preliminary grid-search experiments, giving greater emphasis to Pareto-set improvement while balancing feasible-solution generation and constraint control. The learning rate ρ was set to 0.2. The discount factor ζ was set to 0.9 to retain long-term search feedback without excessively weakening the influence of the current reward. These parameters were fixed in all experiments.

Exploration does not rely on unrestricted random action selection. An ε -greedy policy was adopted, with ε set to 0.30, 0.15, and 0.05 during the exploration, balance, and exploitation stages, respectively. Therefore, most actions are selected according to the current Q-values, while limited randomness is retained to avoid premature convergence. The reward is evaluated at the end of each iteration, and the Q-table is updated immediately with a learning rate of 0.20; consequently, the search strategy can change in the next iteration. A discount factor of 0.90 propagates delayed improvements to previous state-action pairs. If the hypervolume does not improve for five consecutive iterations, the search state is temporarily returned to exploration. The hypervolume reward jointly reflects convergence towards the Pareto front and objective-space coverage. However, the reinforcement learning module does not independently determine solution

uniformity. Distribution homogeneity and boundary-solution preservation are primarily maintained by the reference-vector dual-archive mechanism. The reinforcement learning module adjusts the search strategy according to hypervolume improvement, feasible-solution generation, and constraint satisfaction, thereby promoting promising regions without concentrating the population exclusively around the knee point.

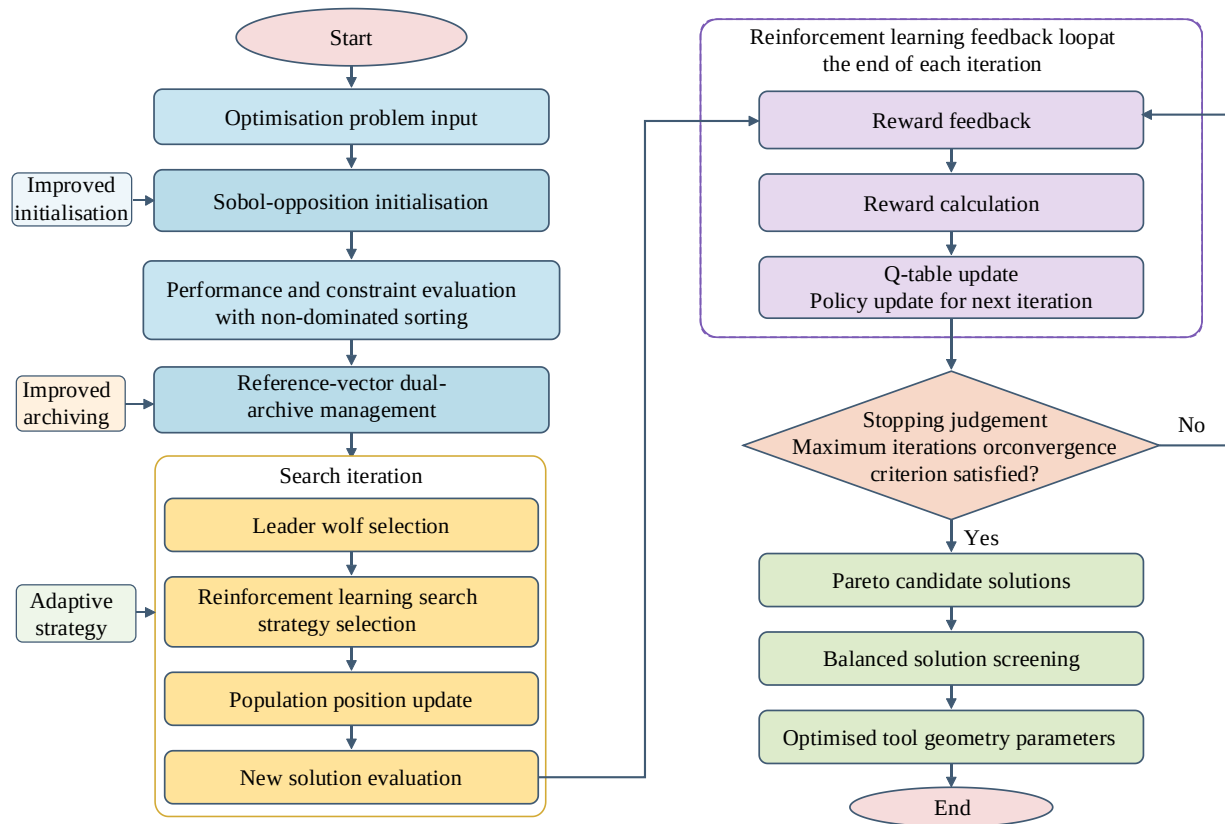


Fig. 6. Flowchart of the SORD-RL-MOGWO algorithm

After reaching the termination condition, the algorithm outputs a Pareto scheme set that satisfies mechanical constraints, and normalizes the evaluation of axial force, cutting torque, and chip removal risk. Combined with structural safety margin, a knee point scheme with comprehensive performance coordination is selected. Finally, the rake angle, clearance angle, point angle, edge radius, chip-breaker width, and chip-groove guide angle are determined.

3. Results

3.1 Practical Cutting Verification of The Mechanical Performance Evaluation Model

The cutting coefficients were calibrated using the 18 tool-geometry schemes described in Section 2.3. Eight additional schemes that were not involved in coefficient identification were used for independent validation. They covered both interior regions and parameter combinations near the allowable boundaries rather than being concentrated in a narrow safe region. All validation tests were conducted under the same machining conditions, and the model validity is therefore limited to the specified geometric ranges and cutting conditions. The calibrated coefficients were fixed throughout the validation process, and the predicted axial force and cutting torque were compared with the stable-stage measured values, as shown in Figure 7.

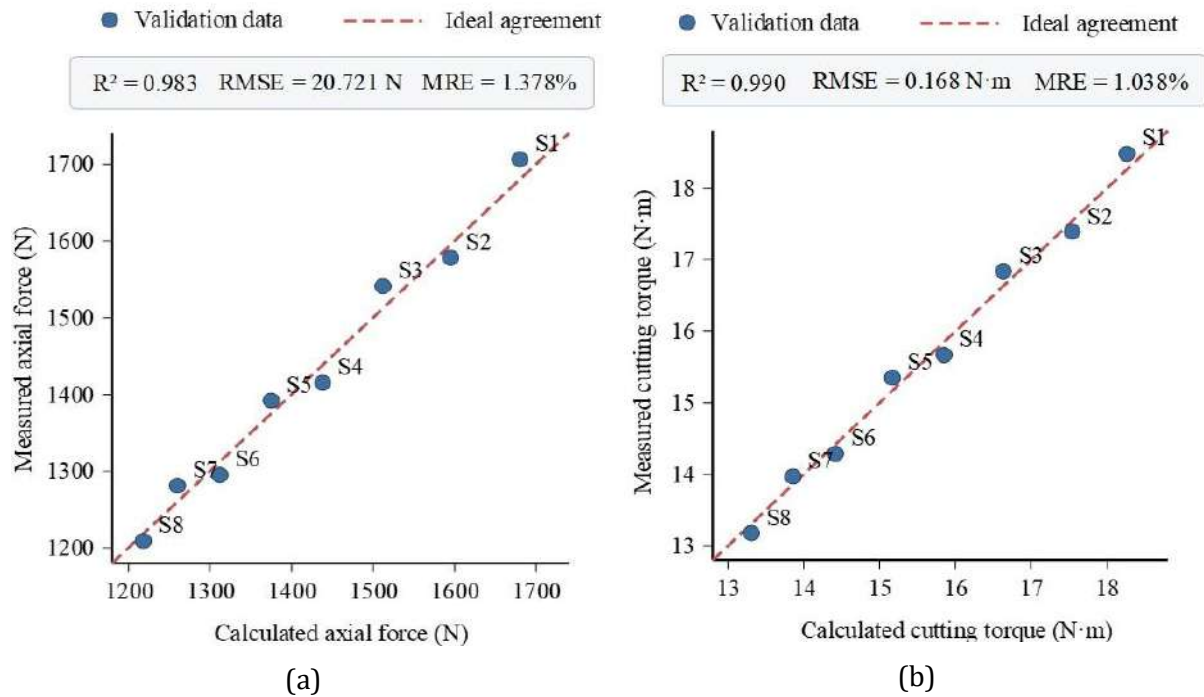


Fig. 7. Actual cutting verification results of the mechanical properties evaluation model (a) Axial force verification and (b) Cutting torque verification

In Figure 7 (a), the calculated values of the axial force were generally close to the ideal line of conformity with the verified values. The coefficient of determination, root mean square error, and mean relative error were 0.983, 20.721 N, and 1.378%, indicating that the model could well reflect the influence of changes in tool geometry parameters on the axial cutting load. In Figure 7 (b), the calculated value of the cutting torque was also highly consistent with the verified value, with a coefficient of determination of 0.990, root mean square error and average relative error of 0.168 N·m and 1.038%. It indicated that the model had a high predictive accuracy for torsional load changes. To sum up, the established mechanical performance evaluation model had good engineering reliability and could provide an effective basis for the subsequent multi-objective optimization of tool geometry parameters. The agreement obtained for both interior and near-boundary schemes indicates that the model maintains satisfactory prediction accuracy across the feasible parameter space.

3.2 SORD-RL-MOGWO Sensitivity Analysis of Key Parameters

To evaluate the influence of key parameter settings on optimization performance, sensitivity experiments were conducted using the algorithm configuration summarized in Table 3.

Table 3. Algorithm settings

Category	Item	Setting
Algorithm Settings	Population size	100
	Maximum function evaluations	10,000
	Number of independent runs	30
	Dual-archive capacity	150
	Reinforcement learning rate	0.2

In SORD-RL-MOGWO, the dual-archive capacity affects objective-space coverage, while the learning rate influences search stability. The tested ranges were determined through preliminary optimization trials and the scale of the current problem. Dual-archive capacities of 50–250 were selected at intervals of 50 to cover undersized, moderate, and relatively large archives with respect to the population size of 100. Capacities below 50 caused frequent deletion of non-dominated solutions, whereas capacities above 250 produced little improvement in Pareto-front coverage but

increased archive-management cost. Learning rates from 0.05 to 0.40 were examined to represent conservative to relatively rapid Q-value updates. Values below 0.05 resulted in slow strategy adaptation, while values above 0.40 caused noticeable oscillations in action selection during preliminary tests. Sensitivity and convergence analyses were conducted over 30 independent runs, as shown in Figure 8.

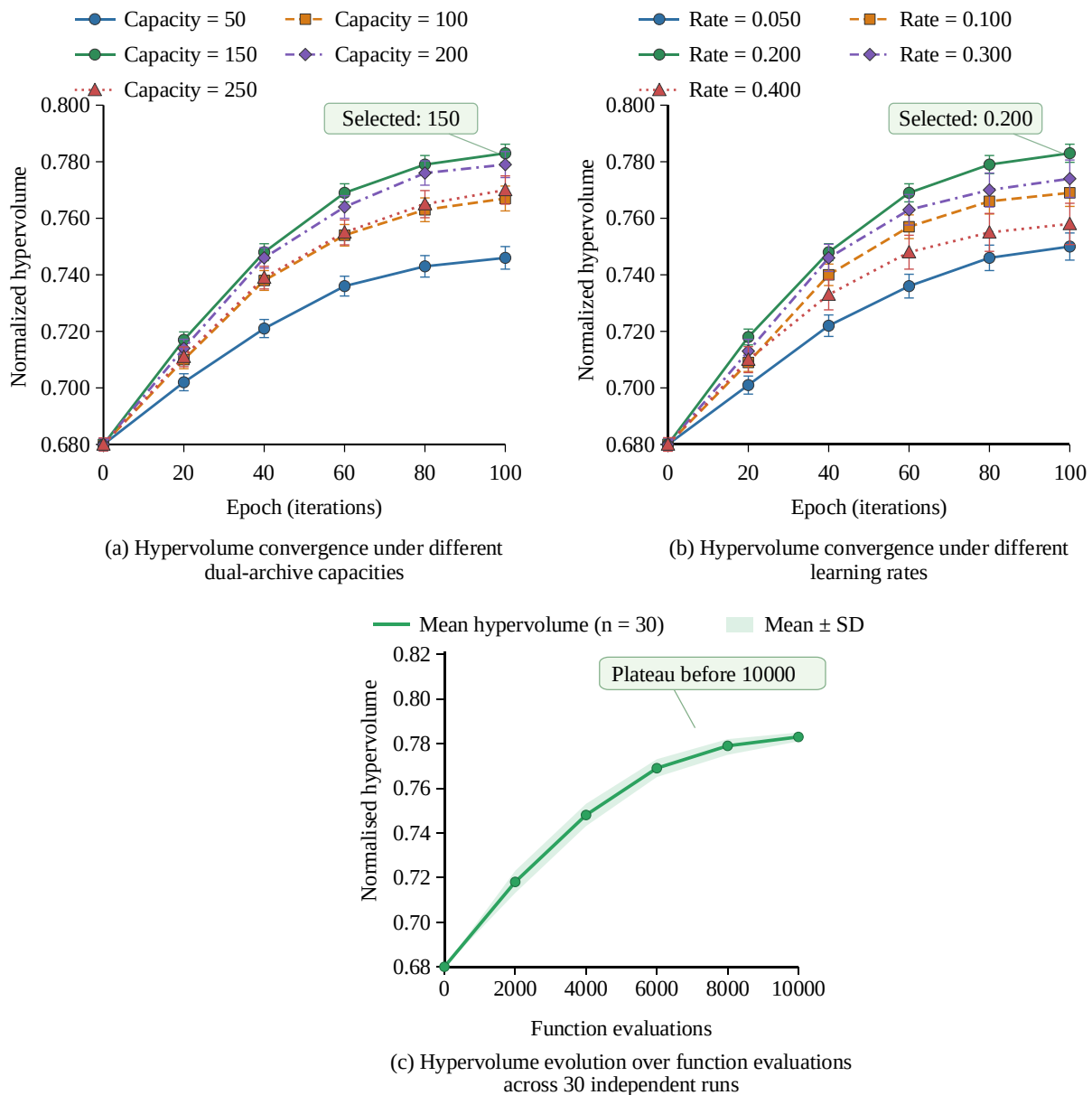


Fig. 8. Results of the sensitivity analysis for key parameters of SORD-RL-MOGWO

Figure 8 presents the sensitivity analysis results of the dual-archive capacity and learning rate. Each data point represents the mean hypervolume over 30 independent runs, and the error bars indicate the corresponding standard deviation. As shown in Figure 8(a), the hypervolume first increases with archive capacity and then tends to stabilize. A capacity of 150 achieves the highest mean hypervolume with relatively small dispersion and is therefore selected. As shown in Figure 8(b), the learning rate of 0.200 yields the best overall performance in terms of both convergence quality and stability. Although higher learning rates such as 0.300 and 0.400 can still improve the hypervolume, their error bars become larger in the middle and later stages of the search, indicating increased run-to-run fluctuation and reduced search stability. Therefore, 0.200 is adopted as the final learning rate. Figure 8(c) presents the mean hypervolume trajectory over 30 independent runs. The curve gradually reached a stable plateau before 10,000 function evaluations, and only limited improvement was observed during the final evaluation stage. Therefore, 10,000 function

evaluations were sufficient to balance convergence quality and computational cost for the six-variable optimization problem.

Table 4. Effect of dual-archive capacity on Pareto-front continuity

Archive capacity	Hypervolume	Spacing	Feasible solution ratio/% ↑	Occupied reference-vector regions (%)	Maximum normalized gap ↓	Extreme-solution retention (%)	Runtime (s)
50	0.742 ± 0.021	0.064 ± 0.009	91.82 ± 2.91	66.7 ± 4.8	0.124 ± 0.018	76.7 ± 6.1	18.62 ± 0.48
100	0.772 ± 0.015	0.045 ± 0.006	94.71 ± 2.08	84.1 ± 3.6	0.081 ± 0.011	91.1 ± 3.9	19.54 ± 0.53
150	0.783 ± 0.012	0.036 ± 0.005	95.93 ± 1.72	91.8 ± 2.7	0.059 ± 0.008	97.2 ± 2.1	20.68 ± 0.61
200	0.781 ± 0.013	0.035 ± 0.005	95.86 ± 1.79	92.5 ± 2.5	0.056 ± 0.007	97.9 ± 1.8	22.18 ± 0.70
250	0.779 ± 0.014	0.037 ± 0.006	95.72 ± 1.88	92.9 ± 2.6	0.055 ± 0.008	98.1 ± 1.7	23.86 ± 0.84

To assess whether an archive capacity of 150 is sufficient, five capacities were compared over 30 independent runs using Pareto-front continuity and convergence indicators, as shown in Table 4. Table 4 shows that an archive capacity of 50 is insufficient to maintain Pareto-front continuity, as indicated by a reference-vector occupancy of only 66.7%, a maximum normalized gap of 0.124, and an extreme-solution retention rate of 76.7%. Increasing the capacity to 150 raises the occupancy to 91.8%, reduces the maximum gap to 0.059, and retains 97.2% of the objective-wise extreme solutions. Although capacities of 200 and 250 provide marginal improvements in coverage and extreme-solution retention, their hypervolume does not increase, while runtime rises by 7.3% and 15.4%, respectively, relative to a capacity of 150. Therefore, a capacity of 150 is sufficient to preserve Pareto-front continuity and boundary solutions while maintaining favorable convergence and computational efficiency.

3.3 Comparison of Optimization Algorithm Performance and Ablation Analysis

The study further introduced three advanced constrained multi-objective optimization methods for comparison: the Constrained Multi-objective Evolutionary Algorithm with an Improved Two-Archive Strategy (C-TAEA2) [23], the Constrained Multi-objective Evolutionary Algorithm with Two-Stage Resource Allocation (CMOEA-TSRA) [24], and the Deep Reinforcement Learning-assisted Operator Selection embedded in Evolutionary Multitasking Constrained Multi-Objective Optimization (DRLOS-EMCMO) [25]. The Pareto-optimal solution sets generated by these algorithms were first compared, as shown in Figure 9.

In Figure 9 (a), the Pareto scheme obtained by SORD-RL-MOGWO was overall located on the lower left side of the contrast method. The axial force and cutting torque of its low-torque scheme were 1387 N and 13.180 Nm, which were lower than those of MOGWO, 1470 N and 13.490 Nm, it showed that the improved search mechanism helped to reduce the cutting load. Among the obtained Pareto solutions, the selected low-risk scheme achieved an axial force of 1387 N and a chip-removal risk value of 0.146. The corresponding point is highlighted in Figure 9(b) for visual identification. In Figure 9 (c), the study method achieved a lower chip removal risk with cutting torque close to that of DRLOS-EMCMO, demonstrating a more reasonable performance trade-off. Overall, SORD-RL-MOGWO could generate a set of tool geometry parameter schemes with better distribution and more reasonable compromise relationships.

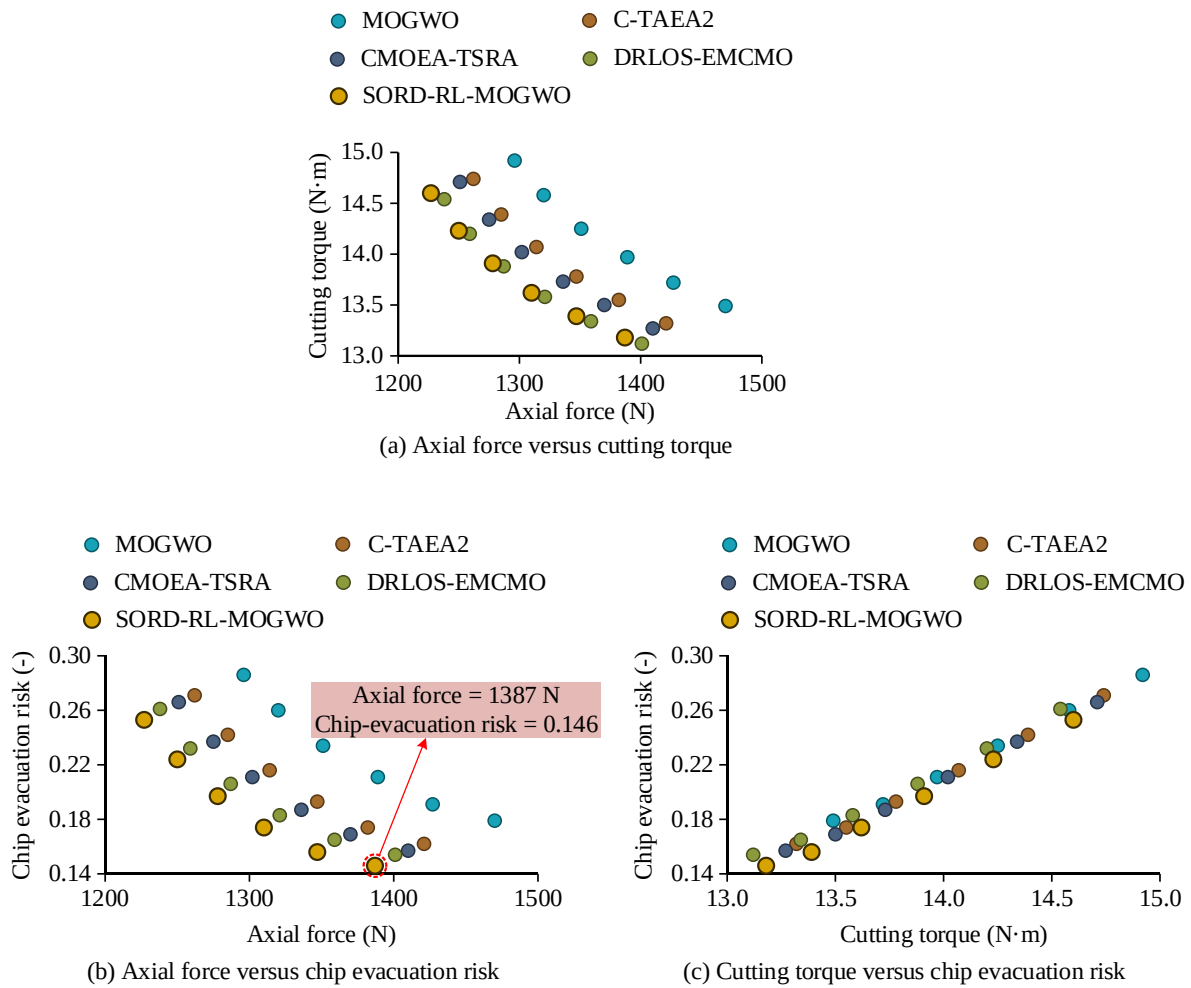


Fig. 9. Distribution of Pareto parameter schemes obtained using different multi-objective optimization algorithms

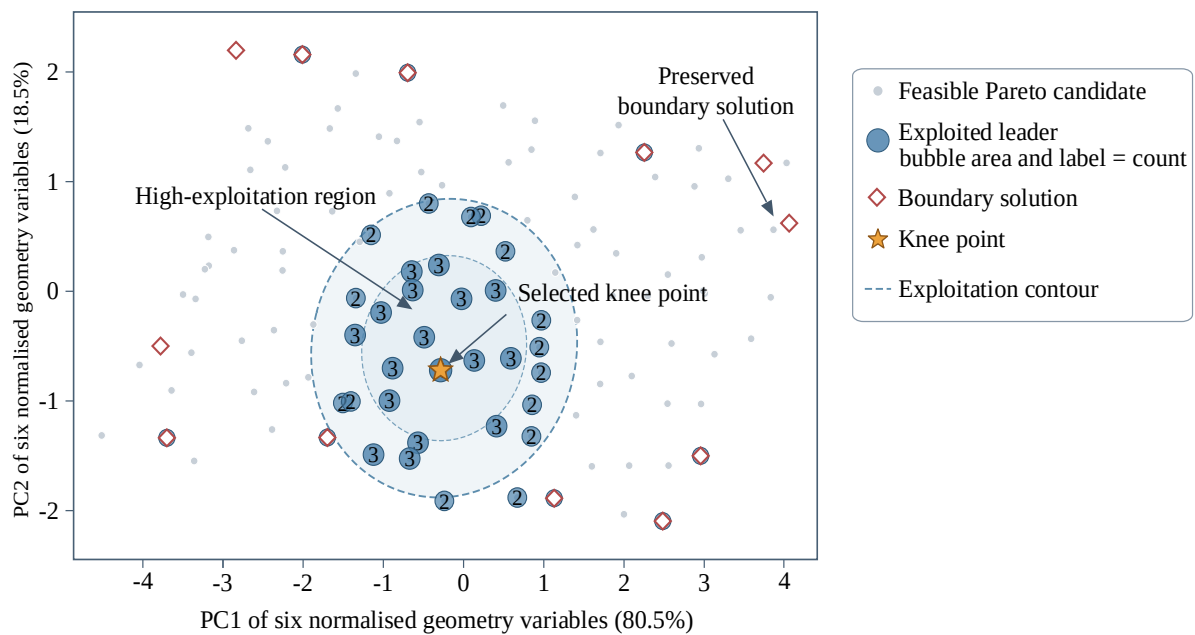


Fig. 10. Exploitation distribution before knee-point selection

Table 5. Performance comparison of different optimizers and ablated configurations of SORD-RL-MOGWO

Method	Hypervolume	Inverted generational distance	Spacing	Feasible solution ratio (%)	Running time (s)
MOGWO	0.689 ± 0.027 $p < 0.001$	0.091 ± 0.013 $p < 0.001$	0.074 ± 0.012 $p < 0.001$	83.73 ± 3.10 $p < 0.001$	14.33 ± 0.85 $p < 0.001$
C-TAEA2	0.754 ± 0.017 $p < 0.001$	0.058 ± 0.008 $p < 0.001$	0.043 ± 0.006 $p < 0.001$	92.27 ± 2.15 $p < 0.001$	19.14 ± 1.02 $p < 0.001$
CMOEA-TSRA	0.760 ± 0.015 $p < 0.001$	0.054 ± 0.007 $p < 0.001$	0.046 ± 0.007 $p < 0.001$	94.03 ± 1.96 $p < 0.001$	18.51 ± 0.96 $p < 0.001$
DRLOS-EMCMO	0.773 ± 0.013 $p = 0.008$	0.050 ± 0.006 $p = 0.039$	0.040 ± 0.006 $p = 0.005$	95.07 ± 1.81 $p = 0.085$	23.73 ± 1.26 $p < 0.001$
SORD-RL-MOGWO	0.783 ± 0.012 —	0.047 ± 0.006 —	0.036 ± 0.005 —	95.93 ± 1.72 —	20.68 ± 1.11 —
w/o SOI	0.759 ± 0.015 $p < 0.001$	0.056 ± 0.007 $p < 0.001$	0.044 ± 0.006 $p < 0.001$	92.87 ± 2.03 $p < 0.001$	19.96 ± 1.03 $p = 0.018$
w/o RVDA	0.731 ± 0.019 $p < 0.001$	0.069 ± 0.010 $p < 0.001$	0.061 ± 0.010 $p < 0.001$	89.60 ± 2.45 $p < 0.001$	17.89 ± 0.92 $p < 0.001$
w/o RL	0.747 ± 0.016 $p < 0.001$	0.061 ± 0.008 $p < 0.001$	0.049 ± 0.007 $p < 0.001$	92.07 ± 2.19 $p < 0.001$	18.42 ± 0.98 $p < 0.001$

Note: Values are reported as the mean $\pm$ standard deviation over 30 independent runs. The p -values were obtained using two-sided Wilcoxon rank-sum tests against SORD-RL-MOGWO and were adjusted using the Holm procedure for multiple comparisons. An adjusted p -value below 0.05 indicates a statistically significant difference. The symbols w/o SOI, w/o RVDA, and w/o RL denote the removal of Sobol–opposition initialization, reference-vector dual-archive management, and reinforcement learning, respectively.

To visualize the search preference before knee-point selection, Figure 10 presents the exploitation distribution of leader solutions in the PCA-projected design space. Figure 10 shows that most high-frequency leader selections are concentrated in the central feasible region, indicating that the reinforcement learning module preferentially exploits solutions that improve hypervolume. Meanwhile, several boundary solutions remain preserved by the dual-archive mechanism, maintaining Pareto-set diversity. The selected knee point lies within the main exploitation region and represents a balanced compromise among axial force, cutting torque, and chip-evacuation risk.

To comprehensively evaluate the solution performance of different optimization algorithms and the independent contributions of each improved module, the study compared SORD-RL-MOGWO with the comparison algorithms. The ablation configurations for removing Sobol - opposition learning initialization (w/o SOI), reference vector dual file management (w/o RVDA), and reinforcement learning policy conditioning (w/o RL) were constructed, and the results are shown in Table 5.

In Table 5, the hypervolume index of SORD-RL-MOGWO was 0.783 ± 0.012 , and the inverted generational distance, distribution uniformity, and feasible solution ratio were 0.047 ± 0.006 , 0.036 ± 0.005 , and $95.933 \pm 1.724\%$. The overall performance was superior to that of the comparison algorithms. Compared with MOGWO, its hypervolume index was increased by 13.64%, and the inverted generational distance and distribution uniformity were reduced by 48.352% and 51.351%, indicating that the improved strategy could enhance the convergence and distribution

quality of the tool parameter scheme. Compared with DRLOS-EMCMO, the study method achieved better quality of the solution set and proportion of feasible solutions while reducing the running time by 12.829%, showing better solution efficiency. The ablation results indicated that the performance degradation was most significant after the removal of the reference vector dual file management, suggesting that the mechanism played a key role in coordinating low load, low chip risk and structural feasibility. The removal of the remaining modules also led to a decline in metrics, verifying the synergistic effectiveness of the three improved strategies. Although SORD-RL-MOGWO contains multiple enhancement mechanisms, their computational overhead is limited. Sobol-opposition sampling is performed only during initialization, reference-vector dual-archive management mainly involves normalization, association, and archive truncation, and the tabular Q-learning module updates only a small Q-table. In contrast, DRLOS-EMCMO requires repeated neural-network inference and parameter updates for operator selection. Therefore, under the same hardware and function-evaluation budget, SORD-RL-MOGWO achieves a shorter running time of 20.68 s than DRLOS-EMCMO at 23.73 s. The Wilcoxon rank-sum test results show that SORD-RL-MOGWO differed significantly from MOGWO, C-TAEA2, CMOEA-TSRA, and all ablated configurations in hypervolume, inverted generational distance, and spacing, with adjusted p-values below 0.05. Compared with DRLOS-EMCMO, significant differences were obtained for hypervolume, inverted generational distance, and spacing, whereas the difference in feasible solution ratio was not statistically significant after Holm correction, with an adjusted p-value of 0.085. This result indicates that the main advantage of SORD-RL-MOGWO over DRLOS-EMCMO lies in Pareto-front convergence and distribution quality rather than a statistically conclusive increase in the feasible solution ratio.

3.4 Manufacturing-Tolerance Sensitivity Analysis

To evaluate the engineering robustness of the selected knee-point design, a manufacturing-tolerance sensitivity analysis was conducted by independently perturbing the angular parameters by $\pm 0.5^\circ$ and $\pm 1.0^\circ$ and the dimensional parameters by ± 0.01 mm and ± 0.02 mm, as summarized in Table 6.

Table 6 shows that the selected knee-point geometry remains relatively robust under normal manufacturing tolerances. A deviation of $\pm 0.5^\circ$ or ± 0.01 mm increases axial force by no more than 1.34% and chip-evacuation risk by no more than 1.89%. The edge radius is the most influential parameter for axial force, whereas the chip-breaker width and chip-groove guide angle have greater effects on chip-evacuation risk. Even under twice the normal tolerance, the maximum increases in axial force and chip-evacuation risk are limited to 2.71% and 4.72%, respectively.

Table 6. Manufacturing-tolerance sensitivity of the selected knee-point geometry

Parameter	Nominal value	Perturbation level	Worst-case deviation	Axial force (N)	Increase (%)	Chip-evacuation risk	Increase (%)
Knee-point design	—	0	—	1441.8 ± 10.6	0	0.212 ± 0.004	0
Rake angle	12.0°	±0.5°	−0.5°	1458.8 ± 11.0	1.18	0.213 ± 0.004	0.47
Parameter	Nominal value	Perturbation level	Worst-case deviation	Axial force (N)	Increase (%)	Chip-evacuation risk	Increase (%)
Rake angle	12.0°	±1.0°	−1.0°	1475.7 ± 11.7	2.35	0.215 ± 0.005	1.42
Clearance angle	8.0°	±0.5°	−0.5°	1454.5 ± 10.9	0.88	0.213 ± 0.004	0.47

Clearance angle	8.0°	±1.0°	-1.0°	1467.8 ± 11.5	1.8	0.214 ± 0.004	0.94
Point angle	118.0°	±0.5°	+0.5°	1455.8 ± 11.2	0.97	0.213 ± 0.004	0.47
Point angle	118.0°	±1.0°	+1.0°	1469.9 ± 11.6	1.95	0.214 ± 0.005	0.94
Edge radius	0.18 mm	±0.01 mm	+0.01 mm	1461.1 ± 11.6	1.34	0.214 ± 0.004	0.94
Edge radius	0.18 mm	±0.02 mm	+0.02 mm	1480.9 ± 12.4	2.71	0.217 ± 0.005	2.36
Chip-breaker width	1.20 mm	±0.01 mm	+0.01 mm	1448.7 ± 10.8	0.48	0.216 ± 0.005	1.89
Chip-breaker width	1.20 mm	±0.02 mm	+0.02 mm	1456.9 ± 11.3	1.05	0.221 ± 0.006	4.25
Chip-groove guide angle	24.0°	±0.5°	+0.5°	1449.3 ± 11.0	0.52	0.216 ± 0.005	1.89
Chip-groove guide angle	24.0°	±1.0°	+1.0°	1457.4 ± 11.8	1.08	0.222 ± 0.006	4.72

3.5 Actual Cutting Validation of Optimized Tools Under Multi-Machining Conditions

To evaluate the adaptability of the optimized tool geometries, cutting tests were conducted at feed rates of 0.040, 0.060, 0.080, and 0.100 mm/r, as shown in Figure 11. The tool denoted as “Original” in Figure 11 refers to the unoptimized baseline design established before geometric parameter optimization rather than a standard off-the-shelf industrial product. It used the same carbide insert material, tool-body structure, outer diameter, retained-core diameter, guide-pad arrangement, and internal-coolant configuration as the optimized tool. The two tools differed only in the six optimized geometric parameters, namely the rake angle, clearance angle, point angle, edge radius, chip-breaker width, and chip-groove guide angle. Because chip blockage is a binary event, its uncertainty was quantified using the 95% Wilson confidence interval. Differences between the original and optimized tools were evaluated using a two-sided Fisher’s exact test, with statistical significance defined at $p < 0.05$.

In Figure 11 (a), the axial force of the SORD-RL-MOGWO optimized tool under high feed conditions was 1814 N, which was 14.232% lower than that of the original tool, indicating that the optimized cutting-edge structure could effectively reduce the axial cutting load. In Figure 11 (b), the cutting torque of the tool was 19.480 N·m, which was 13.996% lower than that of the original tool, indicating that the optimization of the rake face and chip diversion structure helped to improve the chip flow state. Figure 11(c) shows that, at 0.040 mm/r, the optimized tool reduced axial force and cutting torque by approximately 12% and 9%, respectively, while the chip-blockage rate decreased from 5% to 0%. At 0.100 mm/r, chip blockage occurred in 11 of 20 trials for the original tool and 4 of 20 trials for the optimized tool. The corresponding blockage rates were 55.0% with a 95% confidence interval of 34.2%–74.2% and 20.0% with a 95% confidence interval of 8.1%–41.6%, respectively. The absolute reduction was 35.0 percentage points with a 95% confidence interval of 5.0–57.6 percentage points, and the difference reached statistical significance according to Fisher’s exact test, with $p = 0.048$.

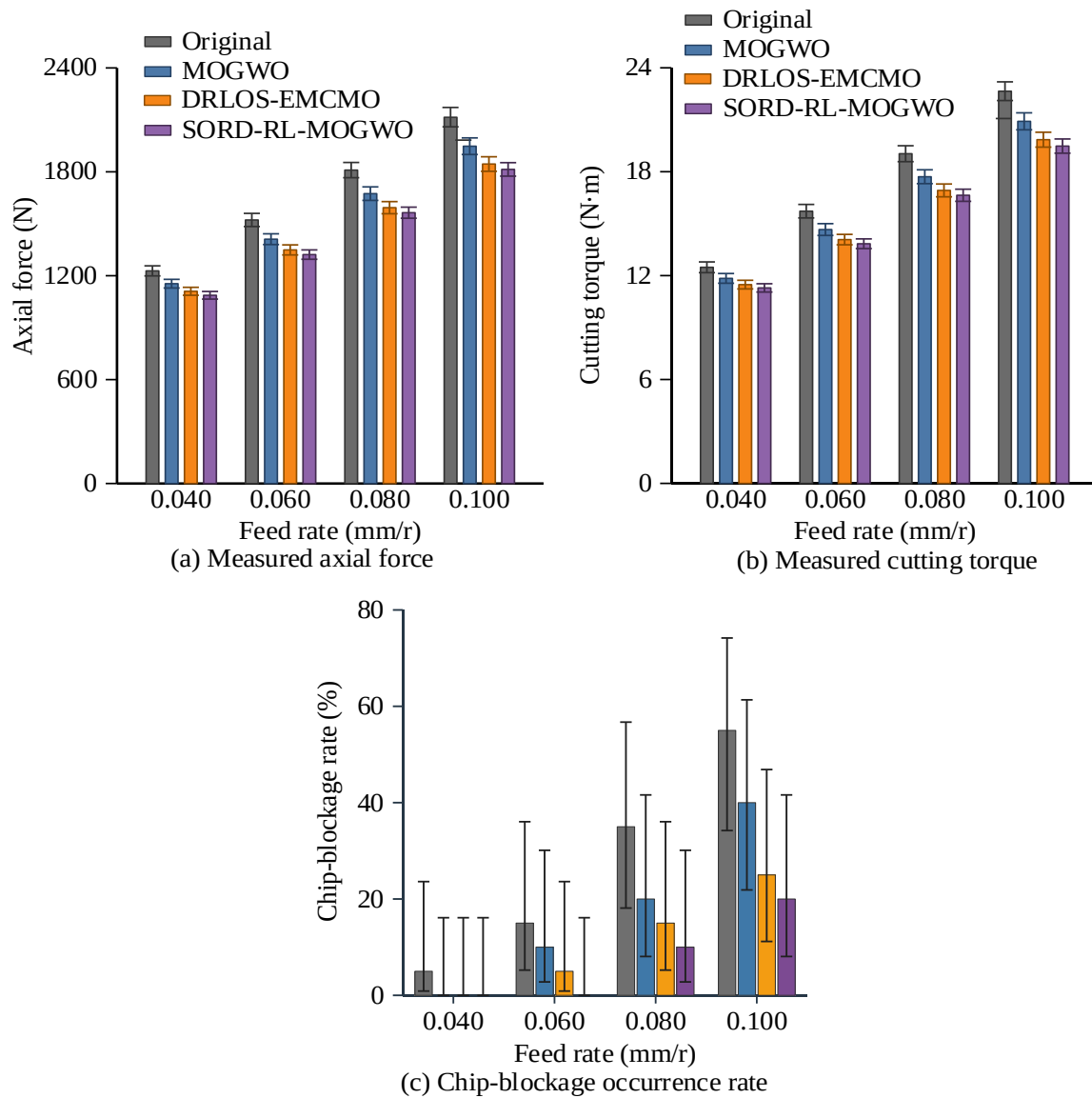


Fig. 11. Verification results of optimized tool cutting under different machining conditions (Error bars in (a) and (b): standard deviation over five repeated tests; error bars in (c): 95% Wilson confidence intervals over 20 trials.)

Table 7. Cutting-load fluctuations of the original and optimized tools

Feed rate (mm·r ⁻¹)	Tool	Mean axial force (N)	Axial- force signal SD (N)	Axial-force CV (%)	Mean torque (N·m)	Torque signal SD (N·m)	Torque CV (%)
0.04	Original	1268	58.3	4.6	14.18	0.57	4.02
0.04	Optimized	1116	35.7	3.2	12.91	0.37	2.87
0.06	Original	1564	84.5	5.4	17.03	0.79	4.64
0.06	Optimized	1372	49.4	3.6	15.34	0.49	3.19
0.08	Original	1847	116.4	6.3	19.86	1.05	5.29
0.08	Optimized	1602	67.3	4.2	17.49	0.65	3.72
0.1	Original	2115	150.2	7.1	22.65	1.36	6.00
0.1	Optimized	1814	81.6	4.5	19.48	0.80	4.11

To further evaluate the dynamic cutting stability of the optimized geometry, the fluctuation characteristics of the stable-stage load signals were analyzed at different feed rates. Entry transients were removed, and identical filtering parameters and sampling windows were applied to both tools. The signal standard deviation and coefficient of variation were calculated from five repeated tests under each condition, as shown in Table 7. Table 7 shows that the optimized tool consistently reduces both the mean load and signal fluctuation. At 0.100 mm/r, the axial-force standard deviation decreases from 150.2 N to 81.6 N, corresponding to a reduction of 45.7%, while the torque standard deviation decreases from 1.36 N·m to 0.80 N·m, a reduction of 41.2%. The axial-force coefficient of variation also decreases from 7.10% to 4.50%. These results indicate that the optimized geometry suppresses intermittent chip impact and load fluctuations, thereby improving cutting stability under high-feed conditions. However, its influence on tool life should be further verified through long-duration wear tests.

4. Conclusion

This study addressed the high cutting load, unstable chip evacuation, and coupled geometric design challenges of deep-hole trepanning tools by establishing a multi-objective optimization framework that simultaneously considers axial force, cutting torque, chip-evacuation risk, and structural feasibility. Beyond numerical performance, SORD-RL-MOGWO identified diverse and manufacturable solutions within the narrow feasible design space, enabling coordinated control of cutting load, chip evacuation, and structural safety. Its practical value lies in supporting geometry screening before physical cutting tests and providing alternative Pareto-optimal designs for different engineering requirements.

Experiments showed that the load evaluation model had determination coefficients of 0.983 for axial force and 0.990 for cutting torque. The hypervolume index of the complete algorithm was 0.783 ± 0.012 , the inverted generational distance was 0.047 ± 0.006 , and the proportion of feasible solutions was $95.933 \pm 1.724\%$. At a feed rate of 0.100 mm/r, the optimized tool reduced axial force and cutting torque by 14.232% and 13.996%, respectively, while the chip blockage rate decreased by 35 percentage points, from 55.0% to 20.0%. The substantial reduction in chip blockage can alleviate repeated chip impact, local heat accumulation, and abnormal load fluctuations, which is expected to reduce cutting-edge damage and unplanned interruptions, thereby improving tool service life and long-term process reliability in deep-hole trepanning. These results indicate that the model can reliably screen candidate geometries before physical cutting tests. By eliminating infeasible or high-load designs at the optimization stage, the proposed framework can reduce repeated trial machining, shorten the parameter-selection cycle, and limit unnecessary tool and material consumption in engineering applications.

The ablation results further show that reference-vector dual-archive management makes the largest contribution to Pareto-front quality among the three enhancement modules. Removing this mechanism reduced the hypervolume from 0.783 to 0.731, a decrease of 6.64%, and produced the most pronounced deterioration in convergence, distribution uniformity, and feasible-solution coverage. This finding confirms that coordinated management of the convergence and diversity archives is the key mechanism enabling SORD-RL-MOGWO to maintain well-distributed feasible solutions within the narrow and highly constrained design space.

Tool-wear evolution was excluded because the study focuses on initial geometry optimization and aims to isolate geometric effects from progressive edge degradation. Although the SORD-RL-MOGWO framework is generally applicable, transferring the model from GH4169 to stainless steels or titanium alloys requires recalibration of the cutting coefficients, chip-evacuation thresholds, and feasible geometric ranges. Future work will therefore include wear-dependent optimization, multi-batch and multi-material validation, temperature and coolant-flow measurements, and verification under higher-load cutting conditions.

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Cross-load generalization of bearing fault diagnosis via prototypical few-shot learning on vibration benchmarks

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Abstract

Limited labelled bearing-fault data are a practical challenge for EV gearbox condition monitoring, motivating diagnostic methods that can learn effectively from only a few examples. EV gearboxes also operate under varying speed and load conditions and may experience torque transients, which can alter vibration signatures and further challenge the generalization of diagnostic models trained with limited data. To investigate this problem in a controlled and reproducible setting, this study evaluates a Prototypical Network for few-shot bearing fault diagnosis using the publicly available CWRU vibration benchmark. A 4-way K-shot protocol is used with $K = 1, 3, 5$, and 10 , and a lightweight 1D-CNN is employed to extract robust embeddings while preventing overfitting on small support sets. In the pooled-load benchmark evaluation, ProtoNet achieves 98.87% accuracy in the 1-shot case, 99.80% in the 3-shot case, 99.58% in the 5-shot case, and 99.78% in the 10-shot case, outperforming SVM, KNN, and a supervised 1D-CNN under the same comparison protocol. Additional leave-one-load-out tests across all three CWRU load permutations and $K = 1, 3, 5$, and 10 maintain mean accuracies above 99.80% over 1,000 independent test episodes per condition. These results demonstrate strong few-shot performance and robustness to the limited load shifts represented by the CWRU benchmark, providing encouraging methodological evidence for the intended EV gearbox monitoring context. Because CWRU does not reproduce the full speed, transient-torque, noise, and installation variability of an EV gearbox, hardware-specific EV validation remains an important next step.

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1. Introduction

1.1 Background and Motivation

Drive train safety for electric vehicles (EVs) is becoming increasingly important as bearing failures inside gearboxes could start off as a minor vibration or noise and develop into catastrophic failure of the whole transmission. Recent reviews of EV fault diagnosis and automotive powertrain monitoring also identify vibration-based condition monitoring as a central route for drivetrain health assessment [1-4]. Current industry practice for drivetrain diagnostics is based on using accelerometer data and processing it through segmentation and feature extraction methods in order to identify impact signatures within the vibration signal that can indicate the presence of inner-ring (IR), ball (B) or outer-ring (OR) faults or healthy bearings. Much of the state-of-the-art in this area has been achieved through the use of deep learning, where models have obtained high classification accuracy on bearing fault diagnosis datasets. However, there is a significant problem of obtaining sufficient representative labelled vibration data to train such models for EV transmission gearboxes. Bearing faults in practice are random events that do not occur very often, which makes labelling sufficient data samples very time consuming and, in practice, it is unlikely

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that more than a few samples per fault type will be available for training. Further complicating the problem are the variable operating conditions under which EVs operate including load, speed, temperature and lubrication, which all affect how a bearing fault signal is propagated to the accelerometer attachment point. In addition, EV traction systems can operate over wide and high rotational-speed ranges, which broaden the frequency range of vibration components and increase the bandwidth and sampling-rate requirements for reliable vibration acquisition. The regenerative braking process also results in abrupt torque transients, leading to non-stationary vibration characteristics and possible distribution shifts between operating conditions. Such variations can reduce the generalization ability of diagnostic models trained under a limited set of conditions, particularly when only a few labelled samples are available for adaptation. To the best of the authors' knowledge, there is no publicly available, comprehensive vibration dataset specifically recorded from an EV gearbox for fault diagnosis. Even if a few samples are available for each fault class at one particular operating load, it is unlikely that enough samples would be available to train a model to generalize across loads. This paper therefore investigates few-shot bearing fault diagnosis and controlled cross-load generalization using an open vibration benchmark, motivated by the data-scarcity and operating-condition challenges encountered in EV gearbox monitoring.

1.2 Research Gaps and Challenges

Few-shot learning (FSL) typically trains a model on a small dataset and bypasses the need for a large labelled training set by learning a metric in some embedding space or network structure to recast recognition as a similarity comparison problem. Hence, FSL has been shown to be effective for bearing and gearbox diagnosis in recent literature using Prototypical and Siamese network architectures. By further employing transfer learning and pseudo-labelling, FSL can even be extended to cross-domain scenarios. Nevertheless, three methodological gaps remain relevant to data-efficient bearing diagnosis for potential EV gearbox monitoring.

First, existing few-shot learning studies have predominantly focused on generic rolling-element bearings or other industrial applications, with limited focus on EV-relevant operational profiles, particularly variations in load and speed. In the present study, CWRU is used as a controlled and reproducible vibration benchmark for evaluating the proposed few-shot learning framework under limited load shifts, rather than as a direct proxy for an EV-specific gearbox. Secondly, although standard evaluation protocols for the CWRU dataset exist, a standardized evaluation protocol specifically tailored for few-shot learning in bearing diagnostics-defining fixed episode construction and data splitting-is still lacking. Even for those papers that do utilize a fixed dataset and split, key hyperparameters-such as segment length, overlap, and K-are typically to be determined by the individual researcher, producing unreliable cross-study comparisons. Finally, models trained at single operating points may exhibit degraded generalization under changes in loading conditions, which vary continuously in real-world EV driving. In addition to linear running between high and low speed, EV gearbox bearings are subjected to start-stop cycling and part-load operation, meaning that single-load evaluation provides little guarantee of in-vehicle performance. Quantitative cross-load generalization performance therefore remains a relatively under researched area in few-shot learning for diagnostic applications. The framework outlined subsequently addresses these methodological gaps through a fixed open-benchmark protocol and explicit cross-load evaluation, while EV gearbox monitoring remains the motivating application for subsequent hardware-specific validation.

1.3 Objectives and Contributions

We seek a reproducible few-shot learning framework for bearing defect detection and explore its pooled-load and cross-load performance using data from CWRU. Specifically, we present a 1D-CNN Prototypical Network (1D-CNN ProtoNet) designed for the aforementioned segmented vibration time series data. For robust model evaluation, we perform 4-way K-shot learning using the CWRU drive-end (DE) bearing vibration data under 1, 2, and 3 HP load conditions, corresponding to approximately 1772, 1750, and 1730 RPM, respectively. We choose a 1024-point window with 50% overlap. We consider Normal and three levels of defect (IR, Ball, and OR) for classification. For the pooled-load evaluation, episodes are sampled from the available CWRU load conditions, with support and query samples kept separate within each episode. For the cross-load evaluation, the

model is trained using two load conditions and tested on the remaining unseen load without target-load fine-tuning. All three leave-one-load-out load permutations are evaluated. We use three supervised learning baselines: SVM, KNN and a supervised 1D-CNN. All experiments in the present study use CWRU as a controlled vibration benchmark rather than data recorded from an EV gearbox. Accordingly, the present work is positioned as a pilot benchmark study of the proposed few-shot learning framework, while its generalization to the wider speed, load, and transient operating conditions of real EV gearboxes requires further hardware-specific validation.

1.4 Paper Organization

Section 2 reviews the relevant state-of-the-art; Section 3 details the proposed 1D-CNN ProtoNet and few-shot learning framework; Section 4 describes the experimental setup; Section 5 presents and discusses the results, including cross-load robustness; and Section 6 concludes the study and outlines future work.

2. Related Work

2.1 Vibration-Based Fault Diagnosis

Impacts of bearing faults in rotating machineries are manifested in vibration signals. The frequency of such impacts is related to the location of faults, i.e., ball pass frequency inner (BPFI), ball pass frequency outer (BPFO) and ball spin frequency (BSF). The fault related frequency components are embedded or masked within the broadbands of vibration signals. Thus, time-frequency analysis and some statistical features are usually employed to improve the fault diagnosis performance. The short-time Fourier Transform (STFT) and wavelet transform are commonly used time-frequency transforms. Several features including Root Mean Square (RMS), kurtosis, spectral centroid, spectral bandwidth, spectral excess kurtosis, spectral kurtosis, Crest factor, and Shape factor are extracted from time-frequency transformed signals. Two of the most commonly used features, namely, RMS and kurtosis have been exploited widely. The energy of a signal band (in spectral domain) is proportional to its RMS value, and it correlates well with wear and overall loading on a machine. The kurtosis feature of a signal captures the Transient Impulses within the signal and is commonly used for incipient fault detection. Although these fixed features have been widely and successfully applied for bearing fault diagnosis, they suffer significant limitations. Beyond conventional feature-based methods, recent studies have also explored advanced signal-processing, simulation-driven learning, and digital-twin-based approaches for bearing and gear diagnostics. For example, variable-speed bearing diagnosis has been investigated through envelope-spectrum characteristic-frequency analysis [5], while simulation-driven machine learning [6] and digital-twin frameworks [7] have been applied to bearing fault classification and gear degradation prognosis. Large variations in operating conditions or in machine dynamics result in variations that are large enough to significantly degrade the performance of classifiers that are trained under a specific operating condition. Due to large variability of images of the same object, there is an increasing interest in data-driven approach capable of learning to recognize an object from very few examples. Deep learning methods drastically changed the rule of feature extraction in time domain. More recently, 1D-CNN, LSTM, TCN and other deep learning architectures have been applied to time-series fault diagnosis in rotating machinery and EV transmission bearings [8,9]. Features can be extracted directly from the amplitude of the raw signal and, unlike traditional methods, feature extraction is no longer a manually designed process. The entire sensing to diagnosis process of automotive powertrains can be reviewed in the light of order analysis techniques and the processing of time varying signals using time-frequency methods to tackle the non-stationary fault characteristics. In particular for EVs, data-driven approaches are becoming increasingly prominent for motor, battery and gearboxes etc. diagnostics. A crucial assumption of the above methods is that a large labelled dataset is available. While this can be achieved through real world EV driving, the EV test rigs can be utilized in the meantime to generate the required data. This may not be true on real EV gearboxes where faults are extremely rare and annotating such faults is very time consuming and expensive.

2.2 Few-Shot Learning for Fault Diagnosis

Foundational few-shot learning formulations include Siamese one-shot learning [10], model-agnostic meta-learning (MAML) [11], and Prototypical Networks [12]. In FSL, many classification methods can be viewed as searching in the embedding space, either fixed prototypes or Siamese networks learning embeddings of small support sets and then classifying a new query sample based on distance or pairwise scores. Recently, such methods have been applied to vibration-based diagnosis. The wavelet-prototypical method in [13] combines wavelet transforms with a prototypical network for few-shot gearbox detection and reports strong small-sample performance. Weighted prototype and anti-noise Siamese variants in [14] and [15] further show that bearing diagnosis can be handled under limited-data conditions. The hydrodynamic-transmission study in [16] extends few-shot diagnosis through transfer learning, while the pseudo-labeling framework in [17] uses time-frequency preprocessing for aero-engine bearings. Cross-domain few-shot diagnosis has also been studied through meta-learning and domain-adversarial graph modeling [18]. The review in [19] shows that metric-learning methods remain competitive at very small data sizes, whereas the industrial-machinery study in [20] compares preprocessing choices across MIMII, CWRU, and MFPT. Meta-transfer and MAML-style approaches further address operating-condition shifts [21,22]. Related motor and EV-powertrain studies include wide-kernel CNN/Siamese models for PMSM faults [23] and residual-current few-shot learning for EV inverter faults [24]. Data augmentation and multimodal fusion have been explored through GAN/CBAM rolling-bearing diagnosis [25] and MMFSL for industrial bearings [26]. Wavelet-prototypical fusion also confirms the usefulness of embedding-based few-shot learning on vibration signals [27]. Compared with wavelet-prototypical and anti-noise Siamese approaches, the present method adopts a lightweight 1D-CNN Prototypical Network operating directly on segmented vibration signals, without additional time-frequency preprocessing or dedicated anti-noise modules. However, systematic few-shot evaluation using a fixed open-benchmark protocol, particularly under cross-load conditions, remains limited in bearing fault diagnosis.

2.3 Open-Access Vibration Benchmarks

The experiments in this study use the publicly available CWRU bearing dataset, including drive-end vibration measurements, EDM-induced bearing faults, a 12 kHz sampling rate, and three motor load conditions. CWRU is retained because it provides a public and widely used vibration benchmark with controlled fault classes and operating conditions, allowing the proposed few-shot protocol to be evaluated reproducibly against established bearing-diagnosis studies. Nevertheless, recognized limitations and anomalies in the CWRU dataset have been documented in previous benchmark analyses [28]. Accordingly, the present study treats CWRU as a controlled methodological benchmark rather than as representative of the broader operating variability of an EV gearbox. Differences in dataset partitioning, preprocessing, and episodic construction make direct comparison across few-shot diagnostic studies difficult. However, due to the lack of a standardized evaluation protocol, it is generally not possible to perform a direct comparison between different approaches in the field, Liang et al. [19]. Using time-frequency prototypical network by Wang et al. on CWRU bearing data, wavelet-based frequency information improves the classification performance when only a small number of training samples are available, which makes this network a good starting point for a few-shot learning protocol. We utilize CWRU at three different motor loads (1, 2 and 3 HP), each having different shaft speeds (approximately 1772–1730 RPM). These operating speeds are determined by the CWRU benchmark and are used here as a controlled starting point for methodological evaluation; they do not represent the wider speed range encountered in real EV gearboxes. We test whether a model trained at one load can generalize to other loads, in addition to the existing fault severity levels and bearing fault types included in the CWRU dataset. We use a single experimental protocol throughout the work to enable reproducibility. This includes fixed window length, overlap, and number of ways for a fixed number of training cases (K-shot learning), as well as the pooled-load and leave-one-load-out cross-load evaluation settings.

2.4 Summary of Research Gaps

Despite substantial progress in vibration-based fault diagnosis and few-shot learning, several methodological issues remain in reproducible few-shot bearing diagnosis. First, existing studies employ different preprocessing procedures, episodic settings, and operating-condition partitions, which makes direct comparison across studies difficult. Existing few-shot learning studies have predominantly focused on generic bearing benchmarks, while systematic evaluation under changes in operating load remains limited. Secondly, open benchmarks have not been consistently utilized in the literature. For example, different window lengths, data split ratios and/or different episode construction methodologies have been used at different times for supposedly the same dataset (e.g. [19]). Thirdly, and most importantly, cross-load generalization has rarely been quantified. As mentioned earlier, EV gearboxes are subject to dynamic changes in load and speed, therefore explicit evaluations of train-on-one-load and test-on-another methodologies (e.g. [16, 20]) are required. The present study evaluates the proposed few-shot learning framework using a fixed protocol on the CWRU bearing benchmark, with EV gearbox monitoring retained as the motivating application context. Results were evaluated under both pooled-load and leave-one-load-out cross-load settings.

3. Methodology

3.1 Overall Framework

As shown in Figure 1, the architecture consists of standard few-shot learning stages to process raw CWRU vibration data: windowing and normalization of the time series. Following this, an episodic few-shot learning loop is implemented with N classes and K support samples per class. A single 1D-CNN is employed as a shared-weight encoder for both support and query samples, such that the same network parameters are used to embed all windowed and normalized time-series segments into the same embedding space. For each class, the mean embedding of the support samples is computed as the class prototype, and each query sample is classified according to its Euclidean distance to these prototypes. The use of an episodic framework as opposed to training on a large fixed dataset is due to the small number of labelled samples per fault type.

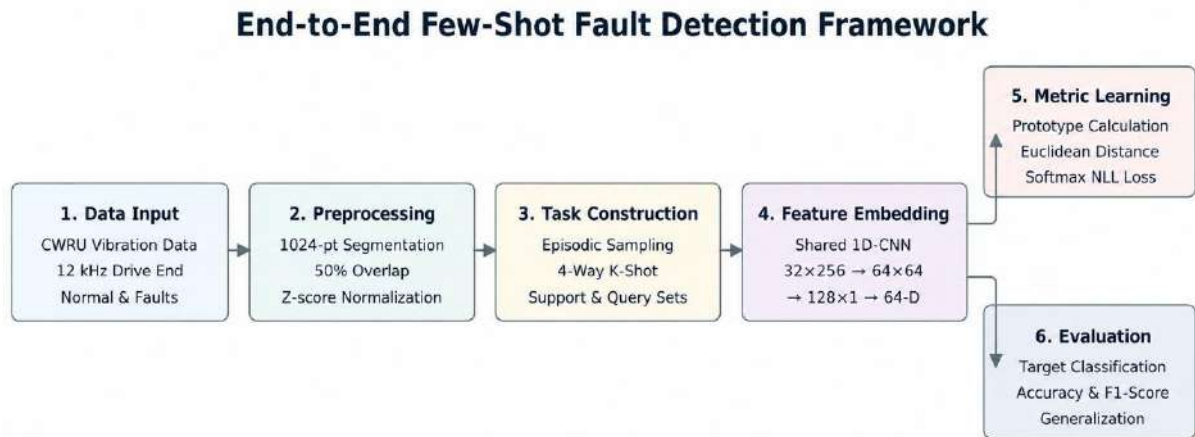


Fig. 1. End-to-end few-shot fault detection pipeline

In practice the CWRU segments are of a fixed window size with 50% overlap between successive segments. Hence all individual segments are first separately Z-score normalized, and then collectively treated as a single set of samples for training a given network. For training a given network, the same four classes are always used, K supports and query samples are drawn for each class, and then passed through the same 1D-CNN, after which prototypes for each class are computed and the negative log-likelihood loss is evaluated. After that Adam is used for back propagation to update the CNN's weights. The test accuracy and F1 score are then computed using the trained CNN according to the evaluation protocol specified in Section 3.3. The codebase for this

work consists entirely of Python, and only uses the following libraries: PyTorch for all of the neural network layers/functions, and scikit-learn for an SVM baseline as well as functions to calculate class weights for K. The functions to calculate the mean and standard deviation of time series samples for Z-score normalization are implemented using SciPy. All results were generated with a fixed random seed for reproducibility.

3.2 Signal Acquisition and Preprocessing

The CWRU Bearing Data Center provides drive-end bearing data for a test rig that includes a variable speed drive motor with a torque transducer, a dynamometer, and coupling to the test bearing trial. The bearing is an SKF 6205-2RS JEM. Three different defects were machined on the IR and OR bands and on the balls using EDM at 4 different diameters (0.007–0.028 in.). Three motor load conditions were considered: 1 HP at approximately 1772 RPM, 2 HP at approximately 1750 RPM, and 3 HP at approximately 1730 RPM. The DE accelerometer was sampled at 12 kHz. The bearing data are provided in the form of MATLAB files with the “.mat” extension. The relevant parameters are listed in Table 1.

Table 1. CWRU bearing test-rig technical parameters

Parameter	Specification
Sensor Type	Drive End (DE) accelerometer
Sampling Rate	12,000 Hz
Defect Introduction	Electrical discharge machining (EDM)
Fault Types	Inner Race (IR), Ball (B), Outer Race (OR)
Defect Diameters (inch)	0.007, 0.014, 0.021, 0.028
Motor Load (HP)/ Rotational Speed (RPM)	1/1772, 2/1750, 3/1730

For this work, 27 fault files and 3 normal time history files have been recorded. The normal files were recorded at three different loads, and the IR, Ball, and OR defects are recorded at three different loads each. All files are recorded on the DE transducer channel. All of the time history files are recorded at full length without any sampling down. The length of the time history files therefore varies, with the fault files being approximately 121k-123k samples (or ~10 s) long, and the normal files approximately 484k-486k samples (or ~40 s) long. In this work, all four defect diameters and all three loads are included, rather than studying diameter effects or load effects in this paper. Window lengths of 512, 1024, and 2048 points, corresponding to approximately 42.7, 85.3, and 170.7ms at the 12kHz sampling rate, were evaluated. A 1024-point window was selected as a compromise between capturing sufficient fault-related temporal information and retaining an adequate number of training segments. At a representative shaft speed of 1750 RPM, one revolution lasts approximately 34.3ms, so a 1024-point window spans approximately 2.5 shaft revolutions. For the CWRU drive-end bearing, the BPFI and BPFO are 5.4152 and 3.5848 times the shaft rotational frequency, corresponding to approximately 157.9 Hz and 104.6 Hz at 1750RPM. Thus, one 1024-point window contains approximately 13.5 BPFI periods and 8.9 BPFO periods, providing multiple fault-related impact opportunities within each input segment. The window steps by 512 points, which is roughly twice the number of segments that would be obtained with 0% overlap. After segments were Z-score normalized (mean subtracted and then divided by standard deviation) within files, the following segment counts per file were used (Table 2).

Table 2. Dataset segmentation summary

Class	Fault Type	Sample Segments	Percentage
0	Normal	2835	30.73%
1	Inner Race (IR) Fault	2841	30.79%
2	Rolling Element (Ball) Fault	2129	23.07%
3	Outer Race (OR) Fault	1422	15.41%

A total of 9,227 segments were produced and labeled for the total data set (2,835 Normal, 2,841 IR, 2,129 Ball, 1,422 OR) and numbered accordingly for classification (0-3). Figure 2 provides some

waveforms from the different fault classes. In the IR and Ball faults, there are strong periodic effects observed within the waveform, whereas the OR faults have effects that are more evenly spaced within the waveform. For segment class definition, this project followed CWRU's guidelines for segmenting roller bearing data so the research can be comparable with past studies.

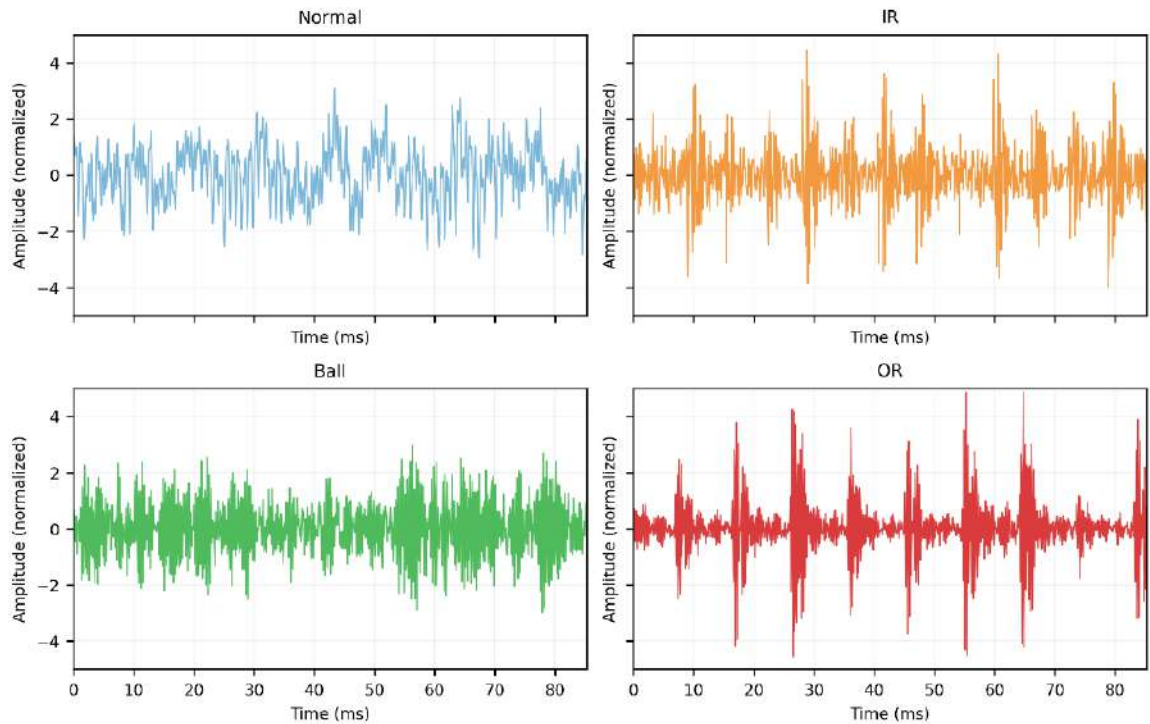


Fig. 2. Time-domain waveforms of Z-score-normalized vibration segments for the four classes

3.3 Few-Shot Task Construction

We fix $N=4$ (i.e., all four classes are in every episode) and vary K in $\{1, 3, 5, 10\}$. Increasing K from 1 to 10 significantly changes both the accuracy and stability of the network as it sees a support set of increasing size. $K=1$ is the most relevant scenario, given the data scarcity assumption of this work. For each class, we use an average of 15 queries per episode (resulting in 60 queries per episode). For each K -shot setting, the ProtoNet is trained using 200 episodically generated training tasks. For the pooled-load evaluation, performance is evaluated over 100 independently sampled test episodes. For the leave-one-load-out cross-load evaluation, each K -shot setting and load permutation is evaluated over 1000 independently sampled test episodes. Accuracy and macro-F1 are reported as the mean and standard deviation across the test episodes.

- $N = 4$: Normal, IR, Ball, OR appear in every episode.
- $K \in \{1, 3, 5, 10\}$: number of support samples per class.
- Query: 15 samples per class per episode.
- Training: 200 episodes; pooled-load testing: 100 episodes; added cross-load testing: 1,000 episodes per condition; results reported as mean $\pm$ std.

3.4 Model Architecture and Metric Learning

To learn segment embeddings, the network uses three sequential 1D convolutional blocks, followed by a fully connected projection layer to obtain the final embedding representation. Large feature maps from early layers capture high level local structure while later layers capture finer details. The first block utilizes a kernel size of 7 to incorporate a slightly longer local region of the signal, corresponding to approximately 0.58 ms at the 12 kHz sampling rate. The following two blocks have kernel sizes of 5 and 3 respectively. The first two convolutional blocks are followed by stride-4 max-pooling, reducing the temporal dimension from 1024 to 256 and then to 64, respectively. The third block is followed by adaptive average pooling, reducing the resulting feature map to 128×1 before the final 128-to-64 linear projection. The 1D-CNN therefore learns an

embedding of a 1024-point segment into a 64-dimensional vector. Step by step the capacity of the network is increased by powers of two, by increasing the number of filters from 32 to 64 to 128 dimensional filters, within three corresponding blocks of convolutional layers. After the third such block, adaptive average pooling is utilized in order to step down the temporal dimension to a single index, the corresponding channel. Then a linear projection from 128 to 64 dimensions follows, without any activation function. We then L2 normalize the 64-dimensional vector, which means that we keep the distances between the vectors scale consistent, which is important for prototype matching.

- Conv Block 1: 32 filters, kernel 7 (padding 3), BN, ReLU, max pool stride 4 $\rightarrow$ (32, 256).
- Conv Block 2: 64 filters, kernel 5 (padding 2), BN, ReLU, max pool stride 4 $\rightarrow$ (64, 64).
- Conv Block 3: 128 filters, kernel 3 (padding 1), BN, ReLU, adaptive avg pool $\rightarrow$ (128, 1).
- FC: 128 $\rightarrow$ 64, output L2-normalized.

Following the standard Prototypical Network formulation [12], the prototype of class n is computed as the mean of its K support embeddings c_n :

$$c_n = \frac{1}{K} \sum_{i=1}^K f_{\theta}(x_{n,i}) \quad (1)$$

A query embedding $f_{\theta}(\cdot)$ is assigned to the class whose prototype is nearest under Euclidean distance:

$$d(q, c_n) = \|q - c_n\|_2 \quad (2)$$

Euclidean distance is retained (rather than a learned metric or cosine) because, at $K = 1$, the prototype is simply the single support embedding and the assignment rule is unambiguous; for small K the mean already provides a low-variance estimate, and preliminary trials showed no benefit from more elaborate metrics. SoftMax over negative distances yields probabilities, and training minimizes NLL:

$$\mathcal{L} = -\log \frac{\exp[-d(q, c_y)]}{\sum_{n=1}^N \exp[-d(q, c_n)]} \quad (3)$$

with y the true class of the query.

3.5 Training Strategy and Implementation

The model is optimized with Adam (lr 0.001, $\beta_1=0.9$, $\beta_2=0.999$) with a StepLR scheduler that halves the learning rate every 50 training steps. For each model setting, training is performed over 200 episodically generated tasks. In each episode, support and query samples are independently sampled according to the predefined 4-way K -shot protocol, and previously generated episodes are not replayed. To ensure reproducibility, fixed random seeds are used for NumPy (42) and PyTorch (123), including the episodic sampling procedure. The code runs without errors with Python 3.10, PyTorch 2.2.2, scikit-learn 1.7.2, SciPy 1.15.3, NumPy 1.26.4. No proprietary software is required.

4. Experimental Setup

4.1 Benchmark Dataset and Partitioning Scheme

The training and evaluation procedures use the 9,227 segments described in Section 3.2. The four classes contain 2,835 Normal, 2,841 IR, 2,129 Ball, and 1,422 OR segments. Although these raw totals are imbalanced, each episode is constructed with equal numbers of samples from all four classes. For the pooled-load benchmark comparison, episodes are sampled from the three motor load levels (1, 2, and 3 HP), with support and query samples kept non-overlapping within each episode rather than using a fixed segment-level train/test ratio. For cross-load testing, a leave-one-load-out scheme is used: Loads 1+2 $\rightarrow$ 3, Loads 1+3 $\rightarrow$ 2, and Loads 2+3 $\rightarrow$ 1. The held-out load is not used for fine-tuning.

4.2 Baseline Methods

Three reference baselines are included to compare metric-based few-shot classification with conventional small-sample classifiers and a supervised neural-network baseline.

- SVM (RBF): $C=10$, $\gamma=\text{`scale}$. Trained per episode on the support set and evaluated on the query set.
- KNN: $K = \min(K_{shot}, 5)$ the support set is used as the reference, Euclidean nearest-neighbor classification. No training step is required.
- Supervised 1D-CNN: shares the same 1D-CNN backbone as the embedding network, with an added linear classifier; fine-tuned on the support set for 50 epochs per episode before evaluation on the query set [8]. This is used as a conventional small-sample supervised reference rather than as an episodically trained FSL method. No early stopping was used for this baseline.

SVM and KNN use the same 1024-point segments as ProtoNet. Fourteen fixed handcrafted features are extracted from each segment: mean, standard deviation, maximum, minimum, RMS, mean absolute value, crest factor, kurtosis, shape factor, skewness, spectral centroid, spectral energy, peak frequency, and spectral standard deviation. The feature set is kept fixed for all comparisons.

4.3 Evaluation Metrics

- Accuracy: mean $\pm$ std over independent test episodes (100 episodes for the pooled-load comparison and 1,000 for each added cross-load condition).
- Macro F1: average of the four per-class F1 scores. Because every episode contains equal query counts from all four classes, Macro F1 and Accuracy can be numerically very close at high classification accuracy.
- Confusion matrix: normalized, ProtoNet 5-shot, to reveal which fault pairs are most often confused.

4.4 Ablation and Sensitivity Analysis

All methods in the pooled-load comparison are evaluated at $K = 1, 3, 5$, and 10. The cross-load analysis is also evaluated at $K = 1, 3, 5$, and 10 for each of the three leave-one-load-out permutations. No additional architecture or dataset ablation is introduced in this study.

5. Results and Discussion

See Tables 3 and 4 for the Accuracy and macro F1 of all methods under the 4-way K-shot setting ($K = 1, 3, 5, 10$).

Table 3. 4-way K-shot classification accuracy comparison (mean $\pm$ std, 100 test episodes)

Method	1-shot	3-shot	5-shot	10-shot
ProtoNet	98.87 $\pm$ 2.04	99.80 $\pm$ 0.59	99.58 $\pm$ 0.79	99.78 $\pm$ 0.56
Supervised 1D-CNN	63.07 $\pm$ 7.75	77.42 $\pm$ 7.81	84.63 $\pm$ 6.71	91.48 $\pm$ 5.03
SVM	57.80 $\pm$ 7.93	67.03 $\pm$ 7.16	71.80 $\pm$ 6.40	78.17 $\pm$ 5.61
KNN	57.75 $\pm$ 7.97	62.13 $\pm$ 5.99	63.68 $\pm$ 7.66	73.45 $\pm$ 6.28

Table 4. 4-way K-shot macro-averaged F1 score comparison

Method	1-shot	3-shot	5-shot	10-shot
ProtoNet	98.87	99.80	99.58	99.78
Supervised 1D-CNN	63.12	77.44	84.64	91.48
SVM	57.83	67.05	71.80	78.16
KNN	57.77	62.10	63.61	73.45

ProtoNet achieves 98.87 $\pm$ 2.04% accuracy in the 1-shot setting, compared with 63.07 $\pm$ 7.75% for the supervised 1D-CNN, 57.80 $\pm$ 7.93% for SVM, and 57.75 $\pm$ 7.97% for KNN. At 3-shot, ProtoNet

reaches $99.80 \pm 0.59\%$. A small decrease to $99.58 \pm 0.79\%$ is observed at 5-shot, followed by $99.78 \pm 0.56\%$ at 10-shot; this small non-monotonic variation is treated as sampling variability rather than as a systematic trend. The strong low-shot performance is consistent with the metric-learning formulation: instead of fitting a new decision boundary from a very small support set, ProtoNet compares queries directly with class prototypes in a learned embedding space. The baselines improve as K increases, but remain below ProtoNet under the reported protocol. Figures 3 and 4 visualize the corresponding accuracy comparisons.

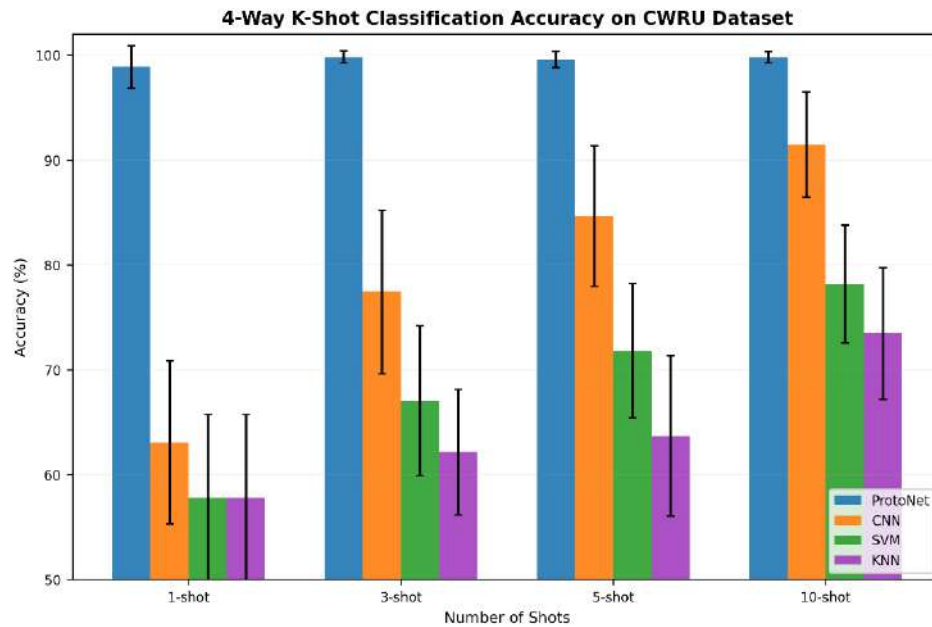


Fig. 3. Classification accuracy comparison bar chart for the four methods under 4-way K-shot

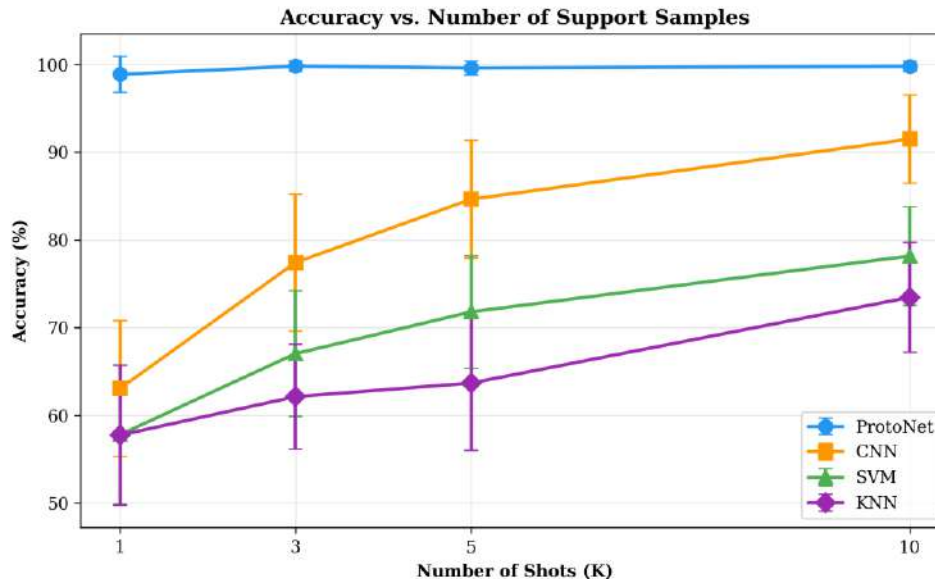


Fig. 4. Accuracy vs. number of support samples for each method

The pooled-load results are obtained on the CWRU benchmark and should therefore be interpreted as methodological evidence for the proposed few-shot bearing-classification framework. In the context of this study, they support the feasibility of a data-efficient diagnostic approach relevant to EV gearbox monitoring, while direct EV gearbox performance remains to be established using hardware-specific data.

5.1 Cross-Load Generalization Performance

Table 5 reports the added leave-one-load-out cross-load evaluation. The model is trained using two CWRU load levels and evaluated on the third held-out load, and all three permutations are tested at $K = 1, 3, 5$, and 10 . Across all three held-out-load configurations, mean accuracy remains above 99.80% for every tested K . For Loads 1+2→3, the accuracy ranges from 99.81% to 99.97%; Loads 1+3→2 yields 100.00% in the reported runs; and Loads 2+3→1 ranges from 99.96% to 99.98%. Macro-F1 values differ from Accuracy by less than 0.01 percentage points because each episode uses balanced query sets. These additional permutations show that the original cross-load observation is not unique to one particular held-out CWRU load.

Table 5. Cross-load generalization under leave-one-load-out evaluation (accuracy %, mean $\pm$ std over 1,000 test episodes)

Training loads	Unseen load	1-shot	3-shot	5-shot	10-shot
1 + 2	3	99.94 $\pm$ 0.32	99.81 $\pm$ 0.56	99.90 $\pm$ 0.40	99.97 $\pm$ 0.23
1 + 3	2	100.00 $\pm$ 0.00	100.00 $\pm$ 0.00	100.00 $\pm$ 0.00	100.00 $\pm$ 0.00
2 + 3	1	99.96 $\pm$ 0.24	99.97 $\pm$ 0.24	99.98 $\pm$ 0.20	99.98 $\pm$ 0.20

The three operating points nevertheless have relatively close shaft speeds and represent controlled laboratory loading. The results therefore support robustness to modest operating-condition shifts within CWRU and provide encouraging evidence for further evaluation under the wider speed, transient-torque, noise, temperature, and installation variability encountered in EV gearboxes. Quantitative extrapolation to those broader conditions would require additional hardware-specific validation.

5.2 Error Analysis

Figure 5 shows the normalized confusion matrix for ProtoNet at 5-shot in the pooled-load evaluation. The diagonal entries dominate, with only a small number of off-diagonal errors. The few Ball-IR errors may be associated with similarities in their transient vibration signatures; however, the present experiment does not establish a specific physical mechanism for these errors. Figure 6 provides a qualitative t-SNE visualization of the 64-dimensional embedding space. The visualization is used only to illustrate the learned representation and is not interpreted as quantitative evidence of K -dependent cluster evolution.

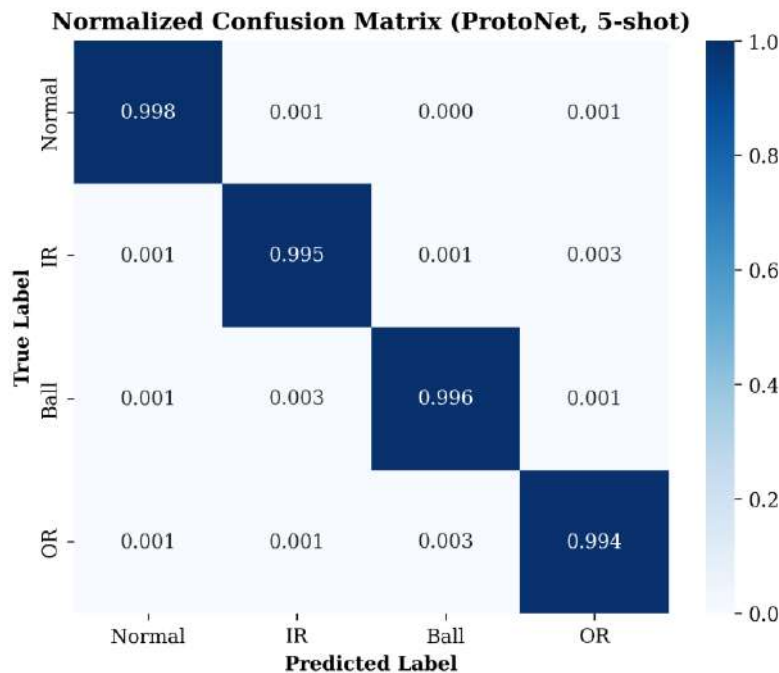


Fig. 5. ProtoNet 5-shot normalized confusion matrix

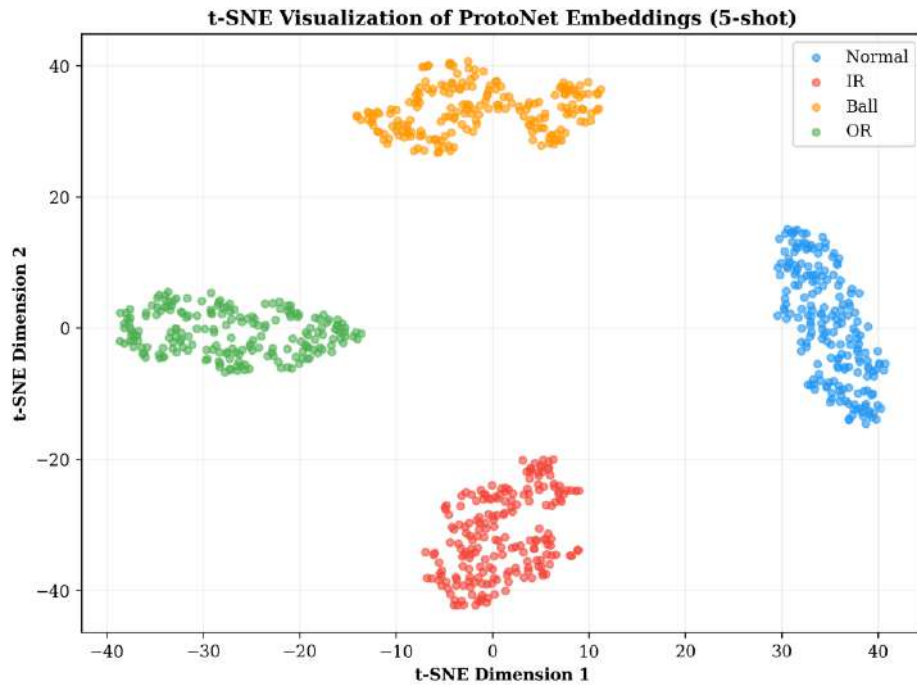


Fig. 6. ProtoNet 5-shot embedding-space t-SNE visualization

5.3 Engineering Implications

The results indicate that, within the controlled CWRU benchmark, the ProtoNet can achieve high classification accuracy with only a few labelled support segments per class. Performance changes little beyond approximately 3-5 shots in the pooled-load results, suggesting diminishing returns from additional support examples for this particular benchmark. This data-efficient behaviour is relevant to the limited-labelled-data motivation of EV gearbox monitoring, but it should not be interpreted as a complete field-deployment requirement. Real gearbox measurements include additional sources of variability, including road and structural noise, temperature, sensor placement, wider speed ranges, and transient loading. These factors define the next stage of hardware-specific validation and, where necessary, domain adaptation.

6. Conclusions and Future Work

6.1 Principal Findings

This study evaluates a 1D-CNN Prototypical Network for 4-way few-shot bearing fault diagnosis on the CWRU benchmark. In the pooled-load comparison, ProtoNet achieves $98.87 \pm 2.04\%$ at 1-shot, $99.80 \pm 0.59\%$ at 3-shot, $99.58 \pm 0.79\%$ at 5-shot, and $99.78 \pm 0.56\%$ at 10-shot, outperforming the reported SVM, KNN, and supervised 1D-CNN baselines. The additional leave-one-load-out evaluation covers all three CWRU load permutations and $K = 1, 3, 5$, and 10, with mean accuracy above 99.80% in every tested condition. Together, these results establish a reproducible benchmark-level demonstration of strong few-shot classification and robustness to the limited load shifts represented by CWRU, and provide encouraging methodological evidence for the intended EV gearbox monitoring context. Direct performance under the broader operating conditions of real EV gearboxes remains to be established through hardware-specific validation.

6.2 Practical Significance

The practical significance of the present result is its data efficiency under a controlled laboratory benchmark. The method requires only a small support set at inference, while the metric-learning representation avoids fitting a separate high-capacity classifier from those few examples. This characteristic is relevant to EV gearbox monitoring scenarios where labelled fault examples may be scarce. Before field application, however, the approach should be validated on hardware-specific data that include realistic speed variation, transient torque, noise, temperature, mounting, and

sensor-location effects. The present results therefore provide a methodological basis for subsequent EV gearbox validation rather than a claim of completed field deployment.

6.3 Limitations and Future Work

Three limitations should be emphasized. First, the experiments use only the CWRU bearing benchmark, whose load and speed variations are limited and whose laboratory fault generation does not represent the full diversity of field faults. Second, no vibration measurements from an EV gearbox are used in the present evaluation, so the influence of road noise, temperature, sensor placement, wide speed ranges, and torque transients remains to be quantified. Third, the present study evaluates only the Normal, IR, Ball, and OR bearing classes and a lightweight 1D-CNN embedding; it does not compare alternative datasets or more complex architectures. Future work will therefore extend the benchmark findings through additional bearing datasets and, importantly, dynamometer and in-service EV gearbox measurements.

Acknowledgements

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Modern approaches to adhesive debonding from smooth substrates with preservation of surface integrity: A review

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Abstract

Adhesive joining has displaced mechanical fastening in a growing share of aerospace, biomedical, electronic and architectural assemblies, mainly because bonded joints spread load more evenly, add little weight and tolerate dissimilar adherends. The reverse operation, separating the adhesive again without marking the part underneath, has attracted far less attention, and it becomes particularly awkward when the substrate is smooth: a polished metal coupon, a float-glass pane, a thinned silicon wafer or an optical-grade polymer presents a uniform, defect-free interface that offers none of the stress concentrators most debonding processes rely on. Removal procedures chosen without regard for the adherend leave micro-scratches, thermally altered layers, chemisorbed residues or dimensional drift, all of which shorten the service life of the recovered component. In this work, we systematically reviewed the literature on mechanical, thermal, chemical, electrochemical and hybrid debonding and assessed each family against the condition of the recovered substrate rather than against removal speed alone. Surface roughness, subsurface hardness, residual stress, chemical state and wettability were adopted as the assessment framework, and the available characterization techniques were compared by the depth of material each actually interrogates. Across the studies surveyed, laser and laser-chemical hybrid routes returned the closest match to the pre-bonding baseline, with roughness increments of roughly 0.05–0.10 μm on metals and composites and below 1 nm in wafer-level release, whereas mechanical scraping produced the largest degradation, up to about 2.3 μm . Chemical routes avoid mechanical damage but leave carbon-rich films that only surface-sensitive spectroscopy detects. We argue that the absence of an agreed post-debonding evaluation protocol, rather than any shortage of removal techniques, is now the main obstacle to progress in the field, and we indicate where responsive adhesives, inline verification and greener chemistries are most likely to develop.

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1. Introduction

1.1 From Animal Glue to Load-Bearing Joints

Joining by adhesion is older than metallurgy. Birch-bark tar held stone points to wooden shafts in the Middle Palaeolithic, and animal glues, casein and natural resins carried cabinetmaking, bookbinding and marquetry through to the industrial revolution. What changed in the twentieth century was the chemistry rather than the idea. Phenolic and phenolic-poly(vinyl formal) systems introduced during the 1940s made it possible, for the first time, to pass primary structural load through a bonded metal joint; epoxies followed within the decade and became the reference chemistry for structural bonding, a position they still hold [1]. Bonded doublers, metal-to-metal laminates and honeycomb panels were routine in airframes by the 1970s, and the later expansion

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of acrylics, polyurethanes, silicones and cyanoacrylates gave designers a chemistry for almost any combination of adherends [1, 2].

1.2 Industrial Importance of Adhesive Bonding

The commercial argument for bonding is straightforward. A bonded overlap transfers load through the whole joint area instead of through a few holes, so peak stresses fall, fatigue performance improves and the fastener holes that act as crack starters disappear [2, 3]. Because no melting is involved, materials that cannot be welded to each other, such as aluminium to carbon-fibre-reinforced polymer (CFRP) or glass to metal, can be joined without distortion [4, 5]. Multi-material light weighting in automotive body structures has become almost entirely dependent on this capability [4], and bonded flush repairs are now an accepted route for restoring damaged composite aerostructure [3, 6].

Bonding is equally embedded in sectors where the joint is never expected to be permanent. Semiconductor manufacturing relies on temporary adhesives to hold device wafers to rigid carriers during thinning and through-silicon-via processing, after which the wafer must be released without cracking or contaminating it [7, 8, 9]. Medical practice attaches dressings, sensors and ostomy appliances to skin that is often fragile [10, 11], and the surgical adhesives now entering clinical use raise the same removal question in a biological setting [12, 13]. Structural silicone carries the dead load of glazed curtain walls [14, 15], and ethylene-vinyl acetate (EVA) encapsulant laminates every crystalline-silicon photovoltaic module in service [16, 17].

1.3 Debonding: The Neglected Half of The Joint Life Cycle

Every one of those applications eventually requires the joint to be opened. The reasons are prosaic and recurrent: rework of misaligned assemblies, warranty repair, failure analysis, replacement of a degraded seal, and, increasingly, recovery of materials at end of life [18, 19, 20]. Regulatory pressure on recyclability has turned what was once a workshop nuisance into a design constraint, and a substantial literature on debondable and debonding-on-demand adhesives has grown up in response [21, 22].

Shop-floor practice, however, has moved much more slowly. Heat guns, chisels, oscillating wire, abrasive discs and solvent soaking remain the standard toolkit. They are cheap and universally available, but they are operator-dependent, essentially uninstrumented, and judged almost exclusively by whether the parts separate [19, 23, 24]. The adherend tends to be treated as a by-product of the operation rather than as the object being recovered. That distinction matters whenever the part is expensive, is to be rebonded, or has to meet an optical, sealing or fatigue specification after reassembly [25, 26].

1.4 Why Surface Integrity is the Correct Measure of Success

Surface integrity, in the sense established for machined components, is not a single number. It bundles together the topography of the outermost surface, the microstructure and hardness of the layer immediately beneath it, the residual stress state, and the chemical composition of the first few nanometers [27]. Each of these has an established engineering consequence, which is why we adopt it here as the axis along which debonding methods are compared.

Roughness is the most frequently reported parameter and the most frequently misused. An increase in R_a after debonding is not merely cosmetic: roughness peaks act as stress concentrators and shorten fatigue life, an effect quantified repeatedly for machined and additively manufactured metals [28, 29]. Roughness also changes the effective bonding area, so a substrate that has been scraped will not rebond with the same interfacial behaviour as the original [30, 31].

Microhardness and the associated subsurface deformation field record whether the removal process worked the material rather than merely the adhesive. Hardness depth profiles are the conventional way of revealing a work-hardened or thermally softened layer, and they respond to processing history well below the depth that a profilometer can see [27].

Residual stress is the parameter most often ignored in debonding studies and arguably the most consequential. Tensile residual stress introduced at the surface accelerates fatigue crack initiation

and stress-corrosion cracking, whereas a compressive state is protective; the relationship between surface roughness, residual stress and fatigue life has been established quantitatively for several alloy systems [28, 27, 29].

Corrosion resistance depends on the continuity of the passive oxide film. Any operation that abrades, thermally alters or chemically strips that film exposes fresh metal, and on aluminium and stainless steel the recovered joint can become the corrosion-critical location of the assembly [32, 33]. Optical performance is even less forgiving: in interferometric and photonic assemblies, surface figure changes of a few nanometers are already out of specification [34, 35]. In biomedical use the requirement shifts from topography to chemistry, since residual solvent or adhesive on a surface that will contact tissue is a contamination issue before it is a mechanical one [12, 13, 10].

1.5 Why Smooth Substrates Require Separate Treatment

We restrict this review to substrates whose as-received arithmetic mean roughness R_a is below approximately $0.8\text{ }\mu\text{m}$, the value conventionally associated with a fine-machined, ground or polished finish. The limit is a working convention rather than a physical constant, but it separates two genuinely different adhesion regimes.

On a deliberately roughened substrate, a significant part of the bond strength comes from mechanical interlocking: adhesive penetrates the relief left by grit blasting or laser texturing and is held by geometry as much as by chemistry. Laser-texturing studies show this directly, with lap-shear strength rising steeply as the processed roughness is increased over a controlled range [30, 31, 5]. Below roughly $0.8\text{ }\mu\text{m}$ the asperity relief becomes small compared with the adhesive layer thickness, real contact area approaches the nominal area, and adhesion is carried instead by wetting, adsorption and chemical interaction across an interface that is close to planar.

Three consequences follow, and together they justify treating smooth substrates as a distinct problem. First, the interfacial stress distribution is nearly uniform, so there is no natural crack starter; separation must be initiated artificially, which is precisely why smooth joints resist gentle methods and invite aggressive ones. Second, the tolerance for damage is far lower. A $2\text{ }\mu\text{m}$ scratch is invisible within the profile of a grit-blasted surface but is the dominant feature on a polished mirror, a float-glass pane or a thinned wafer. Third, damage on a smooth substrate is usually irreversible in practice, because restoring an optical or sealing finish means re-polishing, which removes material and alters dimensions. For these reasons the surface integrity criterion becomes binding on smooth substrates in a way it simply is not on rough ones.

1.6 Relation to Previous Reviews and Contribution of the Present Work

Debonding has been reviewed before, but from other vantage points. Srinivasan and Idapalapati surveyed debonding of adhesively bonded composite structures with sustainability as the organizing theme and with composites as the only substrate class [19]. Mulcahy and co-workers reviewed debondable adhesive chemistries in the context of recycling, treating the adhesive rather than the adherend as the object of study [18]. Blelloch et al. and Liu and Yan covered stimuli-responsive and switchable adhesion from a molecular-design perspective [21, 22]. Mo et al. reviewed temporary bonding and debonding specifically within advanced semiconductor packaging [7], and Ghahremani et al. confined their analysis to delamination of waste photovoltaic panels [23]. Laser cleaning has its own review literature, but it addresses coatings and contamination generally rather than adhesive joints [36, 32].

None of these takes the recovered substrate surface as the primary evaluation axis, and none cuts across substrate classes. The present review is intended to fill that gap, and it differs from its predecessors in five specific ways. It is substrate-centered rather than adhesive-centered and organizes evidence by what happens to the adherend. It covers five substrate families and five method families within a single comparative frame, so that a decision can be made for a given material rather than for a given chemistry. It arranges the characterization toolbox by information depth, which is the only way that roughness data, spectroscopic data and hardness data from different studies can be read together. It consolidates the scattered quantitative roughness

evidence into a single comparison. Finally, it is explicitly bounded to smooth substrates, where the surface integrity constraint is at its most severe.

The remainder of the paper is organized as follows. Section 2 classifies smooth substrates and their sensitivities. Section 3 summarizes adhesive chemistries and the debonding triggers each expose. Section 4 reviews the debonding methods themselves. Section 5 deals with characterization, Section 6 with comparative performance, Section 7 with industrial application, and Section 8 with the open problems we consider most limiting.

2. Classification of Smooth Substrates

For the purposes of this review, a smooth substrate is any solid surface presenting an as-received R_a below approximately $0.8\ \mu\text{m}$ together with good dimensional consistency and a reasonably well-defined surface energy. Classification matters in debonding work because composition, surface free energy, thermal conductivity and chemical reactivity between them decide which removal mechanisms are admissible and what the recovered surface will look like. The six classes below are ordered by how commonly they appear in the debonding literature. Their properties are compared in Table 1 and their relationships summarized in Figure 1.

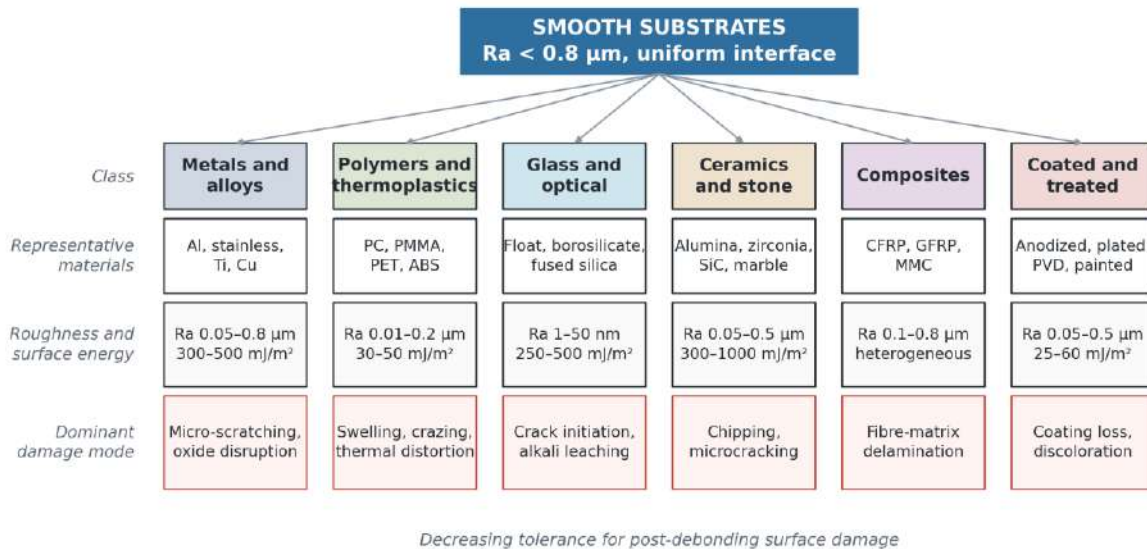


Fig. 1. Classification of smooth substrates used in this review, with the indicative as-received roughness range, surface free energy and dominant damage mode of each class

2.1 Metals and Alloys

Aluminium alloys, stainless steels, titanium alloys and copper form the most extensively studied group. Surface free energies in the range of roughly $300\text{--}500\ \text{mJ/m}^2$ on a cleaned, oxide-covered surface promote excellent wetting and strong adhesion, which is exactly what makes damage-free separation difficult [30, 31]. The characteristic surface integrity concerns are micro-scratching and plastic smearing from contact tools, disruption of the passive oxide, embedded abrasive or tool material, and retained organic contamination [37, 38]. Because metals conduct heat well, thermal methods spread energy away from the bondline efficiently, which is helpful for the substrate but often means the adhesive itself never reaches its softening temperature.

2.2 Polymers and Thermoplastics

Polycarbonate, poly(methyl methacrylate), poly(ethylene terephthalate), poly(vinyl chloride) and acrylonitrile-butadiene-styrene are used in electronic enclosures, optical components and medical device housings, usually with a molded or extruded finish that is already smooth. Their low surface free energy, typically $30\text{--}50\ \text{mJ/m}^2$, limits interfacial adhesion and makes controlled separation easier in principle [4]. In practice they are the most vulnerable class, because the same softening temperature and solvent sensitivity that make debonding possible also damage the substrate:

thermal distortion begins not far above the glass transition, polar solvents induce swelling, crazing and stress cracking, and the low hardness means any contact tool leaves a mark [16, 39].

2.3 Glass and Optical Substrates

Soda-lime float glass, borosilicate and fused silica are used in architectural glazing, display assemblies and photovoltaic front sheets. Fusion-drawn and float surfaces are among the smoothest engineering surfaces available, with R_a of the order of a nanometer and in all cases well below $0.05\ \mu\text{m}$, and a freshly cleaned, hydroxyl-terminated surface presents a high surface energy that silanol-coupled adhesives exploit efficiently [40, 15]. Two properties govern their debonding behaviour. Glass is brittle and has no plastic reserve, so any tensile or bending stress imposed during separation propagates from pre-existing surface flaws and initiates cracking rather than yielding [41]. Glass is also chemically alterable: strongly alkaline media leach network-modifying alkali ions and permanently change the surface composition, so chemical assistance has to be selected with the substrate and not only the adhesive in mind [42].

2.4 Ceramics and Stone

Alumina, zirconia and silicon carbide are used in cutting tools, dental prostheses and wear components, while marble and granite appear as architectural cladding bonded with structural adhesive. These materials combine very high hardness with low fracture toughness, and the combination is unhelpful: they resist abrasive damage better than metals do, but they respond to a locally applied mechanical load by chipping or by initiating a microcrack rather than by deforming [43]. Chemical methods must be screened against the specific composition, since acidic media attack glassy grain-boundary phases in sintered ceramics and carbonate-based stone dissolves outright.

2.5 Composite Materials

Carbon- and glass-fibre-reinforced polymers and metal-matrix composites dominate aerospace and high-performance automotive structures, and their tool-side surfaces are frequently smooth enough to fall within the scope of this review [3, 44]. Three features complicate their debonding. The surface energy is heterogeneous at the scale of the fibre spacing, so adhesion is not uniform. The mechanical response is anisotropic, and a load that is safe in the fibre direction may not be safe transverse to it. Most importantly, the resin-rich surface layer is thin, and removing it exposes fibre and initiates fibre-matrix delamination, a form of damage that is not detectable by roughness measurement and not repairable [19, 45].

2.6 Coated and Surface-Treated Substrates

Anodized aluminium, electroplated components, physical-vapor-deposited coatings and painted panels behave as composite surfaces in which the functional layer may be only a few micrometers thick. Debonding behaviour is set by the coating rather than by the bulk material, and the failure mode of concern is loss of the coating itself through delamination, discoloration or dissolution [46, 24]. Automotive glazing removal is the canonical example: the polyurethane bead must be cut away while the electrocoat and topcoat on the flange survive intact [4].

3. Adhesive Systems and Their Bonding Mechanisms

Choosing a debonding route without knowing the adhesive chemistry is guesswork. Composition, cure mechanism and thermal behaviour together decide what kind of interfacial bond exists and which stimulus can be used to break it, and they also decide what will be left behind [2, 1]. The classes below are summarized in Table 2.

3.1 Epoxy Adhesives

Epoxies remain the reference structural chemistry. The most common formulations are based on the diglyceryl ether of bisphenol A (DGEBA), cured irreversibly with amine or anhydride hardeners through opening of the terminal oxirane rings, which builds a three-dimensional covalent network whose crosslink density is set by the hardener and the cure schedule [47, 48]. Adhesion to a smooth metal arises from a combination of van der Waals interaction, hydrogen bonding to surface

hydroxyls and, on primed surfaces, covalent bonding; reported lap-shear strengths on prepared metallic substrates typically fall between about 15 and 45 MPa [1, 2]. The same network structure that gives epoxies their strength also explains their debonding behaviour: because the network is covalently continuous, a cured epoxy cannot dissolve, it can only swell, and thermal or chemical scission of the network is required before it will release. The DGEBA structure and its amine cure are shown in Figure 2.

Table 1. Comparison of smooth substrate classes relevant to adhesive debonding

Substrate class	Representative materials	As-received Ra	Surface free energy (mJ/m ²)	Typical applications	Dominant debonding risk	Damage susceptibility
Metals and alloys	Al alloys, stainless steel, Ti, Cu	0.05–0.8 µm	300–500	Airframe skins, housings, heat sinks	Micro-scratching, oxide disruption, embedded contamination	Moderate to high
Polymers and thermoplastics	PC, PMMA, PET, PVC, ABS	0.01–0.2 µm	30–50	Enclosures, optical parts, device housings	Solvent swelling and crazing, thermal distortion, abrasion	High
Glass and optical	Soda-lime float, borosilicate, fused silica	1–50 nm	250–500 (freshly cleaned)	Glazing, displays, PV front sheets	Crack initiation from surface flaws, alkali leaching, staining	Very high
Ceramics and stone	Alumina, zirconia, SiC, marble, granite	0.05–0.5 µm	300–1000	Cutting tools, dental prostheses, cladding	Chipping, microcracking, grain-boundary attack	High
Composites	CFRP, GFRP, metal-matrix composites	0.1–0.8 µm	Heterogeneous, 35–60 for matrix	Aerostructure, automotive body panels	Fibre-matrix delamination, loss of resin-rich layer	Very high
Coated and treated	Anodized Al, electroplate, PVD, paint	0.05–0.5 µm	25–60 (coating dependent)	Body flanges, decorative and functional coatings	Coating delamination, discoloration, dissolution	High

Values are indicative ranges compiled from the sources cited in Section 2 and are intended for comparison between classes, not as specification data.

3.2 Acrylic and Methacrylate Adhesives

Two distinct families share this heading. Pressure-sensitive adhesives develop adhesion by viscoelastic flow into contact under light pressure, holding through van der Waals interaction and hydrogen bonding without any chemical reaction; their bond strength is rate- and temperature-dependent and is normally reported as a peel force per unit width rather than as a shear stress [49]. Reactive acrylic and methacrylate systems polymerize by a free-radical mechanism to give tough networks of lower crosslink density than epoxies. That lower density is significant for debonding, because it leaves the network accessible to solvent penetration and, in lightly crosslinked grades, to genuine dissolution [2].

3.3 Polyurethane Adhesives

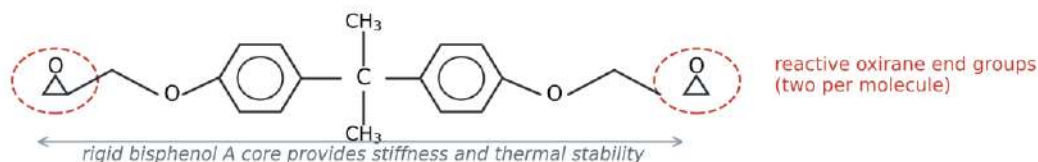
Polyurethanes are chosen where flexibility, impact resistance and accommodation of differential thermal expansion matter, which is why they dominate windscreen bonding and construction sealing [4]. Bonding proceeds through reaction of isocyanate groups with polyols and with adsorbed surface moisture, giving urethane and urea linkages and, on hydroxylated substrates, true covalent interfacial bonds. Thermoplastic grades soften on heating, and this reversibility is the basis of the heated-wire cutting used in automotive glass replacement [50].

3.4 Silicone Adhesives and Sealants

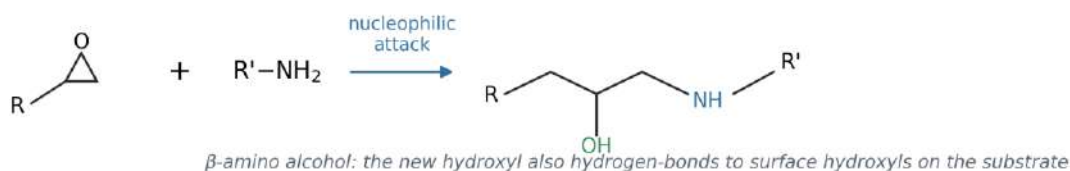
Structural silicones are built on a polydimethylsiloxane backbone and cure by condensation or by addition at ambient temperature. The Si-O-Si chain is unusually flexible and the glass transition lies

near -120°C , which is why these materials remain elastomeric over the whole service range of a building facade and why elongation at break well above 100% is normal for structural glazing grades [14, 40]. Adhesion is predominantly physical, through dispersive interaction, and is supplemented by covalent coupling where organ silane primers have been used. Silicones are the most difficult class to remove completely: their very low surface energy means that the last molecular layers wet the substrate strongly and are not displaced by solvents, so a low-energy siloxane residue typically survives cleaning and prevents subsequent rebonding [14, 51].

(a) Diglycidyl ether of bisphenol A (DGEBA) prepolymer



(b) Ring-opening of the oxirane by a primary amine hardener



(c) Crosslinked network and its consequence for debonding

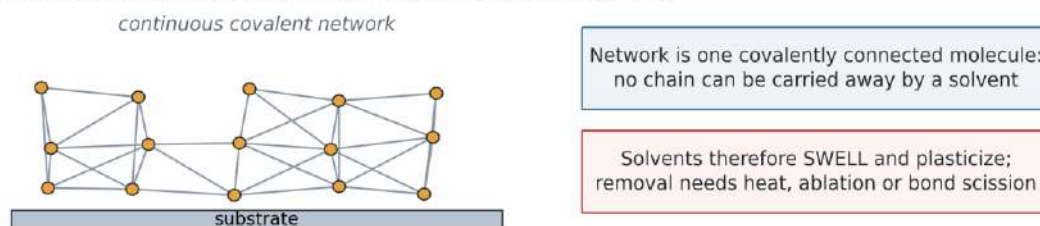


Fig. 2. Chemistry of DGEBA-based epoxy adhesives: (a) the DGEBA prepolymer; (b) ring-opening of the terminal oxirane by a primary amine; (c) build-up of the crosslinked network, whose covalent continuity prevents dissolution and restricts solvent action to swelling

3.5 Cyanoacrylate Adhesives

Cyanoacrylates polymerize anionic ally within seconds, initiated by the water film that is present on any surface exposed to ambient humidity, and form a rigid, essentially linear poly(alkyl cyanoacrylate). Because the polymer is linear rather than crosslinked, it is susceptible to polar aprotic solvents, and acetone, dimethyl sulfoxide and nitromethane dissolve rather than merely swell it. This is the one adhesive class for which true solvent dissolution, as opposed to swelling, is a practical debonding route [2].

3.6 Hot-Melt Adhesives

Hot melts are thermoplastics applied as a melt that develop adhesion purely on solidification, with no chemical reaction involved. Their reversibility is complete in principle: heating above the softening point restores the melt and the joint opens with minimal force [9]. They are consequently the easiest class to debond and the least demanding on the substrate, provided the substrate itself tolerates the temperature.

3.7 Bonding Mechanisms at A Smooth Interface

Five mechanisms are generally recognized, and their relative weight shifts markedly when the substrate is smooth. Mechanical interlocking, which dominates on textured surfaces, is reduced to whatever nanoscale relief remains and contributes comparatively little [30, 31]. Adsorption and wetting, governed by the thermodynamic work of adhesion, become correspondingly more important, as does chemical bonding through primers and coupling agents, which produces the strongest and most durable interfaces [37]. Electrostatic interaction and interdiffusion play a part at polymer-polymer interfaces but are minor at polymer-metal and polymer-glass interfaces [4]. The practical implication for this review is that on smooth substrates the bond is carried by short-range interactions distributed uniformly across the contact area, which is precisely the configuration that resists crack initiation. Figure 3 summarizes these mechanisms.

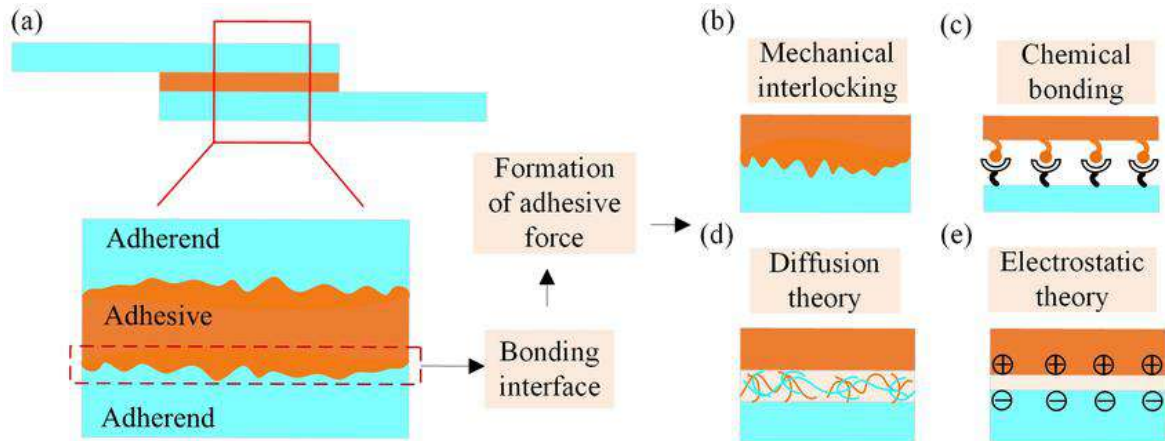


Fig. 3. Bonding mechanisms operating at an adhesive-substrate interface

The corresponding question, and the one that organizes Section 4, is what physical process can be introduced to defeat each mechanism. Figure 4 sets out the three stimuli that recur throughout the debonding literature: thermal relaxation of the polymer network above its glass transition, penetration and swelling of the network by a matched solvent, and electrochemical generation of gas at the interface itself.

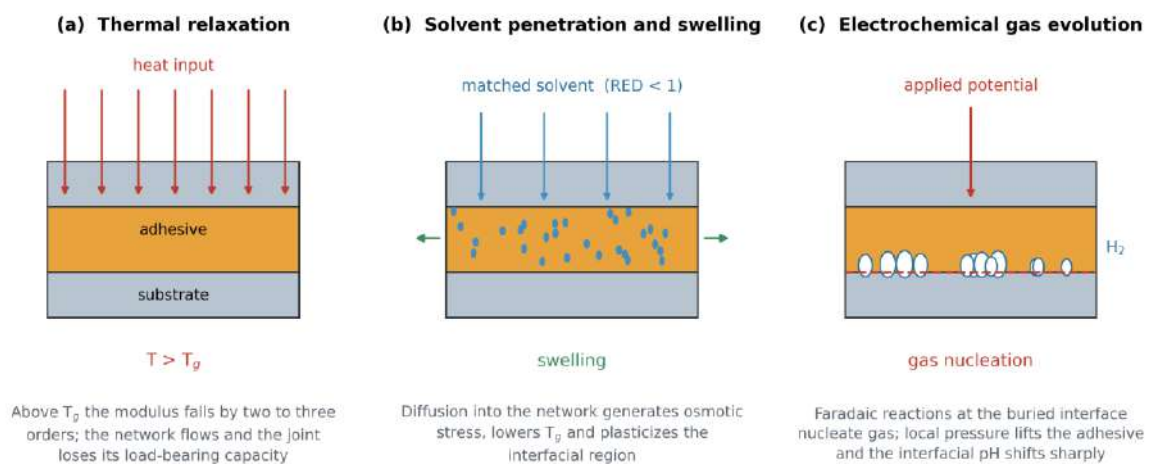


Fig. 4. Principal debonding stimuli at a smooth interface: (a) thermal relaxation, in which heating above the glass transition collapses the modulus and the load-bearing capacity of the network; (b) solvent penetration and swelling, in which diffusion into the network generates internal osmotic stress and plasticizes the interfacial region; (c) electrochemical gas evolution, in which faradaic reactions at the buried interface nucleate hydrogen or oxygen bubbles that lift the adhesive locally

Table 2. Adhesive classes, bonding mechanisms and the debonding triggers each expose

Adhesive class	Bonding and cure mechanism	Indicative strength on smooth substrate	Practical debonding trigger	Residual surface effect
Epoxy (DGEBA-amine)	Irreversible thermoset network; van der Waals, hydrogen and covalent bonding	15–45 MPa lap shear	Heating above T_g , laser ablation, strong alkali or acid; solvents swell only	Thin crosslinked residue; carbon-rich film detectable by XPS
Reactive acrylic / methacrylate	Free-radical chain growth; moderate crosslink density	10–30 MPa lap shear	Polar solvents, moderate heat, ultrasonic assistance	Partly dissolved and swollen residue
Pressure-sensitive acrylic	Viscoelastic contact; van der Waals and hydrogen bonding	0.2–1.5 N/mm peel	Peel angle control, cooling below T_g , solvent	Tackifier and plasticizer transfer film
Polyurethane	Isocyanate-polyol and isocyanate-water reaction; urethane and urea linkages	8–25 MPa lap shear	Thermal softening of thermoplastic grades, heated-wire cutting, alkaline hydrolysis	Soft residue; possible staining of coated substrates
Silicone (RTV and structural)	Condensation or addition cure of PDMS; dispersive interaction plus silane coupling	0.7–2 MPa tensile (structural glazing)	Mechanical cutting with chemical softening; thermally activated debonding-on-demand grades	Low-energy siloxane residue; the hardest class to remove completely
Cyanoacrylate	Anionic polymerization initiated by adsorbed water; linear polymer	15–25 MPa lap shear	Acetone, DMSO, nitromethane; heating above about 150 °C	Brittle flakes; genuine dissolution possible
Hot melt	Physical solidification of a thermoplastic melt; no reaction	1–8 MPa lap shear	Heating above the softening point	Thermoplastic film, readily wiped away

Strength figures are indicative values reported for well-prepared smooth substrates and vary strongly with surface preparation, bondline thickness and test geometry.

4. Debonding Methods

The methods described below differ in operating principle, in the process parameters that control them, in the range of substrates they suit, and above all in what they leave behind. Figure 5 places them in a common framework. We have kept the conventional division into mechanical, thermal, chemical, electrochemical and hybrid families, with one change relative to earlier treatments: cryogenic debonding is discussed under thermal methods, since it operates through the same mechanism of imposed temperature change and differential thermal strain, merely in the opposite direction.

4.1 Mechanical Debonding

4.1.1 Controlled Peeling

Peeling applies a tensile force at a defined angle to the interface and concentrates the resulting stress into a narrow peel front, which then advances along the bondline. The peel angle is the controlling variable. At angles approaching 90 degrees the loading at the front is predominantly

tensile, separation is progressive and substrate deformation is small; at low angles a substantial shear component is superimposed, and on soft or coated substrates this produces micro-tearing and smearing rather than clean separation [49]. Reported peel forces fall steeply as the peel angle is increased from a few degrees towards 90 degrees, after which the dependence weakens considerably, so operating near or above 90 degrees is advantageous both for the force required and for the damage produced [49]. Peeling is only available where at least one adherend is flexible enough to be bent to the peel radius, which excludes glass, ceramics and most thick metallic sections.

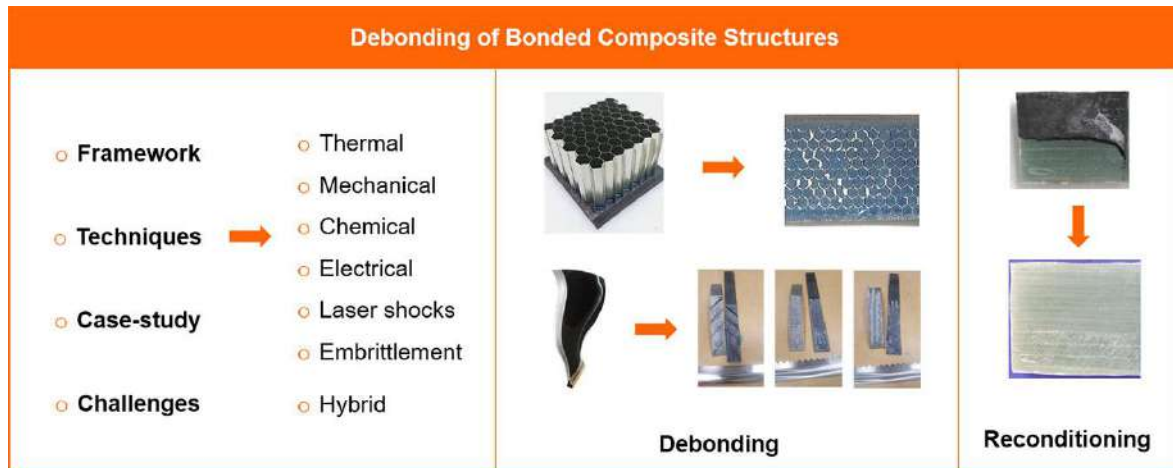


Fig. 5. Debonding techniques in adhesive joints. Adapted from [19]

4.1.2 Scraping and Abrasion

Scraping is the default workshop method and the worst performer in surface integrity terms. Material is removed by combined shear and compression under a hand-held or powered tool, and the damage produced scales with the hardness differential between tool and substrate, the applied normal load, the sliding velocity and the edge geometry of the tool. Where quantified, the differences between tools are large. In dental work, which is one of the few fields in which post-removal roughness is measured routinely, the four common protocols for removing residual orthodontic adhesive from enamel produced significantly different final roughness, with rotary carbide instruments leaving the roughest surfaces and finishing sequences recovering only part of the original condition [52, 43]. The same ordering, if not the same absolute values, holds for engineering substrates: across the studies surveyed here, mechanical scraping raised R_a by roughly 0.3–2.3 μm , an order of magnitude more than any non-contact method [19, 23]. Substituting polymer or wooden tools for steel reduces but does not eliminate the effect, because the abrasive is often the detached adhesive itself, which carries entrained hard particles.

4.1.3 Ultrasonic Vibration Debonding

Ultrasonic methods, generally operated between 20 and 40 kHz, are the most effective mechanical route on smooth substrates and deserve more attention than they have received. Four mechanisms act together, and separating them is important because they respond to different process parameters [53].

The first is acoustic cavitation, which occurs when the process is carried out in a liquid. The negative pressure half-cycle nucleates vapor cavities that collapse asymmetrically near the solid boundary, producing microjets and local shock loading directed at the interface. Cavitation is what makes ultrasonic cleaning effective at removing adherent films, and in a debonding context it delivers energy to the bondline without any tool touching the substrate [39]. The second is cyclic shear: the acoustic field imposes an alternating shear strain across the adhesive layer, and because polymer adhesives are viscoelastic, repeated cycling accumulates damage at stress amplitudes well below the static strength, so the joint fails by a fatigue mechanism rather than by overload [53, 54]. The third is localized heating: the loss modulus of the adhesive converts part of the vibrational energy to heat directly within the bondline, and because the adhesive is a far poorer conductor than

the metal or glass adherend, the temperature rise is concentrated exactly where it is needed while the substrate stays close to ambient [53, 55]. The fourth is resonance. When the excitation frequency is matched to a thickness or layer resonance of the assembly, the displacement amplitude at the interface is amplified substantially for the same input power, and layer-resonance-assisted approaches have been demonstrated for metal plate assemblies [56].

Reported process times range from seconds to a few minutes depending on adhesive class, bonded area and delivered power, and roughness increments as low as about 0.2 μm have been reported on smooth metallic and composite substrates, since no tool contacts the surface [53]. The practical limitations are that brittle substrates can be cracked by resonant amplitudes, and that the acoustic field is difficult to keep uniform over large bonded areas.

4.1.4 Pneumatic and Hydraulic Pressure Methods

Fluid pressure introduced through micro-injection nozzles distributes the separating load over the whole bonded area instead of concentrating it at a tool tip. The approach is useful in large-area assemblies where peeling is impractical, and it minimizes solid contact with the substrate, but it requires an accessible edge or an injection port and generally leaves the adhesive itself in place, so a cleaning step is still needed [6].

4.2 Thermal Debonding

4.2.1 Convective and Conductive Heating

Convective hot air systems and infrared sources operating between about 150 and 350 $^{\circ}\text{C}$ are standard for thermoplastic and hot-melt adhesives. The method is inexpensive and needs no special adhesive formulation, but heat is delivered to the outside of the assembly and has to conduct inwards to the bondline, so the substrate necessarily sees a higher temperature than the adhesive does. On polymers this is the dominant risk, and without good thermal control distortion and discoloration occur before the adhesive softens [9, 4].

4.2.2 Laser Debonding

Laser debonding is the most precise route available and, on the evidence assembled in Section 6, the best performer in surface integrity terms. Its advantage comes from selective absorption: energy is deposited where the optical properties of the material place it, not where the heat happens to conduct. Three wavelength regimes are used and they operate by genuinely different physics, a distinction that is often blurred in the applied literature. The process is illustrated in Figure 6.

Ultraviolet excimer sources, typically KrF at 248 nm or ArF at 193 nm, deliver photons whose energy exceeds the dissociation energy of the carbon-carbon and carbon-oxygen bonds in the polymer backbone. Ablation is therefore largely photochemical: bonds are broken directly and the fragments are ejected before the absorbed energy has time to diffuse as heat, which leaves a very shallow heat-affected zone. This is the basis of laser release through transparent glass carriers in wafer-level packaging, where the absorbing layer is engineered to be a few hundred nanometers thick and the silicon behind it is essentially untouched [7, 57, 42]. Near-infrared Nd:YAG sources at 1064 nm, by contrast, act photothermally: absorption raises the temperature of the adhesive and of any filler within it, and separation follows from thermal expansion, decomposition and the resulting stress wave [58, 59]. Carbon dioxide lasers at 10.6 μm are strongly absorbed by most organic polymers and by glass, which makes them efficient at removing thick organic layers but unsuitable where a glass substrate must survive, since the substrate absorbs as readily as the adhesive.

The surface integrity consequences follow directly from this division. Ultraviolet processing gives the smallest thermal alteration but photochemically oxidizes the exposed surface, which shows up as a marked increase in hydrophilicity rather than as a change in topography. Photothermal processing leaves the topography almost unchanged provided the fluence stays below the substrate ablation threshold, but it can generate a thin recast or oxidized layer. Systematic laser cleaning studies on aluminium alloy show exactly this threshold behaviour, with the substrate

essentially unaffected below a critical fluence and roughness rising once it is exceeded [36, 60, 32]. Post-debonding roughness comparable to the pre-bonding baseline has been demonstrated for composite, glass and semiconductor wafer applications, and in wafer release the recovered surface is held below 1 nm Ra [57, 58].

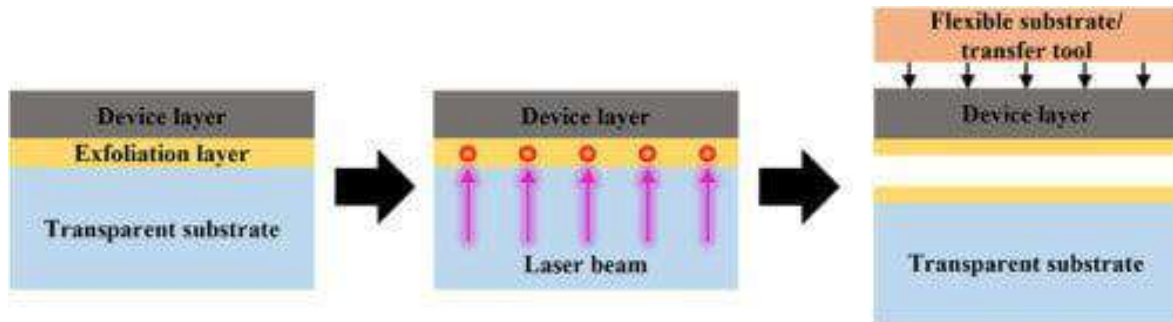


Fig. 6. Schematic of the laser debonding process through a transparent carrier

4.2.3 Induction Heating and Susceptor Fillers

Induction heating generates heat inside the joint rather than outside it, either by eddy currents in a conductive adherend or by hysteresis and relaxation losses in ferromagnetic or superparamagnetic particles dispersed in the adhesive itself. Debonding within 60 seconds has been reported on aluminium alloy substrates with negligible roughness increase, and magnetically responsive nanocomposite interlayers have been demonstrated as a designed debonding-on-demand route for laminated composites [46, 61]. Related concepts use resistive heating of an embedded conductive reinforcement to the same end [62], or expandable graphite fillers that generate separating stress as they intercalate [63].

The reviewer-relevant question is what these fillers cost in joint performance and in surface cleanliness, and the literature is candid about it. Filler loading generally reduces lap-shear strength relative to the unmodified adhesive, because the particles interrupt the continuity of the polymer network and introduce stress concentrations; the loading therefore has to be optimized against the heating rate required [46, 61]. After debonding, the particles do not evaporate. They remain in the residue, and iron oxide or graphite particles adhering to a recovered metal surface constitute both a contamination problem and, in the case of iron on aluminium, a galvanic corrosion risk. Complete removal requires a cleaning step whose own aggressiveness must then be assessed, which is why we regard filler-based routes as well suited to designed disassembly and poorly suited to recovery of optical or electronic surfaces [63].

4.2.4 Cryogenic Debonding

Cooling the assembly with liquid nitrogen or solid carbon dioxide exploits the same physics in reverse. Polymer adhesives have thermal expansion coefficients roughly one order of magnitude larger than metals and two orders larger than glass or silicon, so cooling generates a differential contraction strain that appears as interfacial shear. At the same time the adhesive passes below its glass transition and becomes brittle, so the joint loses the capacity to accommodate that strain by flow. Joints have been reported to separate spontaneously or under very low manual force, without reagents, heat or tool contact [50]. The approach is attractive for temperature-sensitive assemblies and for clean-room use, and the behaviour of adhesives at cryogenic temperature is well documented in the photonic packaging literature [35, 34, 64]. Its limits are equally clear: assemblies with a large thermal expansion mismatch between the adherends themselves may be damaged by the cooling, brittle substrates can crack under thermal shock if the cooling rate is not controlled, and condensation on rewarming introduces a moisture problem of its own.

4.3 Chemical Debonding

4.3.1 Solvent Action: Swelling is not Dissolution

The most common conceptual error in the applied literature is to treat solvent debonding as dissolution. It is worth stating the distinction plainly, because it determines what a solvent can and

cannot achieve. A crosslinked thermoset is a single covalently connected molecule spanning the whole bondline. No solvent can dissolve it, because dissolution requires individual chains to be separated and carried away, and here there are no individual chains. What a well-matched solvent does instead is diffuse into the network and swell it. Swelling generates internal stress, plasticizes the polymer, drops the glass transition temperature and reduces the modulus, and it is this softening, together with the osmotic stress developed at the interface, that opens the joint [65, 39]. Cured epoxies, cured polyurethanes and silicones fall in this category and swell only. Linear or lightly crosslinked polymers, notably poly(alkyl cyanoacrylate), many pressure-sensitive acrylics and hot melts, genuinely dissolve. The practical consequence is that a solvent process designed for a cyanoacrylate joint will disappoint on an epoxy joint no matter how long the immersion, and that solvent treatment of a thermoset should be understood as a softening pretreatment rather than as a removal step in its own right.

Which solvent to choose is not a matter of trial and error. Hansen solubility parameter (HSP) theory splits the cohesive energy density of a liquid into three components: a dispersion term δD from London forces, a polar term δP from permanent dipole interaction, and a hydrogen bonding term δH [66]. Each solvent and each polymer occupy a point in this three-dimensional space, and interaction is predicted by the distance between them,

$$Ra^2 = 4(\delta D1 - \delta D2)^2 + (\delta P1 - \delta P2)^2 + (\delta H1 - \delta H2)^2 \quad (1)$$

where the factor of four on the dispersion term is empirical. Each polymer is characterized by an interaction radius R_0 , and the relative energy difference $RED = Ra/R_0$ predicts behaviour: RED below unity indicates strong interaction and maximum swelling or dissolution, values near unity indicate borderline behaviour, and values well above unity indicate that the solvent will barely penetrate [66]. Representative parameters are given in Table 3. The framework explains several observations that would otherwise look arbitrary, including why dimethyl sulfoxide, with its very high polar term, is so much more aggressive towards cyanoacrylates than toluene is, and why d-limonene, a low-polarity terpene, is effective against EVA encapsulant and is now used as a greener alternative to aromatic solvents in photovoltaic module recycling [39]. It also provides a rational basis for the deep-eutectic solvent systems now being developed for the same purpose [67].

For surface integrity, the risks of solvent methods are chemical rather than mechanical. Polymer substrates may swell, craze or stress-crack in the same solvent that attacks the adhesive; coatings may lift; and a carbon-rich contamination film is left on the recovered surface almost universally, invisible to profilometry and detectable only by surface-sensitive spectroscopy [16, 33].

4.3.2 Acidic and Alkaline Agents

Sodium hydroxide and potassium hydroxide are used at concentrations of roughly 5–20% to soften polyurethanes and epoxies, typically over 30 minutes to several hours depending on bondline thickness and crosslink density. It should be stated clearly that these are strong bases, fully dissociated in aqueous solution, and describing dilute solutions of them as weak is incorrect: a 5% sodium hydroxide solution has a pH above 14. Their action is chemical scission rather than swelling. Hydroxide attacks ester and amide linkages by nucleophilic acyl substitution, which is why anhydride-cured epoxies and polyurethanes are much more susceptible than amine-cured epoxies, whose network contains no readily hydrolysable linkage; subcritical hydrolysis studies show the same selectivity at elevated temperature [48, 68, 65].

The corrosion consequences deserve more attention than they usually receive. Aluminium is amphoteric and dissolves rapidly in alkali, so strongly alkaline debonding on aluminium adherends etches the substrate, destroys the anodic layer where one is present and leaves a measurably rougher surface. Zinc and tin coatings behave similarly. Acidic agents raise the complementary problem, attacking passive films on stainless steel and titanium, dissolving carbonate stone and, in the presence of chloride, initiating pitting. Alkaline agents also leach network-modifying ions from silicate glass and change its surface chemistry permanently. In every case the recovered surface should be checked for both roughness and composition, not roughness alone [33, 32].

Table 3. Representative Hansen solubility parameters used in the selection of debonding solvents

Liquid or polymer	δD (MPa ^{1/2})	δP (MPa ^{1/2})	δH (MPa ^{1/2})	Relevance to debonding
Acetone	15.5	10.4	7.0	Dissolves cyanoacrylates; swells acrylics
Methyl ethyl ketone	16.0	9.0	5.1	General-purpose adhesive softening
Dimethyl sulfoxide	18.4	16.4	10.2	High polar term; aggressive towards cyanoacrylate and cured acrylic
N-methyl-2-pyrrolidone	18.0	12.3	7.2	Strong epoxy swelling agent; regulatory constraints
Dichloromethane	18.2	6.3	6.1	Rapid swelling of epoxy; substantial health and VOC concerns
Toluene	18.0	1.4	2.0	Low polarity; ineffective on polar networks
Ethanol	15.8	8.8	19.4	High hydrogen bonding; limited penetration of epoxy
d-Limonene	17.2	1.8	4.3	Bio-derived; effective on EVA encapsulant
Water	15.5	16.0	42.3	Non-solvent for most adhesives; carrier for alkaline and enzymatic systems
Poly(methyl methacrylate)	18.6	10.5	7.5	Substrate at risk when acrylics are targeted
Cured DGEBA epoxy (typical)	18–20	9–12	8–11	Swells only; cannot dissolve

Solvent values are the standard tabulated parameters [66]; polymer values are indicative, since they depend on formulation, cure schedule and crosslink density.

4.3.3 Enzymatic and Bio-Inspired Routes

Enzymatic debonding uses proteases, esterases and lipases to cleave hydrolysable linkages in the adhesive under mild aqueous conditions, near neutral pH and close to ambient temperature. Because the enzyme is selective for particular bonds, the substrate is essentially unaffected, which makes the approach attractive for biomedical components where residual chemical contamination is unacceptable [12, 13]. An enzymatic delamination process has been demonstrated for crystalline-silicon photovoltaic modules, releasing the glass and cells without the solvent load of conventional routes [69]. The limitations are kinetic and economic: reaction times are long, enzyme cost is high, activity is sensitive to temperature and pH, and only adhesives containing hydrolysable linkages are susceptible, which excludes amine-cured epoxy and silicone.

4.4 Electrochemical Debonding

Electrochemical methods remain the least investigated of the five families, which is unfortunate, because on metallic substrates they combine short process times with essentially no mechanical loading. The configuration is shown in Figure 7.

4.4.1 Electrolytic Debonding

In electrolytic debonding the bonded assembly is made one electrode of an electrochemical cell and a potential is applied across the joint in the presence of an electrolyte that has access to the bondline, either through a conductive adhesive, through the joint edges, or through the substrate itself. Faradaic reactions then occur at the buried metal-adhesive interface. Water electrolysis is thermodynamically possible above 1.23 V and proceeds at a useful rate once kinetic overpotentials are exceeded, so hydrogen is evolved at the cathode and oxygen at the anode directly beneath the

adhesive layer [70]. Two effects follow. Gas nucleates in the confined space at the interface and generates local pressure that lifts the adhesive mechanically, and the pH shifts sharply at the electrode surface, becoming strongly alkaline at the cathode, which degrades the interfacial oxide and hydrolyses susceptible linkages in the adhesive. The combination is efficient: separation is often achieved in seconds to a few minutes [70, 38].

Reported operating windows are summarized in Table 4. They should be read as the range encountered across the studies surveyed rather than as recommended settings, since the optimum depends strongly on the adhesive formulation and on the conductivity of the path to the interface.

Table 4. Operating parameters reported for electrolytic debonding, and their effect

Parameter	Range reported in the surveyed literature	Principal effect
Electrolyte	Aqueous NaCl, Na ₂ SO ₄ or NaNO ₃ , typically 0.1–1 M	Sets ionic conductivity and the available anion chemistry; chloride raises pitting risk
Applied voltage	Approximately 5–50 V DC	Must exceed the water decomposition potential plus overpotentials; higher voltage accelerates gas evolution
Current density	Of the order of 1–50 mA/cm ²	Controls the rate of gas generation and therefore the debonding time
Process duration	Tens of seconds to tens of minutes	Scales inversely with current density and with the accessibility of the interface
Polarity	Cathodic or anodic on the workpiece	Cathodic operation evolves hydrogen and raises interfacial pH; anodic operation risks dissolution of the substrate
Temperature	Ambient to about 60 °C	Raises reaction kinetics and diffusion, at the cost of adhesive softening that may be desirable or not

The risks are specific and, in our reading, insufficiently discussed in the debonding literature. Cathodic operation evolves nascent hydrogen at the metal surface, and on high-strength steels, martensitic stainless steels and titanium alloys, atomic hydrogen absorbed into the lattice causes embrittlement, reducing ductility and fracture toughness without leaving any visible mark on the surface. A component recovered by cathodic debonding and returned to service may therefore be degraded in a way that no roughness or spectroscopic measurement will detect, and a post-process bake-out is the standard mitigation. Anodic operation avoids hydrogen but dissolves the substrate. Where dissimilar metals are present in the same electrolyte, the applied field superimposes on the natural galvanic couple and accelerates corrosion of the less noble member. Finally, localized dissolution and gas-evolution pitting increase roughness in patches rather than uniformly, which is why area-averaged Ra can look acceptable while the surface carries discrete pits [70, 33].

4.4.2 Electrochemically Switchable Adhesives

A separate and more elegant strategy designs the adhesive so that its adhesion can be switched off electrically rather than overcome mechanically. Redox-active polymers, ferrocene-containing and viologen-containing formulations, conducting polymer systems and mussel-inspired catechol chemistries all change their interfacial interaction when oxidized or reduced, and reductions in debonding force exceeding 90% under an applied potential have been reported [22, 38, 71]. Because the adhesion is switched rather than destroyed, the recovered surface is often indistinguishable from the pre-bonding condition, which is the best surface integrity outcome available from any method reviewed here. The constraint is architectural rather than chemical: the joint has to be designed from the outset with electrical contacts and an ionically conductive path, so the approach applies to new products designed for disassembly and not to the legacy assemblies that make up most of the practical debonding workload [21, 20].

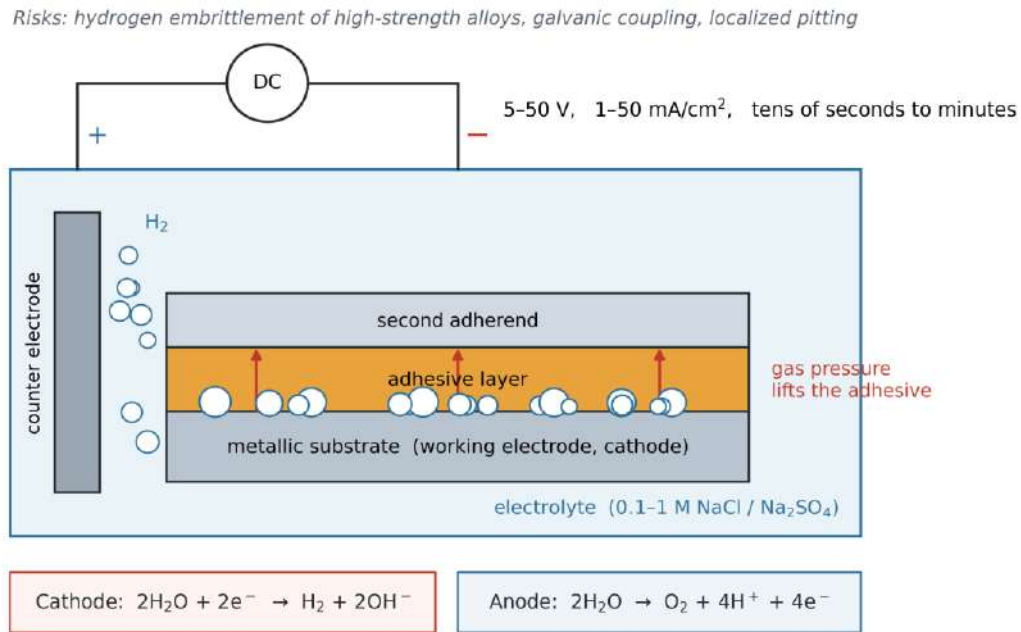


Fig. 7. Electrolytic debonding: the applied potential drives faradaic reactions at the buried interface, hydrogen and oxygen nucleate within the confined bondline, and the resulting local pressure lifts the adhesive while the interfacial pH shift degrades the oxide-adhesive bond

4.5 Hybrid Methods

4.5.1 Laser-Chemical Hybrid Debonding

In this combination a low-fluence laser scan perforates the adhesive layer with an array of micro-channels, and solvent introduced afterwards reaches the interface by diffusion through those channels rather than by the much slower route from the joint edge. Reported reductions in process time relative to solvent immersion alone reach about 70% [19, 36].

The surface integrity outcome depends entirely on the laser fluence, and the apparent contradiction between reports of increased roughness and reports of near-baseline roughness resolves once the two regimes are separated. If the fluence is set high enough for the beam to reach and act on the substrate, the perforations are transferred into the surface and roughness increases measurably. If the fluence is held below the substrate ablation threshold, so that perforation terminates within the adhesive layer, the substrate is not addressed by the beam at all and the recovered roughness stays close to the original, with increments of approximately 0.08 μm reported across metallic, composite and glass substrates [36, 60]. Fluence control, rather than the hybrid concept itself, is therefore the critical process variable, and studies that do not report fluence relative to the substrate threshold are difficult to compare.

4.5.2 Thermal-Mechanical Hybrid Debonding

Moderate heating reduces the adhesive modulus by one to two orders of magnitude as the glass transition is approached, so a mechanical separation that would have damaged the substrate at room temperature can be achieved at much lower force. Reductions in separation force exceeding 60% are reported, with post-debonding roughness within about 15% of the pre-bonding baseline for epoxy removal from carbon-fibre composite substrates [19, 50]. The approach is attractive industrially because it uses equipment that most workshops already have.

4.5.3 Ultrasonic-Chemical Hybrid Debonding

Immersing the assembly in a solvent bath driven at 20–80 kHz couples' cavitation-enhanced mass transport to chemical softening: the acoustic field continually refreshes solvent at the interface and disrupts the diffusion boundary layer, so the rate-limiting step of static immersion is removed.

Reductions in residual surface contamination of up to about 85% relative to static immersion have been reported, with correspondingly high cleaning efficiency on precision optical and medical device surfaces [39, 53]. This combination is, on the evidence surveyed, the most effective route where residue-free recovery matters more than absolute speed.

5. Characterization of the Recovered Surface

No single technique characterizes a debonded surface adequately, and the reason is worth making explicit: the techniques do not look at the same material. A stylus profilometer averages over several micrometers of relief; X-ray photoelectron spectroscopy interrogates the outermost few nanometers; residual stress measured by X-ray diffraction integrates over some tens of micrometers of depth. Two studies reporting contradictory conclusions about the same process may simply be measuring different things. Figure 8 compares the information depth of the principal techniques, and we suggest that any post-debonding assessment should combine at least one topographic, one mechanical and one chemical method chosen from different depth regimes.

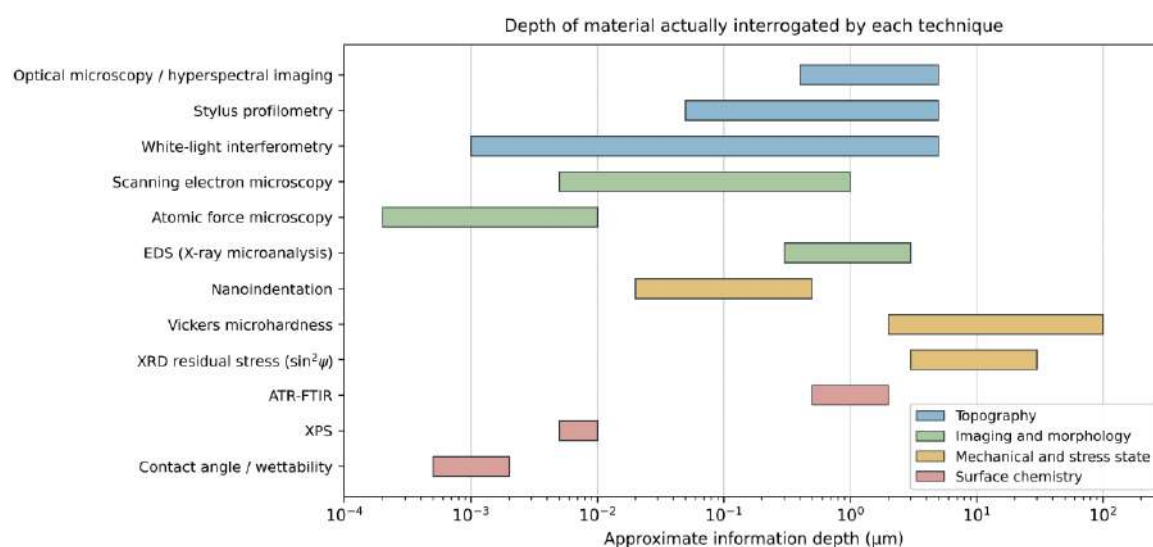


Fig. 8. Information depth of the principal characterization techniques used to assess debonded surfaces, plotted on a logarithmic depth scale. Techniques probing different depths answer different questions and are not interchangeable

5.1 Roughness and Topography

Contact stylus profilometry and non-contact white-light interferometry or confocal microscopy provide the parameters that dominate the debonding literature: R_a , R_q , R_z and the reduced peak height R_{pk} . R_a is the most reported and the least informative, since it is insensitive to the distinction between a uniformly matt surface and a smooth surface carrying isolated deep scratches; R_z and R_{pk} , or a full areal analysis to ISO 25178, capture that difference. Across the studies compiled in Section 6, laser and electrochemical routes produced the smallest roughness increments, of the order of 0.05 μm, while mechanical scraping produced the largest, up to about 2.3 μm [19, 52].

5.2 Atomic Force Microscopy

Atomic force microscopy resolves the nanometer-scale features that profilometry cannot see. It is the appropriate technique for detecting nanoscale islands of residual adhesive, shallow scratches below the profilometer detection threshold, and changes in the fine texture of a polished or as-drawn surface. It has been used directly for exactly this purpose in studies of enamel surface roughness after adhesive removal [43]. The trade-off is scan area: a typical AFM field is tens of micrometers square, so the technique characterizes a sample of the surface rather than the surface, and it should be used alongside, not instead of, an area-averaging method.

5.3 Scanning Electron Microscopy And EDS

Scanning electron microscopy visualizes micro-scratches, the distribution of adhesive residue, pitting and coating delamination zones directly, over fields ranging from the whole joint footprint to individual features. Coupled energy-dispersive X-ray spectroscopy adds elemental mapping and identifies carbon-rich contamination layers that a roughness measurement would miss entirely [33]. Its limitations should be kept in view: the interaction volume of EDS extends roughly one micrometer into the material, so it is not a surface technique in the sense that XPS is, and it cannot detect sub-monolayer contamination. Non-conductive substrates require a conductive coating, which is destructive of the surface being assessed.

5.4 Microhardness and Nanoindentation

Vickers microhardness testing under loads of 10–500 gf allows a hardness depth profile to be constructed and reveals subsurface work hardening or thermal softening produced by the debonding operation [27]. Nanoindentation extends the same measurement to penetration depths of tens to hundreds of nanometers and returns modulus as well as hardness, which makes it the only mechanical technique capable of characterizing the outermost layer where a thermally altered or chemically degraded zone would sit.

5.5 Residual Stress

X-ray diffraction using the $\sin^2\psi$ method provides non-destructive near-surface stress measurement and is the standard technique for the purpose. Reported values distinguish the method families clearly: tensile residual stresses of up to approximately 180 MPa have been reported following mechanical scraping, against compressive or near-zero states after laser and electrochemical debonding [28, 27]. The substrate-specific limitation is fundamental rather than practical, and is discussed in Section 5.8.

5.6 Surface Chemistry: XPS and FTIR

X-ray photoelectron spectroscopy quantifies elemental composition and chemical state within the outermost 5–10 nm and detects adhesive contamination at sub-monolayer coverage. It is the reference technique for answering the question that matters most after chemical debonding, namely whether the surface is actually clean [33]. Attenuated total reflectance FTIR complements it by identifying specific polymer residues and solvent degradation products through their vibrational signatures, at sensitivities well below the EDS detection threshold [72]. Where these two techniques have been applied to solvent-debonded surfaces, they have consistently found residual organic films on surfaces that appeared clean by every other measure.

5.7 Optical Inspection and Wettability

High-resolution optical microscopy in brightfield, darkfield and differential interference contrast modes detects micron-scale surface features rapidly and at low cost, and remains the first-line inspection method in production. Hyperspectral imaging extends optical inspection to chemical discrimination, resolving reflectance spectra spatially and identifying contaminant and oxidation zones; it has been demonstrated for quantifying organic contamination on copper surfaces in electronics manufacture, which is close to the debonding verification problem [72]. Contact angle measurement by the sessile drop method is the simplest and most sensitive routine indicator of surface chemical change: organic solvent debonding leaves hydrophobic contamination that has been reported to increase water contact angles by up to 35 degrees on smooth aluminium alloy, whereas ultraviolet laser debonding produces strongly hydrophilic surfaces through photochemical oxidation [33, 26].

5.8 Substrate-Specific Limitations

Technique selection cannot be separated from substrate class, and several of the inconsistencies in the published data appear to arise from this. X-ray diffraction residual stress analysis requires long-range crystalline order, so it applies to metals and to crystalline ceramics but returns nothing for amorphous polymers, silicate glass or the polymer matrix of a composite; residual stress in those materials has to be inferred by other means, such as curvature methods or photoelasticity for

transparent substrates. Energy-dispersive spectroscopy struggles with light-element organic residues on light-element substrates, which is exactly the polymer-on-polymer case. Conductive coating requirements make SEM awkward for glass and polymers. XPS and ATR-FTIR are the only techniques that work across all six substrate classes and detect residue reliably, and we regard them as the minimum chemical requirement for any credible claim of residue-free debonding. Stylus profilometry, finally, damages soft polymer surfaces during the measurement itself, so non-contact methods should be preferred there.

6. Comparative Analysis

Comparing debonding methods across the literature is difficult for a reason that recurs throughout this review: there is no standard test. Substrates, adhesives, bondline thicknesses, baseline roughness and measurement protocols all differ between studies, and absolute values are therefore not strictly commensurable. What is robust is the ordering between method families, which is consistent across independent studies. The comparison below uses six criteria: debonding efficiency, preservation of surface integrity, substrate compatibility, operational cost, environmental and safety burden, and industrial scalability.

6.1 Mechanical Methods

Mechanical routes are the simplest and cheapest and carry the highest risk to the substrate. Scraping raised Ra by roughly 0.3–2.3 μm across the studies surveyed, and also introduced the highest tensile residual stresses of any family. Ultrasonic vibration is the exception within the family and achieved increments as low as about 0.2 μm on smooth metallic and composite substrates, because no tool touches the surface [53, 19]. Controlled peeling sits in between, and its performance is governed by peel angle more than by any other variable [49].

6.2 Thermal Methods

Thermal performance is the most variable of the five families, which reflects how differently energy is delivered within it. Convective heating is inexpensive and scalable but heats the substrate more than the bondline. Laser debonding consistently returns surface roughness within 0.05–0.15 μm of baseline and, in wafer release, below 1 nm Ra, making it the best single-mechanism performer in the review [57, 58, 36]. Induction heating gives rapid cycles but is restricted to conductive substrates or to formulations carrying susceptor particles, with the residue consequences discussed in Section 4.2.3. Cryogenic debonding is the gentlest thermal route in surface terms and the most constrained by assembly geometry [50].

6.3 Chemical Methods

Chemical routes eliminate mechanical damage entirely, which is their principal attraction on brittle and optically finished substrates, but they substitute a contamination problem for a topographic one. Organic residue is detectable by XPS on essentially every solvent-debonded surface reported, even after cleaning [33]. Strongly alkaline and acidic agents add a corrosion penalty that is substrate-specific and, on aluminium, severe. Enzymatic routes have the best substrate compatibility of any method in this review and the worst kinetics and reagent economics [69].

6.4 Electrochemical Methods

Electrochemical debonding is the least investigated family and the one where the gap between demonstrated potential and industrial use is widest. Assemblies built with switchable adhesives recover surfaces that are, within measurement resolution, indistinguishable from the pre-bonding condition [22, 38]. The requirement for integrated electrical contacts confines the approach to designed-for-disassembly products, and the hydrogen embrittlement and galvanic risks set out in Section 4.4.1 restrict electrolytic variants on high-strength alloys.

6.5 Hybrid Methods

Hybrid methods outperform single-mechanism approaches consistently, and the reason is structural rather than incidental: each component addresses a different rate-limiting step. The laser-chemical hybrid recovered Ra within approximately 0.08 μm of baseline across metallic,

composite and glass substrates when the fluence was held below the substrate threshold [36, 60]. The thermal-mechanical hybrid reduced separation force by more than 60% [50], and the ultrasonic-chemical hybrid reduced residual contamination by up to 85% relative to static immersion [39, 53]. The cost is process complexity and a larger parameter space to control.

6.6 Comparative Summary

Table 5 summarizes the qualitative performance of each method against the six criteria, and Table 6 collects the quantitative roughness evidence on which the surface integrity ratings rest. Figure 9 presents the same qualitative comparison graphically.

Table 5. Comparative performance of debonding methods against six evaluation criteria. All criteria are oriented so that a higher rating is more favorable

Debonding method	Debonding efficiency	Surface integrity	Substrate compatibility	Low operational cost	Environmental and safety	Industrial scalability
Controlled peeling	Moderate	Good	Limited	Excellent	Excellent	Moderate
Mechanical scraping	Good	Poor	Good	Excellent	Excellent	Good
Ultrasonic vibration	Good	Good	Good	Moderate	Good	Good
Pneumatic and hydraulic	Moderate	Excellent	Moderate	Moderate	Excellent	Moderate
Convective heating	Good	Moderate	Moderate	Good	Good	Good
Laser debonding	Excellent	Excellent	Excellent	Limited	Good	Moderate
Induction heating	Good	Excellent	Limited	Moderate	Good	Good
Cryogenic debonding	Good	Excellent	Moderate	Moderate	Excellent	Moderate
Solvent chemical	Good	Moderate	Good	Good	Poor	Good
Acidic and alkaline	Good	Limited	Limited	Good	Poor	Moderate
Enzymatic	Moderate	Excellent	Excellent	Poor	Excellent	Limited
Electrolytic	Moderate	Excellent	Limited	Moderate	Good	Limited
Switchable adhesive	Excellent	Excellent	Moderate	Limited	Excellent	Limited
Laser-chemical hybrid	Excellent	Excellent	Excellent	Limited	Moderate	Moderate
Thermal-mechanical hybrid	Good	Good	Good	Good	Good	Good
Ultrasonic-chemical hybrid	Excellent	Excellent	Good	Moderate	Moderate	Good

6.7 Interpretation and Trade-Offs

Three patterns emerge from Tables 5 and 6, and they are more useful than any single ranking.

The first is that surface integrity and cost are inversely related across almost the whole range. The methods that recover the best surfaces, laser, laser-chemical hybrid, switchable adhesives and enzymatic routes, are also the most capital-intensive or the most demanding in reagent cost, while the methods available in every workshop are the ones that damage the substrate most. The break-even point is set by the value of the recovered part. For a precision optics, a thinned wafer or a titanium aerospace fitting, the capital cost of a laser station is recovered on a small number of components; for a bonded automotive sill destined for material recycling, it never is, and a thermal-mechanical route is the rational choice [24].

The second is that no method is simultaneously fast, gentle and broadly applicable. Laser processing is fast and gentle but line-of-sight, so it cannot reach a buried bondline. Chemical routes reach anywhere but are slow and leave residue. Electrochemical routes are fast and gentle but require a conductive path designed in from the start. The apparent superiority of hybrid methods comes precisely from pairing techniques whose constraints do not overlap.

The third is that the environmental ranking has begun to override the technical one. Solvent and strong-acid or strong-base routes score well on cost and compatibility but poorly on environmental burden, and tightening volatile organic compound regulation is progressively removing them from the available options regardless of their technical merit [73]. Methods that looked uncompetitive on cost alone a decade ago, including enzymatic and deep-eutectic-solvent routes, are becoming attractive on this basis [69, 67].

Table 6. Quantitative surface integrity outcomes reported for the principal debonding methods on smooth substrates

Method	Reported roughness change	Residual stress state	Chemical residue	Typical process time
Mechanical scraping	Ra increase of about 0.3–2.3 μm	Tensile, up to approximately 180 MPa	Embedded tool and adhesive debris	Minutes, operator dependent
Controlled peeling	Small increase; strongly peel-angle dependent	Low, localized at the peel front	Adhesive transfer film	Seconds to minutes
Ultrasonic vibration	Ra increase of about 0.2 μm	Low	Reduced when performed in a liquid bath	Seconds to minutes
Convective and conductive heating	Negligible on metals; distortion risk on polymers	Low; thermal gradients possible	Softened adhesive film	Minutes
Laser debonding	Within 0.05–0.15 μm of baseline; below 1 nm Ra in wafer release	Compressive or near zero	Minimal; photo-oxidized surface after UV processing	Seconds
Induction heating	Negligible increase on aluminium alloy	Low	Susceptor particles remain in the residue	Under 60 s
Cryogenic debonding	No measurable change reported	Low	None; no reagents used	Minutes
Solvent chemical	No mechanical change; swelling of polymer substrates possible	None introduced	Carbon-rich film detectable by XPS	30 min to several hours
Alkaline and acidic	Increase through etching, substrate dependent	None introduced	Ionic residue; altered oxide	30 min to several hours
Electrolytic	Small area-averaged increase; localized pitting possible	Low; hydrogen uptake risk	Electrolyte salts requiring rinsing	Seconds to minutes
Switchable adhesive	Indistinguishable from pre-bonding condition	None introduced	Minimal	Seconds
Laser-chemical hybrid	Within about 0.08 μm of baseline below the substrate ablation threshold	Compressive or near zero	Reduced relative to solvent alone	Reduced by up to about 70% versus solvent alone
Thermal-mechanical hybrid	Within about 15% of baseline on CFRP	Low	Softened adhesive film	Separation force reduced by more than 60%
Ultrasonic-chemical hybrid	No mechanical damage reported	None introduced	Contamination reduced by up to about 85% versus static immersion	Minutes

Values are compiled from the sources cited in Sections 4 and 6. Because test conditions differ between studies, the table should be read as an ordering between method families rather than as a set of directly comparable measurements.

6.8 Method Selection Guidance

For high-value substrates with tight surface specifications, such as optical glass, thinned wafers and precision metallic components, laser and laser-chemical hybrid processing are justified despite their capital cost. For high-volume production with moderate surface requirements, ultrasonic-chemical and thermal-mechanical hybrids offer the better cost-benefit balance. For temperature-sensitive assemblies, cryogenic, electrochemical and enzymatic routes avoid the thermal load altogether. For products still at the design stage, designing in a debonding trigger, whether thermal,

electrical or photochemical, converts the problem from one of removal to one of actuation and gives by far the best surface integrity outcome [21, 22, 20].

	Debonding efficiency	Surface integrity	Substrate compatibility	Low operational cost	Environmental and safety	Industrial scalability
Controlled peeling	Moderate	Good	Limited	Excellent	Excellent	Moderate
Mechanical scraping	Good	Poor	Good	Excellent	Excellent	Good
Ultrasonic vibration	Good	Good	Good	Moderate	Good	Good
Pneumatic and hydraulic	Moderate	Excellent	Moderate	Moderate	Excellent	Moderate
Convective heating	Good	Moderate	Moderate	Good	Good	Good
Laser debonding	Excellent	Excellent	Excellent	Limited	Good	Moderate
Induction heating	Good	Excellent	Limited	Moderate	Good	Good
Cryogenic debonding	Good	Excellent	Moderate	Moderate	Excellent	Moderate
Solvent chemical	Good	Moderate	Good	Good	Poor	Good
Acidic and alkaline	Good	Limited	Limited	Good	Poor	Moderate
Enzymatic	Moderate	Excellent	Excellent	Poor	Excellent	Limited
Electrolytic	Moderate	Excellent	Limited	Moderate	Good	Limited
Switchable adhesive	Excellent	Excellent	Moderate	Limited	Excellent	Limited
Laser-chemical hybrid	Excellent	Excellent	Excellent	Limited	Moderate	Moderate
Thermal-mechanical hybrid	Good	Good	Good	Good	Good	Good
Ultrasonic-chemical hybrid	Excellent	Excellent	Good	Moderate	Moderate	Good

Fig. 9. Comparative performance of debonding methods across the six evaluation criteria of Table 5. Darker cells indicate more favourable performance for that criterion

7. Applications and Industrial Relevance

The requirements placed on a debonding process differ so widely between sectors that a method which is standard practice in one is inadmissible in another. Table 7 summarizes the combinations that recur in industrial practice.

7.1 Aerospace and Defence

Aerospace presents the most demanding combination of requirements in the review. Aluminium alloy, titanium and CFRP substrates must be freed of adhesive completely, without any increase in roughness or microstructural change that would compromise fatigue performance, and usually so that the component can be rebonded [3, 6]. Laser methods are the most promising route for composite repair preparation, since they remove the adhesive without touching the fibre and leave a surface whose chemistry favors rebonding [25, 5]. The recurring difficulty is access: bond lines inside a wing box are not in line of sight.

7.2 Automotive

Windscreen replacement is the highest-volume debonding operation carried out anywhere, and it is instructive because both adherends must survive. The polyurethane bead has to be severed while the electrocoat and topcoat on the painted flange remain intact. Optimized heated-wire cutting confines paint damage to under 2 mm of flange width, against 8–15 mm for cold-knife methods, which is the difference between a repair and a repaint [4, 50]. Multi-material body structures combining steel, aluminium and CFRP are substantially harder, because a single stimulus applied to the whole assembly cannot be optimal for every interface within it [24].

7.3 Electronics and Microelectronics

Semiconductor manufacture applies effectively zero-tolerance contamination standards. Device wafers thinned to 50 µm or less are bonded to transparent carriers and released optically after

processing, with the recovered surface held below 0.3 nm Ra, a level of surface preservation no other method approaches [7, 57], [58, 8]. Thermally releasable polyimide systems and laser-sensitive release layers now compete for the same process step [9, 42].

7.4 Biomedical and Healthcare

Here the criterion shifts from topography to chemistry and to the substrate's own fragility. Medical adhesive-related skin injury is a recognized clinical problem, and silicone-based adhesive removers cause substantially less disruption of the stratum corneum than mechanical peeling does, which matters particularly for neonatal and elderly patients [10, 11]. For implantable and surgical adhesives, freedom from residual chemical contamination is a regulatory requirement rather than a quality target [12, 13].

7.5 Construction and Architecture

Structural silicone removal from glass and aluminium curtain-wall systems is the archetypal case of a smooth, brittle, high-value substrate. Combining chemical softening with plastic scraping tools has been reported to limit Ra increments to about 0.05 μm , against more than 0.8 μm where steel tools are used alone [14, 40]. Laser methods preserve historic substrate surfaces without measurable alteration, which is why they have been adopted in conservation practice [32]. In-service inspection of these joints has its own developed literature based on laser-excited thermography [15, 41].

7.6 Renewable Energy

Photovoltaic module recycling is the fastest-growing application of debonding technology, driven by the volume of modules now reaching end of life. Thermal separation of the EVA encapsulant recovers more than 99% of the glass by mass, while controlled swelling under sonication, particularly with bio-derived limonene, offers a lower-temperature and greener alternative [39, 17], [74, 75]. Deep-eutectic solvents and enzymatic delamination are both under active development for the same purpose [67, 69]. At the other extreme of scale, removal of leading-edge protection film from wind turbine blades exceeding 80 m in length requires robotic implementation simply to achieve consistent results over the length of the blade [76]. Figure 10 illustrates the debonding-on-demand concept in a recycling context.

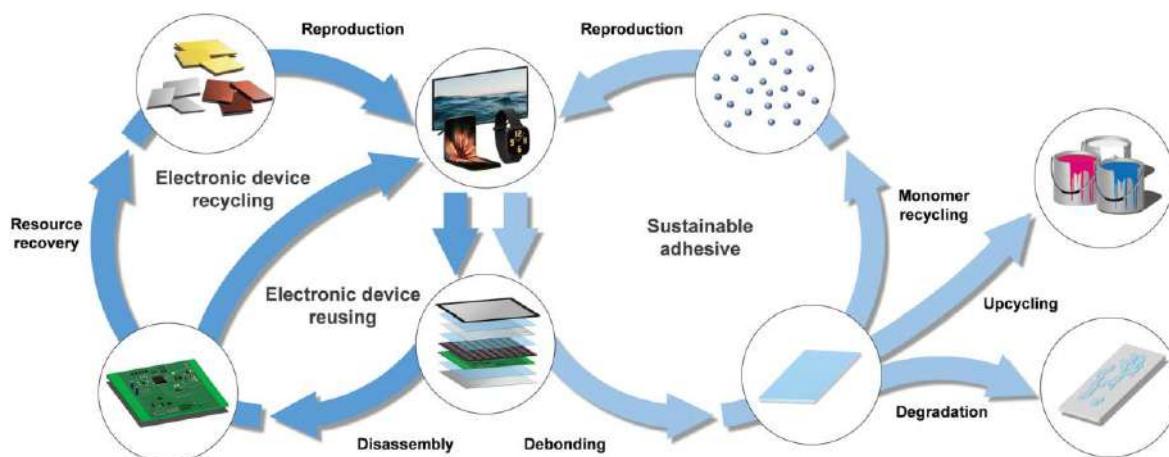


Fig. 10. Debonding-on-demand adhesives applied to disassembly and recycling

7.7 Optics and Precision Instrumentation

Optical instrumentation imposes the tightest specification of all, with Ra below 0.5 nm required and effectively no tolerance for surface damage, since a change in surface figure of a few nanometers is directly visible in an interferometric measurement. Thermal cycling protocols and cryogenic separation have both been shown to release adhesives without measurable modification of surface figure [34, 35, 64].

Table 7. Recommended debonding routes by industrial sector

Sector	Typical substrate	Typical adhesive	Recommended debonding route	Governing constraint
Aerospace and defence	Al and Ti alloys, CFRP	Structural epoxy, film adhesive	Laser; thermal-mechanical hybrid	Fatigue performance and rebondability
Automotive glazing	Painted steel or aluminium flange, glass	Polyurethane	Heated-wire cutting; low-temperature debonding	Survival of the paint system
Automotive body structure	Multi-material steel, Al, CFRP	Toughened epoxy, structural acrylic	Thermal-mechanical; induction with susceptor fillers	Multi-material compatibility and cost
Microelectronics	Thinned Si wafer, glass carrier	Temporary bonding polymer, release layer	Laser release through the carrier	Sub-nanometer roughness and zero contamination
Biomedical devices and skin	Polymer housings, skin	Silicone and acrylic PSA	Silicone-based remover; enzymatic	Chemical contamination and tissue trauma
Construction and glazing	Float glass, anodized aluminium	Structural silicone	Chemical softening with plastic tooling; laser for heritage work	Brittleness and appearance
Photovoltaic recycling	Solar glass, silicon cells	EVA encapsulant	Thermal separation; limonene swelling under sonication; deep-eutectic solvents	Throughput and environmental burden
Wind energy	GFRP blade shell	Leading-edge protection adhesive	Robotic mechanical and thermal-mechanical	Scale and consistency over 80 m
Precision optics	Fused silica, optical glass	Optical epoxy and cement	Cryogenic and controlled thermal cycling	Surface figure at the nanometer level

8. Challenges and Future Directions

8.1 Current Challenges

8.1.1 Absence of Standardized Evaluation

This is, in our assessment, the single most limiting problem in the field. There is no agreed protocol for what should be measured after debonding, on what area, with which technique, or against what baseline. Studies report Ra without specifying the sampling length; roughness without residual stress; residue by EDS where XPS is required. The consequence is that quantitative comparison across studies is unreliable, as Table 6 makes clear, and that genuinely superior methods cannot be distinguished from well-reported ones. An internationally recognized evaluation framework, defining a minimum set of parameters and reporting conventions, would do more for the field than any new removal technique.

8.1.2 Multi-Material Assemblies

As multi-material construction spreads through aerospace, automotive and electronics, the compatibility requirement increasingly prevents the use of the method that would be optimal for any individual interface. A stimulus applied to a whole assembly must be tolerable for its most sensitive component, which means the process is set by the weakest link rather than by the joint

being separated. Spatially selective and adaptive stimuli, of the kind that scanned laser processing and locally addressed induction can in principle provide, are the obvious response [4, 61].

8.1.3 Verification of Complete Removal

Residual contamination detectable only by XPS or secondary ion mass spectrometry persists across every method family reviewed here. Both techniques are laboratory instruments requiring vacuum, and neither can be placed inline in a production environment. There is therefore no practical way to verify, at production rate, that a debonded surface is clean enough to rebond. Optical and hyperspectral approaches are the most plausible route to closing this gap, since they are fast, non-contact and already used for contamination quantification in electronics [72, 33].

8.1.4 Aged and Environmentally Degraded Joints

Almost all published debonding data are obtained on freshly prepared laboratory specimens, whereas almost all industrial debonding is performed on joints that have been in service for years. Environmental ageing changes mechanical properties, chemical composition and interfacial condition, generally in the direction of increased brittleness and reduced predictability, and moisture-degraded joints behave differently again [25]. Process protocols that adapt to the measured condition of the joint, informed by non-destructive assessment before the operation begins, are needed and largely absent [77].

8.1.5 Regulatory Pressure on Chemical Routes

Tightening volatile organic compound regulation is progressively closing off organic solvent-based methods, independently of their technical performance [73]. The replacement candidates, water-based systems, bio-derived terpenes, deep-eutectic solvents and supercritical fluids, are at very different levels of maturity, and the field needs performance data on them measured against the same surface integrity criteria used for the incumbents [39, 67].

8.2 Future Research Directions

8.2.1 Stimuli-Responsive Adhesive Systems

Designing the debonding trigger into the adhesive rather than applying it from outside is the most promising direction in the field, because it converts an aggressive removal problem into a controlled actuation problem. Photocleavable molecular units, thermally reversible Diels-Alder networks, vitrimer chemistries with exchangeable bonds, responsive microcapsules and magnetically addressable formulations have all been demonstrated [21, 78], [79, 80], [81, 51]. The open questions are no longer whether the chemistry works but whether these systems retain structural performance over a full-service life, and whether the trigger can be guaranteed not to fire accidentally in service, which is the concern that has kept them out of primary structure so far [82, 83].

8.2.2 Machine Learning in Process Optimization

Two applications are developing in parallel. Convolutional networks trained on electron micrographs have been used to classify post-debonding surface condition automatically, with reported accuracies around 94% for wafer surfaces, which points towards inline machine vision assessment. Separately, reinforcement learning and Bayesian optimization have been applied to the selection of laser and plasma process parameters, converging on effective settings with far fewer experiments than a conventional design of experiments requires [26, 84]. The limiting factor in both cases is the availability of labelled datasets, which is itself a consequence of the standardization gap discussed above.

8.2.3 Nanotechnology-Enhanced Routes

Carbon nanotube and graphene-reinforced formulations create percolating conductive pathways through an otherwise insulating adhesive, which extends electrochemical and resistive-heating debonding to non-conductive substrates that could not otherwise be addressed [62, 63]. In a second approach, nanoparticle carriers deliver debonding agents directly to the adhesive-substrate interface, so that the effective local concentration is high while the bulk solvent inventory, and

therefore the environmental burden, stays low. Both raise the residue question addressed in Section 4.2.3 and should be evaluated with it in view.

8.2.4 Robotics and Automation

Large-area debonding in aerospace, automotive and renewable energy applications is currently limited less by the physics of removal than by the variability that manual operation introduces over large areas. Robotic implementation with force-torque feedback and inline process monitoring addresses this directly by holding the process parameters constant regardless of position on the component, and it is a prerequisite for any operation on the scale of a wind turbine blade or a fuselage panel [76, 24].

8.2.5 Sustainable and Bio-Inspired Approaches

Biological adhesion offers working models of reversible attachment that engineered systems have not yet matched. Marine organisms release catechol-based adhesion in response to environmental cues, and the dry adhesive systems inspired by gecko setae attach and detach thousands of times without residue [37, 82, 71]. Translating these mechanisms into structural adhesives that survive an industrial service environment is the outstanding challenge. In the nearer term, integrated platforms combining laser debonding, atmospheric plasma reactivation and inline spectroscopic verification appear to us the most realistic route to a closed-loop process in which a component is separated, verified and rebonded without leaving the production line [26, 72].

9. Conclusion

This review examined adhesive debonding from smooth substrates with the condition of the recovered surface, rather than the speed of separation, as the criterion of success. Taking that view changes the ranking of the available methods considerably.

Substrate class and adhesive chemistry jointly determine what is admissible. Smooth substrates are a distinct problem because adhesion is carried by short-range interactions distributed uniformly across a nearly planar interface, which removes the natural crack initiation sites that debonding processes exploit, while simultaneously reducing the tolerance for damage to a level at which a single scratch is a failure. Adhesive chemistry sets the available trigger: covalently crosslinked thermosets can be swollen but not dissolved, thermoplastic systems remain reversible, and silicones resist complete removal in every method surveyed.

Among the method families, mechanical routes are simple and cheap but degrade the surface most severely, with ultrasonic vibration the clear exception within the family. Thermal methods vary widely, and laser debonding is the best single-mechanism performer, returning roughness within 0.05–0.15 μm of baseline and below 1 nm in wafer release. Chemical methods eliminate mechanical damage but substitute a contamination problem and, in strongly alkaline or acidic form, a corrosion problem. Electrochemical methods, particularly switchable adhesive systems, produce the best surfaces of all but require the joint to be designed for them, and electrolytic variants carry hydrogen embrittlement and galvanic risks that are not always acknowledged. Hybrid methods consistently outperform single-mechanism approaches because they pair techniques whose limitations do not overlap, with laser-chemical and ultrasonic-chemical combinations the strongest performers.

Assessment of the recovered surface requires several complementary techniques chosen from different depth regimes, since no single parameter captures the range of modification that debonding can produce, and since techniques that probe different depths answer different questions. We would argue that a credible claim of surface-preserving debonding requires at minimum an aerial topographic measurement, a subsurface mechanical or residual stress measurement, and a surface-sensitive chemical measurement.

Industrial demand across aerospace, automotive, electronics, biomedical, construction, renewable energy and precision optics continues to grow, and the technical literature is expanding with it. What is not expanding at the same rate is the methodological basis for comparing results between studies. Until a standardized post-debonding evaluation protocol exists, the field will continue to accumulate empirical, application-specific procedures that cannot be transferred between

substrates or sectors. Establishing that protocol, and coupling it to the developing work on responsive adhesives, inline verification and greener chemistries, is in our view the most productive direction for the next several years of research.

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Multi modal early warning for mountain bridge structural defects using CBAM enhanced ResNet Mask R CNN

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Abstract

To address poor feature extraction adaptability and the gap with practical engineering requirements for mountain bridge structural defect early warning under complex geological backgrounds and variable illumination, this paper proposes a CBAM-ResNet-Mask R-CNN instance-segmentation early warning framework for mountain-bridge disasters. A dual-path residual block is designed for the Stages 3-4 of the ResNet-101 backbone, where dilated convolution is adopted in the main path and 1×1 dimensionality-reduction convolution is introduced in the auxiliary path, so as to suppress complex background noise and enhance weak disaster features under uneven illumination. The CBAM attention module is embedded inside residual blocks of Stages 3-4 to realize joint channel-spatial feature screening. Since Mask R-CNN only outputs two-dimensional pixel-level segmentation masks from RGB images, the visual quantitative features calculated from segmentation masks are further fused with auxiliary structural monitoring data including vibration and strain time-series signals, to realize inversion of three-dimensional structural deformation and disaster severity assessment. The reduction rate of economic loss is estimated by feeding disaster-severity quantification results into a bridge life-cycle cost model. On the independent test set, the proposed model achieves mAP@0.5 of 92.17%, mAP@0.5:0.95 of 76.42% and multi-disaster comprehensive F1-score of 0.93. Its recall rate for small-scale rockfall early warning reaches 93.64%, with an early-warning false-alarm rate of 2.56%. The monitoring accuracy for bearing deformation and concrete spalling proportion both exceeds 90%, and the maximum economic-loss reduction rate reaches 58.46% under test-set conditions. The proposed framework can realize accurate disaster-feature segmentation and multi-source-data joint quantification for mountain-bridge hazards, which provides feasible technical support for mountain-bridge safety monitoring and graded early-warning decision-making.

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1. Background

Mountain bridges are susceptible to disasters such as rockfall impact, concrete spalling, and crack propagation due to long-term exposure to complex geological environments and variable climatic conditions [1-2]. Untimely disaster early warning can cause structural damage, traffic disruption, and even casualties. Therefore, constructing accurate and efficient mountain bridge disaster early warning models is of great engineering significance for ensuring bridge operational safety [3-4]. Current early warning methods still have limitations in disaster feature segmentation accuracy and structural stress prediction [5]. Some models tend to mis extract features or miss disasters when dealing with complex backgrounds and weak features in mountain bridge images [6-7]. Residual Network (ResNet) can specifically enhance weak disaster features and suppress redundant background information. Mask Region-based Convolutional Neural Network (Mask R-CNN) can

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restore high-resolution segmentation masks through a four-layer transposed convolution up sampling in the segmentation branch. The Convolutional Block Attention Module (CBAM), due to its flexible modular design, can seamlessly integrate into ResNet backbone networks and Mask R-CNN feature extraction pipelines. Against the background of civil-infrastructure structural health monitoring, vision-based inspection has become an important technical route for bridge condition assessment [8-9]. Nevertheless, mountain-bridge monitoring suffers from complex geological background interference and variable illumination, which easily trigger false detection. Moreover, the logical linkage between image segmentation outputs and disaster quantitative evaluation remains insufficiently addressed in existing research. Accordingly, this paper proposes a mountain-bridge disaster early warning model integrating CBAM, ResNet, and Mask R-CNN. Dual-path residual blocks together with multi-scale feature fusion and CBAM attention modules are adopted to suppress complex mountain background noise and reduce false alarms caused by redundant background features. Pixel-level segmentation masks yielded by the visual model are further combined with vibration and strain time-series sensor data to realize disaster severity quantification, establishing the correlation between image processing results and structural disaster assessment. This work intends to satisfy the requirements of weak-disaster feature extraction and early warning for mountain bridges.

2. Literature Review

Structural health monitoring is essential to guarantee the safe service of civil infrastructures, including contact sensor based and non-contact sensing approaches. Contact sensors can collect one dimensional time series vibration and strain signals; one dimensional convolutional neural network combined with transfer learning are widely used to extract damage related features, while generative adversarial networks are adopted to mitigate the scarcity of real-world damage samples [10-11]. Sensor placement optimization is also a key issue for dynamic structural monitoring of civil infrastructures [12]. Apart from contact monitoring techniques, InSAR time series remote sensing can realize large scale infrastructure deformation monitoring but cannot capture fine grained local damage details of bridges [13]. Non-contact computer vision-based monitoring possesses flexible deployment merits, and vision driven real time structural health monitoring systems have been developed for offshore infrastructures. However, few investigations focus on mountain bridge scenarios disturbed by uneven illumination and complicated geological backgrounds. As an instance segmentation algorithm with multi-task capability and high-precision segmentation, Mask R-CNN can simultaneously output target categories, bounding boxes, and segmentation masks in tasks. Some scholars have conducted explorations based on its characteristics. Setrayana A et al. proposed the application of YOLOv11 and Mask R-CNN models for real-time water level detection to address the problem of slow response in manual systems for urban flood monitoring in Indonesia [14]. Instance segmentation approaches can be divided into two stage and one stage paradigms. Two stage detectors represented by Mask R CNN achieve superior mask boundary precision benefited from RoI Align operation, yet bring non negligible computational overhead. By comparison, one stage segmentation models such as YOLOv8 seg deliver faster inference speed suitable for edge device deployment, at the cost of slight degradation in segmentation boundary accuracy, and have been applied for post seismic structural damage segmentation tasks [15]. Besides conventional ResNet backbones, Swin Transformer and other vision Transformer architectures exhibit advantages in global context modeling and capturing long range dependencies for complex scenes, while suffering higher computational cost. Unlike Mask R-CNN, CBAM can precisely enhance key features and suppress redundant information through dual attention mechanisms of channel and spatial dimensions. Some domestic and international scholars have conducted different studies using this method. Huang Y et al. developed a model that integrated multi-scale convolution with a self-attention mechanism to address seismic event classification tasks, while introducing a voting mechanism to integrate multi-component data. Experiments verified that this method outperformed traditional methods [16]. Wenxuan Z et al. proposed a lightweight neural network method to solve the problems of low accuracy and slow speed in traditional fire detection methods. The results indicated that this approach demonstrated significant superiority in identifying small-scale fire events [17].

In addition, effective disaster early warning can minimize the impact of disasters on life, property, and social systems. Some scholars have proposed different opinions on this issue. Yue Y put forward an analytic hierarchy process optimized by entropy weighting and fuzzy numbers to assess construction risks of highway bridges in plateau mountainous regions. The experimental results showed that this method could effectively identify key risk points such as building materials and provide quantitative basis for precise prevention and control of high risks [18]. Yang P's team proposed a dual-threshold method for the uncertainty problem of critical rainfall thresholds in mountain flood warning. The results indicated that this method had higher reliability compared to traditional single-threshold methods [19]. Rawat D et al. proposed the use of wavelet synchro squeezing transform to analyze seismic signals from station networks for the Chamoli 2021 ice-rock avalanche disaster. The experimental results showed that this method could effectively identify high-frequency collapse signals at 16-18 Hz and low-frequency seismic precursors at 0.01-0.2 Hz [20]. Zhao Y's team innovatively proposed an adversarial interpretive structural modeling method for factors influencing mountain community resilience. The results showed that this method could systematically identify the action paths of key factors such as infrastructure, information access, and emergency planning [21]. Ding Y et al. proposed a comprehensive rockfall scenario simulation method for the difficult problem of rockfall risk simulation in mountain bridges. Experiments showed that this method could effectively analyze rockfall risks under complex coupling effects by constructing virtual risk environments [22].

From the conclusions drawn by domestic and international scholars, although current disaster early warning methods have made certain progress, there still exist problems of insufficient model adaptability and scenario compatibility. Some early warning methods are disconnected from actual engineering needs, and the entropy weight-based analytic hierarchy process and dual-threshold rainfall methods are difficult to directly provide precise technical support for bridge disaster emergency response. Therefore, to improve the adaptability of disaster early warning methods, this study proposed a mountain bridge disaster early warning model combining CBAM, ResNet, and Mask R-CNN. The study expects to provide effective technical support for property loss control and ecological facility protection through this model.

3. Mountain Bridge Disaster Early Warning Model Construction

3.1 Design of Mask R-CNN Disaster Recognition Model Combine Resnet

ResNet introduces residual block structures that allow inputs to directly propagate across layers to outputs. This advantage enables effective network training even when the depth increases. Mask R-CNN provides precise instance segmentation and multi-task capabilities, which can simultaneously identify different disaster types in mountain bridge early warning scenarios. Therefore, this study designs a Mask R-CNN disaster recognition model integrating ResNet, which strengthens weak feature extraction for bridge disasters through ResNet and achieves integrated disaster category recognition, location detection, and pixel-level segmentation with Mask R-CNN. The instance segmentation process is shown in Figure 1.

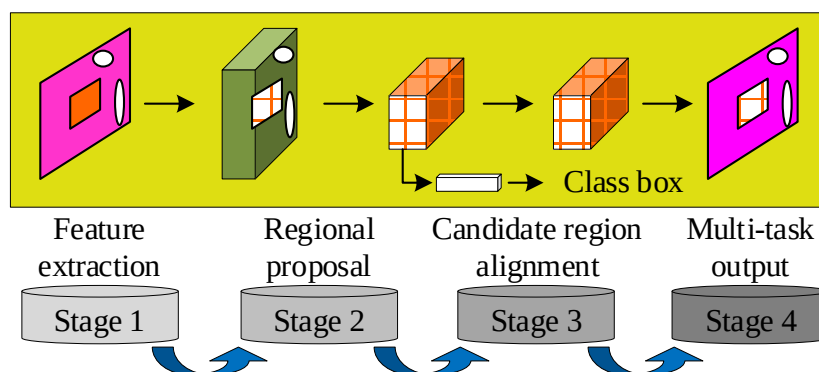


Fig. 1. Instance segmentation process of Mask R-CNN

In Figure 1, Stage 1 first standardizes and augments the original bridge images, then uses the backbone network to gradually reduce image size and increase feature dimensions. Stage 2 presets

twelve anchor boxes on the C4 feature map for diverse bridge disaster forms, evaluates whether each anchor box contains a disaster using 1×1 convolution, and outputs confidence scores. Candidate regions are filtered through non-maximum suppression to remove redundant overlapping anchor boxes, producing high-confidence candidate regions. Stage 3 maps candidate rockfall regions from the original image onto the C4 feature map proportionally, divides sampling points in the mapped feature regions, and calculates feature values for each point using bilinear interpolation. Finally, it averages the feature values to generate a fixed-dimension feature vector. The total loss function is shown in Equation (1).

$$L_{total} = L_{cls} + \lambda_1 \cdot L_{reg} + \lambda_2 \cdot L_{mask} \quad (1)$$

Equation (1) shows L_{cls} and L_{reg} representing classification loss and bounding box regression loss, L_{mask} representing Mask segmentation loss, and λ_1 and λ_2 representing loss weights. The classification loss calculation is shown in Equation (2) [23].

$$L_{cls}(p, p^*) = - \sum_{k=0}^K p_k^* \cdot \log(p_k) \quad (2)$$

In Equation (2), $p = [p_0, p_1, \dots, p_K]$ indicates the predicted class probability distribution, p_k represents the probability that a candidate region belongs to class k , and K is the total number of classes. Stage 4 performs multi-task output. The input feature vector passes through two fully connected layers to compress it to 1×4 dimensions. The Softmax function outputs class probabilities, and another fully connected layer outputs coordinate parameters. The Mask branch enlarges the feature map with four transposed convolution layers, removes low-confidence regions, and overlays the segmentation mask of remaining high-confidence regions onto the original bridge image. The study selects ResNet-101 as the backbone to capture finer-grained disaster features. The ResNet-based feature enhancement module is shown in Figure 2.

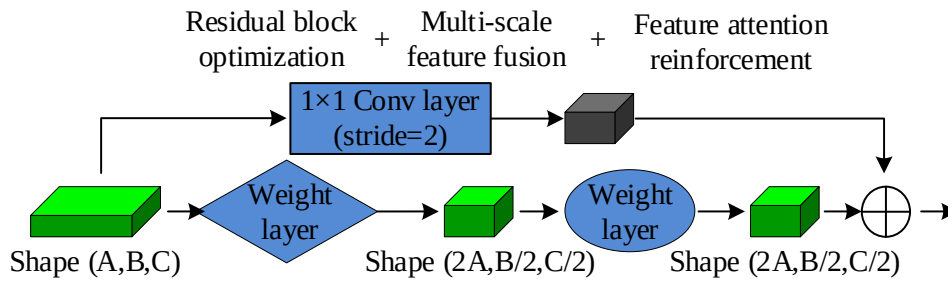


Fig. 2. ResNet-based feature enhancement module

In Figure 2, the model adds an adaptive contrast enhancement module at the ResNet input layer. It uses the Retinex algorithm to separate image illumination and reflection components and applies contrast-limited adaptive histogram equalization to the reflection component. A dual-path residual block structure is designed, where the main path introduces dilated convolution to the original residual block, and the auxiliary path adds a 1×1 convolution for dimensionality reduction and expansion. The two paths are fused through element-wise addition. Different stages of ResNet-101 use different residual block configurations: stages 1–2 use standard residual blocks, while stages 3–4 replace all standard blocks with dual-path residual blocks. Residual block feature propagation is calculated as shown in Equation (3).

$$H(x) = F(x, \{W_i\}) + W_s(x) \quad (3)$$

In Equation (3), $H(x)$ is the output feature map of the residual block, $F(x, \{W_i\})$ is the residual function, $\{W_i\}$ is the convolution kernel weights in the residual function, and $W_s(x)$ represents the shortcut connection. Global feature extraction is calculated as in Equation (4).

$$F_{main}(x) = BN(W_2 \cdot ReLU(BN(W_1 \cdot x))) \quad (4)$$

Equation (4) uses W_1 and W_2 as 3×3 convolution kernels, with BN representing batch normalization. High-frequency weak feature enhancement is calculated in Equation (5).

$$F_{aux}(x) = W_4 \cdot ReLU(W_3 \cdot x) \quad (5)$$

Equation (5) shows W_3 and W_4 as 1×1 convolution kernels, W_3 reduces to one-fourth of input channels, and W_4 expands to the main path output channels. The overall framework of the Mask R-CNN-based disaster recognition model integrated with ResNet is illustrated in Figure 3.

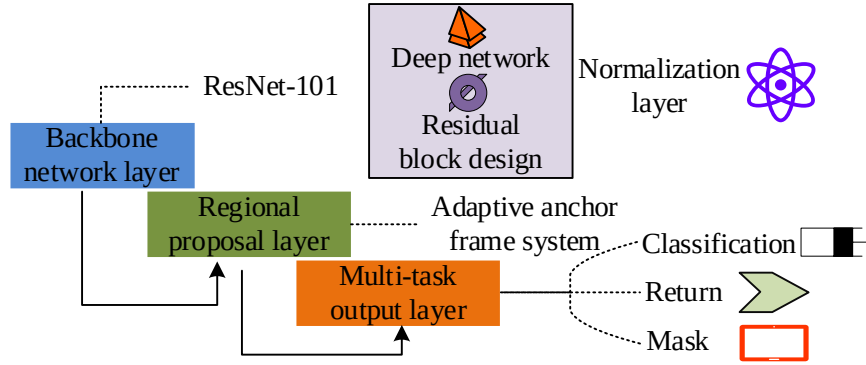


Fig. 3. Mask R-CNN disaster recognition model integrating ResNet

As shown in Figure 3, the backbone network uses ResNet-101 as the core for feature extraction. The 101-layer deep network with residual block design addresses weak feature extraction deficiencies in traditional convolutional neural networks. Batch normalization layers are added in the residual blocks of ResNet stages 3 and 4. The weighted feature fusion is calculated as shown in Equation (6) [24].

$$F_{fusion} = \alpha \cdot C3' + \beta \cdot C4 + \gamma \cdot C5' \quad (6)$$

In Equation (6), F_{fusion} indicates the fused enhanced feature map, α , β , and γ are fusion weights. $C3'$, $C4$, and $C5'$ represent aligned multi-scale feature maps. Channel attention weights are calculated as in Equation (7).

$$M_c(F) = \sigma(W_2' \cdot ReLU(W_1' \cdot GAP(F))) \quad (7)$$

Equation (7) shows F as the input feature map to the attention module. W_1' and W_2' are fully connected layer weights, and c represents the number of feature map channels. The region proposal layer designs an adaptive anchor system for diverse bridge disaster forms. The multi-task output layer includes classification, regression, and Mask branches. The model outputs disaster probabilities through activation functions and optimizes bounding box coordinates using Smooth L1 loss. The regression branch outputs pixel-level disaster locations. The Mask branch up samples to the original image size for disaster quantification, using 1×1 convolution and Sigmoid activation to output 28×28 binary segmentation masks.

3.2 Optimization of Disaster Recognition Model with CBAM

Although the Mask R-CNN disaster recognition model combined with ResNet can effectively extract disaster features, it still suffers from background interference that leads to false detections in complex mountain environments. CBAM can accurately enhance key features without significantly increasing the number of model parameters. This advantage makes CBAM well suited for mountain bridge disaster recognition tasks [25]. Therefore, to further

improve model performance, this study introduces CBAM to optimize the constructed disaster recognition model. The process of how CBAM adapts to bridge disaster requirements is shown in Figure 4.

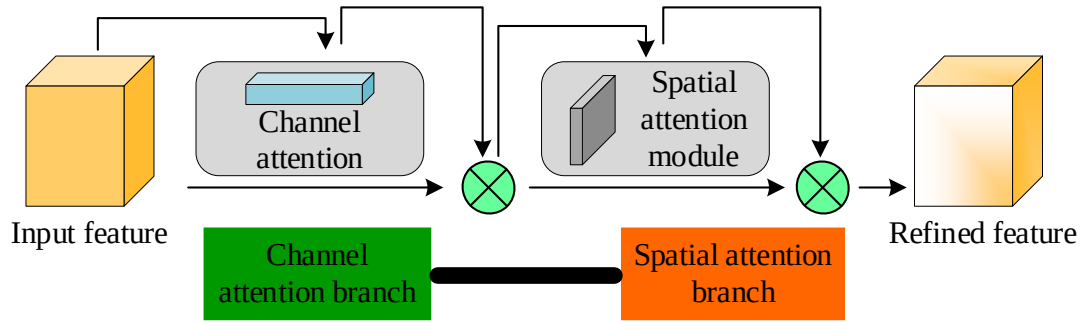


Fig. 4. Schematic diagram of CBAM adapting to bridge disaster requirements

In Figure 4, the channel attention branch in CBAM applies global average pooling to condense the feature map into a $1 \times 1 \times C$ vector, which is subsequently processed through two fully connected layers followed by a Sigmoid activation function to produce channel weights. These weights amplify the disaster-related feature channels while attenuating background information. This process enables adaptation to the mixed-channel scenes in bridge disasters. The spatial attention branch addresses the issue that disaster regions occupy a small proportion of the image and are easily covered by backgrounds. It compresses the feature map through channel pooling, and after compression to $2 \times H \times W$, applies a 1×1 convolution to generate spatial weights, achieving localization of disaster pixels. The calculation of the final enhanced feature map output by CBAM is shown in Equation (8) [26].

$$F'_{final} = F' \otimes M_c(F') \otimes M_s(F'_c) \quad (8)$$

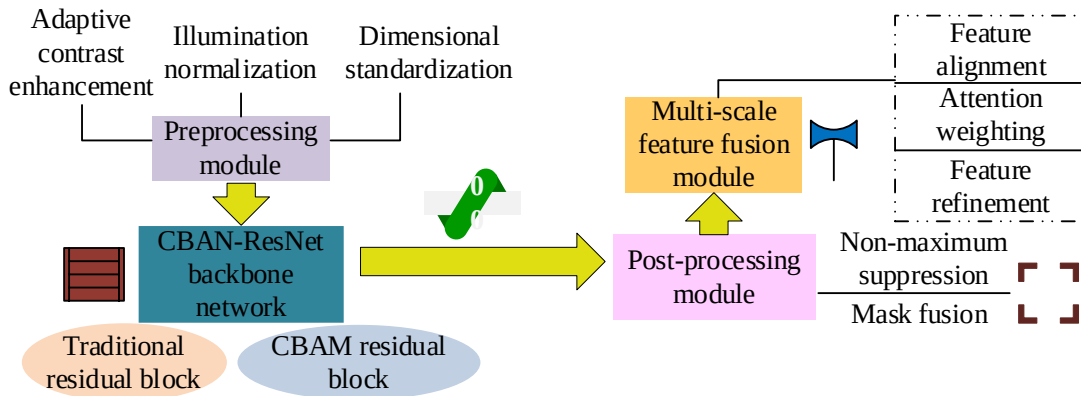


Fig. 5. Architecture of CBAM-ResNet-Mask R-CNN

In Equation (8), F' is the input feature map of CBAM, $M_c(F')$ is the channel attention weight vector, and the two $\otimes$ operations respectively perform channel-level feature selection and spatial-level feature focusing. It should be noted that Mask R-CNN as a two-stage detector imposes higher computational burden compared with one-stage YOLO-seg series models. For mountain-bridge disaster quantitative early-warning tasks, the segmentation boundary accuracy directly governs the calculation reliability of crack width, rockfall volume and subsequent structural-damage evaluation. Boundary errors from segmentation will propagate and trigger incorrect early-warning decisions. Therefore, pixel-level segmentation accuracy is prioritized in this study. Meanwhile, the dual-path residual blocks and embedded CBAM attention module optimize feature extraction and suppress redundant background computation, enabling the model to satisfy the latency requirement for mountain-bridge early warning under given hardware conditions. The study then combines CBAM, ResNet, and Mask R-CNN to construct a new algorithm named CBAM-ResNet-

Mask R-CNN, which serves as the foundation of the disaster early warning model. The architecture is shown in Figure 5.

In Figure 5, the algorithm enhances the local contrast of weak features such as crack edges, while the Retinex decomposition removes uneven lighting effects and scales the image. The CBAM-ResNet backbone serves as the feature extraction core. Stages 1–2 use traditional residual blocks to output C2 and C3 feature maps, while stages 3–4 are replaced with CBAM residual blocks to output C4 and C5 feature maps. Each CBAM residual block includes convolutional feature extraction and CBAM attention selection. The crack width is calculated as shown in Equation (9).

$$W' = \frac{N \times Re\ s}{S} \quad (9)$$

Equation (9) shows N as the pixel length of the crack in the segmentation mask, $Re\ s$ and S as the image resolution and image scaling factor, respectively. Equation (9) is derived under the premise of calibrated near-orthogonal shooting perspective. For practical inclined camera viewpoints, camera intrinsic parameters and planar homography transformation should be introduced to compensate perspective distortion. In this paper, image scaling factors are calibrated for each sampling scene using reference scale markers on bridge surfaces to mitigate measurement bias induced by oblique shooting angles. The rockfall volume is calculated as shown in Equation (10) [27].

$$V = Area \times h \times \rho \quad (10)$$

In Equation (10), $Area$ is the pixel area of the rockfall in the segmentation mask, h and ρ are the average height and shape coefficient of the rockfall. Accurate 3-D volume calculation requires depth information from LiDAR or RGB-D sensors, which are not available in this monocular RGB imaging system. Therefore, the rockfall pixel area calculated from segmentation mask serves as the primary quantitative indicator. The volume calculated in Equation (10) is an estimated reference value derived from empirical average height and shape-coefficient assumptions, rather than a strictly measured true 3-D volume. The average height and shape-coefficient parameters are derived from field statistical data of rockfall events in local mountain-bridge zones, rather than real-time depth measurement from each input image. These empirical parameters are suitable for general-scale rockfall samples in the study area. The estimated rockfall volume carries an inherent relative uncertainty of $\pm 22\%$ under different actual rockfall morphologies. Volume output serves only as auxiliary reference, and rockfall pixel area from segmentation mask acts as the primary quantitative indicator. In the multi-scale feature fusion module, the region proposal network generates disaster candidate regions based on the fused feature maps. RoI Align performs pixel-level alignment through bilinear interpolation. The classification branch outputs disaster probabilities, the localization branch outputs bounding box coordinates, and the segmentation branch generates a 28×28 binary mask through four transposed convolution layers, which is up sampled to produce pixel-level segmentation results. The post-processing module removes overlapping candidate regions using non-maximum suppression and overlays the segmentation masks onto the original image. The Mask R-CNN only outputs two-dimensional pixel-level segmentation masks of disaster regions from RGB images. Structural indicators such as bearing deformation and structural stress, as well as economic-loss estimation, cannot be directly derived from visual segmentation results alone. In this framework, visual quantitative features computed from masks (including crack length, crack width, spalling area and rockfall area) are fused with multi-source auxiliary structural monitoring data (vibration frequency and strain growth rate collected by sensors, see Equation (11)) for joint calculation, so as to invert three-dimensional structural bearing deformation and stress state. The percentage of economic-loss reduction is evaluated by inputting disaster-severity quantification results into the bridge life-cycle cost analysis model, rather than directly predicted by the instance-segmentation network. The workflow of the mountain bridge disaster early warning model is illustrated in Figure 6.

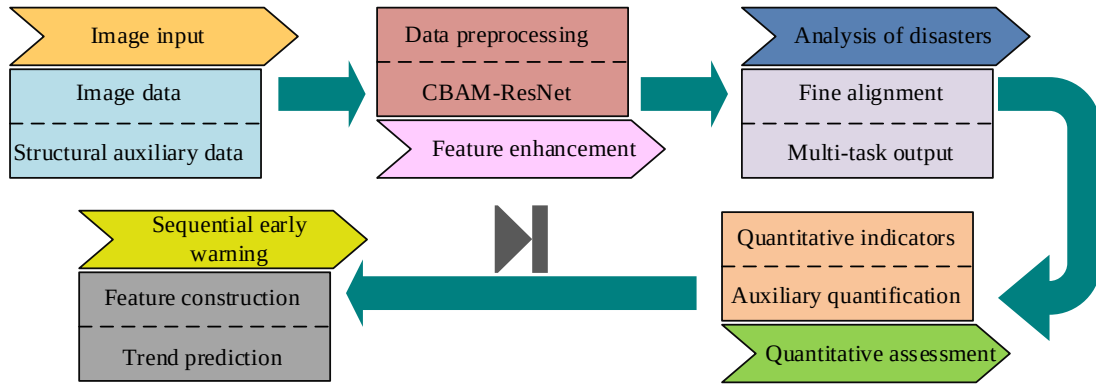


Fig. 6. Workflow of mountain bridge disaster early warning model

Figure 6 shows that the preprocessing and feature enhancement layers mainly solve the problems of complex backgrounds and weak feature extraction in mountain bridge images through data preprocessing and CBAM attention enhancement. This part optimizes images using illumination normalization, contrast enhancement, and size standardization, and filters and normalizes vibration and strain data. The calculation is shown in Equation (11).

$$I_{norm}(i, j) = \frac{I(i, j) - \mu}{\sigma' + \varepsilon} \quad (11)$$

In Equation (11), $I(i, j)$ is the grayscale value of the original image, $I_{norm}(i, j)$ represents the normalized pixel value, μ is the global mean of the image, σ' is the global standard deviation. The calculation of mask morphological filtering is shown in Equation (12).

$$m_{filtered} = Close(Open(m, k_1), k_2) \quad (12)$$

In Equation (12), m is the original segmentation mask, $m_{filtered}$ represents the filtered mask, and $Open(m, k_1)$ and $Close(m, k_2)$ are the opening and closing operations, respectively. The calculation of crack length is shown in Equation (13).

$$l = \sum_{i=1}^N Dist(s_i, s_{i+1}) \times Re s \quad (13)$$

In Equation (13), $Dist(s_i, s_{i+1})$ represents the Euclidean distance between adjacent sampling points, and s_i represents the sampling points on the crack skeleton line. CBAM-ResNet uses ResNet-101 as the backbone network. The disaster instance segmentation layer, based on the enhanced feature map, provides accurate spatial information for subsequent quantitative analysis. In the disaster quantification analysis layer, the model performs auxiliary quantification using structural data, integrating vibration frequency changes, strain growth rate, and image-based quantification results. The time-series trend prediction layer predicts disaster development trends based on multi-period quantitative data. The early warning decision output layer combines the current disaster level and future trend prediction to set multi-level warning thresholds, finally outputting visualized early warning results. Image instance-segmentation outputs visual damage features. The complete early-warning workflow follows sequential steps. Damage geometric parameters derived from segmentation are combined with multi-source sensor time-series data to calculate structural-state indicators. Disaster development trends are predicted using multi-period quantitative results, and multi-level warning thresholds are triggered to generate early-warning outputs. Pixel-level segmentation acts as the core intermediate step of the whole pipeline, not the final early-warning output. The transformation from pixel-level defect geometric features to structural stress and economic indicators adopts a data-driven correlation mapping strategy, without introducing external simulation software or digital-twin models. The defect geometric parameters (crack width, spalling area, rockfall area) output from segmentation masks are correlated with structural stress by linear fitting built upon labeled sensor monitoring datasets. The calculated

structural-stress results are further mapped to economic-loss indicators according to empirical maintenance-cost statistics. This mapping assumes a linear correlation between defect geometric size and local structural stress within the service-state range of mountain bridges, and the economic calculation keeps unit repair cost and traffic-interruption loss coefficient unchanged.

4. Performance Verification of Mountain Bridge Disaster Early Warning Model

4.1 Comparison of Model Simulation Performance

To verify the performance of the proposed mountain bridge disaster early warning model, the study selected the following three early warning models as comparison models, which were early warning models based on ResNet-Mask R-CNN-CNN, ResNet-Mask R-CNN, and Mobile Network Version 3-Mask R-CNN (MobileNetV3-Mask R-CNN). These three comparison models are set as ablation baselines. ResNet-Mask R-CNN serves as the original basic framework without our dual-path residual and CBAM improvement. ResNet-Mask R-CNN-CNN adds extra ordinary convolution branches on the basis of the basic framework to verify the effectiveness of our dual-path residual design. MobileNetV3-Mask R-CNN replaces the ResNet backbone with lightweight MobileNetV3, which is widely adopted for edge-end vision tasks in infrastructure monitoring. The study named them as Model 1, Model 2, and Model 3. Two kinds of datasets are adopted in this work. The input of Mask R-CNN is optical RGB image dataset, which is derived from the public dataset of Mountain Bridge Structure Health Monitoring Laboratory of Chongqing Jiaotong University. This image dataset contains 8264 annotated images, covering 11 mountain-bridge structures, with four damage categories: crack, concrete spalling, rockfall, and bearing deformation-related surface features. The Long-Term Guided-Wave Dataset Under Real-World Conditions contains 7.6 million one-dimensional ultrasonic guided-wave (UGW) time-series signals collected by 14 piezoelectric sensors from simulated bridge metal components. Note that these UGW signals are not fed into the Mask R-CNN network. Instead, they are fused with visual segmentation outputs in the post-processing disaster-quantification module for auxiliary structural-state inversion, and do not provide image-level labels for instance-segmentation training. Ground-truth labels for visual damage categories including crack, concrete spalling, rockfall and surface features of bearing deformation are manually annotated by two professional bridge-inspection engineers with LabelMe software. Ground-truth values of crack width, spalling area and bearing deformation are acquired through total-station and high-precision laser-ruler field measurements at real mountain-bridge sites. Structural-stress ground-truth originates from strain sensor readings of test components. Manual annotation error for image masks is within 3 pixels, and physical measurement uncertainty for crack width reaches $\pm 0.5 \mu\text{m}$. The reported $0.68 \mu\text{m}$ prediction error of crack width is calculated on scenes with high-precision on-site reference measurements. The image dataset is split into training set and test set at a ratio of 7:3 by whole bridge-scenario units instead of random sampling of individual images. All images collected from one specific mountain-bridge scenario are entirely allocated to either the training set or the test set. Images originating from the same bridge-scenario do not appear in both subsets, avoiding data leakage induced by inter-correlated samples from identical bridges, monitoring periods or damage events. The ultrasonic guided-wave dataset follows the identical scenario-based splitting rule. The experimental hardware adopted equipment with CPU of Intel Core i9-13900K (3.0GHz) and GPU of NVIDIA RTX 4090 (24GB video memory), with memory and storage of 64GB and 2TB SSD respectively. In terms of software, it was based on Ubuntu 22.04 system, used Python 3.9 as the development language and employed PyTorch 2.0 deep learning framework, adopted OpenCV 4.8.0 to process image data, and used Matplotlib 3.7.1 for visualization of results. The bounding box regression loss weight and Mask segmentation loss weight were 1.0 and 2.0 respectively, with 100 iterations. The learning rate decayed linearly in the first 50 epochs, and the learning rate for the last 50 epochs was $1\text{e-}5$. Detailed training hyper-parameter settings are summarized in Table 1 for full experimental reproducibility.

There is no feature fusion inside Mask R-CNN network backbone between optical image and UGW signals; multi-source fusion is implemented only in the post-processing disaster-quantification module. All test images adopt the resolution of 1920×1080 pixels. Under the given experimental hardware, the average time consumption of each link is as follows: image pre-processing takes

28 ms, model inference takes 53 ms, post-processing including mask morphological filtering and multi-source data fusion takes 41 ms. In offline laboratory testing without field data-transmission overhead, the total processing latency is 122 ms. Data-transmission delay is not included in this offline test; in practical on-site deployment, additional sensor-to-device transmission latency will be introduced. The reported sub-one-second early-warning response time refers to offline laboratory processing latency excluding remote wireless transmission. The study first compared the Pearson correlation coefficients of each model under different early warning lead times, and the results were shown in Figure 7.

Table 1. Comparison of smooth substrate classes relevant to adhesive debonding

Hyper-parameter	Value
Optimizer	SGD with momentum (momentum=0.9)
Weight decay	0.0001
Total epochs	100
Batch size	2
Initial learning rate	0.001
Learning rate scheduler	Linear decay: decay in first 50 epochs, final lr =1e-5 for epoch 51-100
Anchor box aspect ratios	[0.5, 1, 2]
NMS threshold (proposal)	0.7
NMS threshold (detection output)	0.5
RoIAlign sampling points	4
Classification loss weight	1.0
Bounding-box regression loss weight	1.0
Mask segmentation loss weight	2.0

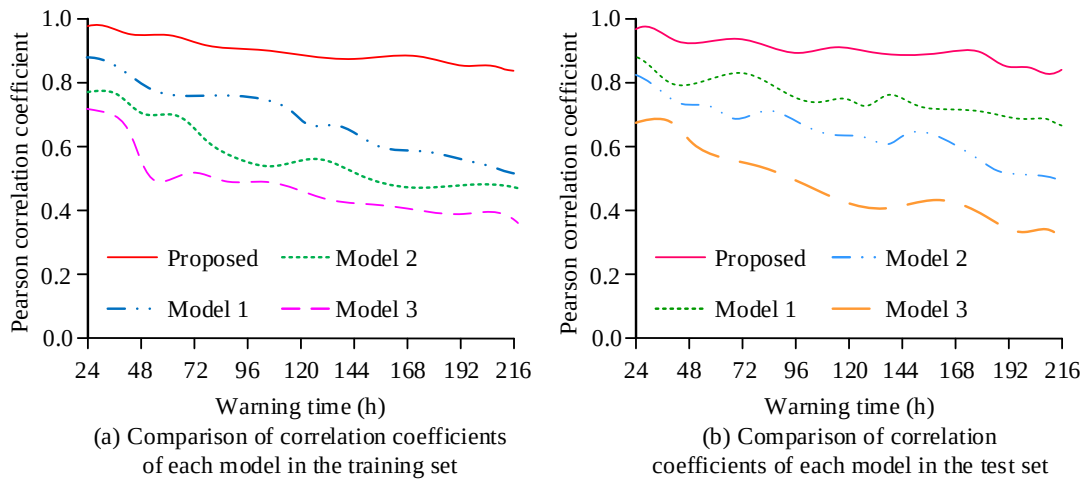


Fig. 7. Comparison of correlation coefficients

Figure 7(a) showed that in the training set, when the early warning lead time was 144 h, the Pearson correlation coefficient of the proposed model was 0.916. At this time, the Pearson correlation coefficients of Model 1 and Model 2 were 0.682 and 0.547, and the Pearson correlation coefficient of Model 3 was 0.473. In Figure 7(b), the Pearson correlation coefficient of the proposed model remained at a high level under different early warning lead times and was more stable compared to the comparison models. The above results indicated that the proposed model performed best in terms of correlation in disaster early warning and demonstrated stronger stability and precision. Subsequently, the study tested the economic loss reduction of the four models in mountain bridge disaster early warning tasks, and the results were shown in Figure 8. The percentage of economic-loss reduction reflects the relative improvement of expected economic loss under the model-driven early-warning mode compared with the traditional periodic manual-inspection warning mode. The baseline reference corresponds to conventional regular manual inspection for mountain bridges. The comprehensive economic loss covers bridge maintenance expenses, traffic interruption losses and structural repair costs. Relevant economic parameters adopt statistical data of mountain-bridge operation and maintenance in Southwest China. This

indicator is calculated under fixed assumptions of disaster occurrence probability and unit repair cost, and it only represents relative improvement effect instead of absolute monetary loss value.

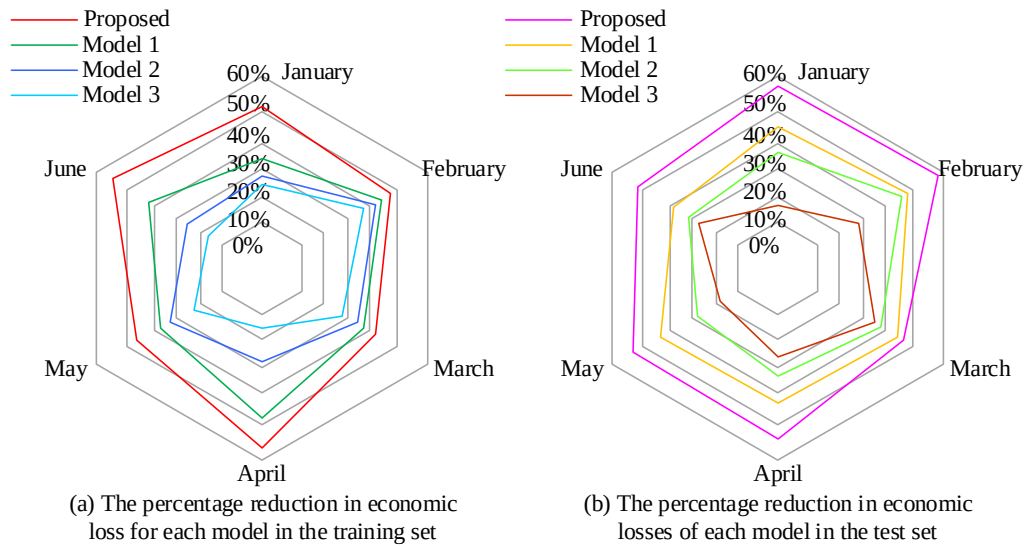


Fig. 8. Economic loss reduction in training set and test set

In Figure 8(a), in the training set, the economic loss reduction of the proposed model reached 51.23% in January. The economic loss of Model 1 decreased by 34.78% in January, while Model 2 and Model 3 decreased by 28.64% and 24.89% respectively, all lower than the proposed model. This indicated that the proposed model had a positive effect on reducing the economic impact of disasters. As shown in Figure 8(b), the proposed model had the highest economic loss reduction percentage in different months in the test set, especially in February, reaching 58.46%, while the highest economic loss reduction percentage of the comparison models was only 48.21%. The above results showed that the proposed model could more effectively reduce the economic losses caused by mountain bridge disasters. Subsequently, the study compared the prediction results of bridge structural stress for the four models, as shown in Figure 9.

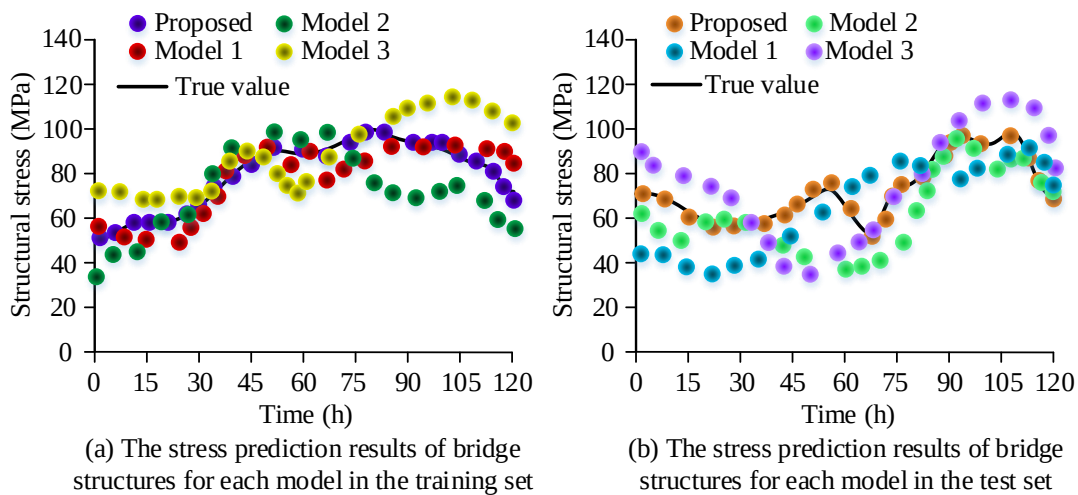


Fig. 9. Bridge structural stress prediction results of each model

From Figure 9, it could be seen that the bridge structural stress prediction values of the proposed model were closer to the true values. In the training set, the proposed model exhibited only a 0.53 MPa deviation between the predicted and actual bridge structural stress at 90 h, whereas Model 2 showed a much larger deviation of 23.47 MPa. In the test set, the prediction points of the proposed model closely fitted the true value curve and performed better in capturing stress patterns. From this, it could be known that the proposed model had more superiority in the stress prediction task of mountain bridge disaster early warning and could provide more reliable technical support for structural safety assessment. Finally, the study compared the mountain bridge disaster early warning effects of the four models, and the results were shown in Table 2.

Table 2. Comparison of mountain bridge disaster early warning effects

Dataset	Index	Proposed	Model 1	Model 2	Model 3
Training	Disaster early warning response time (s)	0.85	1.23	1.07	1.52
	Early warning false alarm rate (%)	3.21	8.57	11.34	15.68
	Rockfall early warning recall rate (%)	92.35	81.62	75.88	68.45
Test	Disaster early warning response time (s)	0.69	1.35	1.18	1.62
	Early warning false alarm rate (%)	2.56	8.98	12.84	16.82
	Rockfall early warning recall rate (%)	93.64	80.54	73.25	65.84

From Table 2, it could be seen that in the training set, the disaster early warning response time of the proposed model was only 0.85 s, and its bearing deformation over-threshold early warning false alarm rate and small-volume rockfall early warning recall rate were 3.21% and 92.35% respectively, all superior to the comparison models. In the test set, all performance indicators of the proposed model were still superior to the comparison models, and its small-volume rockfall early warning recall rate was the highest, reaching 93.64%. Here, the recall rates of Model 1 and Model 2 were 80.54% and 73.25% respectively, and the recall rate of Model 3 was the lowest at 65.84%. The above results indicated that the proposed model could quickly and accurately provide early warning for mountain bridge disasters and had significant efficiency.

4.2 Analysis of Model Practical Application Effects

To further verify the performance effects of the disaster early warning model in practical applications, the study selected the public dataset from the Mountain Bridge Structure Health Monitoring Laboratory of Chongqing Jiaotong University to conduct performance tests on each model. The collection of this dataset strictly matched the actual service scenarios of mountain bridges and was synchronously associated with disaster quantification annotation information of key parts of bridges. The study first compared the crack width prediction results of different models in mountain bridge disaster early warning tasks, and the results were shown in Figure 10.

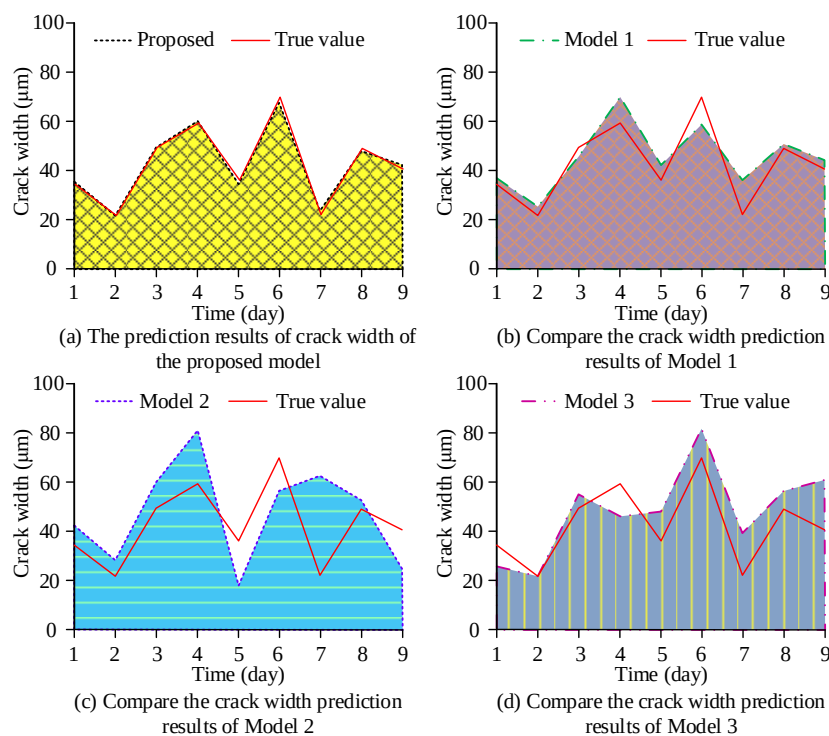


Fig. 10. Comparison of crack width prediction results

In Figure 10, when the time was at the 9th day, the proposed model showed a difference of only 0.68 μm between the predicted and actual crack widths. The prediction range of Model 1 was

relatively wide, and the difference reached 1.53 μm . The prediction values of Model 2 differed greatly from the true values in most time periods. The prediction values of Model 3 fluctuated greatly, and when the time was at the 9th day, the difference reached 20.06 μm . The above results indicated that the proposed model could more accurately provide key data on crack development for bridge disaster early warning. Next, the study tested the monitoring accuracy of each model for two indicators: bridge bearing deformation and the proportion of concrete spalling area on the bridge surface, and the results were shown in Figure 11.

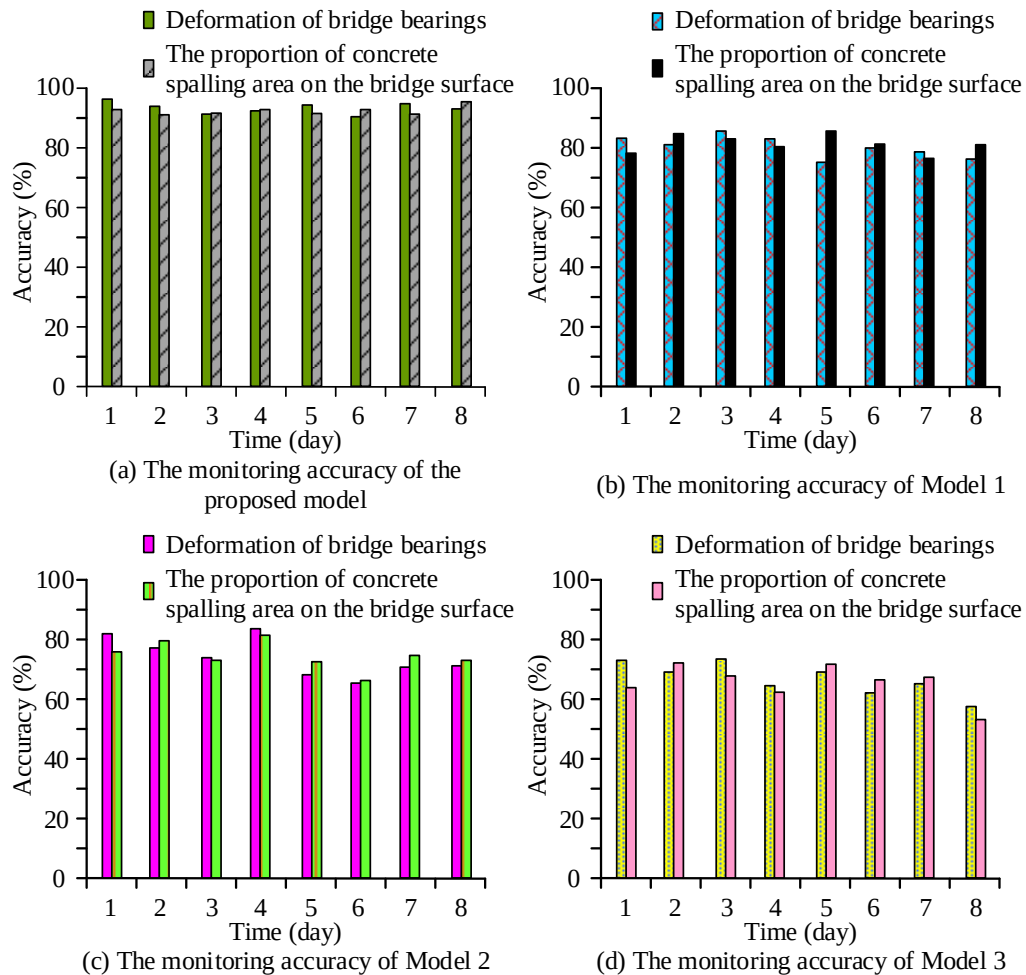


Fig. 11. Comparison of monitoring accuracy for different indicators

In Figure 11, the monitoring accuracy of the proposed model for both bridge bearing deformation and the proportion of concrete spalling area on the bridge surface reached over 90%. When the time was the 5th day, the monitoring accuracy of Model 1 for the proportion of concrete spalling area on the bridge surface was only 85.13%. From the perspective of bridge bearing deformation monitoring accuracy, when the time was at the 6th day, the monitoring accuracy of Model 2 and Model 3 were 64.91% and 62.82% respectively. The above data results indicated that the proposed model could more accurately monitor mountain bridge bearing deformation and concrete spalling on the bridge surface, and performed better in bridge structural damage monitoring tasks. Finally, the study compared the comprehensive performance of each model in mountain bridge disaster early warning tasks, and the results were shown in Table 3.

From Table 3, it could be seen that from the perspective of model inference speed, the proposed model reached 18.72 fps, and its crack identification accuracy in weak light environments was also the highest at 88.56%. Meanwhile, the bridge bearing deformation prediction error of the proposed model was only 0.32mm, indicating that this model could effectively ensure the accuracy of structural safety assessment.

Table 3. Comparison of comprehensive disaster early warning indicator results

Indicator	Proposed model	Model 1	Model 2	Model 3
Inference speed (fps)	18.72	15.36	16.89	12.75
mAP@0.5 (%)	92.17	84.33	80.21	74.58
mAP@0.5:0.95 (%)	76.42	67.25	63.14	58.77
Crack identification accuracy in weak light environments (%)	88.56	76.23	72.91	69.37
Bridge bearing deformation prediction error (mm)	0.32	0.85	1.21	1.57
F1 score for concurrent multi-disaster identification	0.93	0.82	0.75	0.68

The proposed model had the highest F1 score for concurrent multi-disaster identification at 0.93, while the concurrent multi-disaster identification F1 score of Model 3 was only 0.68. In terms of standard instance segmentation evaluation on test set, the proposed model achieves mAP@0.5 of 92.17% and mAP@0.5:0.95 of 76.42%, demonstrating its competitive capability in accurate boundary localization for bridge disaster targets. The above data results indicated that the proposed model had superior comprehensive performance and had good accuracy and efficiency in mountain bridge disaster early warning. Detailed precision, recall and pixel-level F1 for each bridge defect category are listed in Table 4. YOLO11-seg is introduced as a modern one-stage instance-segmentation baseline for comparative evaluation to analyze the trade-off between mask boundary accuracy and inference speed.

Table 4. Instance-segmentation performance comparison on test set

Damage category	Metric	Proposed model	Model 1	Model 2	Model 3	YOLO11-seg	Swin-Transformer-Mask R-CNN
Crack	Precision (%)	90.12	81.77	77.24	70.16	85.41	87.33
	Recall (%)	92.88	79.64	72.17	64.22	86.27	89.15
	Pixel-level F1	0.89	0.79	0.74	0.67	0.83	0.85
Concrete spalling	Precision (%)	91.34	82.45	78.31	71.25	86.52	88.46
	Recall (%)	94.15	81.22	74.36	66.13	87.44	90.31
	Pixel-level F1	0.91	0.80	0.76	0.69	0.84	0.86
Rockfall	Precision (%)	89.66	80.83	76.42	69.47	84.77	86.62
	Recall (%)	93.64	80.54	73.25	65.84	87.52	90.33
	Pixel-level F1	0.88	0.78	0.73	0.66	0.82	0.84
Bearing-deformation-related surface feature	Precision (%)	88.74	79.91	75.58	68.61	83.94	85.77
	Recall (%)	91.26	78.43	71.08	63.17	84.86	87.42
	Pixel-level F1	0.87	0.77	0.72	0.65	0.81	0.83

Five independent repeated training and evaluation runs are conducted with different random initialization. Key segmentation metrics exhibit limited fluctuation: the standard deviation of Mask-AP is 0.72 %, and the standard deviation of pixel-level F1 is 0.013. Such small variability demonstrates the stability of the proposed model under the given dataset and hyper-parameter setting. From Table 4, per-defect-category metrics consistently show that the proposed two-stage

CBAM-ResNet-Mask R-CNN achieves higher pixel-level F1 than YOLO11-seg across all damage categories, which demonstrates its advantages in mask boundary segmentation accuracy benefited from RoI-Align operation. By comparison, YOLO11-seg as a modern one-stage model exhibits faster inference speed but suffers degradation in fine boundary segmentation performance. Swin-Transformer-Mask R-CNN obtains competitive segmentation results, yet it brings heavier computational overhead. For mountain-bridge scenes with small-size and irregular defects, higher-quality mask boundaries are critical for subsequent geometric quantification of crack width and rockfall area.

5. Summary

This study addresses the adaptability and real time engineering requirements for mountain bridge structural defect monitoring. A multi modal early warning framework based on CBAM enhanced ResNet Mask R CNN is proposed for instance segmentation and quantitative evaluation of bridge local defects including cracks, concrete spalling and rockfall impact. Benefiting from dual path residual blocks and CBAM attention enhancement, the proposed method suppresses complex mountain background interference and improves detection performance for weak small-scale defects. Under test set conditions, the structural stress prediction deviation reaches only 0.53 MPa, and the recall rate for small scale rockfall events achieves 93.64%. By fusing visual segmentation outputs and ultrasonic guided wave time series data, the framework realizes the mapping from pixel level defect geometry to structural state indicators and provides technical support for mountain bridge early warning practice.

This work acknowledges that insufficient depth of multi-source data fusion may lead to incomplete disaster-trend prediction. Beyond that, this method exists inherent hardware-driven limitations originating from monocular optical imaging. Without active depth sensors such as LiDAR or RGB-D cameras and stereo-vision systems, accurate absolute three-dimensional rockfall-volume quantification cannot be guaranteed. Volume estimation calculated from 2-D segmentation masks relies on empirical statistical parameters and will produce non-negligible estimation uncertainty. In addition, the optical-vision-dominated framework is vulnerable to harsh mountain environmental conditions. Heavy fog, rainfall and snow cover will degrade image quality and may reduce the detection and segmentation reliability of visual models. Although ultrasonic guided-wave sensor signals are introduced as auxiliary inputs, they cannot fully compensate the performance drop of visual branches under extreme weather.

Future work will further construct correlation models between multi-dimensional monitoring data and structural-defect evolution trends. To tackle the insufficient multi-source-fusion depth, two promising computational frameworks will be considered. The structural-health-monitoring digital-twin (SHMDT) framework can couple visual defect measurements with physical structural simulation, mapping short-term visual damage observations to long-term structural bearing-capacity evolution. Physics-informed neural networks (PINNs) will also be adopted, embedding structural mechanical prior knowledge into data-driven model training, to reduce the dependence of quantitative assessment on purely empirical fitting relations. Further field-site tests under foggy, rainy and snowy weather will also be carried out to improve the robustness of the whole early-warning system.

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Algorithm improvement and simulation verification for mobile robot path planning in complex environments

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Abstract

Mobile robots are increasingly used, but navigation in complex dynamic environments suffers from issues like local optima, poor obstacle avoidance, and difficulty in coordination. This paper proposes a field swarm cooperative algorithm that integrates improved potential field and Particle Swarm Optimization. It uses simulated annealing to escape local optima, leverages the potential field force to enhance obstacle avoidance, and establishes a three-objective cooperative framework. Simulation outcomes indicate that in simple obstacle environments, the proposed algorithm successfully overcomes the local minima problem in U-shaped traps using the traditional Artificial Potential Field method. In the complex static library scenario, the path length of the algorithm proposed by the study is 218.7 m, the average safe distance is 0.56 m, and the smoothness is 3.89 rad. All the indicators have achieved balanced and superior results. Compared with the single path length optimization mode, the multi-objective collaborative optimization mode, at the cost of a slight increase in path length ($p > 0.05$), has achieved significant improvements in the average safe distance and smoothness ($p < 0.001$). The simulation results show that the algorithm proposed in this paper is significantly superior to the comparison methods, and multi-objective equilibrium, and has important practical value for improving the intelligence level and task execution efficiency of robots.

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1. Introduction

With the swift advancement of AI, the Internet of Things and automation technology, mobile robots are gradually becoming core equipment in realms like intelligent manufacturing, logistics and distribution, medical services and home services [1-2]. Their autonomous navigation ability in complex environments, especially their ability to dynamically avoid obstacles and plan multi-objective collaborative paths, has become a key indicator for measuring the level of robot intelligence. In dynamic and unstructured environments, robots not only need to be able to quickly plan a collision-free path from the starting point (SP) to the target point (TP), but also need to achieve a balance between multiple objectives like path length, safety, smoothness and real-time performance to improve task execution efficiency, lower energy consumption and ensure operational safety [3-4].

However, traditional path planning (PP) algorithms still have significant shortcomings when dealing with complex environments. For example, the computational burden of the graph search-based A* algorithm increases sharply after grid refinement; the Artificial Potential Field (APF) method guides robot movement by constructing a virtual potential field (PF), but it is prone to getting trapped in local minima; while the Rapidly-exploring Random Tree (RRT) algorithm based on sampling performs path search by randomly sampling the state space, which can effectively handle high-dimensional space and complex constraints and has probabilistic completeness, but

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the planning results are random, the path is usually non-optimal and not smooth, and the sampling efficiency may decrease in complex environments [5]. Furthermore, existing research often focuses on a single objective (such as the shortest path), neglecting path safety, smoothness, and energy consumption coordination, making it difficult to achieve stable, efficient, and smooth navigation in real dynamic environments [6-7].

To address the aforementioned problems, this study focuses on PP for mobile robots in complex environments. An Artificial Potential Field and Swarm Optimization (ASO) fusion optimization algorithm is designed, combining Particle Swarm Optimization (PSO) with APF to optimize the path length, safety, and smoothness. To verify the algorithm's effectiveness and practicality, a simulation model of a Mecanum wheel omnidirectional mobile robot was built based on a Robot Operating System (ROS). Multi-scenario experiments were conducted in the Gazebo environment, and the results were compared with traditional methods.

This study addresses robot PP in complex environments by proposing an ASO, combining PSO with APF to optimize paths, using ROS modeling, and conducting multi-scenario experiments with Gazebo to compare with traditional methods.

2. Related Work

2.1 PP Method Based on Graph Search and Sampling

Graph search and sampling methods are widely used in robot PP due to their clear structure and ease of implementation. T. Ni et al. proposed the adaptive weighted jump Theta algorithm, which combines the advantages of Theta algorithm and jump point search to improve the real-time performance and robustness of complex scenarios [8]. T. T. C. Giang et al. proposed the fast travel fireworks algorithm to solve the problems of multi-objective PP tree expansion conflict and time consumption, and reduce cost and time [9]. Y. Huang et al. proposed the multi-strategy bidirectional RRT algorithm, which integrates multiple strategies to shorten the execution time and number of nodes [10]. Y. Kim et al. studied the energy optimization PP of differential drive wheeled robots, derived the optimal conditions, and provided theoretical support [11].

2.2 PP Methods Based on Intelligent Optimization and Machine Learning

In recent years, intelligent optimization algorithms and deep reinforcement learning have become research hotspots. X. Wang et al. [12] introduced a multi-strategy improvement to the ant colony algorithm, introducing APF repulsion field and other methods to optimize indicators such as path length; B. Wang et al. [13] introduced a multi-strategy improvement to the golden jackal optimization algorithm, introducing nonlinear factors and other methods to improve global convergence and dynamic obstacle avoidance (OA) capabilities. In the field of deep reinforcement learning, C. Y. Sheng et al. [14] addressed the problem of low exploration efficiency of Deep Q-Network (DQN) in complex environments by introducing an improved action selection strategy and a multi-objective fusion reward function, achieving faster convergence speed and higher learning efficiency. J. P. Carvalho et al. [15] proposed a zero-shot coverage PP framework based on deep reinforcement learning. Through course learning and environment randomization, the agent could generalize to different map sizes and subtasks, and its performance far exceeded that of traditional heuristic algorithms. In view of the problems of long path, slow convergence, non-smoothness and oscillation in existing algorithms, S. Kumar et al. [16] introduced an intelligent sparrow search algorithm. By integrating Steiner midpoint smoothing strategy and predator distance neighborhood search, it performed well in terms of path smoothness and convergence speed. D. Lestari et al. [17] proposed a genetic algorithm with dynamic crossover and mutation rate to address the efficiency problem of path search for wheeled mobile robots in grid space. By adaptively adjusting the probability of genetic operation, the algorithm could better balance global exploration and local development in the search process, thereby improving the efficiency and effect of path search. Simulation results showed that the method could effectively shorten the path length and improve the convergence speed in different grid environments.

In summary, most PP algorithms focus on single-objective optimization such as minimizing path length or computation time, lacking a systematic trade-off mechanism for the contradictory

relationships between multiple indicators like path length, safety, and smoothness. Secondly, while existing improved algorithms perform well in static obstacle scenarios, their real-time response capability to dynamic obstacles is insufficient, making it difficult to achieve fast and smooth OA trajectory replanning in complex dynamic environments. To address the problems of local minima, insufficient dynamic OA capability, and difficulties in multi-objective collaborative optimization in mobile robot PP under complex environments, this research proposes a PP method that integrates improved APFs and multi-objective PSO: At the algorithm level, Simulated Annealing (SA) is introduced to solve the local minima problem of traditional APFs, and dynamic OA capability is enhanced by incorporating the PF force into the particle swarm velocity update equation; at the optimization mechanism level, an optimization system is constructed with path length, safety, and smoothness as objectives, and roulette wheel selection is used to achieve multi-objective collaborative optimization

3. Methods and Materials

3.1. Improved APF Method

APF is a classic local PP method [18]. It realizes collision-free PP from the SP to the TP. In two-dimensional space, the gravitational PF function is typically formulated as a quadratic function that depends on the distance separating the robot's current location and the TP, as presented in Equation (1).

$$U_{att}(X) = \frac{1}{2} K_{att} \cdot \rho^2(X, X_g) \quad (1)$$

In Equation (1), $U_{att}(X)$ is the gravitational PF value of the robot at position X ; K_{att} is a positive proportionality coefficient, called the gravitational gain constant; $\rho(X, X_g) = \|X - X_g\|$ is the Euclidean distance between the robot's current position X and the TP X_g . Gravity $F_{att}(X)$ can be obtained from the negative gradient of the gravitational PF, as shown in Equation (2).

$$F_{att}(X) = -\nabla U_{att}(X) = -K_{att} \cdot \rho(X, X_g) \cdot \frac{\partial \rho}{\partial X} \quad (2)$$

The repulsive PF function is usually designed to only work when the robot enters the range of the obstacle's influence, and its form is shown in Equation (3).

$$U_{rep}(X) = \begin{cases} \frac{1}{2} K_{rep} \left(\frac{1}{\rho(X, X_o)} - \frac{1}{\rho_0} \right)^2, & \rho(X, X_o) \leq \rho_0 \\ 0, & \rho(X, X_o) > \rho_0 \end{cases} \quad (3)$$

In Equation (3), $U_{rep}(X)$ is the repulsive PF value; K_{rep} is the repulsive gain constant; X_o is the obstacle position; $\rho(X, X_o)$ is the distance from the robot to the obstacle; ρ_0 is the maximum influence radius of the obstacle. The precise expression of the repulsive force (RF) vector is given by Equation (4).

$$F_{rep} = K_{rep} \left(\frac{1}{\rho(X, X_o)} - \frac{1}{\rho_0} \right) \frac{1}{\rho^2(X, X_o)} \nabla \rho, \quad \rho(X, X_o) \leq \rho_0 \quad (4)$$

In Equation (4), $\nabla \rho$ is the unit direction vector pointing from the obstacle to the robot. The $F_{att}(X)$ direction points from the robot's current position to the TP.

When the robot is within the influence range of the obstacle, the repulsive PF generates an RF $F_{rep} = -\nabla U_{rep}(X)$ pointing in the opposite direction to the obstacle. The total PF $U(X)$ experienced by the robot in the environment is the sum of the gravitational PF and the repulsive PFs of all obstacles. The corresponding total resultant force $F(X)$ can also be expressed similarly, and the calculation method is presented in Equation (5).

$$\begin{cases} U(X) = U_{att}(X) + \sum_{j=1}^n U_{rep}^j(X) \\ F(X) = F_{att}(X) + \sum_{j=1}^n F_{rep}^j(X) \end{cases} \quad (5)$$

The robot moves along the direction of the total resultant force until it reaches the TP. Although APF is widely used due to its simple structure and high computational efficiency, it may become unreachable when there are dense obstacles or obstacles near the target [19-20]. Figure 1 summarizes three typical cases of this problem. In Figure 1(a), the robot stops moving because the RFs cancel each other out; in Figure 1(b), when there are obstacles near the TP, the attraction and repulsion forces may reach a balance, making the target unreachable; in Figure 1(c), when there are U-shaped obstacles in the environment, the robot may be trapped in the concave area and unable to escape.

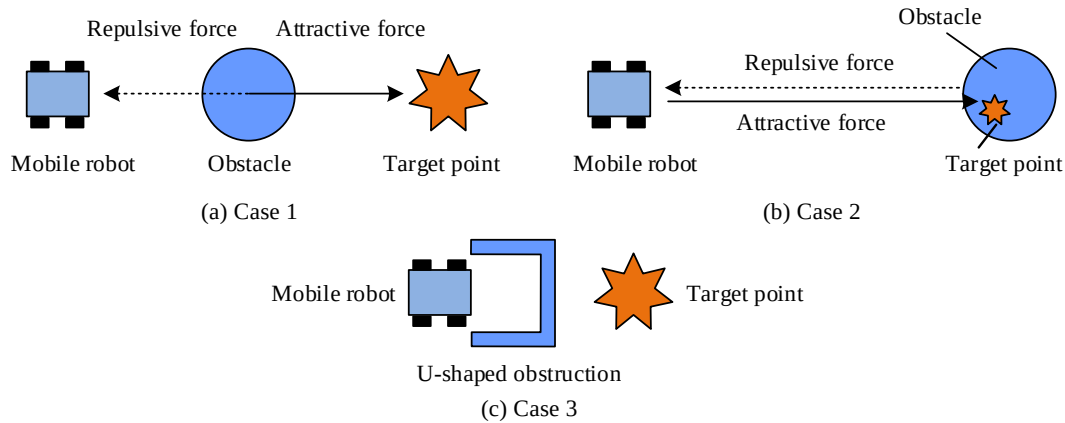


Fig. 1. The occurrence of local minima

To overcome the local minima problem, the SA algorithm was introduced into the APF method, and the SA-APF algorithm was introduced. In the SA-APF algorithm, when the robot is detected to be trapped in a local minimum (such as the position change being less than the threshold and the resultant force being close to zero in several consecutive iterations), the algorithm randomly generates multiple disturbance points near the current robot position [21]. Each disturbance point will exert a virtual RF on the robot, and the magnitude and direction of the RF are controlled by the temperature parameter of SA. When the initial temperature is high, the amplitude of the RF generated by the disturbance point is large, and the probability of the algorithm accepting unfavorable disturbances is also high, thus effectively breaking the original force balance state. As the iteration proceeds, the temperature decreases exponentially, as shown in Equation (6).

$$T_{k+1} = \alpha \cdot T_k \quad (6)$$

In Equation (6), T_k represents the temperature at the k -th iteration; α is the annealing rate constant. After the temperature decreases, the amplitude of the random perturbation gradually reduces, ensuring that the algorithm can converge stably in the later stage. The perturbation acceptance adopts the Metropolis criterion, with the acceptance probability being $P_{accept} = \exp(-\Delta F/T_k)$, where $\Delta F = \|F_{perturbed}\| - \|F_{current}\|$ and $F_{perturbed}$ are the resultant forces acting on the robot after the perturbation, and $F_{current}$ is the current resultant force. When $\Delta F < 0$, the acceptance probability is 1. At each temperature, a maximum of 8 perturbations are attempted. If it is not accepted for 3 consecutive times, the temperature is reduced earlier. As the temperature decreases, the amplitude of the random perturbation gradually reduces, ensuring that the algorithm can converge stably in the later stage. The force analysis of the random disturbance point is shown in Figure 2. The direction of the resultant force has changed, guiding the robot away from the local minima.

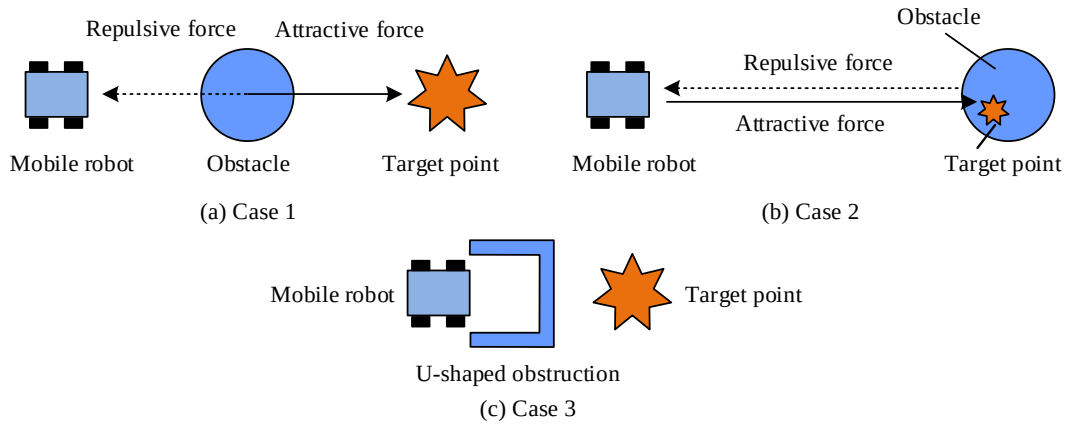


Fig. 2. Force analysis of random TPs

The complete process of the SA-APF algorithm is presented in Figure 3. The algorithm first initializes the robot's position, TP, obstacle information and SA parameters. In each iteration, the resultant force acting on the robot is calculated. If the resultant force is zero and the robot has not reached the TP, it is determined that the robot has fallen into a local minimum and the random perturbation point generation and temperature update mechanism are activated; otherwise, the robot moves forward in the direction of the resultant force [22]. When the perturbation is activated, the total resultant force is $F_{total}^{SA} = F_{att} + \sum F_{rep} + \sum F_{dist}$, where the magnitude of the repulsive force of the perturbation point is $\|F_{dist}\| = 0.5 \cdot T_k$ and the direction is from the perturbation point to the robot. The above process is repeated until the robot reaches the TP or reaches the maximum number of iterations.

The path generated by SA-APF may still have local inflections or unevenness, which can lead to frequent acceleration, deceleration, and turning during robot movement, increasing energy loss and mechanical wear. To address this, the study further introduces a moving spline smoothing algorithm to optimize the path post-processing. The moving spline smoothing algorithm first uses the moving average method to perform coarse smoothing on the original path points. For a sequence $P = \{p_1, p_2, \dots, p_n\}$ of n path points, the coordinates of the smoothed points are calculated by the average of the five adjacent points, as shown in Equation (7).

$$p_i^{\text{smoothed}} = \frac{p_{i-2} + p_{i-1} + p_i + p_{i+1} + p_{i+2}}{5}, \quad i = 3, 4, \dots, n-2 \quad (7)$$

Boundary points are averaged with fewer points or the original value is retained. Then, the smoothed path points are used as control points, and cubic spline interpolation is used to generate a continuous smooth curve. The cubic spline function constructs a cubic polynomial between every two adjacent control points and ensures that the first and second derivatives are continuous at the connection points, thereby obtaining a globally smooth path [23]. The general form of the spline interpolation function is shown in Equation (8).

$$S_i(x) = a_i(x - x_i)^3 + b_i(x - x_i)^2 + c_i(x - x_i) + d_i, \quad x \in [x_i, x_{i+1}] \quad (8)$$

In Equation (8), x_i is the x -coordinate of the i th path point; a_i , b_i , c_i , and d_i are undetermined coefficients, determined by the continuity of function values, the continuity of the first derivative, the continuity of the second derivative, and the natural boundary conditions (the second derivative at the TPs is zero). By solving the linear equation system, the spline functions of each segment can be obtained, thereby generating dense, smooth path points.

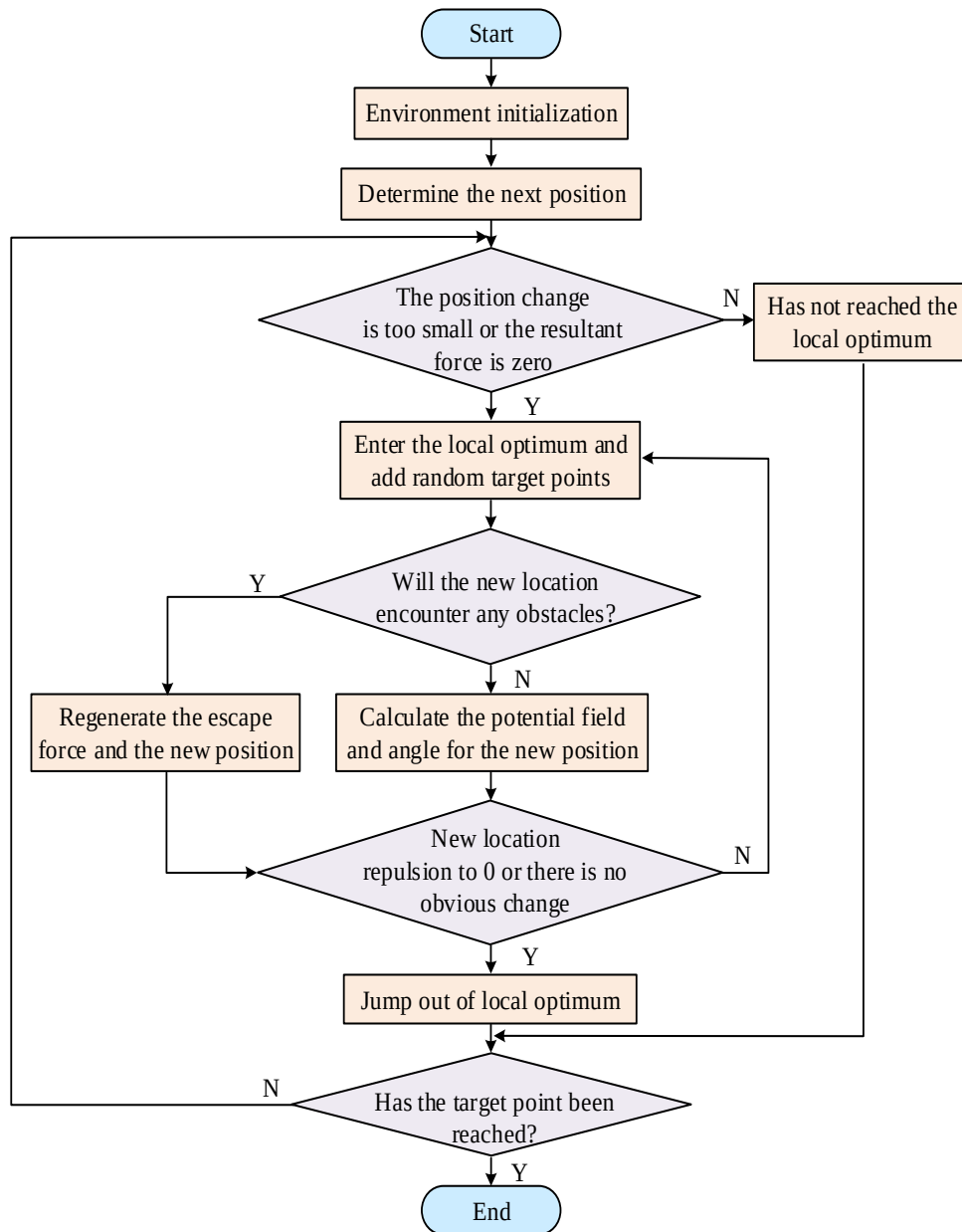


Fig. 3. Flowchart of the SA-APF algorithm

3.2 Field-Group Fusion Multi-Objective Optimization (MOO) Algorithm

Based on the improvement of the APF method to effectively overcome the local minima problem, in order to achieve the collaborative optimization among multiple objectives such as path length, safety and smoothness, a field swarm fusion algorithm integrating APF and PSO is further proposed. The fundamental concept behind this algorithm is to organically combine the local real-time OA capability based on physical PF in the APF algorithm with the global heuristic search capability based on swarm intelligence in the PSO algorithm. An MOO framework is established, and a population-based efficient solution search is introduced to obtain the optimal solution set for the balance of the three objectives, thus enabling the planning of optimal paths for robots in complex scenarios [24-25].

To quantitatively evaluate the merits of each candidate path (i.e., the path represented by the particle position) during the PSO iteration process, it is necessary to integrate the three objectives of path length, safety, and smoothness into a scalar fitness function. The study uses a linear weighted sum method to construct the fitness function, and its expression is shown in Equation (9).

$$\begin{cases} F = \omega_1 \cdot L + \omega_2 \cdot \frac{1}{S_1} + \omega_3 \cdot S_2 \\ L = \sum_{i=1}^{n-1} \sqrt{(x_{i+1} - x_i)^2 + (y_{i+1} - y_i)^2} \\ S_1 = \sum_{i=1}^n d_i \\ S_2 = \sum_{i=2}^{n-1} \theta_i \end{cases} \quad (9)$$

In Equation (9), F represents the fitness value of the particle; the smaller the value, the better the overall performance of the corresponding path. L is the path length objective function, representing the total Euclidean distance of the path. S_1 is the path safety objective function, representing the sum of distances d_i between each point on the path and the nearest obstacle. Since a larger safety value indicates a better path, and the fitness function aims to minimize it, its reciprocal is used for conversion. S_2 is the path smoothness objective function, representing the sum of all turning angles θ_i in the path. ω_1, ω_2 , and ω_3 are the weight coefficients for the corresponding objectives.

Due to the significant differences in the dimensions and numerical magnitudes among path length, path safety, and path smoothness, each target needs to be normalized to the maximum-minimum range before weighted summation, mapping each target to the interval [0,1] to eliminate the weight imbalance problem caused by the dimensional differences. After normalization, the linear weighted sum method is used to construct the fitness function, as shown in Equation (10).

$$\begin{cases} F = \omega_1 \cdot L' + \omega_2 \cdot \frac{1}{S'_1 + \varepsilon} + \omega_3 \cdot S'_2 \\ f'_k = \frac{f_k - f_{k,min}}{f_{k,max} - f_{k,min}} \end{cases} \quad (10)$$

In Equation (10), F represents the fitness value of the particle, and the smaller the value, the better the comprehensive performance of the corresponding path. ω_1, ω_2 , and ω_3 are the weight coefficients of the corresponding targets. L', S'_1 , and S'_2 are the path length, path safety, and path smoothness targets after normalization. ε is a very small constant, set to 10^6 , to prevent division by zero errors when S'_1 is 0 after normalization. f'_k is the maximum-minimum normalization function, and $f_{k,min}$ and $f_{k,max}$ are the minimum and maximum values of the k th target within the population, which are updated dynamically during each generation of population iteration.

The traditional PSO algorithm guides particle flight through individual historical optimality P_{best}^i and group global optimality g_{best} , but its update mechanism lacks perception of real-time environmental information [26]. To enhance the algorithm's local OA and real-time response capabilities in complex obstacle environments, the virtual force generated by the APF is directly introduced into the particle velocity update equation. The improved velocity update formula is shown in Equation (11).

$$v_i(t+1) = \omega \cdot v_i(t) + c_1 \cdot r_1 \cdot (P_{best}^i - x_i(t)) + c_2 \cdot r_2 \cdot (g_{best} - x_i(t)) + F_{att} + F_{rep} \quad (11)$$

In Equation (11), $v_i(t)$ and $x_i(t)$ represent the velocity and position vectors of the particle i in the t th iteration, respectively. ω is the inertial weight, used to balance the particle's global exploration and local development capabilities. c_1 and c_2 are learning factors, controlling the step size of the particle's learning towards its individual optimal and global optimal positions, respectively. r_1 and r_2 are random numbers uniformly distributed in the interval [0, 1], used to increase the randomness of the search. Gravity F_{att} always points towards the global TP, causing particles to converge towards the target area; repulsion F_{rep} is generated by all obstacles in the environment, its direction is away from the obstacles, forcing particles away from dangerous areas.

The choice of g_{best} directly affects the convergence direction of the entire population. In MOO, there exists a Pareto front composed of numerous non-dominated solutions, rather than a single global optimum. To select a suitable guide from the non-dominated solution set while maintaining population diversity during the iterative process, this study uses fitness based reciprocal calculation of roulette wheel selection probability to dynamically determine g_{best} .

Specifically, in each iteration, all non-dominated solutions are extracted from the current population to construct a temporary elite solution set [27-28]. The Pareto dominance screening is performed based on the three original optimization objectives, namely, path length, path safety, and path smoothness, rather than the weighted-sum composite fitness F defined in Equation (10). The composite fitness F serves solely as an aggregated performance metric for executing selection operations on the obtained non-dominated candidate solutions. Following the minimization principle, a smaller value of F indicates better overall path quality. Therefore, the reciprocal of fitness is used to calculate the roulette wheel selection probability, thereby preventing inferior solutions with larger fitness values from being assigned excessively high selection probabilities. The selection probability of each individual in the elite set is computed by Equation (12).

$$P_i = \frac{1/F_i}{\sum_{j=1}^{N_e} (1/F_j)} \quad (12)$$

In Equation (12), N_e denotes the cardinality of the current elite non-dominated solution set; F_i represents the composite fitness value of the i th elite candidate. F_j denotes the composite fitness value of the j th elite candidate within the elite set for summation operation. P_i corresponds to the probability that the i th elite solution is selected as the global optimum g_{best} for the current iteration.

The algorithm generates a random number in the interval [0,1] and selects a solution from the elite solution set as the current iteration based on the aforementioned probability distribution g_{best} . This random selection mechanism allows the population to explore along different directions of the Pareto front, which helps to obtain a more evenly distributed set of non-dominated solutions, thus providing decision-makers with a series of optimization path schemes with different focuses.

3.3 ROS-Based Mobile Robot System Modeling

To confirm the capability and practicality of the introduced PP algorithm, a complete mobile robot simulation system was built based on the ROS system. The mobile robot designed in the research adopts a four-wheel independent drive Mecanum wheel structure and has omnidirectional mobility. To accurately describe the relationship between the robot's center of mass velocity and the velocities of each wheel, a kinematic analysis model of the robot, as shown in Figure 4, was established.

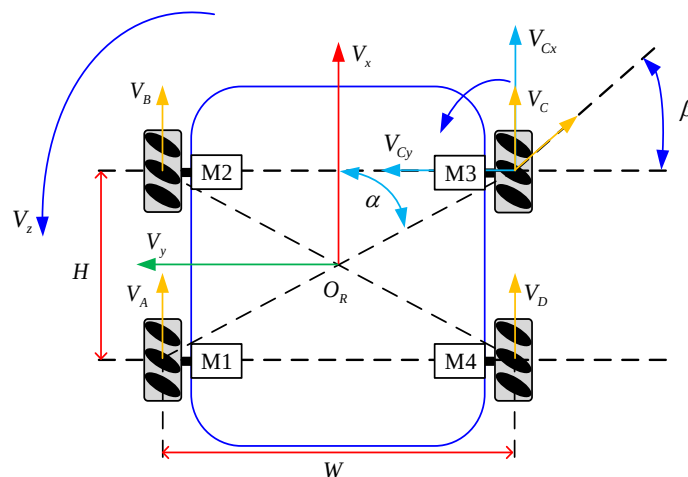


Fig. 4. Kinematic analysis diagram of the four-wheel drive wheel-driven robot

Figure 4, the robot's geometric center is set as the origin, and a robot coordinate system $X_R O_R Y_R$ is established. The robot's wheelbase is defined as H , the wheelbase as W , the robot's center of mass velocities in the X and Y directions as V_x and V_y , and the rotational angular velocity around the center of mass as V_z (counterclockwise is positive). The linear velocities of the four Mecanum wheels are V_A, V_B, V_C , and V_D . Taking wheel C as an example, assuming that there is no slippage during the robot's movement and that the center of mass is located at the geometric center, the velocity of the center of mass of wheel C can be decomposed according to Equation (13).

$$\begin{cases} V_{CX} = V_x + V_z \cdot I \cdot \sin \alpha_C \\ V_{CY} = V_y + V_z \cdot I \cdot \cos \alpha_C \end{cases} \quad (13)$$

In Equation (13), $I = \sqrt{\left(\frac{H}{2}\right)^2 + \left(\frac{W}{2}\right)^2}$ is the distance from the robot center to the wheel center, and α_C is the angle between wheel C and the robot coordinate system X_R axis. Considering the velocity synthesis of the contact point between the Mecanum wheel roller and the ground, Equation (14) is further obtained.

$$V_C = V_x + V_y + V_z \cdot \left(\frac{H}{2} + \frac{W}{2}\right) \quad (14)$$

Similarly, the velocity expressions of the other three wheels can be derived, and finally the inverse kinematics formula of the robot is obtained, as shown in Equation (15).

$$\begin{cases} V_A = V_x + V_y - V_z \cdot \left(\frac{H}{2} + \frac{W}{2}\right) \\ V_B = V_x - V_y - V_z \cdot \left(\frac{H}{2} + \frac{W}{2}\right) \\ V_C = V_x + V_y + V_z \cdot \left(\frac{H}{2} + \frac{W}{2}\right) \\ V_D = V_x - V_y + V_z \cdot \left(\frac{H}{2} + \frac{W}{2}\right) \end{cases} \quad (15)$$

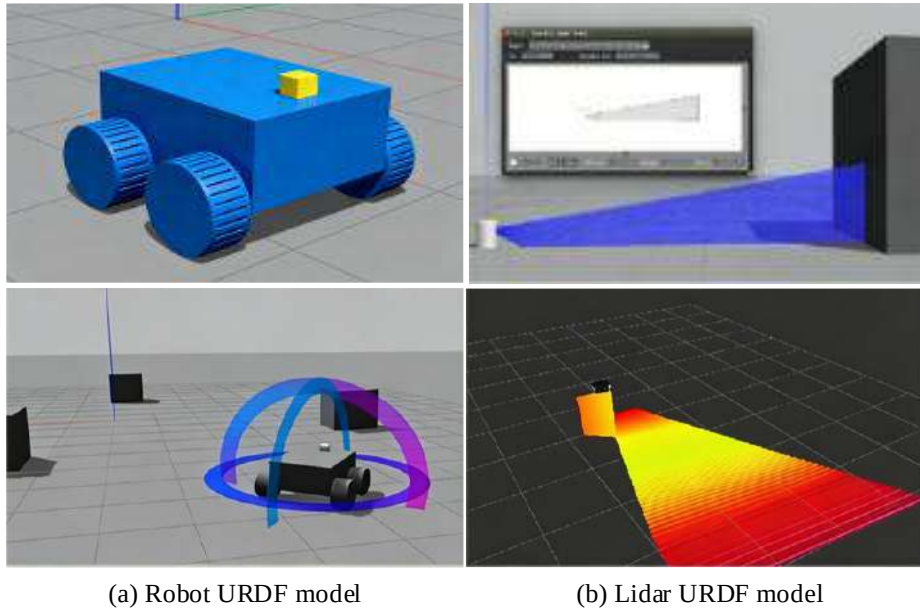


Fig. 5. Construction of the simulation model for mobile robots (Image source: Self-drawn by the author)

At the hardware simulation level, the Unified Robot Description Format (URDF) is used to define the robot's physical structure, including chassis dimensions, wheel mounting positions, mass, inertial parameters, and joint types. The simulation model of the LiDAR sensor is described using

a separate URDF file and integrated into the robot model to simulate the point cloud data output of the real sensor. Figure 5 shows the robot simulation model visualized in Rviz.

The overall structure of the mobile robot motion control system is shown in Figure 6. In terms of software functionality, the system integrates core modules such as coordinate transformation, odometry, SLAM, PP, and navigation. The coordinate transformation module publishes coordinate system transformations through the "robot_state_publisher" node, unifying the coordinates of the LiDAR, robot base, and global map. The odometry module subscribes to the rotational speed information of each wheel, calculates the robot's pose changes by combining the kinematic model, and publishes this information via "nav_msgs/Odometry" messages. The SLAM module uses the gmapping algorithm, fusing LiDAR data and odometry information to build an environmental map in real time and simultaneously estimate the robot's pose. The navigation module, based on the "move_base" framework, integrates a global path planner and a local path planner, autonomously plans the path after receiving the TP, and controls the robot's movement.

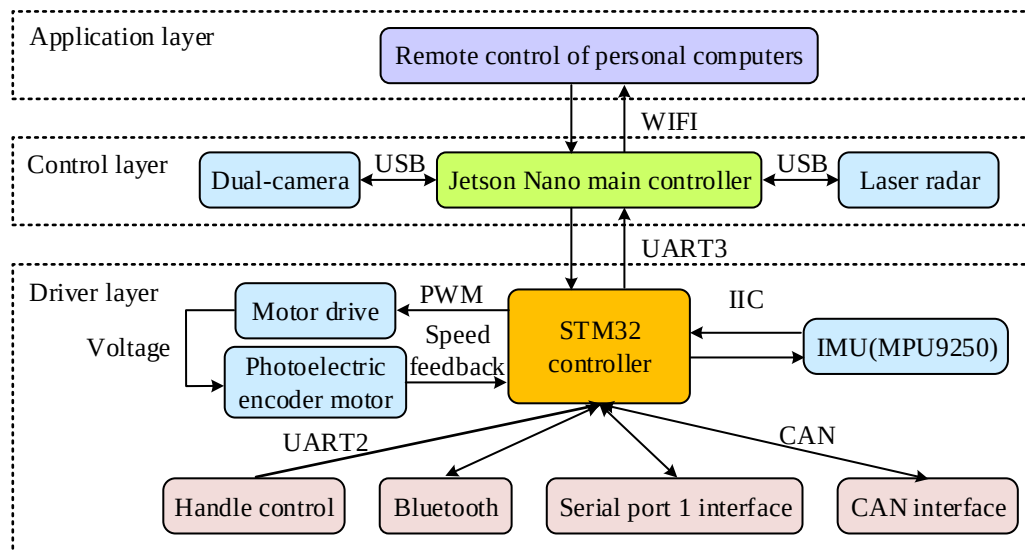


Fig. 6. Overall structure diagram of the robot motion control system

4. Experimental Results

4.1 Simulation Experiment Setup

The simulation experiments were conducted on a laptop equipped with an Intel Core i7-12700H processor (2.30GHz, 14 cores, 20 threads) and 16GB of DDR4 RAM, running Ubuntu 20.04 LTS. Algorithm development and verification relied on the following software platforms: MATLAB R2022a for algorithm prototype development and comparative verification in a 2D grid environment; ROS Noetic as the robot software framework, responsible for robot model description, sensor data transmission and reception, coordinate transformation management, and navigation function integration; Gazebo 11 as a 3D physical simulation environment, used to construct high-fidelity simulation scenes to simulate real-world dynamic characteristics and sensor noise; and Rviz for visualizing robot status, sensor data, planned paths, and map information, assisting in algorithm debugging and result display.

The detailed configuration of the ROS/Gazebo simulation environment was as follows: The ROS version was Noetic Ninjemys, and the workspace was built using catkin_tools; Gazebo 11 used the Open Dynamics Engine as the physics engine, and the FCL library performed collision detection. The simulation step size was set to 0.001 s, and the real-time factor was set to 1.0. The maximum linear velocity of the robot was 0.5 m/s, the maximum angular velocity was 1.0 rad/s, the upper limit of linear acceleration was 0.2 m/s², and the upper limit of angular acceleration was 0.5 rad/s²; The laser radar simulation used the RayPlugin in the gazebo_plugins, with an angle range of $[-\pi, \pi]$, a sampling frequency of 10 Hz, and an effective measurement range of 0.1~30 m.

The particles of the PSO algorithm were encoded using a sequence of control points, and each particle corresponded to a complete path. The dimension of the particles was determined by the number of control points in the path. The population was initialized using a combination of two strategies: a portion of individuals were quickly generated feasible paths using APF as the initial particles; the other portion of individuals randomly sample control points in the free grid space to ensure the diversity of the initial population. All initialized particles were pre-executed for basic collision detection, and invalid individuals that collide with obstacles in the initial stage were eliminated. In the SA-APF algorithm, when the robot's position change was less than 0.15 m for 8 consecutive iterations and the magnitude of the resultant force was less than 0.08, it was determined that the robot had fallen into a local minimum, triggering the simulated annealing random perturbation mechanism.

After the moving spline smoothing was completed, dense sampling was carried out at a step size of 0.05 m on the smoothed path, and each sampling point was checked to see if it fell within the expansion layer of the obstacle (with an expansion radius of 0.3 m). If there were collision points, the path was returned to the original path point before the smooth section and the local path was re-smoothed with a smaller smoothing intensity until the entire path satisfied the no-collision constraint. If after 3 local retries still could not meet the safety requirements, the candidate path was abandoned and the smoothed path before the smooth operation was retained.

To ensure the reproducibility of the experiment, all algorithms involving randomness were set with fixed random seeds. In the MATLAB environment, `rng` (2024) was used to fix the random number generator state; in the Python/ROS environment, `random.seed` (2024) and `np.random.seed` (2024) were used to fix the random seeds. All comparative experiments were run under the same random seed and were independently repeated 20 times to calculate the mean and standard deviation. In the dynamic obstacle experiment, the movement trajectory of the obstacles was also generated under the fixed random seed to ensure that the trajectory was the same for each run.

To systematically evaluate the adaptability of the algorithm to environments with different complexities, three types of raster maps were designed.

- Simple environment map: Environment A (obstacles near the target): The grid size is 30×30. The SP is located at the bottom left corner (2,2), and the TP is located at the top right corner (25,25). 3-5 obstacles are randomly placed near the TP.
- Environment B (U-shaped obstacle): The size is 30×30 grid. A U-shaped trap area is set up with obstacles. The opening is facing downwards. The SP is located at the bottom left of the U (2,2) and the TP is located at the outside right of the U (25, 20).
- Complex static environment map: Environment C (simulated library scene): Based on the floor plan of a university library, the size is 200×200 grid, and includes various static obstacles such as bookshelves, tables and chairs, and pillars.
- Dynamic obstacle environment map: Environment D (Library scene with moving obstacles): Based on Environment C, set 2-3 dynamic obstacles (simulating pedestrians) that move in a straight line or back and forth in key areas of the path, with the movement speed set to 0.5m/s.

In all grid maps, black grids represent obstacles (occupancy probability ≥ 0.8), white grids represent free space, and gray grids represent unknown areas. In the simulation environment, the robot model size is set to a diameter of 0.5m, and the obstacle expansion layer radius is set to 0.3m to ensure a safe distance. The parameters of each comparative algorithm were determined based on recommended values from the literature or through pre-experimental optimization. The core parameter settings of the introduced ASO are presented in Table 1.

To fully quantify the performance of the algorithm, five indicators were used to compare and evaluate it with four algorithms: RRT [29], Rapidly-exploring Random Tree Star (RRT*) [30], a framework for multiple populations for multiple objectives based on ant colony optimization (MPMO-ACO) [31], and DQN [32]. (1) Path length (unit:m): The total Euclidean distance of the path traversed by the robot from the SP to the TP. (2) Computation time (unit:s): The CPU time consumed by the algorithm from inputting the information of the SP and the TP to outputting the complete path. Each experiment was conducted 20 times, and the mean value was calculated to

minimize the impact of randomness. (3) Path safety (unit:m): Collect several discrete points along the planned path, calculate the Euclidean distance from each sampling point to the nearest obstacle, and then take the arithmetic mean of these distances.

Table 1. ASO algorithm parameter configuration

Parameter name	Symbol	Value
Gravitational gain coefficient	K_{att}	15.0
RF gain coefficient	K_{rep}	8.0
Obstacle influence radius	ρ_0	3.5
Initial temperature of SA	T_0	100.0
SA termination temperature	T_{min}	1.0
Annealing rate	α	0.92
Number of local minimum detection steps	N_{local}	8
Moving average window size	w_{smoot}	5
Spline interpolation density	N_{spline}	10
Particle swarm size	N_p	50
Maximum iteration count	N_{iter}	200
Inertia weight	ω	0.7~0.3
Learning factor	c_1, c_2	1.5, 1.5
Multi-objective weighting	$\omega_1, \omega_2, \omega_3$	0.4, 0.3, 0.3
Pareto solution set capacity	$N_{archive}$	100
Random seed	-	2024

Table 2. Comparison of PP results under different weight combinations

Weighted combination ($\omega_1, \omega_2, \omega_3$)	Path length (m)	Path safety degree (m)	Smoothness (rad)	Calculation time (s)
This study (0.4, 0.3, 0.3)	218.7±2.1	0.56±0.04	3.89±0.11	1.43±0.05
95% CI	(217.2, 220.2)	(0.54, 0.58)	(3.84, 3.94)	(1.41, 1.45)
Emphasizing length (0.6, 0.2, 0.2)	215.3±1.9	0.48±0.05	4.21±0.13	1.39±0.04
95% CI	(214.0, 216.6)	(0.46, 11.50)	(4.15, 4.27)	(1.37, 1.41)
Emphasizing safety (0.2, 0.6, 0.2)	223.1±2.3	0.63±0.04	4.05±0.12	1.46±0.05
95% CI	(221.5, 224.7)	(0.61, 0.65)	(3.99, 4.11)	(1.44, 1.48)
Emphasizing smoothness (0.2, 0.2, 0.6)	221.8±2.0	0.45±0.04	3.82±0.10	1.45±0.04
95% CI	(220.4, 223.2)	(0.43, 0.47)	(3.77, 3.87)	(1.43, 1.47)
Equal weight (0.33, 0.33, 0.33)	220.5±2.2	0.52±0.05	3.95±0.11	1.44±0.05
95% CI	(219.0, 222.0)	(0.50, 0.54)	(3.90, 4.00)	(1.42, 1.46)

The larger the average distance, the farther the entire path is from the obstacles, indicating higher safety. (4) Path smoothness (radians): The sum of the absolute values of the turning angles of adjacent line segments on the path. A smaller value indicates a smoother turn, which conforms to kinematic constraints. (5) OA success rate: The percentage of times the robot successfully reaches the target without collision under dynamic obstacles. 50 tests are conducted per scenario to record the success rate.

To verify the impact of the weight coefficient values on the planning results, five different weight combinations were set in environment C for comparative experiments. Each group of experiments was independently run 20 times, and the average values of each indicator were taken. The results are shown in Table 2. From Table 2, it can be seen that when the weight of the path length was increased, the path length was shortened to 215.3 m, but the safety and smoothness decreased. When the safety weight was increased, the average safety distance was raised to 0.63 m, but the path length increased accordingly, and when the smoothness weight was increased, the smoothness improved to 3.82 rad, but the safety decreased significantly. The weight combinations set in this paper achieved balanced and superior results in all indicators. The path length was 218.7 m, the average safety distance was 0.56 m, the smoothness was 3.89 rad, and the calculation time

was 1.43 s. The above results indicated that the algorithm was not sensitive to the weight coefficients within a reasonable range, and the weight selection had good robustness. The weight set in this paper could effectively balance the three conflicting optimization objectives.

4.2 Algorithm Performance Comparison in Simple Environments

Figure 7 shows the path length convergence curves of the four algorithms in environments A and B. In environment A, ASO converged first after about 25 iterations, with a path length of 28.3 meters, which was better than RRT (30.1 meters) and RRT* (29.5 meters), and the curve was smooth with slight fluctuations. In environment B, ASO stabilized after 30 iterations, with a path length of 10.4 meters, which was better than RRT (10.9 meters) and RRT* (10.7 meters), and converged the fastest. RRT and RRT* converged slowly in both environments and fluctuated greatly in the later stages. Overall, ASO had significant advantages in convergence efficiency and path quality, verifying its global search and optimization stability.

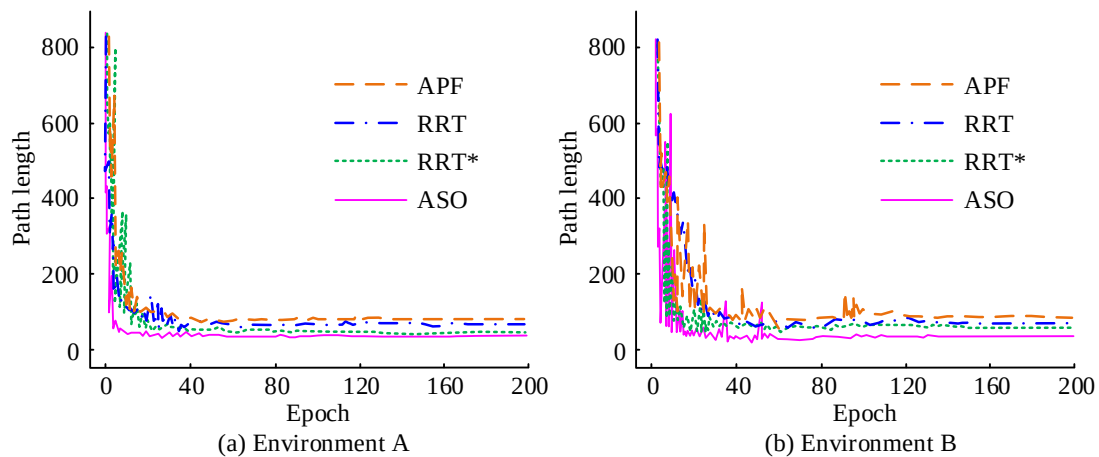


Fig. 7. Path length convergence curve

Table 3 indicates the experimental outcomes of PP in simple environments. From Table 2, the traditional APF algorithm failed to reach the TP in both scenarios due to local minima. The proposed ASO algorithm not only successfully completed all planning tasks but also significantly outperformed the comparative algorithms in regard to computational efficiency and path quality. In environment A, the computation time of ASO was 3.02 s; its path length was 28.3 m, which was 6.0% and 4.1% shorter than RRT and RRT*, respectively. In the more complex environment B, the advantages of ASO were even more pronounced. Its computation time was only 0.86 s; its path length was 10.4 m, which was 4.6% and 2.8% shorter than RRT and RRT*, respectively. This result fully verified the effectiveness of the SA mechanism and moving spline smoothing strategy in solving local minima and improving path quality.

Figure 8 shows the convergence curves of the fitness values of the four algorithms in environments A and B as a function of the iteration count. In Figure 8(a), ASO converged to the lowest fitness value first after about 40 iterations, with a smooth and stable curve, demonstrating strong global search capability; RRT converged quickly but had a slightly higher fitness value; RRT converged slowly and exhibited slight fluctuations in the later stages; APF had the highest fitness curve but large fluctuations, making it difficult to escape local optima. In Figure 8(b), ASO still converged to the lowest fitness value fastest, with a smooth curve; RRT and RRT* had lower convergence speeds and fitness values than ASO; the APF curve fluctuated at a high level, highlighting the limitations of the U-shaped obstacle environment. Overall, ASO significantly outperformed other algorithms in regard to convergence efficiency and optimization quality.

Table 3. Comparison of PP performance of different algorithms in simple environment

Environmental type	Algorithm	Calculation time (s)	Path length (m)	Whether successfully reached the target
Environment A	APF	2.96 ± 0.18	29.8 ± 0.6	No
	RRT	5.35 ± 0.62	30.1 ± 0.8	Yes
	RRT*	3.61 ± 0.41	29.5 ± 0.6	Yes
	ASO	3.02 ± 0.28	28.3 ± 0.5	Yes
Environment B	APF	1.61 ± 0.17	11.4 ± 0.2	No
	RRT	1.95 ± 0.23	10.9 ± 0.5	Yes
	RRT*	1.33 ± 0.15	10.7 ± 0.4	Yes
	ASO	0.86 ± 0.09	10.4 ± 0.3	Yes

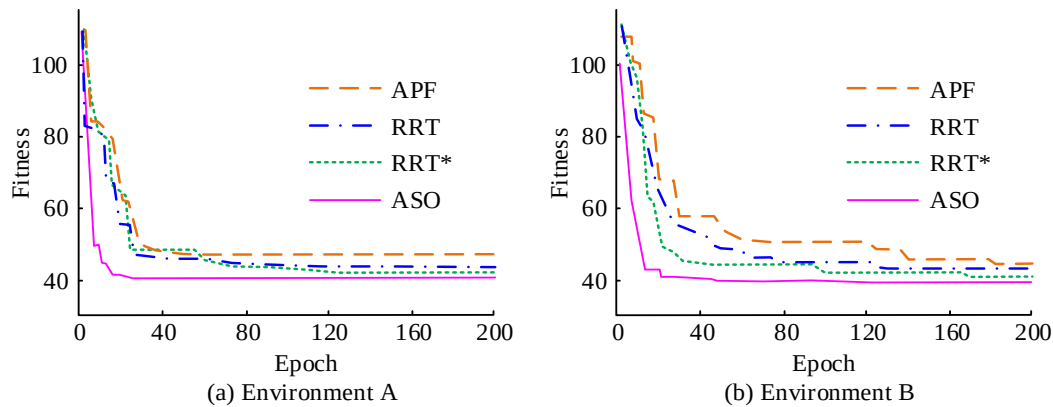


Fig. 8. Fitness iteration curve

4.3 Comparison of Algorithm Performance in Complex Static Environments

In the comparative experiment in environment C (indicators are shown in Table 4). All algorithms were run on the same hardware platform and with the same random seed (2024); for RRT and RRT*, the maximum number of iterations for each planning was set to 5000. If no path was found, the process would terminate. DQN adopted a fixed training step size (100,000 steps) and was pre-trained on a static map identical to the test environment until convergence. After training, the network parameters were fixed for 20 tests. The environment maps, starting and ending positions, and obstacle distributions of all algorithms were exactly the same to ensure the fairness of the comparison. Each set of experiments was run independently 20 times. The ASO algorithm's computation time was only 1.43 seconds, far exceeding RRT (17.25 seconds) and RRT* (21.88 seconds), demonstrating high planning efficiency. Its path length was 218.7 meters, better than SA-APF, but slightly longer than MPMO-ACO, which sacrificed safety (its average safety distance was 0.39 meters, lower than ASO's 0.56 meters). Regarding path smoothness, ASO's cumulative turning angle was 3.89 radians, lower than RRT and RRT*. DQN's smoothness was similar to ASO, but its computation time (3.41 seconds) and path length (285.7 meters) were inferior. ASO achieved the best balance in computational efficiency, path length, safety, and smoothness.

Figure 9 illustrates the impact of four different optimization objectives on PP results in a complex static environment. Figure 9(a) shows the shortest path was close to the obstacle, which had a high risk of collision; Figure 9(b) shows the path was far from the obstacle, which was safe but longer; Figure 9(c) shows the path had gentle turns, but excessive smoothness made the detour longer; Figure 9(d) shows the path was moderate, safe and smooth when using multi-objective collaborative optimization to balance path length, safety and smoothness.

Table 4. Comparison of comprehensive performance of various algorithms in complex static environment

Algorithm	Calculation time (s)	Path length (m)	Path safety degree (m)	Path smoothness (rad)
APF	$1.62 \pm 0.08a$	$285.3 \pm 4.2d$	$0.06 \pm 0.02d$	$8.52 \pm 0.35b$
95% CI	(1.58, 1.66)	(283.3, 287.3)	(0.05, 0.07)	(8.36, 8.68)
RRT	$17.25 \pm 1.86c$	$247.8 \pm 5.1c$	$0.53 \pm 0.06b$	$13.05 \pm 1.52c$
95% CI	(16.38, 18.12)	(245.4, 250.2)	(0.50, 0.56)	(12.34, 13.76)
RRT*	$21.88 \pm 2.34d$	$224.6 \pm 3.8b$	$0.57 \pm 0.05a$	$9.47 \pm 0.89b$
95% CI	(20.78, 22.98)	(222.8, 226.4)	(0.55, 0.59)	(9.05, 9.89)
MPMO-ACO	$1.45 \pm 0.06a$	$203.2 \pm 2.9a$	$0.39 \pm 0.04c$	$3.05 \pm 0.14a$
95% CI	(1.42, 1.48)	(201.8, 204.6)	(0.37, 0.41)	(2.98, 3.12)
DQN	$3.41 \pm 0.35b$	$285.7 \pm 5.8d$	$0.58 \pm 0.05a$	$3.93 \pm 0.23a$
95% CI	(3.25, 3.57)	(283.0, 288.4)	(0.56, 0.60)	(3.82, 4.04)
ASO	$1.43 \pm 0.05a$	$218.7 \pm 2.2b$	$0.56 \pm 0.04a$	$3.89 \pm 0.11a$
95% CI	(1.41, 1.45)	(217.3, 220.1)	(0.54, 0.58)	(3.84, 3.94)

Note: Different superscript letters (a, b, c, d) in the same column indicate statistically significant differences between groups (Tukey HSD post-hoc test, $\alpha=0.05$), while the same letter indicates no significant difference. The same as below.

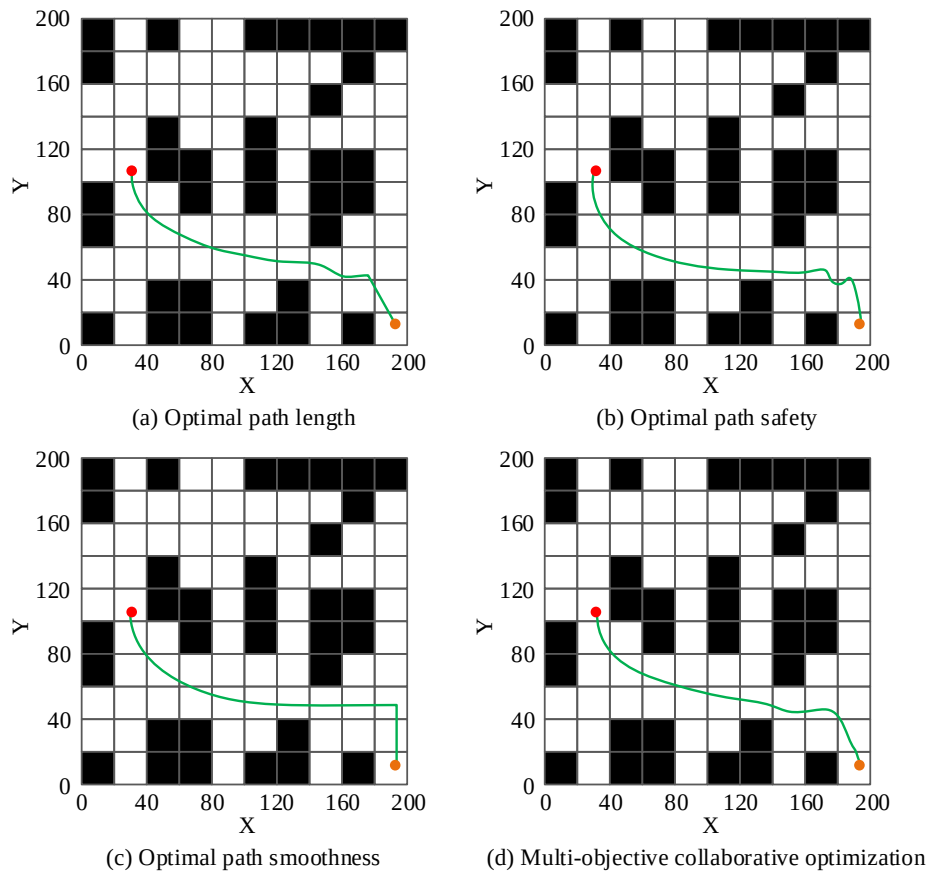


Fig. 9. Comparison of algorithm performance in environment C

The performance evaluation results of multi-objective optimization are presented in Table 5. The results of single-objective optimization exhibit typical target interdependence: when pursuing the optimal path length, the path length reaches the shortest 215.8 m, but the average safety distance was only 0.42 m, and the smoothness was 4.83 rad. When pursuing the optimal safety level, the average safety distance increased to 0.68 m, but the path length increased to 231.2 m, and the smoothness deteriorated to 5.92 rad. When pursuing the optimal smoothness, the smoothness improved to 3.67 rad, but the average safety distance dropped significantly to 0.35 m. The proposed

multi-objective collaborative optimization mode in this paper achieved a comprehensive result of 218.7 m for path length, 0.56 m for average safety distance, and 3.89 rad for smoothness within 2.86 seconds of computation time. Compared with the path length optimal mode, the multi-objective collaborative optimization only required a 2.9 m increase in path length as a trade-off, while achieving significant improvements in safety level and smoothness ($p < 0.001$), fully verifying the effectiveness of the proposed multi-objective collaborative strategy in balancing various conflicting objectives.

Table 5. Comparison of PP results under different optimization objectives

Optimization objectives	Calculation time (s)	Path length (m)	Path safety degree (m)	Path smoothness (rad)
Optimal path length	2.68±0.06	215.8±2.3 ^a	0.42±0.05 ^c	4.83±0.15 ^c
95% CI	(2.65, 2.71)	(214.4, 217.2)	(0.40, 0.44)	(4.76, 4.90)
Optimal path safety	2.71±0.07	231.2±2.6 ^b	0.68±0.04 ^a	5.92±0.18 ^d
95% CI	(2.68, 2.74)	(229.5, 232.9)	(0.66, 0.70)	(5.84, 6.00)
Optimal path smoothness	2.73±0.06	229.7±2.4 ^b	0.35±0.04 ^d	3.67±0.12 ^a
95% CI	(2.70, 2.76)	(228.2, 231.2)	(0.33, 0.37)	(3.61, 3.73)
Multi-objective collaborative optimization	2.86±0.08	218.7±2.2 ^a	0.56±0.04 ^b	3.89±0.11 ^b
95% CI	(2.82, 2.90)	(217.3, 220.1)	(0.54, 0.58)	(3.84, 3.94)
ANOVA F-value (p -value)	F(3,76)=3.12, $p=0.031$	F(3,76)=9.85, $p<0.001$	F(3,76)=12.41, $p<0.001$	F(3,76)=8.67, $p<0.001$

4.4 Comparison of Real-Time OA Performance of Algorithms in Dynamic Obstacle Environments

Figure 10 illustrates the PP results of the proposed algorithm in a complex environment D containing dynamic obstacles. Figure 10 shows that the robot followed the planned path through two mandatory points, demonstrating that the algorithm satisfied multi-task constraints. When encountering dynamic obstacles, the algorithm locally replanned its path to bypass them, and returned to the original path to reach the target after overcoming the obstacles. The entire process was collision-free and the trajectory was smooth, confirming the algorithm's excellent OA and path tracking capabilities in dynamic and unknown environments.

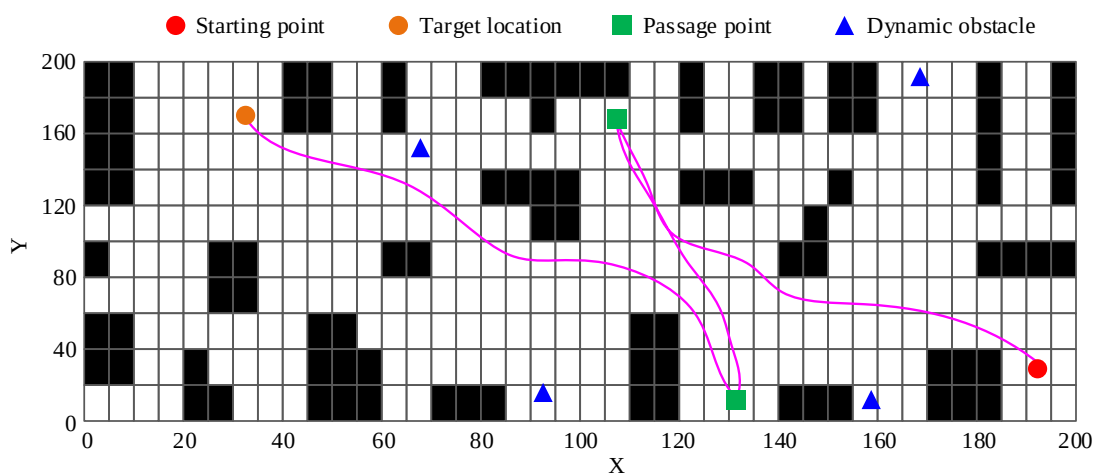


Fig. 10. Comparison of algorithm performance in environment D

The results in Table 6 showed that the performance indicators of the ASO algorithm in the dynamic OA environment were significantly superior to those of MPMO-ACO and APF. The success rate of ASO was 96.0%, with a collision rate of only 4.0%, while MPMO-ACO was 82.0% and 18.0% respectively, and APF was only 34.0% and 66.0%. The gap was extremely obvious. In terms of re-planning time, ASO only took 0.72 s, which was much faster than MPMO-ACO's 1.86 s and APF's

2.15 s, demonstrating its rapid response capability. The minimum obstacle clearance gap of ASO was 0.48 m, with the highest safety, traversal time of 28.6 s, the shortest path length of 226.5 m, and the smoothness of 4.12 rad being the best. One-way analysis of variance indicated that there were extremely significant differences in all indicators among the three algorithms, with p -values all less than 0.001. Tukey's post-hoc test further confirmed that pairwise comparisons between ASO and the other two algorithms were all at a significant level, confirming that ASO had the superiority of efficient, smooth and reliable real-time OA in dynamic environments.

Table 6. Performance comparison of various algorithms in dynamic obstacle environments

Algori thm	Success rate (%)	Collision rate (%)	Re- planning time (s)	Minimum obstacle clearance (m)	Traversal time (s)	Path length (m)	Smoothne ss (rad)
APF	34.0 ± 6.8^c	66.0 ± 6.8^c	2.15 ± 0.32^c	0.08 ± 0.03^c	45.6 ± 5.2^c	312.5 ± 18.4^c	11.23 ± 1.54^c
95% CI	(31.1, 36.9)	(63.1, 68.9)	(2.06, 2.24)	(0.07, 0.09)	(44.2, 47.0)	(307.3, 317.7)	(10.80, 11.66)
MPMO -ACO	82.0 ± 5.1^b	18.0 ± 5.1^b	1.86 ± 0.28^b	0.21 ± 0.05^b	38.2 ± 4.1^b	258.3 ± 15.7^b	7.85 ± 0.92^b
95% CI	(80.6, 83.4)	(16.6, 19.4)	(1.78, 1.94)	(0.20, 0.22)	(37.1, 39.3)	(253.9, 262.7)	(7.59, 8.11)
ASO	96.0 ± 2.8^a	4.0 ± 2.8^a	0.72 ± 0.11^a	0.48 ± 0.06^a	28.6 ± 2.3^a	226.5 ± 10.2^a	4.12 ± 0.35^a
95% CI	(95.2, 96.8)	(3.2, 4.8)	(0.69, 0.75)	(0.46, 0.50)	(27.9, 29.3)	(223.6, 229.4)	(4.02, 4.22)

4.5. Performance Verification of ASO Algorithm PP

Figure 11 indicates a comparison of the PP performance of the three algorithms when facing dynamic obstacles in the Gazebo simulation environment. In Figure 11(a), the trajectory of APF showed obvious reciprocating oscillations. The robot repeatedly adjusted its direction near the dynamic obstacle but could not effectively avoid it, and eventually failed to plan its path due to getting trapped in a local minimum. In Figure 11(b), although MPMO-ACO could identify dynamic obstacles and complete OA, the path had many unnecessary detours and turns, and the overall smoothness was poor, reflecting its insufficient response speed and real-time adjustment capability. In Figure 11(c), ASO quickly identified and avoided dynamic obstacles with a smooth and continuous curve. The path length was the shortest and there were no obvious turning points. The entire motion process was stable and efficient.

Table 7. Ablation experimental results of ASO algorithm

Variation	Calculation time (s)	Path length (m)	Path safety degree (m)	Path smoothness (rad)
w/o SA	1.28 ± 0.04^a	235.6 ± 3.8^c	0.32 ± 0.05^e	5.82 ± 0.28^d
95% CI	(1.26, 1.30)	(233.8, 237.4)	(0.30, 0.34)	(5.69, 5.95)
w/o PF	1.52 ± 0.06^b	225.4 ± 3.2^b	0.48 ± 0.04^c	4.45 ± 0.21^c
95% CI	(1.49, 1.55)	(223.9, 226.9)	(0.46, 0.50)	(4.35, 4.55)
w/o PSO	1.35 ± 0.05^a	241.8 ± 4.5^d	0.41 ± 0.05^d	6.18 ± 0.32^e
95% CI	(1.33, 1.37)	(239.7, 243.9)	(0.39, 0.43)	(6.03, 6.33)
w/o Spline	1.41 ± 0.05^{ab}	220.5 ± 2.5^{ab}	0.54 ± 0.04^b	5.86 ± 0.24^d
95% CI	(1.39, 1.43)	(219.3, 221.7)	(0.52, 0.56)	(5.75, 5.97)
w/o Leader	1.45 ± 0.06^b	222.8 ± 2.8^b	0.50 ± 0.05^c	4.22 ± 0.18^b
95% CI	(1.42, 1.48)	(221.5, 224.1)	(0.48, 0.52)	(4.14, 4.30)
ASO	1.43 ± 0.05^{ab}	218.7 ± 2.2^a	0.56 ± 0.04^a	3.89 ± 0.11^a
95% CI	(1.41, 1.45)	(217.3, 220.1)	(0.54, 0.58)	(3.84, 3.94)

Table 7 reveals the differentiated contributions of each algorithm component to the performance of ASO through the results of the ablation experiments. After removing SA, the path length and safety distance significantly deteriorated by 7.7% ($p<0.001$) and 42.9% ($p<0.001$), respectively, indicating that the SA mechanism played an irreplaceable role in avoiding local minima. The removal of PF led to a 14.3% ($p<0.01$) decrease in the safety distance and a 14.4% ($p<0.01$) deterioration in smoothness, confirming the dual regularization effect of PF force on OA and trajectory smoothness. The absence of the PSO framework increased the path length by 10.6% ($p<0.001$) and the safety distance by 26.8% ($p<0.001$), verifying the necessity of it as the core engine for global search. The removal of spline smoothing significantly worsened the smoothness from 3.89 rad to 5.86 rad ($p<0.001$), highlighting the optimization effect of the post-processing stage on the continuity of path curvature. The removal of the leader selection mechanism led to a decrease in the safety distance from 0.56 m to 0.50 m ($p<0.05$) and a deterioration in smoothness from 3.89 rad to 4.22 rad ($p<0.01$). In summary, the complete ASO significantly outperformed any ablation variant in all indicators, verifying the effectiveness of the coordination of algorithm components and the necessity of the coupling between modules.

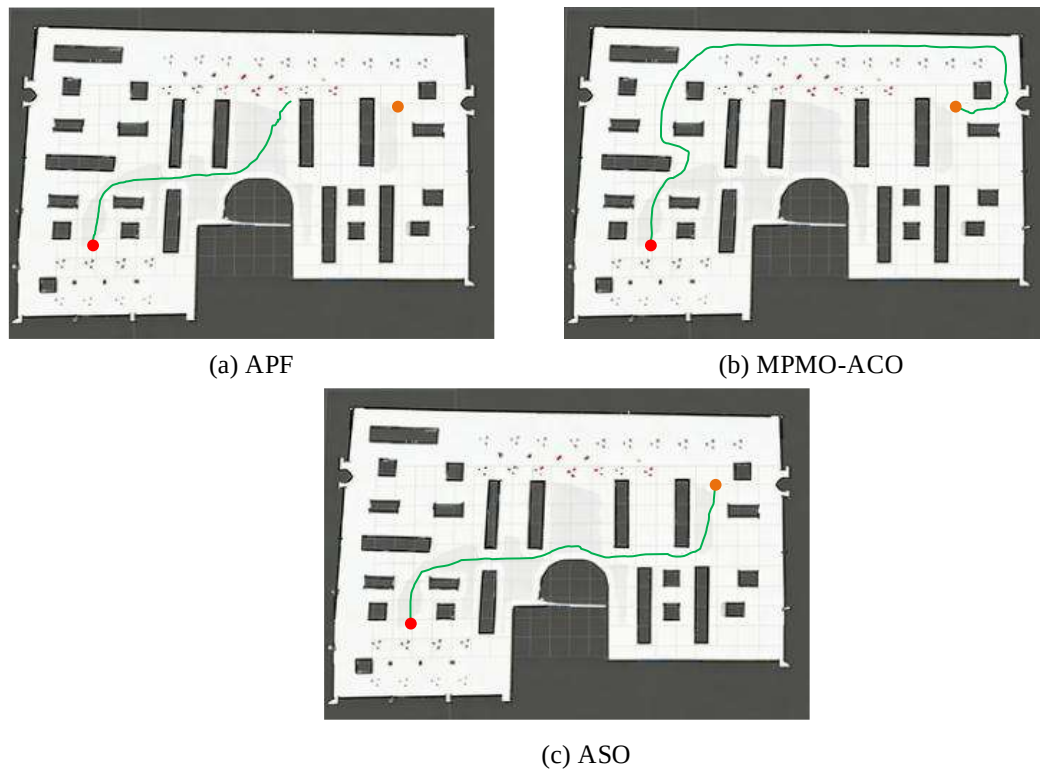


Fig. 11. Comparison of algorithm performance in environment D

5. Discussion

This study addressed the challenges of local minima trapping, insufficient dynamic obstacle-avoidance capability, and difficulties in multi-objective cooperative optimization for mobile-robot PP in complex environments. A field-swarm cooperative algorithm termed ASO, which integrated the improved APF and PSO, was proposed. The ASO algorithm achieved prominent performance advantages within simple-obstacle environments. In Environment A, where obstacles were distributed near the TP, ASO yielded a computation time of only 3.02s, which was shorter than the 5.35s consumed by the RRT algorithm and the 3.61s consumed by the RRT* algorithm; its resulting path length was 28.3m, outperforming the path-length results obtained from RRT and RRT*. More evident performance gains were observed for ASO in the U-shaped-obstacle scenario of Environment B. The computation time of ASO reached 0.86s, which was lower than the 1.95s obtained with RRT, and ASO produced a shorter path of 10.4m compared with the 10.9m path generated by RRT. It is noteworthy that the conventional artificial-potential-field method cannot reach the TP in both simple test scenarios owing to local-minima issues. Oscillation occurs around

the TP in Environment A, whereas the robot becomes trapped inside the U-shaped trap and forms a reciprocating oscillation loop in Environment B. These results verify that the embedded simulated-annealing mechanism effectively breaks the force-balance state of the APF. Once small positional variations over successive iterations and near-zero net force are detected, virtual repulsive forces produced by randomly generated perturbation points guide the robot to escape from local-minima regions. Exponential temperature decay enables stable convergence toward the TP in the later-stage iterations.

The comprehensive performance of the ASO algorithm was verified in a complex static environment experiment. In environment C, the ASO computation time was 1.43 seconds, far exceeding RRT (17.25 seconds) and RRT* (21.88 seconds), demonstrating high efficiency. The path length was 218.7 meters, better than SA-APF, but slightly longer than the safety-sacrificing Bezier smoothing algorithm (safety distance of 0.39 meters, lower than ASO's 0.56 meters). In terms of path smoothness, ASO's cumulative turning angle was 3.89 radians, lower than RRT and RRT*, better conforming to robot kinematic constraints. The moving spline smoothing algorithm first coarsely smooths the path points, then uses cubic spline interpolation to generate curves, eliminating sharp turns and preserving shape, which is beneficial for robot control.

Experiments in a dynamic obstacle environment validated the real-time OA capability of the ASO algorithm. In environment D containing moving obstacles, the robot started from the SP and passed through two pre-set necessary points in sequence. When a dynamic obstacle moved in a straight line and intruded into the planned path, the algorithm triggered a local replanning mechanism to adjust the course and detour around it. After passing the obstacle, the robot swiftly reverted to its initial path and arrived at the TP. No collisions occurred during the entire process, and the trajectory was smooth and continuous. Comparative experiments in the Gazebo environment further confirmed that ASO quickly identified and avoided dynamic obstacles with a smooth and continuous curve, achieving the shortest path length and no obvious turning points. In contrast, APF showed obvious reciprocating oscillations and got stuck in local minima, while MPMO-ACO, although able to complete OA, had several unnecessary detours and turns. This result is attributed to the design of directly introducing artificial potential force into the particle swarm velocity update equation. Gravity always points towards the global TP, causing particles to converge towards the target area, while RF is generated by all obstacles in the environment, forcing particles away from dangerous areas. This gives the algorithm both global heuristic search capability and local real-time OA capability.

The ASO algorithm has made progress but still has shortcomings: MOO relies on linear weighted integration, and the weight coefficients have a significant impact, potentially requiring dynamic adjustment when the environment changes; SA parameters are set based on experience, and adaptive adjustment could be studied in the future; currently, it mainly focuses on simulation verification and has not fully considered interference from actual robot systems. Subsequent experiments will be conducted on a real Mecanum wheel robot platform to verify its engineering practicality.

6. Conclusion

The research focused on the key issues of mobile robot PP in complex environments, such as the susceptibility to local minima, insufficient dynamic OA capability, and difficulties in multi-objective collaborative optimization. The ASO algorithm was proposed to address these problems. This algorithm introduced the SA mechanism to overcome the local optimal defect of the traditional APF, integrated the PF force into the particle swarm velocity update equation to enhance the adaptability to dynamic environments, and constructed a multi-objective framework for the collaborative optimization of path length, safety degree, and smoothness. The roulette wheel selection based on fitness was adopted to explore the Pareto frontier. A simulation platform for omnidirectional mobile robots with mechanic wheels was built based on the ROS system. Comparative experiments were conducted in grid maps and Gazebo environments. The results showed that the ASO algorithm effectively overcame the local minimum problem of APF. In the U-shaped obstacle environment, the calculation time was only 0.86s, the path length was 10.4m, and

it could successfully escape the U-shaped trap and plan a smooth path. In complex static library scenarios, the ASO algorithm could achieve a good coordination and balance among the three conflicting indicators of path length, security, and smoothness. Compared with the single pursuit of the optimal path length, the multi-objective collaborative optimization strategy, at the cost of a slight increase in path length, significantly improved security and smoothness, fully demonstrating the effectiveness of the proposed crowding distance roulette wheel selection mechanism in guiding the population to uniformly distribute along the Pareto frontier. This mechanism effectively avoided the population from prematurely converging to a specific target area by giving higher selection probabilities to non-dominated solutions in sparsely distributed regions, thereby ensuring the diversity and coverage of the solution set across the entire frontier. Moreover, in dynamic obstacle environments, the algorithm achieved effective real-time OA, with smooth trajectories and no collisions. This verified its excellent dynamic response capability under simulation conditions. The simulation results showed that ASO had excellent effectiveness and application potential in autonomous navigation under complex dynamic environments, providing a reliable algorithm foundation for the subsequent deployment of actual systems.

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Multi-objective position layout optimization of CNC milling fixture structure integrating remaining useful life prediction model

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Abstract

CNC tool accessories are affected by cutting vibration and load impact for a long time, and are prone to fatigue damage. However, traditional optimization methods often ignore the synergy between life and performance, making it difficult to balance service life, structural stiffness, deformation control and other indicators. Therefore, this study proposes a multi-objective topology optimization method that integrates life prediction models. To address the correlation characteristics between vibration signals and life degradation, this study first preprocesses the vibration signals through interpolation method and 3σ criterion. It builds a complete integrated empirical mode decomposition-Fine-to-Coarse signal processing framework with adaptive noise, extracts multi-scale sensitive features, and then designs a CNN-BiLSTM life prediction model. Experiments show that the MAE and RMSE of this model are as low as 0.007 and 0.008, and the average prediction accuracy is 96.1%, which is better than the comparative method. In the test using milling machine fixtures as the object, the proposed multi-objective optimization method of BPNN-PSO converges after 90 iterations, the generation distance is 0.011, the proportion of remaining life after optimization is up to 0.96, and the deformation amount is smaller than other methods. Experimental data demonstrate that the proposed method realizes favorable coordination between precise remaining life prediction and structural performance improvement. Under tested working conditions, this approach attains a satisfactory trade-off between fixture service life and deformation control, offering an effective technical reference for the design of CNC tool attachments.

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1. Background

In the context of the transformation of modern manufacturing to high precision, high efficiency, and high reliability, CNC tools are core processing equipment, and the performance of their accessory structures (especially fixtures) directly determines processing accuracy, production efficiency, and operational safety. During long-term service, fixtures are easily affected by complex working conditions such as cutting vibration and load impact, and are prone to fatigue damage and performance degradation. This not only shortens their service life but may also increase workpiece machining errors or cause safety accidents [1,2]. Therefore, how to achieve collaborative optimization of life and performance during the structural design stage has become a key technical problem in the research and development of CNC tool accessories. At present, the structural optimization of CNC tool accessories mostly focuses on improving mechanical properties, often ignoring the correlation between life prediction and optimal design, making it difficult for the optimization plan to take into account core indicators such as service life, structural stiffness, and

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deformation control. Meanwhile, traditional life prediction methods rely on empirical models or single signal features, which are difficult to effectively capture the multi-scale sensitive information related to life degradation in vibration signals, and the prediction accuracy is insufficient [3,4]. Existing structural optimization algorithms often have problems such as slow convergence speed and easy falling into local optima, making it difficult to efficiently solve Multi-Objective Optimization (MOO) problems. In response to the above challenges, there is an urgent need to establish a Multi-Objective Position Layout Optimization (MPLO) framework that integrates accurate life prediction.

Bidirectional Long Short-Term Memory (BiLSTM) enhances the model's ability to understand sequence data by simultaneously processing past and future information at each time step. Duan et al. proposed a parallel BiLSTM MHA prediction framework that integrates multi head attention mechanism to address the issue of insufficient accuracy in predicting the remaining service life of lithium-ion batteries. This framework first constructs multidimensional features of the battery's entire lifecycle to complete capacity estimation, and then combines the CEEMDAN module to achieve remaining life prediction based on the estimated capacity, fully mining the multidimensional temporal information of the battery and taking into account the capacity regeneration effect. Based on the NASA and CALCE battery datasets, case validation was conducted, and the results showed that this method has better performance in predicting the remaining service life of lithium-ion batteries compared to traditional deep learning models [5]. Zhang et al. proposed the VMD-CNN BiLSTM prediction model to address the issue of low accuracy in traditional methods for predicting the remaining life of turbofan engines under variable operating conditions. This method first processes data through variational mode decomposition to extract multidimensional temporal features and eliminate abnormal samples. Then, CNN is used to complete feature mapping and BiLSTM is combined to mine temporal dependencies. At the same time, sparrow algorithm is introduced to optimize model hyperparameters and improve prediction performance. Based on the C-MAPSS dataset, experiments were conducted and the results showed that the model predicted a root mean square error of 13.74 ± 0.51 and an average absolute error of 11.24 ± 0.49 , demonstrating excellent prediction accuracy and generalization ability even in noisy data [6]. Yuan et al. proposed an integrated life prediction framework that combines an improved whale optimization algorithm with adaptive variational mode decomposition and CNN BiLSTM predictor to address the problem of difficulty in achieving reliable signal decomposition and accurate mapping of health indicators to life for predicting the remaining service life of rolling bearings. This framework utilizes IWOA and entropy guided objective function to optimize VMD parameters, screen effective intrinsic modal components, and construct a lightweight self-attention health index SA-HI to characterize the degradation trend of bearings. Based on the XJTU-SY bearing dataset, multiple experiments were conducted, and the results showed that SA-HI had better monotonicity, trendiness, and robustness than traditional indicators. The entire framework was able to achieve more accurate and stable remaining life prediction in both same and cross condition tests, and its performance was superior to mainstream deep learning comparison models [7].

Particle Swarm Optimization (PSO) algorithm can simulate the foraging behavior of flocks of birds and fish. The core idea is to find the optima in the solution space through the collaboration and information sharing of individuals in the group. Hrechanyi explores the method of determining equipment detection intervals based on non-stationary Poisson flow emission theory for predicting the timing of maintenance start-up of metallurgical equipment during degradation processes. The research focuses on analyzing the process of equipment degradation, using emission theory to plan periodic detection cycles, and completing parameter determination by comparing paired correlation coefficients, requiring the correlation coefficient to approach 1. The research results show that when using the emission theory to allocate equipment mutual control cycles, a correlation coefficient close to 1 should be used as the criterion for discrimination, in order to support the construction of detection interval planning algorithms [8]. The Backpropagation Neural Network (BPNN) inverse system decoupling control of the permanent magnet-assisted bearingless synchronous reluctance motor has problems such as slow convergence, easy falling into local optimum, low fitting accuracy and poor generalization ability. Liu et al. proposed a full-speed domain dynamic decoupling control strategy based on enhanced chaotic PSO. The dynamic

decoupling performance between velocity, x- and y-direction radial displacement has been improved, and the static stability and dynamic responsiveness of the system have verified the effectiveness of this scheme [9]. Lin and Sheu proposed an optimization method combining PSO and Simulated Annealing (SA) algorithms for the magnetic delay optimization problem of Fe₃O₄ ferrofluid coating. This method conducted experiments through the uniform design method, used stepwise regression analysis to make the formula consistent with the experimental data, and then used two intelligent search methods, PSO and SA, to find the maximum delay. The optimized parameter combination could significantly improve the magnetic delay [10].

To sum up, existing research on the intrinsic relationship between multi-segment life prediction and structural design makes it difficult to take into account indicators such as service life, stiffness, and deformation control. Moreover, single signal processing is difficult to fully exploit the multi-scale sensitive features of vibration data, traditional prediction models are insufficiently accurate, and conventional optimization algorithms are prone to falling into local optima. Given this, this study combines BiLSTM and PSO to propose a CNC tool accessory structure MPLO method that integrates life prediction models to enhance the structural performance of CNC tool accessories while ensuring its service life. The innovation points are: first, a signal processing method grounded on Complete Ensemble Empirical Mode Decomposition with Adaptive Noise (CEEMDAN) and Fine-to-Coarse (FTC) is proposed, and fused with Convolutional Neural Network (CNN) and BiLSTM for life prediction. Secondly, a Fixture Positioning Layout (FPL) optimization model is constructed with the objectives of minimum maximum deformation, minimum overall deformation, and maximum service life. BPNN is used to accurately map the layout and deformation relationship, integrate the life prediction into the fitness function of the PSO algorithm, and adopt the ideas of Dynamic Inertia Weight (DIW) and Pareto Optimal Solution (POS) to achieve collaborative optimization of performance and life, avoiding traditional optimization that ignores the life problem.

2. Research Design

2.1 Structural Life Prediction Method of CNC Tool Accessories Based on CNN-BiLSTM

CNC tool accessories (the core is the fixture) are subjected to alternating loads and vibration impacts for a long time during cutting processing, and their life degradation process is highly related to the timing characteristics of the vibration signal. Accurately capturing this correlation is the core prerequisite for structural life prediction. Before optimizing the structure of CNC tool attachments, its lifespan needs to be predicted first. By accurately predicting the remaining life of a structure, key data support can be provided for structural optimization, helping to design more efficient, safer, and more economical structures. Vibration is a primary factor causing fixture damage, which induces fatigue crack initiation and aggravates wear. As the core stress-bearing component of CNC tool accessories, the amplitude and frequency changes of its vibration signal directly reflect the initiation and expansion process of fatigue damage, and are a key data source for life prediction. Therefore, this study constructs a vibration signal processing method that integrates CEEMDAN and FTC. First, the vibration signal needs to be preprocessed. For the problem of missing vibration signals, this study uses the interpolation method to approximately supplement the missing values, so that the signal fluctuation tends to normal. For outliers, this study uses the 3σ criterion for detection and processing through manual deletion or interpolation. The above preprocessing operation can effectively eliminate irrelevant noises such as electromagnetic interference and equipment start-stop impact in the cutting environment, retain effective vibration information directly related to fixture life degradation, and provide guarantee for the accuracy of subsequent feature extraction. After preprocessing the vibration signal, it can be decomposed by CEEMDAN to obtain six intrinsic mode function (IMF) components. Among them, IMF1 has the highest frequency and decreases in sequence. The high-frequency IMF component mainly corresponds to the instantaneous impact vibration during the cutting process, and is directly related to the fixture surface wear and micro-crack expansion. The low-frequency IMF component reflects the overall vibration characteristics of the structure and is related to the fatigue degradation trend of the fixture matrix. By analyzing the mean value of the IMF components, vibration characteristics can be revealed. After decomposing the signal, the FTC algorithm can be

used to extract its features. This method first screens high- and low-frequency IMFs, then constructs a pool of them, then calculates the fusion weight, and finally fuses and reconstructs the IMF components. Through these steps, the sensitive features in the IMFs component data can be fully obtained. The instantaneous impact load during milling processing will cause the local stress of the fixture to instantly exceed the yield limit of the material, resulting in plastic strain accumulation. A single overload will form micro defects on the surface of the component, and under multiple impacts, micro cracks will continue to propagate along the stress concentration points; Under the coupling effect of alternating vibration and overload, damage accumulates nonlinearly, and the damage rate increases exponentially with the peak load. From a mathematical perspective, the Miner linear damage accumulation criterion can be used to quantify the incremental damage caused by a single impact, which is positively correlated with the load amplitude and duration. When the total damage accumulates to a critical value, the fixture experiences fatigue failure, and this mechanism is also the core cause of high-frequency components in vibration signals [11]. The energy calculation of the IMF component is shown in Equation (1).

$$E_i = \int |IMF_i(t)|^2 dt = \sum_{j=1}^J |x_{ij}^2| \quad (1)$$

In Equation (1), E_i means the energy of the i -th IMF component. $IMF_i(t)$ is the i -th IMF component at time t . J is the number of IMF component data points. x_{ij} is the amplitude of each point in the IMF component. At this time, the calculation of the total energy of IMF E is shown in Equation (2).

$$E = \sum_{i=1}^n E_i \quad (2)$$

Since IMF components with small amplitude energy will be overwhelmed by IMF components with large amplitude energy, to avoid the above problems, the IMF energy also needs to be normalized to obtain weights. The weight formula is Equation (3).

$$\omega_i = \frac{E_i}{E} \quad (3)$$

In Equation (3), ω_i is the weight. The relationship between weight and IMF energy is shown in Equation (4).

$$\omega_1 : \omega_2 : \dots : \omega_k = E_1 : E_2 : \dots : E_k \quad (4)$$

In Equation (4), the weight is proportional to the IMF energy. At this time, the IMF component fusion result is defined as Equation (5).

$$F(t) = \omega_1 IMF_1 + \omega_2 IMF_2 \dots + \omega_k IMF_k \quad (5)$$

In Equation (5), $F(t)$ is the IMF component fusion result. Finally, the vibration signal characteristics can be obtained by reconstructing the features using the phase space reconstruction method. After extracting the vibration signal features, CNN-BiLSTM can be used to predict the structural life of CNC tool accessories. In view of the nonlinear and long-term characteristics of the life degradation of CNC tool attachment fixtures, the CNN module can effectively extract local key information in the vibration characteristics. The BiLSTM module can deeply mine long-term dependencies in feature sequences and accurately capture the gradual law of life decay. The structure of CNN-BiLSTM is shown in Fig.1.

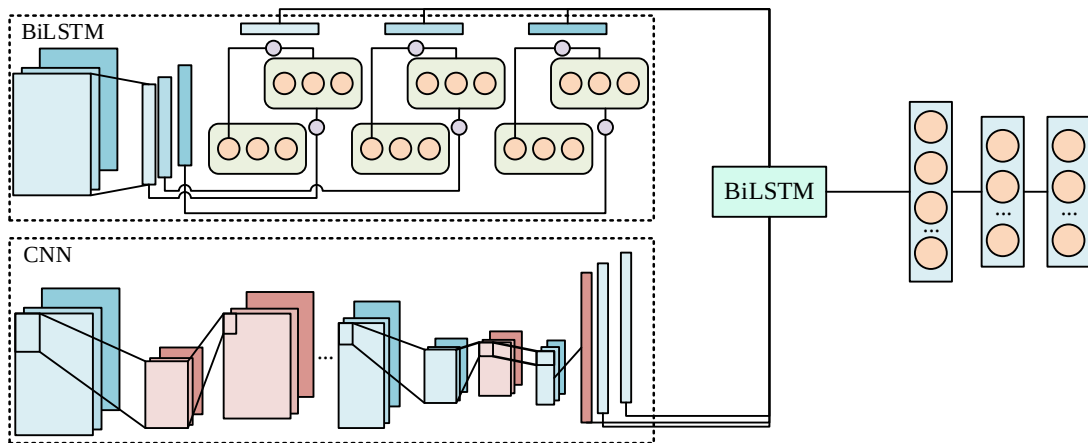


Fig. 1. Framework of CNN-BiLSTM

In Fig.1, the model adopts a dual-branch structure: CNN extracts the lifetime degradation characteristics of low-frequency data, and BiLSTM extracts the lifetime degradation characteristics of high-frequency data. After the features are fused, the Fully Connected Layer (FCL) predicts the remaining service life. The model architecture is specifically adapted to the complex characteristics of the fixture vibration signal. The collaborative design of the dropout layer and the FCL can effectively avoid prediction overfitting caused by fluctuations in cutting conditions, ensuring that the remaining life prediction results of the fixture can be stably output under different processing loads. Fig.2 shows the framework of the designed CNN model.

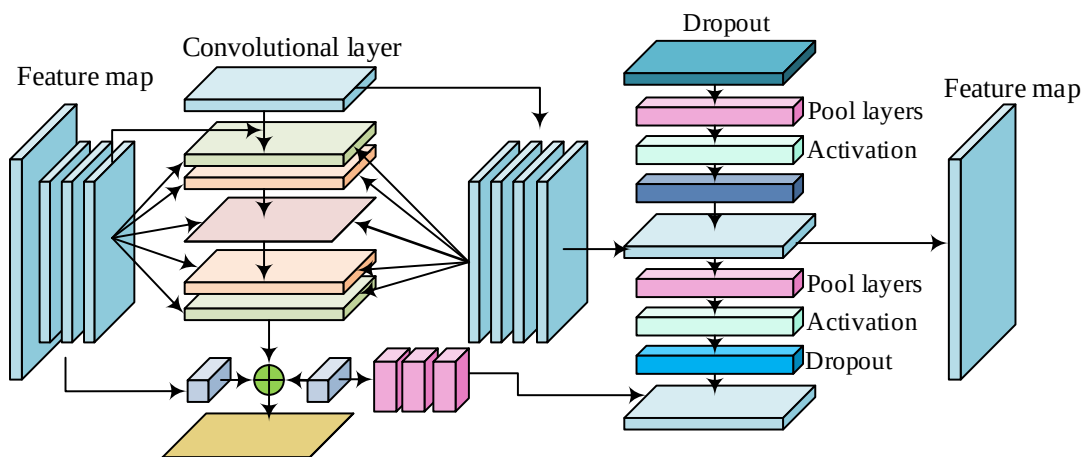


Fig. 2. Structure of CNN

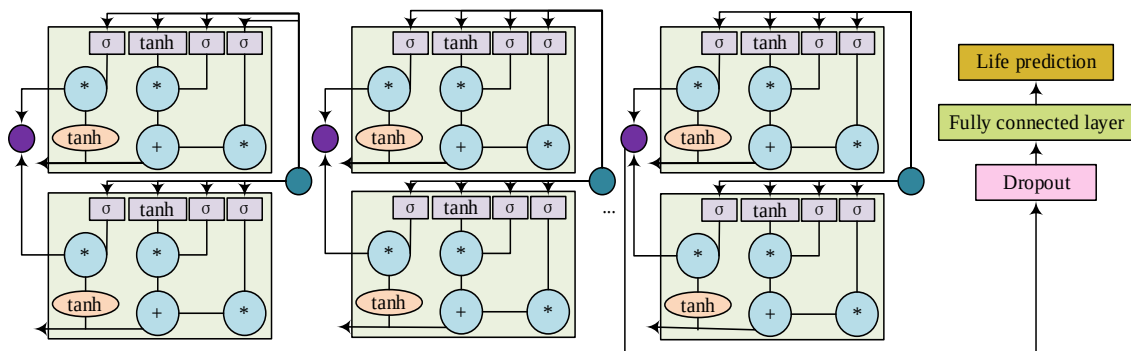


Fig. 3. Structure of BiLSTM

In Fig.2, the constructed deep CNN model contains a total of 4 convolutional blocks in series. Each convolutional block uses a 1D convolutional layer as the core computing unit, equipped with a ReLU activation function for nonlinear transformation, followed by a 1D maximum pooling layer for

downsampling. To balance feature extraction capability with sequence length preservation, the first and fourth convolutional blocks retain pooling operations to compress the data dimension and enlarge the receptive field. The second and third convolutional blocks deliberately remove the maximum pooling layer, and only extract local deep features through the convolutional layer to avoid the loss of key timing information caused by excessive downsampling, thereby more precisely capturing the pattern features in the signal [12]. The structure of BiLSTM is shown in Fig.3.

In Fig.3, the structure includes an input layer (IL), a BiLSTM layer, an FCL, and an output layer (OL). Among them, the number of neurons in the 1st and 2nd BiLSTM layers is 300 and 150, the number of neurons in the FCL is 128, the activation function is ReLU, and the Dropout ratio is 0.5. For the number of predicted branches, this study uses the grid search method to determine [13-15]. For the prediction results of each branch, this study uses a weighted voting method to integrate them. The voting weight is calculated as shown in Equation (6).

$$w_i^j = \frac{P_i^j}{\sum_{k=1}^K P_{r_i}^k} \quad (6)$$

In Equation (6), w_i^j and P_i^j are the weight and prediction percentage of test sample i in prediction branch j . The final prediction results are shown in Equation (7).

$$R = w_i^1 R_i^1 + w_i^2 R_i^2 + \dots + w_i^k R_i^k \quad (7)$$

In Equation (7), R and R_i^k are the prediction results of the final and branch paths. Through the above method, the structural life of CNC tool accessories can be accurately predicted.

2.2 CNC Tool Attachment Structure Optimization Method Based on BPNN-PSO

After life prediction is performed through CNN-BiLSTM, the CNC tool attachment structure can be optimized based on the prediction results. As an important accessory structure of CNC tools, the performance of the fixture has a crucial impact on production efficiency and product quality. FPL plays a vital role in mechanical processing. It not only directly affects the processing accuracy, efficiency and final quality of the workpiece, but also has a significant impact on the service life of the fixture. Therefore, to improve the processing quality and service life, this study proposes the FPL-MOO method based on BPNN-PSO. The FPL-MOO model is Equation (8).

$$\begin{cases} Findx = [x_1, x_2, \dots, x_N, \alpha_1, \alpha_2, \dots, \alpha_N] \\ min f_1(x) = \max\{|u_i(x)|\} \\ min f_2(x) = \sqrt{\frac{\sum_{i=1}^K u_i^2(x)}{K}} \\ max f_3 = L(x) \\ s.t. x_i \in \Omega, x_j \in \Omega \text{ and } x_i \neq x_j, \alpha_i \in \Gamma \end{cases} \quad (8)$$

In Equation (8), x is the design variable. x_i is the coordinates of the anchor point. α_i is the contact characteristic parameter of the positioning point. $f_1(x)$, $f_2(x)$, and $f_3(x)$ are the maximum deformation, overall deformation and service life. u_i is the Z-direction displacement of the node. K is the number of nodes. $L(x)$ is the remaining service life. Ω is the set of feasible regions. Γ is the feasible region of contact parameters. The candidate fixture layout design variable x is input into finite element simulation to obtain vibration timing signals, which are preprocessed by CEEMDAN-FTC to extract multi-scale features and fed into the trained CNN BiLSTM life prediction model to output the $L(x)$ corresponding to the layout. Complete calculation chain: layout parameters $\rightarrow$ simulated vibration data $\rightarrow$ feature extraction $\rightarrow$ CNN BiLSTM inference $\rightarrow L(x)$. Construct a PSO fitness function by combining the remaining service life with the maximum and

overall deformation, to achieve quantitative solution of fitness for each generation of particles and ensure algorithm reproducibility. The objective function of minimizing the maximum and overall strain in this article is directly related to the flow stress of the fixture material: the lower the yield flow stress of the material, the more significant the plastic deformation under the same clamping load. Simultaneously milling the thermo mechanical coupling stress will intensify strain accumulation, and the cutting temperature and feed rate in the machining process will change the instantaneous stress field of the component. As a core structural factor, the positioning layout can change the stress concentration area. Reasonable arrangement of positioning points can disperse the thermal mechanical composite load, reduce local stress peaks, suppress plastic deformation beyond flow stress, and achieve strain index optimization [16,17]. To optimize FPL, this study adopts the POS idea, generates samples through uniform experimental design method, establishes a BPNN model, and maps the relationship between FPL and thin plate deformation. Finally, MOO is performed to get POS. The optimization process based on BPNN-PSO is shown in Fig.4.

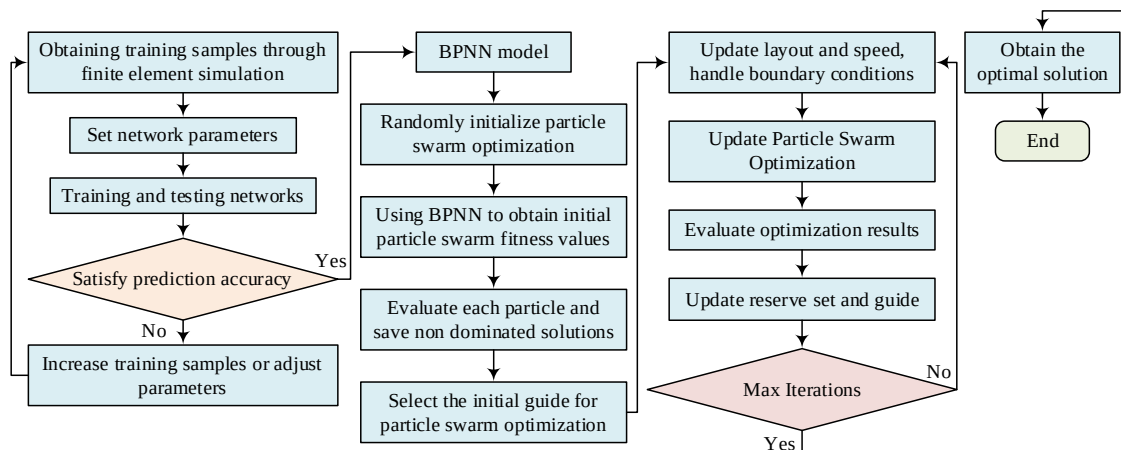


Fig. 4. Optimization process based on BPNN-PSO

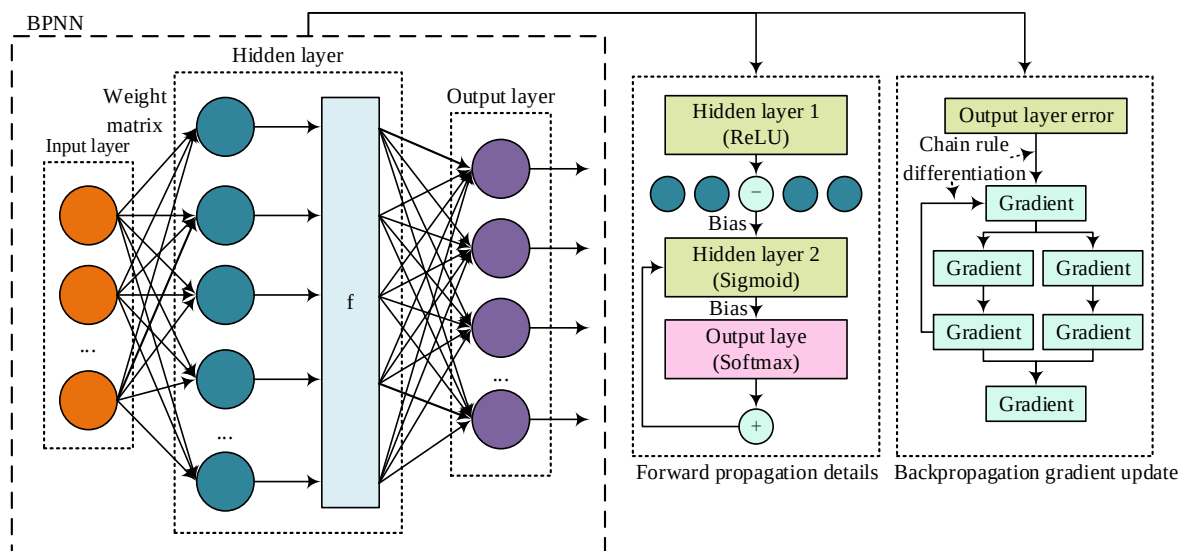


Fig. 5. Structure of BPNN

In Fig.4, first, the layout samples are uniformly selected within the positioning constraint range, and the deformation data is obtained through finite element simulation, and a three-layer BPNN prediction model is constructed at the same time. The network parameters are adjusted by comparing the prediction errors, the positioning layout is encoded, the initial particle swarm is generated, and BPNN is used to evaluate the fitness. By optimizing the solution set through the Pareto dominance relationship and the GD evaluation index, the optimal positioning point coordinate solution set and the corresponding deformation are obtained. The fixture material for

the finite element model is 45 steels, with an elastic modulus of 210GPa and a Poisson's ratio of 0.3; Adopting a solid hexahedral structured grid and encrypting the grid in key positioning areas. Boundary conditions: The fixture base is fully fixed with constraints, and milling equivalent alternating cutting loads are applied to each positioning point. After completing the grid independence verification, batch simulation is conducted to extract the Z-direction displacement of nodes under each layout, calculate the maximum deformation and overall deformation mean, and use them as input sample data for the BPNN network. The structure of BPNN is shown in Fig.5.

In Fig.5, BPNN includes an IL, one or more hidden layers, and an OL. Each node (neuron) in the network is connected by weights and contains an activation function to introduce nonlinearity. The IL receives data, the hidden layer processes and extracts features, and the OL generates prediction results. Since the performance of BPNN is affected by the number of hidden layers and nodes, too much will take more time. Therefore, this study uses two layers of hidden layers and determines the number of nodes through a trial-and-error method [18]. The number of IL nodes of the model is the number of parameters that affect the fixture layout, and the number of OL nodes is determined based on the prediction target. At the same time, this study reduces the prediction error to the target value by adjusting the weights, and builds a multi-objective neural network based on the sim function of the MATLAB toolbox. The expression of the number of hidden layer nodes h is shown in Equation (9).

$$h = \sqrt{l + m} + c \quad (9)$$

In Equation (9), l means the number of neurons in the IL. m denotes the number of output nodes. c is a positive integer. Then, the PSO algorithm can be used to implement the fixture layout. The MOO process based on PSO is shown in Fig.6.

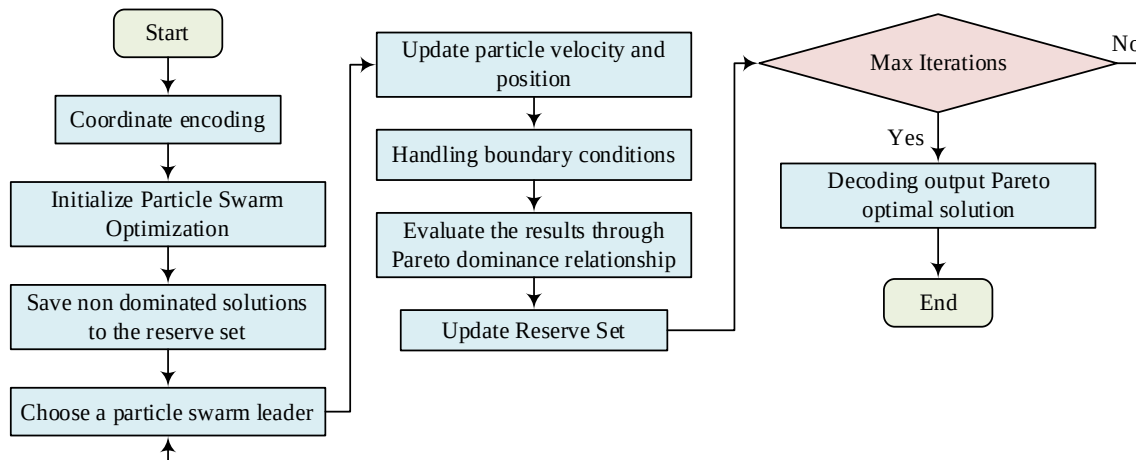


Fig. 6. MOO process based on PSO

In Fig.6, the optimization variables such as the fixture positioning coordinates are first encoded in real numbers, the search boundary and population size are preset, and the particle parameters are randomly initialized. Then, based on multi-objective functions such as clamping error and stability, individual fitness is calculated. The Pareto dominance relationship is used to filter non-dominated solutions to construct elite files, and the particle speed and position are updated accordingly. If a boundary violation occurs during the iteration, the boundary absorption strategy can be used to limit the particles to the feasible region to ensure the convergence of the optimization process and the validity of the solution [19,20]. The iterations are repeated until convergence, and the POS set is obtained, which is the optimal solution of FPL. The speed update formula of PSO is shown in Equation (10).

$$s_i(t + 1) = \beta s_i(t) + m_1 r_1(t)(v_{si}(t) - x_i(t)) + m_2 r_2(t)(v_{gi}(t) - x_i(t)) \quad (10)$$

In Equation (10), $s_i(t+1)$ is the particle speed when iterating $t+1$ times. β is the inertia weight coefficient. m_1 and m_2 are acceleration constants. Both r_1 and r_2 are random numbers. $x_i(t)$ is the particle position at $t+1$ iterations. v_{si} and v_{gi} are individual optimal values and global optimal values. The position update formula is shown in Equation (11).

$$x_i(t+1) = x_i(t) + s_i(t+1) \quad (11)$$

Since the inertial weight has a significant impact on the particle speed, to avoid falling into a local optimum and ensure the convergence speed, this study uses DIW. Equation (12) is the inertia weight formula.

$$\beta = \beta_{\max} - \frac{(\beta_{\max} - \beta_{\min})t}{T_{\max}} \quad (12)$$

In Equation (12), $\beta_{\max}$ and $\beta_{\min}$ are the maximum and minimum inertial weights. $T_{\max}$ means the maximum iterations. For the convergence of the optimal solution set, this study uses generation distance to evaluate. Equation (13) is the generation distance formula.

$$D = \left(\frac{1}{n} \sum_{i=1}^n d_i^p \right)^{1/p} \quad (13)$$

In Equation (13), D is the generation distance. n is the number of Pareto front solutions. d_i is the Euclidean distance between individual i and the closest individual to the real Pareto front. p takes the value 2. MOO for FPL can be achieved through the above method.

3. Results

3.1 Analysis of Life Prediction Results

To verify the proposed life prediction model, this study tests and compares it with Wasserstein Generative Adversarial Network-LSTM (WGAN-LSTM) and Residual Convolutional Block Attention Time Convolutional Network-Vision Transformer (RCATCN-ViT). The experimental dataset is CMAPSS, which contains four subsets, with a total of 709 training samples and a total of 707 test samples. The CMAPSS dataset contains 4 sets of aircraft engine timing samples, recording degradation monitoring signals such as temperature, pressure, and vibration throughout the entire lifecycle. It includes 709 sets of training samples and 707 sets of testing samples. The validation of the life prediction model was completed using the CMAPSS dataset for aircraft engines, both of which belong to mechanical degradation objects under cyclic alternating loads, and the degradation mechanisms have commonalities. The degradation mode of fatigue peeling and performance degradation of aviation turbine blades is highly consistent with the evolution law of fatigue cracks and surface wear caused by long-term cutting vibration and impact loads on milling machine fixtures. Time series signals that characterize performance degradation, such as turbine speed and exhaust temperature, can be compared to the vibration amplitude and frequency characteristics of fixtures; The trend of capacity attenuation throughout the entire lifecycle of the turbine corresponds to the degradation curve of the remaining life of the fixture. The prediction model trained on this dataset can be transferred and adapted to the task of predicting the life of fixture vibration signals. In the experiment, the learning rate and batch size are 0.0001 and 128, and the regularization parameter and maximum number of iterations are 1.2 and 200. To determine the optimal number of predicted branches, this study first analyzes the prediction results of different branch numbers. Experimental procedure: 1 Data preprocessing: interpolation to complete missing values, 3σ criterion to remove outliers, CEEMDAN decomposition of signals, FTC fusion of sensitive IMF components; 2. Model input: Normalize fusion features and divide fixed length sample sequences in chronological order; 3. Training process: Learning rate of 0.0001, batch size of 128, 200 iterations, using 10 branch weighted voting to output lifetime results, Dropout=0.5 to suppress overfitting, fixed random seeds throughout the process to ensure reproducibility of the

experiment. Prediction accuracy: Based on the true remaining lifespan, if the single sample error is within the allowable threshold, the prediction is considered valid. The proportion of valid samples to the total sample is the accuracy. Score is a specialized evaluation index for life prediction, and the larger the error, the more points will be deducted. The calculation formula is constructed based on the relative deviation between the predicted value and the true value; The higher the values of the two, the better the prediction performance. Table 1 is a comparison of predictions for different branch numbers.

In Table 1, as the Number of Predicted Branches (NoPB) increases, both MAE and RMSE first decrease and then increase. When NoPB is 10, both MAE and RMSE reach the minimum, which are 0.007 and 0.008. This shows that the optimal NoPB is 10. The comparison of remaining life predictions of different methods is shown in Fig.7.

Table 1. Prediction results for different numbers of branches

Number of branches	MAE	RMSE
3	0.043	0.045
4	0.037	0.040
5	0.031	0.032
6	0.025	0.025
7	0.013	0.017
8	0.012	0.013
9	0.008	0.011
10	0.007	0.008
11	0.010	0.013
12	0.010	0.014

Note: MAE Mean Absolute Error; RMSE Root Mean Square Error.

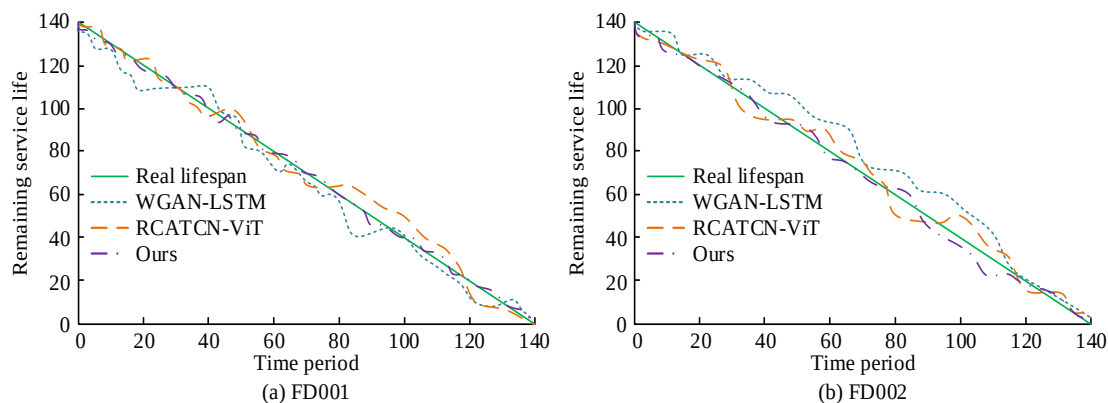


Fig. 7. Remaining life prediction results of different methods

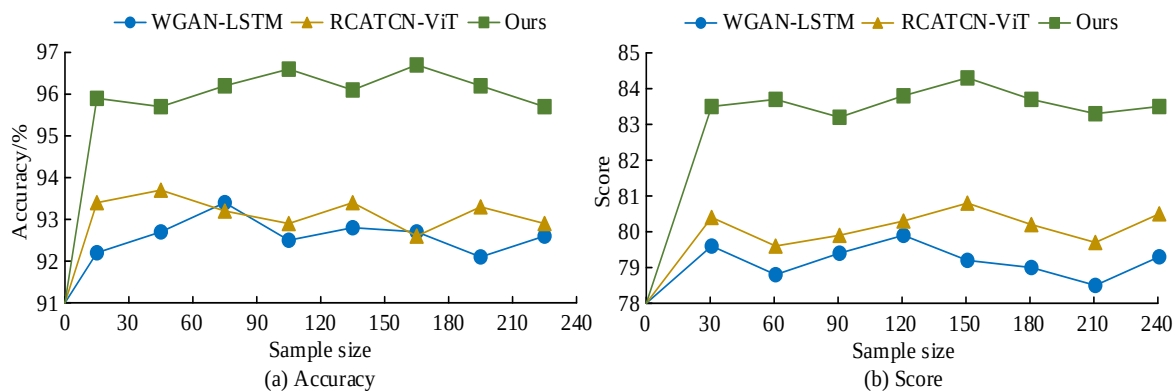


Fig. 8. Prediction accuracy and Score of different methods

In Fig.7(a), for the FD001 subset, the remaining life prediction curves of WGAN-LSTM and RCATCN-ViT fluctuate significantly, and the fit with the real life is poor. The curve of the research method

fluctuates less and basically fits the real-life span. In Fig.7(b), in the FD001 subset, compared with WGAN-LSTM and RCATCN-ViT, the curve of the research method also fluctuates less and has a higher degree of fit with the real-life span. This shows that the research method can achieve accurate prediction of remaining life. The prediction accuracy and Score of different methods are shown in Fig.8.

In Fig.8(a), the lifespan prediction accuracy of WGAN-LSTM and RCATCN-ViT is the highest at 93.4% and 93.7%, and the average accuracy is 92.6% and 93.2%. The highest prediction accuracy of the research method is 96.7%, and the average accuracy is 96.1%, which is higher than other methods. In Fig.8(b), the highest Scores of WGAN-LSTM and RCATCN-ViT are 79.9 and 80.8, and the average Scores are 79.2 and 80.2. The highest Score of the research method is 84.3, and the average Score is 83.6, which is also higher than the comparative methods. Therefore, the life prediction accuracy of the research method is higher. The comparison of predictive performance with integrated machine learning is shown in Table 2.

Table 2. Prediction performance comparison with ensemble machine learning

Model	MAE	RMSE
Random Forest (RF)	0.034	0.036
XGBoost	0.021	0.023
LightGBM	0.018	0.020
Proposed CNN-BiLSTM	0.007	0.008

According to Table 2, the MAE and RMSE of these three ensemble algorithms are significantly higher than those of the CNN BiLSTM framework. Thanks to the multi-scale feature fusion of CEEMDAN-FTC and the dual branch deep network architecture, this method can automatically mine long-term fatigue degradation rules, providing more accurate remaining life prediction for CNC tool accessories. To further explore the performance of the proposed method, this study conducts an ablation experiment on it, as listed in Table 3.

Table 3. Results of experiment

Model	CEEMDAN	FTC	CNN	RMSE
1	×	×	×	0.059
2	√	×	×	0.042
3	×	√	×	0.040
4	×	×	√	0.037
5	√	√	×	0.031
6	√	×	√	0.025
7	×	√	√	0.016
8	√	√	√	0.008

In Table 3, after introducing each module, the RMSE of the model is significantly reduced. CNN has the most significant impact on model performance. After introducing this module, the RMSE of the model drops by 37.3%. This shows that the improved method can effectively improve the life prediction accuracy of CNC tool attachment structures.

3.2 Analysis of Optimization Results of CNC Tool Attachment Structure

To verify the performance of the optimization method, this study tests it and compares it with the Multi-objective Gray Wolf Optimization Algorithm - Radial Basis Function (MOGWO-RBF) and the Improved PPSO-SA Algorithm (IPSO-SA). This study takes the fixture of a CNC milling machine as an example to test it. The iteration curves and POS sets of different methods are shown in Fig.9.

In Fig.9(a), compared with MOGWO-RBF and IPSO-SA, the research method has better convergence and converges after 90 iterations. Its generation distance is only 0.011, which is lower than other methods. In Fig.9(b), the Pareto optimal overall deformation and maximum deformation of the research method are smaller. Therefore, the optimization results of the research method are better. Table 4 shows the fixture layout optimization results under different weights.

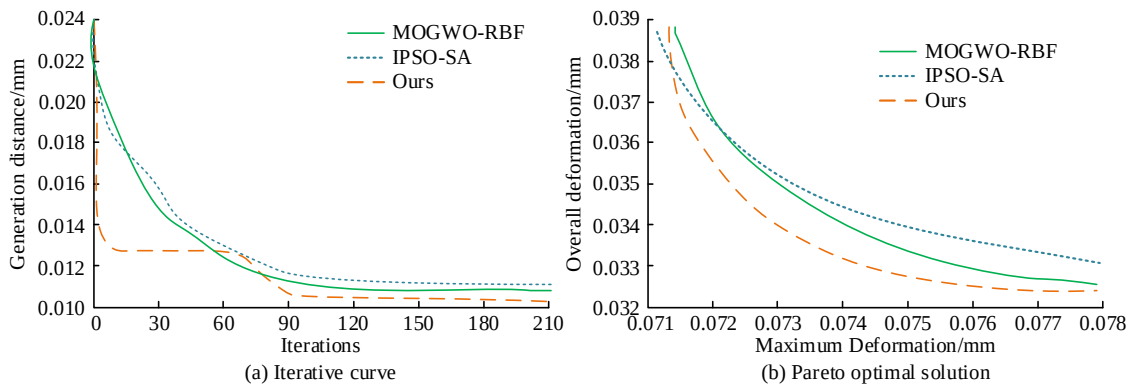


Fig. 9. Iterative curves and POS set for different methods

Table 4. Optimization results of fixture layout under different weights

Anchor point	Weight	MOGWO-RBF	IPSO-SA	Ours
1	0.5	(362.74,235.89)	(356.84,230.92)	(358.94,232.93)
	1.0	(360.22,232.51)	(356.15,234.34)	(358.17,235.47)
2	0.5	(166.38,341.20)	(165.18,337.27)	(164.53,338.18)
	1.0	(169.25,337.53)	(166.73,337.55)	(167.07,338.94)
3	0.5	(63.21,143.74)	(60.12,143.86)	(61.82,144.53)
	1.0	(63.05,148.16)	(62.76,148.97)	(61.06,147.07)
4	0.5	(257.24,43.56)	(257.11,42.36)	(255.47,41.82)
	1.0	(254.67,42.95)	(253.97,40.85)	(252.93,41.06)
Proportion of remaining lifespan	0.5	0.88	0.83	0.96
	1.0	0.84	0.78	0.93

In Table 4, compared with other methods, the layout optimization results of the research method have a larger proportion of remaining life. Taking a weight of 0.5 as an example, the remaining life proportions of MOGWO-RBF, IPSO-SA, and research methods are 0.88, 0.83, and 0.96. The research method can effectively balance the remaining life and performance of the structure when optimizing the CNC tool attachment structure.

4. Discussion and Interpretation

This paper focuses on the life prediction and MPLO of CNC tool attachment structures. By constructing an integrated method that integrates signal processing, DL, and intelligent optimization, it achieves a dual improvement in prediction accuracy and optimization performance. Relevant experimental data provide sufficient support for the effectiveness of the method. In life prediction experiments, the choice of NoPB has a significant impact on model performance. As NoPB increases from 3 to 10, MAE and RMSE continue to decrease and reach the minimum values (0.007, 0.008), while the error increases after NoPB exceeds 10. This shows that the weighted voting mechanism of 10 prediction branches can optimally integrate multi-dimensional degradation features and avoid the prediction deviation of a single branch and redundant calculations caused by too many branches. Compared with WGAN-LSTM and RCATCN-ViT, CNN-BiLSTM performs better in remaining life fitting, prediction accuracy, and Score indicators: The average prediction accuracy reaches 96.1%, which is 3.5% and 2.9% higher than the comparative method; The average Score is 83.6, which is 3.4~4.4 higher than the comparative method, verifying the model's capacity to capture long-term degradation patterns.

In the CNC tool attachment structure optimization experiment, with milling machines as the research object, the BPNN-PSO method is superior to MOGWO-RBF and IPSO-SA in terms of convergence and optimization effects. This method reaches stable convergence after 90 iterations, and the generation distance is only 0.011, which is lower than the comparison algorithm, indicating that the combination of DIW and POS ideas effectively balances global search and local convergence

efficiency. The optimization results show that under different weight settings, the FPL optimized by the research method can achieve smaller overall deformation and maximum deformation, and the proportion of remaining life is significantly increased: When the weight is 0.5, the proportion of remaining life reaches 0.96, which is 8.9% and 15.7% higher than that of MOGWO-RBF (0.88) and IPSO-SA (0.83). When the weight is 1.0, the proportion of remaining life still maintains a high level of 0.93, proving that this method can achieve the optimal balance between structural performance and service life. This result benefits from BPNN's accurate mapping of the relationship between positioning layout and deformation, and the efficient solution of the MOO problem by the PSO algorithm, which offers a quantitative basis for the engineering design of FPL.

Comparing this article's CNN BiLSTM with traditional probabilistic life prediction methods, the probabilistic model relies heavily on fixed degradation distribution assumptions and is difficult to adapt to complex coupled vibration conditions of fixtures, resulting in poor generalization. And the machine learning method in this article does not require a pre-set degradation model and can adaptively mine multi-scale sensitive vibration features; Paired with BPNN-PSO multi-objective optimization, it can simultaneously consider multiple indicators such as lifespan, stiffness, and deformation. Experiments have shown that compared to probabilistic methods such as Gaussian processes and Wiener processes, this approach has lower prediction errors, faster optimization convergence speed, and is more suitable for the collaborative design requirements of CNC accessories under multiple working conditions.

In summary, the proposed structural optimization method for CNC tool accessories can effectively improve the mechanical properties and service life of the structure. However, this study still has certain limitations, that is, the optimization process does not fully consider the constraints of processing costs and assembly processes. In the future, it should be considered to incorporate indicators such as economic cost and process feasibility into the optimization goals at the same time to build a MOO model that is more in line with engineering reality.

4. Summary

Aiming at the prominent contradiction that traditional structural topology optimization of CNC tool attachments ignores the coupling relationship between fatigue life and mechanical performance, this study constructs an integrated multi-objective topology optimization framework embedded with high-precision remaining useful life prediction model, and systematically verifies the effectiveness and superiority of the whole technical route through signal processing experiments, deep learning comparative tests and multi-objective optimization engineering cases. In the life prediction module, this paper combines CEEMDAN signal decomposition and FTC multi-scale sensitive feature fusion to fully mine degradation information hidden in cutting vibration signals, and builds a dual-branch CNN-BiLSTM prediction model with weighted multi-branch voting strategy. Grid search experiments confirm that setting 10 prediction branches can achieve optimal prediction accuracy, where the model's MAE and RMSE reach ultra-low values of 0.007 and 0.008 respectively, with an average prediction accuracy up to 96.1%. Ablation experiments further prove that CEEMDAN, FTC and CNN-BiLSTM modules produce obvious synergistic effects, and each component can independently reduce prediction error to a certain extent. Compared with WGAN-LSTM and RCATCN-ViT benchmark models, the proposed method shows higher fitting consistency with real life curves and superior comprehensive evaluation scores. For fixture positioning layout multi-objective optimization, this research adopts BPNN to establish an accurate nonlinear mapping between positioning coordinate layout and structural deformation, and introduces dynamic inertia weight improved PSO to solve Pareto optimal solutions with remaining service life, maximum deformation and overall deformation as joint optimization objectives. Taking milling machine fixture as the test object, the BPNN-PSO algorithm converges stably after only 90 iterations with a generation distance of merely 0.011. Under different weight coefficients, the optimized fixture layout achieves smaller structural deformation, and the remaining life ratio peaks at 0.96, obviously outperforming MOGWO-RBF and IPSO-SA contrast algorithms. The integrated scheme realizes coordinated balance of service life and structural stiffness, provides a complete digital design technical path for CNC tool accessories, and possesses great guiding value for actual machining fixture development. Future work will introduce manufacturing cost and assembly

process constraints to further fit practical industrial design demands. In the next research cycle, new datasets of fixture vibration for different types of machine tools such as lathes and drilling machines will be collected to conduct cross machine testing on the CNN BiLSTM life prediction and BPNN-PSO multi-objective optimization model proposed in this paper. By replacing different processing conditions and material samples to verify the generalization ability of the model, comparing the residual life prediction error and structural optimization convergence effect across devices, and comprehensively testing the adaptability and engineering practical value of the entire method in actual processing conditions of multiple types of machine tools.

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Experimental investigation on the stabilization of clayey soil using waste palm fiber and tire ash for geotechnical applications

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Abstract

Soil qualities need to be improved to address numerous geotechnical engineering issues. Improvements are needed in areas where soil cannot support loads. Soil improvement is essential in areas where the existing ground cannot support structural loads, particularly in problematic soils characterized by high water content, high compressibility, and low shear strength. This study evaluates the geotechnical performance of clayey soil stabilized with waste palm fiber (PF) and tire ash (TA). Soil samples made from clayey soil were used for this study; four percentages (0.2, 0.4, 0.6, and 0.8%) of palm fiber and three percentages (1, 3, and 5%) of tire ash were employed. The test findings show that increasing the content of palm fiber in clayey soil increases the plastic limit value while decreasing the liquid limit and plasticity index values. Additionally, the liquid limit and clay plasticity index decrease when tire ash is added to mixed soil with the optimum percentage of palm fiber (0.8%). As the amount of palm fiber increases, the optimum moisture content increased while the maximum dry density decreased. By adding more tire ash, the maximum dry density rises. The optimum moisture content of clayey soil continues to decline as the amount of tire ash increases. The treated clay's unconfined compression strength gradually improved as the amount of palm fiber increased, the UCS increased from 60 kPa for untreated clay to 83 kPa representing an increase of 38.33%. Additionally, compared to untreated specimens, the addition of tire ash and the optimum amount of palm fiber resulted in an extra improvement strength; with 0.8% palm fiber and 5% tire ash, the UCS reached 170 kPa, corresponding to a 183.33% increase relative to untreated soil.

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1. Introduction

Besides the foundation materials for projects, the soil properties are also significant in terms of construction material for many other projects. In order to solve many technical problems, soil improvement is necessary in areas where the existing soil cannot support imposed loads. For example, airfield runways and subgrade soil in highways need such improvements [1]. As the fine clay particles retain water and settle down more tightly than sand or loam. Clayey soils tend to retain water for extended periods due to their fine particle size. Incorporating granular materials, such as sand, can help achieve a balanced gradation and overcome these limitations.

Previous studies have investigated the variations in the hydrological properties of clayey soil with different proportions of sand particles. This process involving mixing of certain material with soil or blending of certain soils to enhance certain properties is referred to as stabilization [3]. According to [4], one of the most common techniques of fine soil improvement is stabilization through the addition of certain additives which modify the soil properties both physically and chemically. However, it is important to mention that the behavior of fine soils needs to be well understood prior

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to determining the soil improvement technique. The primary objectives of soil stabilization are to increase mechanical strength and bearing capacity, prevent structural subsidence, and reduce the plasticity index and adhesion of highly plastic soils.

Several studies have investigated the use of industrial and agricultural waste materials to improve the engineering properties of clayey soils. Singh and K.S.Gill found that the application of geogrid led to an enhancement in properties of the clayey foundation soil, where CBR increased from 2.9% to 9.4% at grid at 0.2 H depth [5]. Fattah et al. found that enhancement of three Iraqi clayey soils with the help of rice husk ash (RHA) caused a decrease in the liquid limit (11-18%) and plasticity modulus (32-80%) at 9% RHA, along with a reduction in the maximum dry weight and increase in the optimum moisture content. An increase in the unconfined compressive strength was also observed with 6-8% RHA contents [6]. Similarly, Kadhim et al. have proved that the addition of sludge ash (SSA) in percentages 10%, 15% and 20% to weak clayey soil leads to increase in unconfined compressive strength with curing times (3, 14, and 28 days). However, there was decrease in initial bearing capacity due to time curing; the best CBR ratio was 10% SSA. Other waste-based stabilizers have also demonstrated potential for improving clayey soils [7].

Cement was more effective in comparison with lime; stress-strain curves increased with fiber content increase. Strength enhancement of Kaolinite clayey soil through the addition of different percentages and mixes of Silica Fume and Cockle Shell Ash was studied by [8]. While Zaini et al. conducted a study on strengthening problematic soils by using Palm Oil Fuel Ash (POFA) combined with lime as a catalyst. They carried out various tests, including Atterberg limits, particle size distribution, compaction, and unconfined compression tests. The study involved adding 4%, 8%, and 12% POFA along with 6% hydrated lime to stabilize kaolinitic clayey soil over different curing periods (1, 7, 14, and 30 days). Compared to untreated kaolin, the mixture of POFA and lime resulted in an increase in undrained shear strength. The highest undrained shear strength value recorded was 32.68 kN/m² on the 30th day of curing, achieved when the kaolin was optimally stabilized with 6% lime and 12% POFA. The obtained UCS of 65.36 kN/m² was 185.04% higher than that of the untreated kaolinitic clayey soil [9].

Hasan and Shafiqu proved that the application of high-density polyethylene (HDPE) (6%, 9%, 12%) as non-traditional stabilizers leads to enhancement of physical characteristics of expansive clayey soils and decrease their swelling capacity [10]. In addition to particulate waste materials, fiber reinforcement has been investigated as a means of improving the mechanical behavior of soils. Shen et al. performed experimental investigations on clayey soil with reinforcement using polyester fibers and adding 5% of lime or cement. The findings indicated substantial improvements in soil strength in accordance with fiber content increase [11]. Sarkar et al. used waste plastic to support the clayey soil. Waste plastic and bottle strips are used as soil reinforcement to improve the engineering properties of subgrade soil. Waste plastic strips were manufactured for incorporation in two different proportions (1% and 2%) after being chopped in two distinct aspect ratios (5 mm x 5 mm and 5 mm x 10 mm). Unconfined Compressive Strength test was performed. The results show that the soil's shear strength increased when small bottle strips were combined in the bottle strip case [12]. Snodi et al. investigated the use of palm fibers to improve the collapsibility and shear strength of gypseous soil. Palm fibers were added at contents of 0.1%, 0.3%, and 0.5% by dry soil weight. The results showed that increasing the palm fiber content reduced the collapse potential and improved the shear strength parameters of the soil. At 0.5% palm fiber, the collapse potential decreased by about 40%, while cohesion and the internal friction angle increased by 52.78% and 18.57%, respectively. The study demonstrated that small amounts of natural palm fibers can effectively improve the engineering behavior of gypseous soil [13]. Qu and Zhu investigated the use of palm fibers as a sustainable material for improving clayey soil. The results showed that palm fiber reinforcement increased the soil's unconfined compressive strength, ductility, and energy absorption capacity, with the best performance obtained at 0.4% fiber content and 15 mm fiber length. However, the secant modulus decreased as the fiber content and length increased. The addition of palm fibers also changed the failure behavior from brittle to more ductile. Overall, palm fiber was found to be an environmentally friendly, economical, and effective material for soil stabilization in geotechnical applications [14].

While various additives have been explored, the synergistic effect of combining palm fibre (PF) and tire ash (TA) remains under researched. Therefore, this study aims to evaluate the mixing impact of these waste materials on the geotechnical performance of clayey soils.

2. Materials

2.1 Soil Properties

The clayey soil used in this study was collected from the Al-Taji site (East of Baghdad). The soil was first stored outdoors until it dried out, then stored inside an oven for 24 h to ensure complete drying at a temperature of 100–110 °C. The laboratory tests were performed on the soil to measure the physical and chemical characteristics of the soil. The main characteristics and classification of the soil are listed in Table 1. The grain size distribution of the soil was 2.8% sand, 40% silt and 57.2% clay. Based on USCS, the soil is classified as CL. ASTM D422-63 [15] was used for determination of the grain size distribution of the clayey soil.

Table 1. Characteristics of the soil used

Properties of soil	Values	Units
Specific gravity, G _s	2.68	-
Gravel	0	%
Sand	2.8	%
Silt	40	%
Clay	57.2	%
Classification by USCS		CL
Liquid Limit, L.L	39	%
Plastic Limit, P.L	20.5	%
Plasticity Index, P.I	18.5	%
Maximum dry density, MDD	16.4	(kN/m ³)
Optimum moisture content, OMC	19.5	%
SO ₃ content	0.015	%
Total soluble salts (TSS)	1.012	%
pH value	7.91	-

2.2 Palm Fiber

There are more than 140 million palm trees in the Middle East and North Africa. Some of the countries which lead in the production include Egypt, Iran, Saudi Arabia, United Arab Emirates, Pakistan, and Sudan.



Fig. 1. Palm Fiber used in this study

The production of this palm fiber used in agriculture leads to generation of over 48.8 million tons of waste every year. As such, the production of this palm fiber has become an achievement to note. Such countries utilize sustainable textile fibers derived from different palm fiber wastes like fruit twigs, branches, leaves, and flyers. In this case study, the palm fiber (PF) had a diameter 0.2 cm and a length of 2cm, corresponding to an aspect ratio (L/d) of 10 obtained from the palm trees around Tikrit City, Fig. 1. The physical parameters of the fibers are high strength and have a high natural capacity for water absorption, often holding up to several times their dry weight due to their porous structure and hydrophilic nature.

2.3 Tire Ash

Waste tire materials can be made from waste tires that are generated on a daily basis. Tire ash was produced through the process of incineration of those tires that were discarded or used before. Incineration of such waste tires was carried out under normal atmospheric pressure and temperature conditions (burning air) at a combustion temperature of 600 °C for 2–3 hours to obtain the ash. After combustion, the resulting tire ash was allowed to cool and sieved through sieve No.200. The rubber ash was kept in sealed containers right after sieving in order to prevent pre-hydrating.

3. Experimental Program

3.1 Sample Preparation

Sample preparation is one of the most significant processes in every experimental research work. After oven drying, the clay was reduced to a powdered form and then sieved using a sieve having pore size of 4.75 mm. In addition, clay and PF were mixed and homogenized for five minutes. In order to obtain an optimal percentage of PF to be used in the stabilization process, the percentage of Palm fibers used in the mixing process included (0.2%, 0.4%, 0.6%, and 0.8 %) with respect to the dry weight of the clay. The soil samples were separated into four equal portions whereby each part of the soil samples was mixed thoroughly with different percentage of PF. After identifying 0.8% PF as the optimum content, tire ash (1%, 3%, and 5% by dry weight) was blended into the optimum PF-clay mixture. Finally, they were compacted based on "standard proctor test " at optimum moisture content and maximum dry density.

3.2 Testing Procedures

A series of laboratory tests was conducted to evaluate the effects of palm fiber (PF) and tire ash (TA) on the geotechnical properties of the clayey soil. The testing program included the determination of the Atterberg limits, standard Proctor compaction characteristics, and unconfined compressive strength (UCS). The liquid limit and plastic limit were determined in accordance with ASTM D4318 [16], while the standard Proctor compaction test was performed according to ASTM D698 [17]. The unconfined compressive strength (UCS) test was conducted in accordance with ASTM D2166/D2166M [18]. Cylindrical soil specimens were prepared with a diameter of 38 mm and a height of 76 mm, giving a height-to-diameter ratio of 2:1. The specimens were compacted at the corresponding maximum dry density (MDD) and optimum moisture content (OMC). During the UCS test, an axial strain rate of 1% per minute was applied continuously until the specimen reached its peak stress. The maximum axial stress recorded during the test was taken as the unconfined compressive strength (UCS) of the specimen. The experiment flowchart for the study shown in Fig. 2.

4. Results and Discussion

Based on the experimental work done with PF, 0.8% was the optimum value of PF percentage. After that, experimentations were carried out with the optimum percentage of PF and different percentages of TA as follows.

4.1 Atterberg Limit Test

The effect of PF on the Atterberg limits of the soil was evaluated using the measurement of the Atterberg limits for PF-clayey soil. The Atterberg limits provide an indication of the presence of clay particles within the soil. The higher values of P.I and L.L indicate that the soil is more likely to expand

and contract due to changes in the moisture content of the soil. The Atterberg limits are shown in Table 2 and Fig. 3.

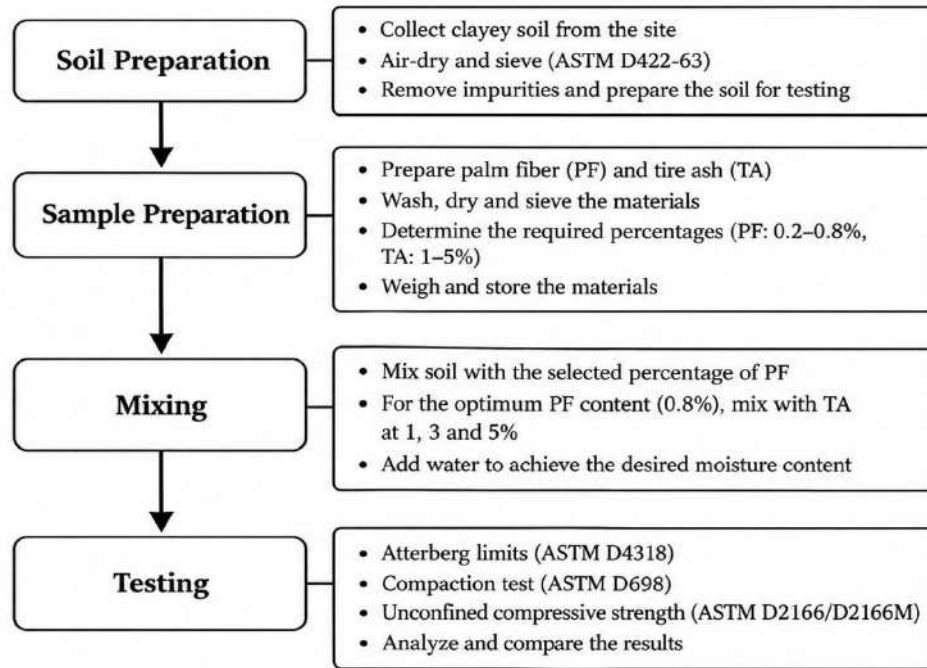


Fig. 2. Flowchart for the study

Table 2. Influence of PF on Atterberg limit

PF (%)	L.L (%)	P.L (%)	P.I (%)
0	39.0	20.5	18.5
0.2	37.9	24.4	13.5
0.4	36.3	25.1	11.2
0.6	33.8	25.2	8.6
0.8	32.4	26.0	6.4

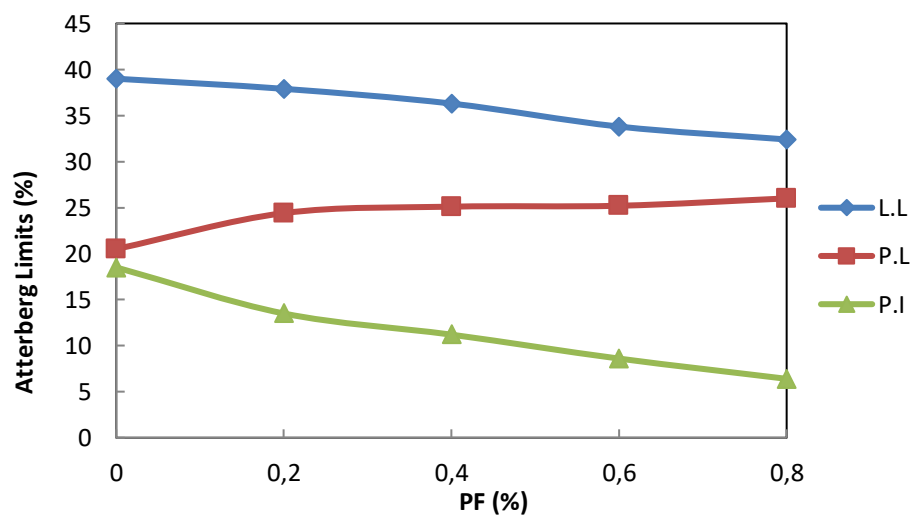


Fig. 3. Variation of atterberg limits with different percentage of PF

Based on the significant decrease in PI, 0.8% was the optimum value of PF percentage used. The above behaviors have also been noted in Ref. [19]. Results of Liquid Limit and Plastic Index test for clay with addition of PF and TA are listed in Table 3, where with increase in percentage of additive

used, there is decrease in liquid limit and plastic index of clay. From the above results, it can be inferred that the plasticity of the soil sample is decreasing with increase in the percentage of PF and TA.

Table 3. Influence of TA on Atterberg Limits of clayey soil treated with optimum of PF

% TA with optimum Palm Fiber	L.L. (%)	P.L. (%)	P.I.(%)
0.8 % PF+ 1% TA	32.1	25.8	6.3
0.8 % PF+ 3% TA	30.7	24.7	6.0
0.8 % PF+ 5% TA	29.8	24.3	5.5

4.2 Standard Proctor Test

Effect of PF on MDD and OMC is calculated from the compaction test of PF-clayey soil. Results for MDD and OMC of soil samples mixed with PF are given in Table 4 below and presented in Fig. 4.

Table 4. Influence of PF on (MDD) and (OMC)

PF (%)	MDD (kN/m ³)	OMC (%)
0	16.4	19.5
0.2	16.0	22.7
0.4	15.6	23.9
0.6	15.3	25.7
0.8	15.2	27.6

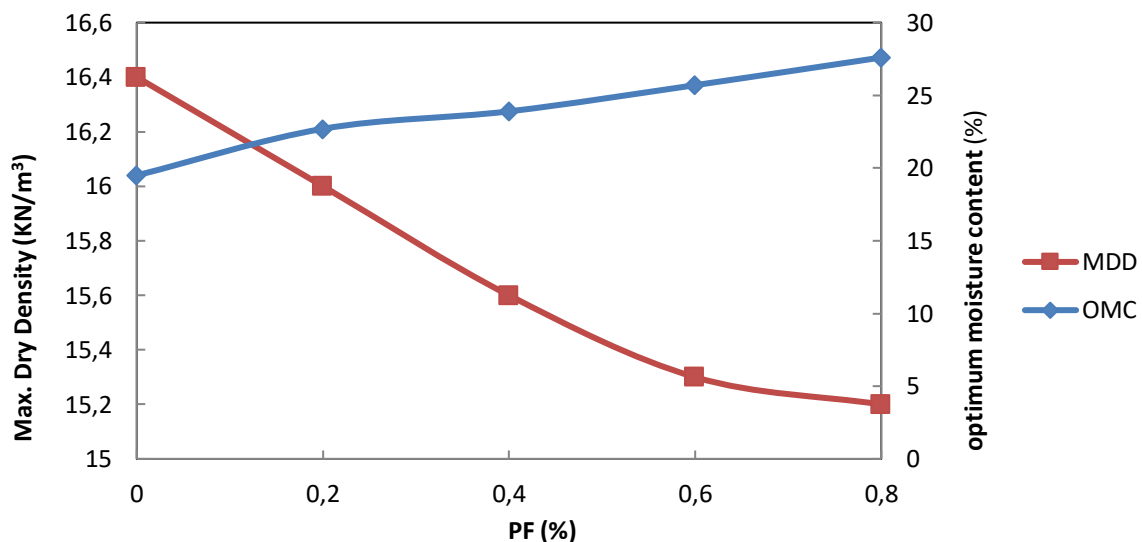


Fig. 4. Variation of MDD and OMC with different percentage of PF

From the result, the increment in OMC and decrement in the amount of MDD with an increasing proportion of PF addition can be seen. Similar trend has been observed in another study [20] in various materials mixture and stabilized soils. The increase in OMC with increasing palm fiber (PF) content can be attributed primarily to the relatively high-water absorption and water-retention capacity of the fibers. Palm fibers are coarse, porous organic fibers rather than fine-grained particles. Their lingo cellulosic structure contains hydrophilic functional groups that facilitate water absorption and retention within the fiber structure. Consequently, an increasing amount of mixing water is required to wet and partially saturate the fibers, in addition to the water needed for lubrication and rearrangement of the soil particles during compaction. The reduction in MDD of soil sample is because of partial substitution of heavier soils with lightweight PF.

As can be seen from Table 5, the MDD value increases with an increase in the percentage of TA. The observed increase may be associated with the combined effect of particle rearrangement and improved packing of the soil–PF. The fine TA particles may have occupied some of the void spaces between the soil and palm-fiber particles, resulting in a denser packing structure under compaction. While with the increase in the percentage of TA, the OMC of the soil continuously decreases. This is due to the decrease in the percentage of clay in the soil due to the substitution by TA. The addition of TA replaces a portion of the highly water-demanding clay fraction with relatively coarser and less water-absorbing particles. Consequently, the specific surface area and the amount of water required to coat the soil particles and facilitate lubrication during compaction are reduced. In addition, the fine TA particles can modify the soil fabric by promoting particle rearrangement and reducing the volume of micropores that retain water. Therefore, less water is required to achieve the maximum compacted state as the TA content increases.

Table 5. Influence of TA on (MDD) and (OMC) of clayey soil treated with optimum of PF

% TA with optimum Palm Fiber	MDD (kN/m ³)	OMC (%)
0.8 % PF+ 1% TA	16.8	28
0.8 % PF+ 3% TA	17.3	20
0.8 % PF+ 5% TA	18.0	19

4.3 Unconfined Compressive Strength Test

Cylindrical compacted samples with maximum dry unit weight (MDD) and optimal moisture content (OMC) were subjected to unconfined compression tests. Fig. 5 shows the test setup for the UCS test.



Fig. 5. Test setup for unconfined compressive strength

Table 6 provides specific results from these tests. As shown in Fig. 6 and mentioned by reference, it is clear that the unconfined compressive strength increases as the amount of palm fiber increases [21].

Table 6. Influence of PF on unconfined compression strength

PF %	Unconfined Compression Strength(kPa)
0	60
0.2	62
0.4	70
0.6	80
0.8	83

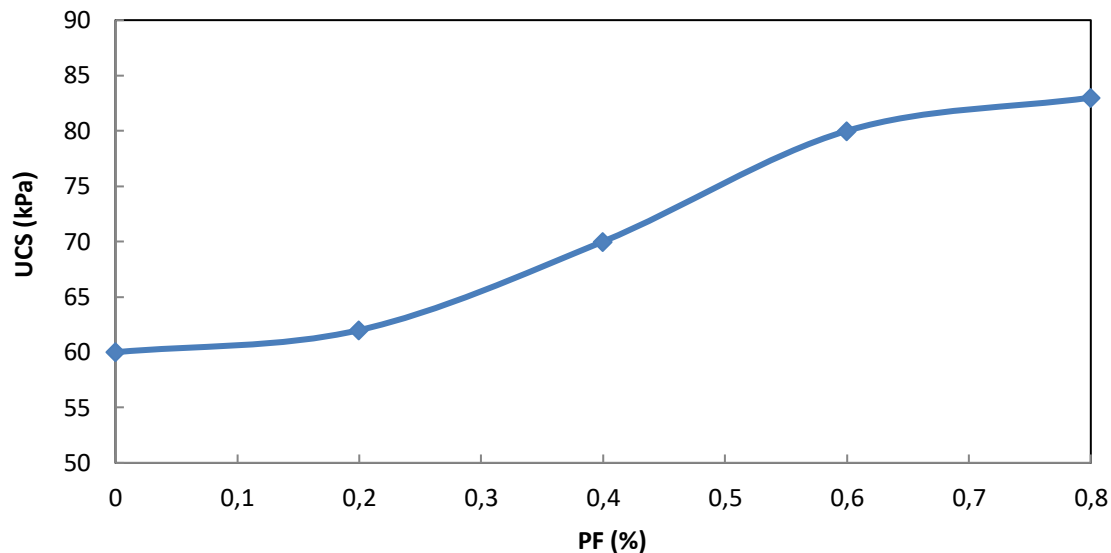


Fig. 6. The relationship between different percentage of PF and unconfined compressive strength values

It can be seen from Table 7 that the addition of Tire ash increased the UCS of the soil samples for the optimum palm fiber percentage. These behaviors were also observed by researchers for different materials lime, volcanic ash, rice husk ash, sugarcane straw ash, scrap tire rubber and synthetic fibers [22].

The pronounced improvement in UCS obtained with the palm fiber–tire ash (PF+TA) treatment can be attributed to the complementary mechanisms produced by the two additives. Tire ash primarily modifies the soil through particle rearrangement, void filling, and improvement of interparticle contact. Depending on its chemical composition, reactive mineral phases in the ash may also contribute to physicochemical bonding between the soil constituents. Such effects can produce a denser and more resistant soil. In contrast, palm fibers provide a discrete reinforcement mechanism. The fibers are randomly distributed within the soil and interact with the surrounding particles through friction and mechanical interlocking. Under axial loading, the fibers bridge developing cracks and potential shear surfaces, transfer tensile stresses, and restrict relative displacement of soil particles. Previous investigations of palm fiber-reinforced clay have demonstrated that these mechanisms can increase UCS and change the failure behavior from relatively brittle to more ductile [23].

Table 7. Influence of TA on (UCS) strength of clayey soil treated with optimum of PF

% TA with optimum Palm Fiber	Unconfined Compression Strength(kPa)
0.8 % PF+ 1% TA	120
0.8 % PF+ 3% TA	142
0.8 % PF+ 5% TA	170

5. Conclusions

The main conclusions of the present study could be summarized as follows:

- The addition of PF to the clayey soil reduced the liquid limit (L.L) and plasticity index (P.I) while raising the plastic limit (P.L). Moreover, the addition of TA to the 0.8% PF mixture results further suppressed soil plasticity, reducing the P.I to 5.5%. This demonstrates that (PF+TA) treatment successfully converts highly swell-prone clay into a more stable, workably inert soil.
- The Optimum Moisture Content (OMC) increased progressively with increasing palm fiber (PF) content, rising from 19.5% for the untreated clayey soil to 27.6% at the highest PF content. In contrast, the Maximum Dry Density (MDD) showed a decreasing trend with

increasing PF content. This behavior can be attributed to the relatively low density, low specific gravity and high-water absorption capacity of the palm fibers. Conversely, adding TA counteracted this effect by increasing MDD and reducing OMC, indicating enhanced structural packing and reduced water demand for field compaction.

- While the unconfined compressive strength (UCS) increases with an increase in the palm fiber content alone from 60 kPa to 83 kPa (a 38.33% increase), the blend of 0.8% PF and 5% TA achieved a peak UCS of 170 kPa representing a 183.33% strength enhancement relative to untreated clay. This synergism combines discrete fiber tension bridging with ash induced densification. The addition of tire ash increases the UCS of the soil samples at the optimum palm fiber percentage.
- The proposed stabilization method (PF+TA) offers compelling practical advantages for geotechnical field applications, particularly regarding environmental sustainability and cost-efficiency. Utilizing waste palm fibers and tire ash directly mitigates environmental landfill burdens by recycling abundant agricultural and industrial byproducts into high-value construction materials. Economically, replacing traditional, carbon heavy chemical stabilizers (such as cement or lime) with these readily available waste materials substantially lowers subgrade preparation and soil stabilization costs for infrastructure projects.
- Practical engineering and environmental implications the proposed stabilization technique offers a viable, eco-friendly alternative to traditional cementations binders (e.g., lime or cement) for pavement subgrades and foundation beds. Recycling regional agricultural waste (palm fibers) and industrial waste (tire ash) reduces landfill burdens while lowering material costs in subgrade preparation. The resulting soil exhibits improved bearing capacity and reduced moisture susceptibility, making it highly suitable for sustainable geotechnical infrastructure projects.
- Future research should evaluate long-term durability under diverse environmental conditions. Additionally, microstructural and mineralogical investigations using advanced techniques, such as XRD and SEM, are recommended to better understand the reaction mechanisms. Investigating the impact of PF and TA on other geotechnical properties, such as consolidation and hydraulic conductivity, is also suggested.

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Performance evaluation of modified self-compacting concrete incorporating UFBFS and nano-silica across varying aggregate densities and strength grades

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Abstract

Self-Compacting Concrete (SCC) mixes typically require higher binder content to address segregation that could hinder passing ability and increase the shrinkage probability; therefore, a modified approach is required to address these challenges. Building upon previous findings on M60 grade SCC designed with ultrafine blast-furnace slag (UFBFS) and nano-silica (NS), this study expands the scope of the modified SCC mix design approach to M50 and M70 SCC mixes, using granite-type aggregates as done in the earlier study. The experiment was extended to M50, M60, and M70 SCC mixes with basalt-type aggregates, using the same binder contents of 520 kg/m³, 565 kg/m³, and 610 kg/m³, respectively. Verified for flow properties, all the mixes achieved slump flow values between 650mm and 680mm, T₅₀₀ less than 6 seconds, a slump to J-ring flow difference less than 50mm, and a V-flow duration of 11 seconds or less. The 28-day compressive strength and flexural strength results for M50, M60, and M70 grade SCC fell in the ranges of 59.0 MPa to 83.4 MPa and 5.5 MPa to 9.20 MPa, respectively. Durability tests at 28 days showed water penetration of less than 10 mm and water absorption of less than 5.5%, and at 56 days, rapid chloride penetration results were less than 800 coulombs. This modified SCC mix design approach could be used across a wider region of India.

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1. Introduction

The ability of self-compacting concrete (SCC) to flow and fill formwork is considered both a material and a solution in time-bound construction projects. The major difference between a normal concrete and an SCC is the conversion of a plastic mass into a flowable material. However, such a conversion has its own challenges, such as resisting bleeding and segregation within a flowable mass, and maintaining homogeneity throughout manufacturing, transportation, and placement. Hence, SCC mixes are invariably designed with various filler materials, such as supplementary cementitious materials (SCMs), lime powder, and marble powder to achieve flowability with uniformity [1–7]. Construction of modern day high-rise residential apartments with a tunnel-housing system involves casting RCC slabs along with a series of interconnected, thin walls with cutouts for doors, windows, electrical/plumbing service requirements, and architectural projections. Other applications where SCC is required include critical structures such as shear wall structures, columns with heavy reinforcement, composite columns with hollow sections, and encased pre-engineered building (PEB) structural members, etc., where both placing and compaction of concrete are difficult. To overcome the casting challenges in these structures, designing and applying SCC mixes with flowability, optimal viscosity and cohesiveness is essential [8]. Equally essential is optimizing the binder content in SCC mixes so that heat of hydration and

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probability of shrinkage, especially at early age, could be limited and durability of the structure is enhanced.

In 1986, Okamura first proposed the idea of SCC to augment durable construction practices. In 1988, Ozawa and Maekawa at the University of Tokyo developed unique methods to design and validate fresh concrete properties for SCC. To demonstrate the functioning of SCC, a prototype application was carried out in 1988. In a pioneering effort, Ozawa et al. developed a deformable (flowable) mix containing lower amounts of coarse aggregate and higher proportions of powder materials, combining OPC, slag, and fly ash. Their experimental work found that, to achieve a flowable mix without segregation, a paste content sufficient to provide adequate viscosity was necessary. Their recommendation was to limit the coarse aggregate content to 50% by solid volume, with the remaining volume occupied by mortar with sufficient deformability to enable the fresh mix to flow without segregation [9].

To design and qualify an SCC mix, fresh concrete properties such as flowability, passing ability, segregation resistance, and viscosity are used as criteria [10]. Okamura and Ouchi noted that a highly viscous paste is required to prevent blockage of coarse aggregates as they flow through obstacles. They mentioned when concrete is deformed, paste with a high viscosity also prevents localized increases in internal stress due to the approach of coarse aggregate particles. High deformability can be achieved only by employing a superplasticizer, while keeping the water-powder ratio very low [11]. To achieve the fresh concrete properties, designing an SCC mix with higher binder content, more than that required in a normal mix, tends to be the concept as it increases the quantity of the paste medium to transport the aggregates with uniformity [12, 13].

However, designing an SCC mix using a powder-based approach is not an efficient approach, as increasing the binder content to achieve uniformity and prevent segregation requires numerous permutations and combinations. The investigation by Miao Lv et al. on various powder-based SCC mix designs concluded that “it might require lots of experiments and waste of material to obtain the SCC mix proportion with satisfying performance” [14]. Their approach, too, was to recommend only a modified powder-based mix design, with rigorous experimental analysis. A powder-based mix design approach increases the shrinkage behavior at both early and later ages of concrete. Various researchers have reported an increased probability of cracking in SCC mixes with higher paste content [15–17]. Hilmioglu et al., reported that cracking in SCC occurred at lower stresses than in normal concrete [18]. A powder-based SCC mix design approach could: increase the probability of shrinkage at both early and long-term ages, require a high number of trial mixes with varying permutations and combinations, rely on additional resources for bulk materials, and increase carbon footprint.

An alternative approach in designing SCC mix is to incorporate a viscosity-modifying agent (VMA) without requiring a much higher binder content. Khayat, K.H et al in 1997 tried VMA in high slump concrete mixes and concluded that “incorporation of a viscosity-modifying admixture is shown to significantly enhance the resistance to bleeding, settlement, and segregation [19]”. Although fillers such as limestone powder were also used to prevent segregation in SCC, their effectiveness is limited. Garcia L et al., reported that additional limestone fines reduced the mix's robustness during the T_{500} and V-Flow tests. They also reported that the mix cohesiveness, as measured by segregation resistance, improved when VMA replaced limestone fines [20]. An experimental observation by Gettu R et al., demonstrated that the robustness of the SCC mix improved with the addition of VMA [21]. However, reports indicated that VMA could adversely affect the hardened properties of concrete. J. Golaszewski reported that a viscosity-enhancing agent (VEA), another term for VMA, might affect compressive strength and recommended that the evaluation of SCC mixes not be limited to rheological properties alone [22]. Wojciech K et al., concluded that VMA use negatively affected the air permeability of concrete. Furthermore, the strength of CEM III concrete decreased with increasing VMA content [23]. Beata Lazniewska Piekarczyk mentioned that “superplasticizers (SPs) and VMAs from different sources cannot be used interchangeably, even if they appear to have a similar chemical composition” [24]. VMA availability in India is limited to very few suppliers, and its production is import-dependent. Apart from receiving the material on time, ensuring the compatibility between superplasticizers and VMAs from different

manufacturers is a major challenge. Moreover, issues such as the air-entraining tendency of VMA, which reduces the strength and durability of concrete, and its retarding effect, which results in delayed setting/hardening of concrete, are considered adverse impacts of VMA on SCC mixes, especially at higher dosages.

When designing SCC mixes, replacing OPC partially with fly ash is essential to achieve an economical mix and reduce the heat of hydration [25, 26]. Partial replacement of OPC is achieved through the addition of one of the two Supplementary Cementitious Materials (SCMs), namely, fly ash and ground granulated blast furnace slag (GGBS). It is also essential to reduce the total binder content, which is a combination of OPC and an SCM, to reduce the probability of concrete shrinkage. The reduction of OPC can be achieved by incorporating micro silica (MS) or ultrafine blast furnace slag (UFBFS). While MS, a by-product of the silicon and ferrosilicon industries, is imported into India, UFBFS is produced by grinding blast furnace slag (BFS), a by-product of the steel manufacturing process. UFBFS is produced and supplied in India either directly by steel manufacturers or by dedicated manufacturers.

Both MS and UFBFS have higher OPC replacement potential due to their higher pozzolanic index. MS, with its higher percentage of reactive silica (SiO_2) exceeding 85% and BET fineness of more than 15,000 m^2/kg , can replace OPC at a ratio of more than 1:1. Its ability to enhance mechanical properties such as compressive strength has been published through various research works, including that of Smarzewski, P [27]. Similarly, UFBFS with higher fineness, in excess of 1500 m^2/kg (BET method), and with reactive compounds in the form of calcium (CaO), silica (SiO_2) and alumina (Al_2O_3) can contribute to the strength with more than 100% efficiency when it replaces OPC in 5% to 10% range. Various studies indicated the beneficial effects of UFBFS on properties such as strength and durability [28–31].

Among MS and UFBFS, the latter can be considered more suitable in the Indian context. The reason is that MS, apart from its limited availability stemming from import dependence, can undergo partial de-densification of its densified particles during mixing, which can have a negative impact, as reported by Diamond S et al. [32]. In terms of influence on workability, while micro silica reduces the workability [33], ultra-fine slag enhances the workability [34], making UFBFS addition advantageous. Positive effect of UFBFS on workability could be attributed to its smooth surface texture derived from the parent granulated slag. MS supplied in bags poses a burden and a health hazard during storage and handling. UFBFS is supplied through bulk carriers directly from the manufacturing units. It can be stored in a silo and used in 'as produced form' through a batching system, a process that removes the dependence on manual storage and addition.

The reduction in binder content, including OPC, is compensated for by an increase in aggregate content to achieve the mix's design yield. Aggregates, especially coarse aggregates, are inert and act as internal restraints that mitigate shrinkage in concrete mixes [35]. However, this could increase friction among aggregate particles in the mix, leading to segregation. Though VMA is found to address these challenges, limitations of VMA in the form of technical challenges as mentioned above, coupled with a lack of VMA manufacturers to ensure quality and consistent supply of VMA in India, necessitate an alternate methodology. Such a methodology shall aid in designing an SCC mix as a uniform, flowing concrete mass with adequate viscosity and optimal binder content.

Nano-silica (NS), a novel material, has attracted the attention of many researchers in the fields of medicine and engineering due to its nanoscale efficacy. Its role in modifying the rheology of the cement slurry is significant in SCC mixes that require robustness and adequate yield stress. Kontoleon et al. reported that the yield stress increased considerably when NS was incorporated into the paste; on the other hand, changes in plastic viscosity were less pronounced [36]. [Click or tap here to enter text.](#), indicating ability of NS in making the paste medium viscous. The experimental investigations of G. Quercia et al. confirmed that the addition of nano-silica reduced the workability, indicating an increase in the mix's yield stress [37]. Xiuzhi Zhang et al. observed that the viscosity of SCC increased with increasing NS (less than 0.3%), but the yield stress values changed little [38]. The dispersion of NS with particle sizes less than 40 nm in fresh concrete mixes requires attention. While discussing the dispersion of NS in concrete mix, Kaffayatullah Khan et al., highlighted that poly carboxylate ether (PCE) based admixture is capable of NS dispersion in concrete mix [39].

Farid Alvansaz et al. concluded that the loss of workability in concrete mix with the addition of NS was addressed with superplasticizers [40]. Dispersion of NS ensures that larger nanoparticle agglomerates do not compromise the intended performance of the SCC mix. PCE-based superplasticizing admixture, which has become mandatory in SCC mixes, plays an additional role as a dispersing agent through its steric hindrance behavior and thus, dispersion of NS is addressed.

A previous study by the authors compared conventional powder-based M60-grade SCC with modified M60-grade SCC incorporating UFBFS and NS. The experimental findings indicated that, despite a reduction in total binder and paste contents compared with conventional SCC, the modified SCC achieved uniformity and met all flow criteria. The compressive strength results of modified SCC were comparable to those of the conventional SCC, and it even performed better in shrinkage mitigation and durability enhancement [41]. This study showed that while UFBFS was useful in reducing a higher quantity of OPC, NS, when used as the final ingredient, enhanced the uniformity and cohesiveness of M60 SCC mix that exhibited segregation due to reduced binder content, at an appropriate dosage. The learnings indicate the likelihood of adopting this modified approach in SCC mixes with varying grades, binder contents, and aggregate densities. In India, SCC has become a necessity in infrastructure projects, and in the construction of both commercial buildings and high-rise residential apartments. The types of rocks used to manufacture concrete aggregates in India can be broadly divided into 'granite' in the southern part and 'basalt' in the northern part. The properties of aggregates in these two types of rocks, such as relative density, shape, size, texture, particle distribution, strength, and brittleness, vary from one another. Among these, the higher relative density values of the aggregates would increase aggregate content and fresh concrete density, thus making SCC mix susceptible to segregation. If the modified approach to SCC mix design that validated the performance of M60 SCC with granite-type aggregates could contribute to similar performances in other grades of SCC mix and for other types (basalt) of aggregates, it could be considered as a unique solution for designing SCC mixes for the wider area of the country. Hence, the authors aim to provide an alternative method for the industry by examining and validating the modified SCC mix design approach for a wider range of SCC mixes through this experimental study. There are three objectives set in this experimental research work.

- The first objective is to incorporate UFBFS and NS into the design and evaluation of SCC mixes for two other grades, namely M50 and M70, with granite-type aggregates. The design of these two SCC mixes is based on the modified M60 SCC mix design approach.
- The second objective is to evaluate the performance of UFBFS and NS-based modified M50, M60 and M70 grade SCC mixes with basalt-type aggregates. The binder content and its combination are kept the same as in the mixes designed with granite-type aggregates.
- The third objective, inherent to the modified SCC mix design approach, is to simplify the mix design procedure, which helps in adopting SCC mixes without a large number of trials and difficult algorithms. The potential efficiency of NS in enhancing the uniformity of the SCC mix is considered a key contributor to this simplification objective.

The experimental work has been formulated with the following scope: designing SCC mixes using a modified approach incorporating UFBFS and NS, with NS added last. Validating the SCC mix designs by measuring the fresh concrete properties (Slump Flow, T_{500} and V-Flow tests as per EFNARC 2005, and J-Ring Flow and Visual stability index (VSI) as per ASTM C1621 and ASTM C1611), hardened concrete properties (compressive and flexural strength) and durability properties (rapid chloride penetration test, water penetration resistance and water absorption). The objectives of the present research work, as mentioned above, are to arrive at M50, M60, and M70 SCC mix designs with optimal binder content for two types of aggregates through a modified approach. The ability of the three mixes in meeting fresh concrete requirements and the strength and durability requirements is considered validation for such a modified approach.

2. Materials

2.1 Aggregates

Granite-type aggregates, both coarse and fine, were sourced from the crusher unit at Walaja, near Chennai, Tamil Nadu. Basalt-type aggregates, both coarse and fine aggregates, were sourced from

the crusher unit located near Indore in Madhya Pradesh. Both crushers consisted of a three-stage crushing process, with a vertical shaft impact (VSI) crusher being the third-stage crusher. Nominal maximum size (NMS) for both types of coarse aggregates was 12.5 mm. Crushed stone sand (CSS) was considered 100% of fine aggregates in experiments with granite-type aggregates. A combination of river sand (RS) and crushed stone sand at a 50:50 volumetric ratio was considered in experiments with basalt-type aggregates to achieve particle size distribution as per the Zone II requirement of IS 383. Particle size distributions of all aggregates are given in Table 1 and in Figs. 1 and 2. Impact value, crushing value, flakiness indices, elongation indices, relative density and water absorption values for both granite-type and basalt-type aggregates are given in Table 2. Fig. 3 shows the differences in appearance between granite-type and basalt-type CSS, with the latter being darker in color. Fig. 4 shows samples of coarse and fine aggregates from both sources used in the experiments.

Table 1. Particle size distribution of coarse and fine aggregates

S.I. No	Sieve Size	Granite-type, Chennai Cumulative passing (%)		Basalt-type, Indore Cumulative passing (%)			
		12.5 mm	CS sand	12.5 mm	CS sand	River sand	Combined CSS & RS
1.	20 mm	100	-	100	-	-	-
2.	12.5 mm	98	-	98	-	-	-
3.	10 mm	65	-	36	-	-	-
4.	6.30 mm	8	-	10	-	-	-
5.	4.75 mm	2	100	1	100	98	99
6.	2.36 mm	-	98	-	78	96	87
7.	1.18 mm	-	68	-	37	92	65
8.	0.60 mm	-	48	-	21	75	48
9.	0.30 mm	-	26	-	9	13	11
10.	0.15 mm	-	13	-	5	2	3
Fineness modulus of fine aggregates (IS 2386: Part I)			2.47	-	3.50	2.24	2.87

Table 2. Relative density and water absorption values of coarse and fine aggregates

Properties of aggregates	Granite-type, Chennai		Basalt-type, Indore		
	12.5 mm	CS sand	12.5 mm	CS sand	R sand
Relative density, SSD (IS 2386: Part III)	2.70	2.64	2.88	2.77	2.67
Water absorption (IS 2386: Part III)	0.50%	1.52%	1.42%	2.48%	1.35%
Flakiness Index (IS 2386: Part I)	8.40%	-	8.80%	-	-
Elongation Index (IS 2386: Part I)	11.20%	-	14.50 %	-	-
Impact value (IS 2386: Part IV)	23%	-	10.90 %	-	-
Crushing value (IS 2386: Part IV)	27%	-	17.20 %	-	-
Los Angeles Abrasion value (IS 2386: Part IV)	29%	-	26%	-	-

Note: In trial mix design calculations, water absorption values were used to adjust water and aggregate contents, depending on the moist/dry condition of the aggregates, to maintain free water to binder ratio and calculated aggregate contents of the mixes.

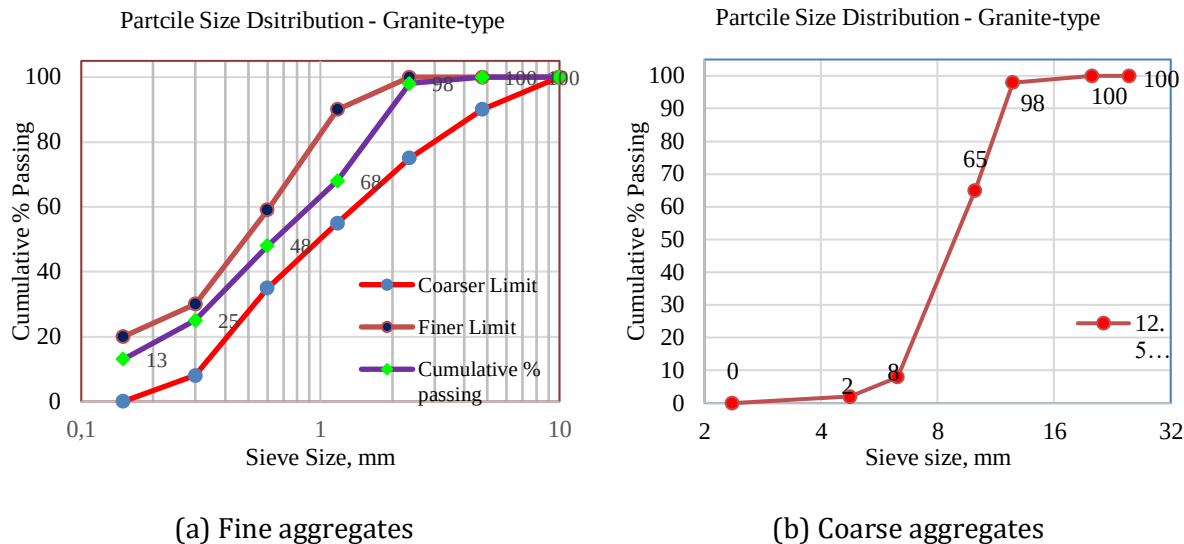


Fig. 1. Particle size distribution of granite-type aggregates

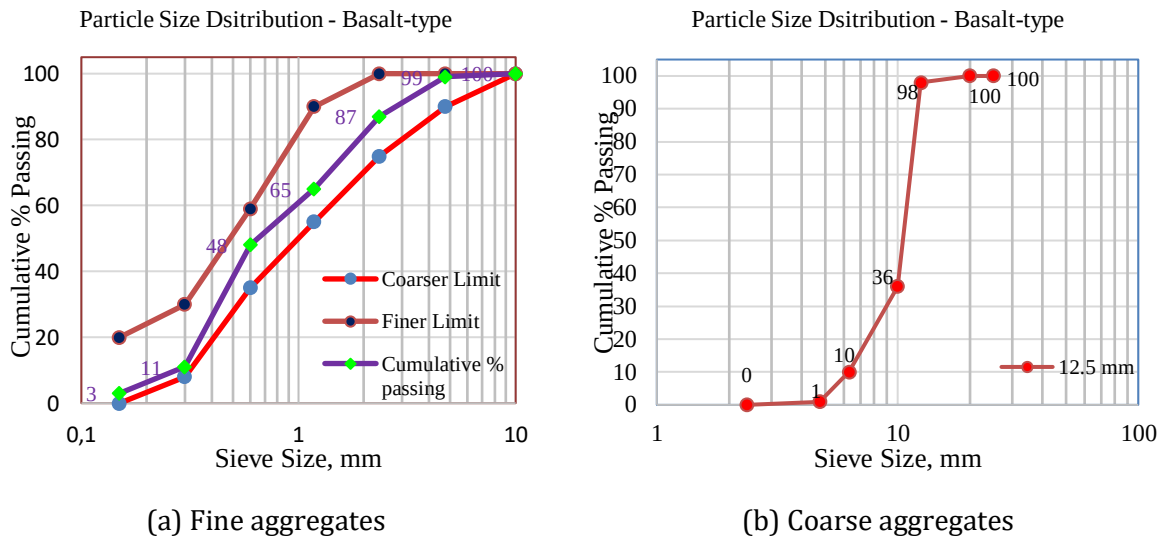


Fig. 2. Particle size distribution of basalt-type aggregates



(a) Granite-type CSS, (Chennai source) (b) Basalt-type CSS (Indore source)

Fig. 3. Granite-type and basalt-type (CSS) images in the stock yard, distinguishing the difference



(a) Granite-type 12.5mm & CSS, Chennai source



(b) Basalt-type 12.5mm, CSS and RS, Indore source

Fig. 4. Images of aggregate samples used in the trial mixes

2.2 Binder Materials, NS and Superplasticizer

The binder materials used to develop simplified SCC consisted of Ordinary Portland Cement (OPC) grade 53, class F fly ash, and ultrafine blast-furnace slag (UFBFS), commercially available as Alccofine 1208. For the consistency of properties, OPC procured from a single source, "UltraTech Cement" from Chennai, was used throughout the experimental program. The North Chennai Thermal Power Plant (NCTPP) was the source of Class F fly ash, which was collected from the electrostatic precipitator (ESP). The UFBFS material chosen was Alccofine (by Gujarat Ambuja Limited), finely ground blast-furnace slag with BET fineness of 2730 m²/kg, meeting the requirements as per IS 16715:2018 (amended 2019). Nano-silica (NS) was supplied by ASTRRA Chemicals, Chennai, in powder form with BET fineness of 202 m²/g and silicon dioxide (SiO₂) content 99.90%. The physical and chemical characteristics of the binder materials used in this present study are summarized in Table 3. A high-range water reducing admixture (HRWR), CONXL PCE 2635, a polycarboxylate ether (PCE) based superplasticizer, was sourced from Chemcon TecSys, Chennai, and was employed as the admixture for SCC trial mixes. The solids content, relative density, and pH of the CONXL PCE 2635 superplasticizer were measured at 40%, 1.10, and 6.1, respectively.

Table 3. Physical and chemical characteristics of constituent binding materials

Property	OPC	Fly ash	UFBFS	NS
Specific gravity (IS 2386 – Part III)	3.15	2.10	2.90	2.40
Blaine fineness (IS4031- Part II)	274 m ² /kg	326 m ² /kg	-	-
BET fineness (IS 15388/IS 16175)	-	-	2,730 m ² /kg	202 m ² /g
Average particle size	-	-	4.30 μm	0.017 μm
SiO ₂	20.40 %	64.20 %	32.30%	99.90 %
Al ₂ O ₃	4.90 %	26.64%	20.60%	-
Fe ₂ O ₃	4.90 %	4.96%	-	-
CaO	59.70 %	2.11 %	34.20%	-
SO ₃	2.80 %	0.19 %	0.26 %	-
MgO	1.80%	-	7%	-
MnO	-	-	0.35%	-
Chloride content	0.011%	-	0.015%	-
Equivalent Na ₂ O	0.41%	0.83%	0.48%	-
Loss on ignition	2.80 %	0.20 %	0.11 %	0.66 %

3. Mix Proportion Details of the SCC Mix

The SCC mixes were proportioned and tested in accordance with the European Guidelines for SCC mixes-2005 [10]. The mix design criteria adopted for the present investigation and its corresponding fresh property requirement are summarized in Tables 4 and 5, respectively. The mix proportions used in this investigation were obtained from the authors' earlier study [41], in which the simultaneous use of UFBFS and NS was employed to optimize the binder content to meet the fresh properties requirements of SCC. To meet the desired strength requirements, three SCC grades (M50, M60, and M70) were proportioned by selecting appropriate binder and free-water contents. The water binder ratio of M50, M60 and M70 grades was considered as 0.33, 0.29 and 0.25, respectively. The coarse and fine aggregates were kept at the same solid volume ratio (50:50), while the binder system comprised ordinary Portland cement (OPC), Class F fly ash, and UFBFS. The purpose of adding NS was to enhance the cohesiveness and stability of fresh SCC, and mixing was performed using a laboratory mixer; no sonication of NS was employed.

Table 4. SCC mix design parameters

Description	SCC mix		
Grade mix	M50	M60	M70
Target mean strength	$50+1.65\times5 = 58.25$ MPa	$60+1.65\times5 = 68.25$ MPa	$70+1.65\times6 = 79.50$ MPa
Maximum Water to Binder Ratio	0.33	0.29	0.25
Nominal Maximum Size of coarse		12.5 mm	
Assumed Air content		1.5%	
Workability Test	Flow (600-700mm), T500, V-Flow, J-Ring Flow & VSI		

Table 5. Fresh concrete property requirements for SCC

Fresh Concrete Properties	Test Method	Requirement
Flowing Ability (EFNARC)	Slump Flow	650 +/- 50mm
Viscosity (EFNARC)	T500	< 6 seconds
Viscosity (EFNARC)	V-Flow	< 12 seconds
Passing Ability / Segregation resistance (ASTM C1621)	J-Ring Flow	Difference between Slump Flow and J-Ring Flow < 50mm
Segregation Resistance (ASTM C1611)	VSI	0

3.1 Mixing Process

In the trial mix process, aggregates were placed in the pan mixer first, followed by the addition of UFBFS, OPC and Fly ash in sequence. Dry mixing was performed for 2 minutes, followed by another 2 minutes of mixing after adding the total water. PCE admixture was then added, and mixing was continued for 4 minutes to achieve the mix's workability/flowability. NS was added manually as the final ingredient, and mixing was continued for 2 minutes; this duration was sufficient to disperse NS and enhance the uniformity of the mix. Visual inspection before the addition of NS showed segregation in all the mixes, invariably. Visual inspection was performed after adding NS to ensure the flowable mix was uniform. The appearance of coarse aggregates on the surface of the fresh mix, with no bleeding after mixing NS, was considered a requirement for a uniform and cohesive mix. Appropriate moisture correction for fine aggregates and water absorption correction for coarse aggregates were carried out based on the moisture test results. Since the maximum water-to-binder ratio was 0.33, the entire mix water was added to make the mix as wet as possible, ensuring effective dispersion of the PCE admixture. Trial mixes for each grade were prepared in batches of 40 liters to ensure adequate volume and better uniformity for fresh concrete property tests and sampling for hardened concrete property tests.

4. Experimental Investigation

The modified SCC mix design approach used in the present study was developed from the authors' previous research on M60 SCC with granite-type aggregate. In that study, the combined

incorporation of NS and UFBFS aided in optimizing the binder content. The performances of that modified M60 SCC mix, in terms of fresh, mechanical, durability, and shrinkage properties, were measured to be satisfactory. Results showed that NS increased cohesiveness in the fresh mix, while UFBFS replaced a higher amount of OPC content without reducing hardened properties. The positive results from the modified mix design approach demonstrated its feasibility and functioned as a basis for the present study. This method has yet to be applied to SCC with basalt-type aggregate, however. Compared with granite-type aggregates, basalt-type aggregates have higher relative densities, which lead to higher fresh concrete density and a greater tendency towards segregation. The first phase of the present study is to extend the modified approach to M50 and M70 grade mixes designed with granite-type aggregates. The second phase is to evaluate the modified SCC mix design method for three grades of SCC: M50, M60, and M70, using basalt-type aggregate. Prior to the SCC mix study, paste studies were conducted with and without NS to gain insight into the role of NS in early-age strength development.

4.1 Influence of NS on Setting Time and Early Strength of Paste

In the present study, an experiment was conducted to measure the setting time pattern and compressive strength at 12 and 24 hours age for paste samples with and without NS. The purpose is to identify the early setting and hardening patterns of paste with NS compared with paste without NS, factors that are considered mitigating early-age cracking of concrete. Two pastes, one with pure OPC (0% NS) and another with a combination of OPC and 0.25% NS by weight of OPC, were prepared for both tests. The Vicat apparatus test set up shown in Fig. 5 was used to verify the normal consistency and setting times. Cube specimens measuring 50 mm × 50 mm × 50 mm were cast to determine compressive strength.

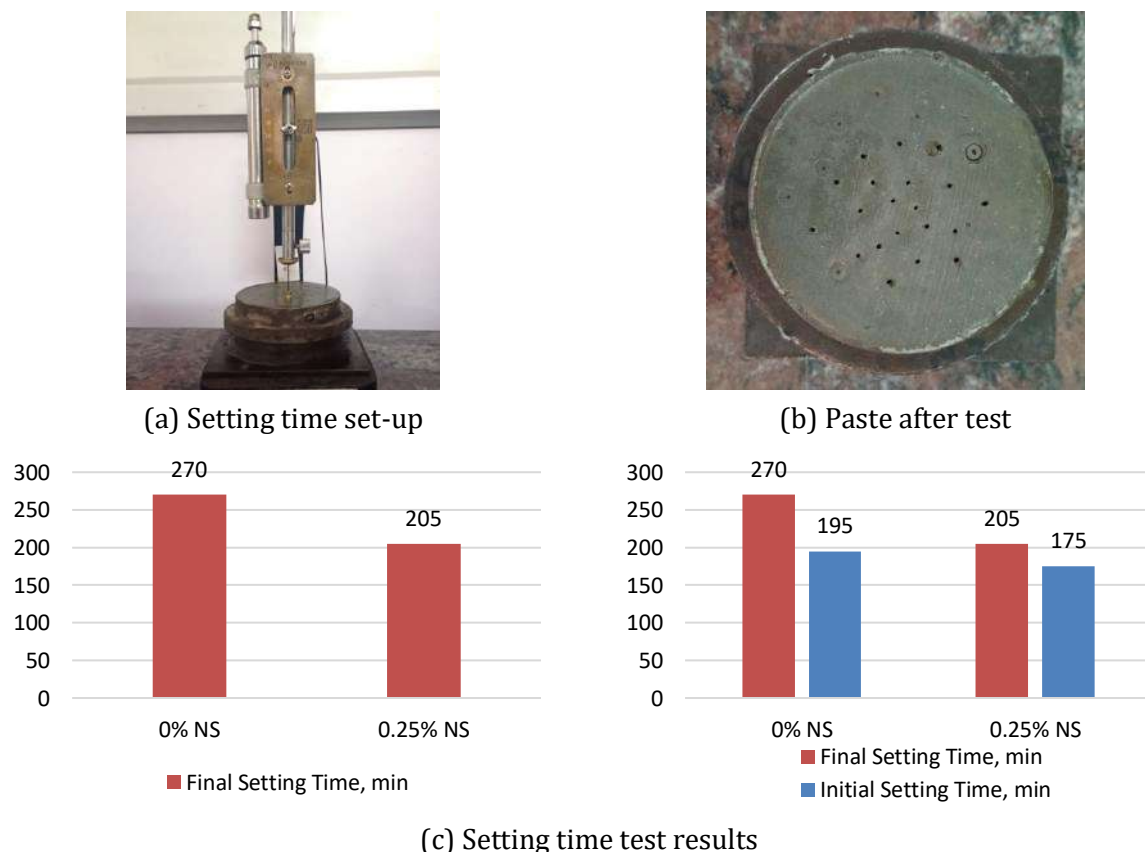


Fig. 5. Setting time test on pastes with and without NS

Pure OPC Paste was mixed in a 5-liter capacity laboratory digital paddle mixer. Initial mixing was done for 60 seconds at lower rpm followed by 60 seconds mixing at higher rpm. For the paste with 0.25% NS, after mixing of pure OPC paste for 120 seconds at lower and higher rpm, NS was added into the paste and was mixed at higher rpm for 60 seconds. Uniform mix was obtained for both the pastes with and without Nano-Silica. Both the sets of specimens in the mold

were kept inside the humidity chamber with top surface covered, at the temperature of $27 \pm 2^\circ\text{C}$ and at humidity of $95 \pm 5\%$. Specimens were removed from the molds just before testing. Water-to-cement ratio was maintained at 0.29 for both samples. The initial setting time and final setting time tests of pastes containing 0% NS and 0.25% are shown in Fig. 5 (c), and the corresponding cube compressive strength results are presented in Table 6.

Setting time tests conducted on both pure OPC paste and paste with 0.25% NS indicated that the paste with 0.25% NS reached the final setting 65 minutes earlier. Compressive strength tests conducted on these two paste samples showed that the paste with 0.25% NS achieved higher strength at 12 and 24 hours, with 35.60% increase at 12 hours, a significant difference, as shown in Fig. 6. The paste with 0.25% NS exhibited enhanced viscosity and rigidity, reaching setting time earlier and achieving higher strength in the early hours compared with pure OPC paste. Lavergne F et al., investigation showed that paste with NS promoted hydration and contributed to early strength [42]. As reported by Sun et al that, “nano-silica with small particle size provides more nucleation sites to form calcium silicate hydrate (C-S-H) seeds and to promote the hydration of C_3S and the formation of C-S-H [43]”. An experimental investigation by Paktiawal A and Alam M concluded that the utilization of nano- SiO_2 by partial replacement of cement significantly increased the hydration of the cement matrix and influenced the setting times of concrete [44]. Early setting time and early strength achievement of paste with NS are very similar to the findings of the above-mentioned research works. Early age strength gain in paste with the addition of NS is anticipated to enhance potential resistance against early age shrinkage cracking. It must be noted that the paste study was conducted with 100% OPC to isolate the pure nucleation effect of NS on OPC; pozzolanic synergy in the concrete matrix was investigated through concrete trial mixes in subsequent stages.

Table 6. Compressive strength results at the end of 12 hours and 24 hours

Compressive strength	0% NS	0.25 % NS	Strength gain of 0.25% NS
@ 12 hours	13.20 MPa	17.90 MPa	35.60%
Standard deviation	1.80 MPa	1.90 MPa	
@ 24 hours	30.20 MPa	34.40 MPa	13.90%
Standard deviation	1.90 MPa	1.60 MPa	

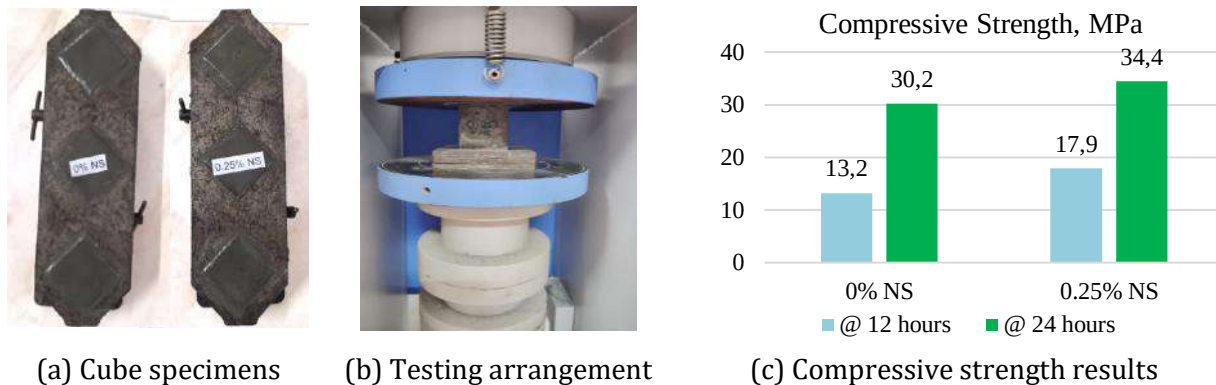


Fig. 6. Early age compressive strength outcomes from paste studies with and without NS

4.2 Design and Validation of M50 and M70 SCC Mixes with a Modified Approach Using Granite-Type Aggregates

Since the test results for both fresh and hardened concrete properties of M60 SCC (G) Mod, designed using the modified M60 SCC mix design approach in the earlier study, were satisfactory, the same approach was extended to design and validate M50 and M70 grade SCC mixes with granite-type aggregates. Binder contents were decreased and increased with respect to M60SCC (G) Mod in M50 and M70 SCC mixes, respectively, to meet the strength requirements.

Both these mixes are identified as M50 SCC (G) Mod and M70 SCC (G) Mod, where ‘G’ denotes granite-type aggregates. The volume of fine aggregates was maintained at 50% of the total aggregate content in both mixes. The mix proportions for M60, M50 and M70 modified SCC mixes

with granite-type aggregates are presented in Table 7. Flow tests conducted for both mixes are shown in Fig. 7 and Fig. 8. The test results for fresh and durability concrete properties of modified SCC mixes prepared using granite-type aggregates are presented in Table 8 and Table 9, respectively. The compressive strength and flexural strength progression of modified SCC mixes (M50 SCC (G) Mod, M60 SCC (G) Mod, and M70 SCC (G) Mod) at the age of 7 and 28 days, are presented in Figs. 9 and 10, respectively.

Table 7. M60, M50 and M70 modified SCC mix designs with granite-type aggregates

Type of materials	*M60 SCC(G) Mod	M50 SCC(G) Mod	M70 SCC(G) Mod
OPC	370	330	410
Fly ash	170	170	170
UFBFS	25	20	30
12.5 mm	827	840	821
CS Sand	809	821	803
Water	165	170	155
Total	2366	2351	2389
CONXL PCE 2635	4.8	4.40	5.30
Nano-Silica	1.2	1.40	1.0
UFBFS / OPC	6.80%	6.10%	7.30%
W/B ratio	0.29	0.33	0.25
Total binder content by weight, kg	565	520	610
Paste content by volume, liters	372.10	362.70	376.60
NS/OPC	0.32%	0.42%	0.24%

*Mix design of M60 SCC (G) Mod obtained from earlier study

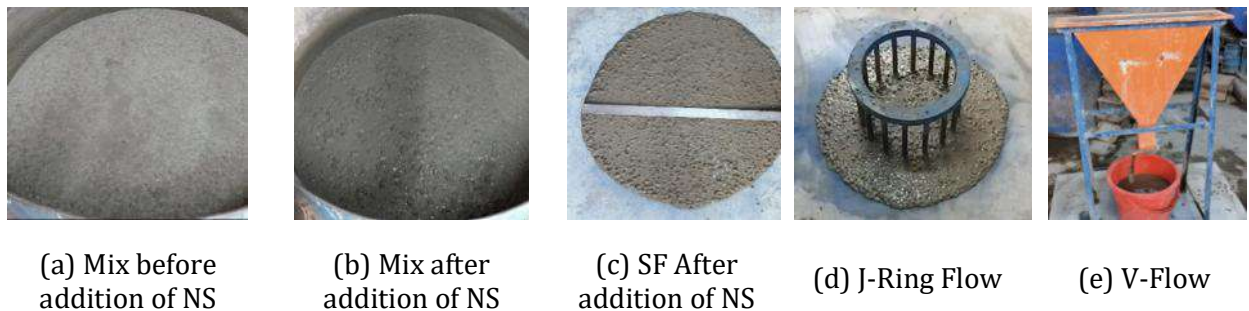


Fig. 7. Fresh concrete properties testing on M50 SCC (G) Mod

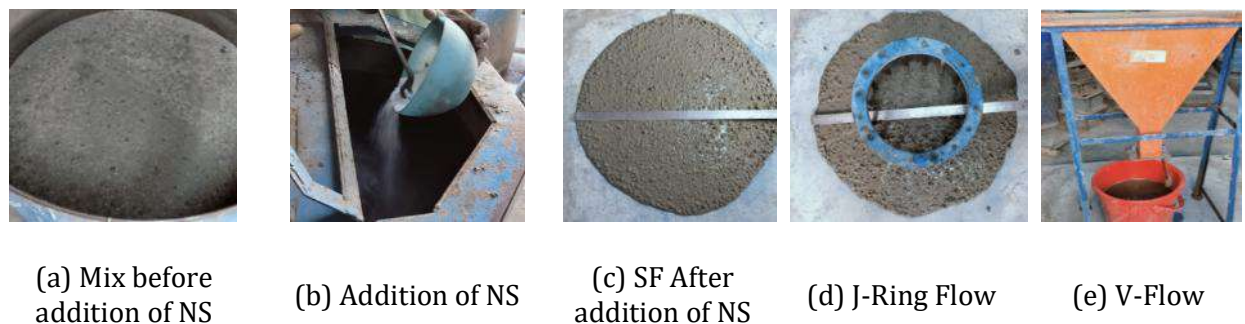


Fig. 8. Fresh concrete properties testing on M70 SCC (G) Mod

Both M50 SCC (G) Mod and M70 SCC (G) Mod became uniform and cohesive after the addition of NS as shown in Fig. 7 and Fig. 8. The dosage of NS required for M50 is measured at 1.4 kg/m³, and for M70 at 1.0 kg/m³. M50 SCC (G) Mod requires a higher NS dosage owing to its relatively lower binder content. It is evident that an appropriate dosage of NS made the M50 SCC (G) Mod mix more robust, despite its relatively low binder content. Compressive strength increment in the three grades of SCC mixes from 7 days to 28 days is noted to be between 70% to 80%, and flexural strength increment is between 70 and 75% in the same period. The reduction in RCPT coulomb

values from 28-day to 56-day age is between 49% and 59% for these three mixes (including M60 SCC (G) Mod evaluated in the earlier study), indicating increased resistance to rapid chloride penetration with age. The results of other durability tests, such as water penetration (WP) and water absorption (WA) at 28 days, are satisfactory, with WP less than 10 mm and WA not more than 5% in all three mixes.

Table 8. Fresh properties of modified SCC mixes prepared from granite-type aggregates

Tested properties	*M60 SCC (G)	M50 SCC (G) Mod	M70 SCC (G)
Fresh concrete properties			
Slump flow	650 mm	675 mm	670 mm
T500	4.80 seconds	4.20 seconds	4.90 seconds
V-Flow	11.0 seconds	7.0 seconds	9.0 seconds
J-Ring flow	620 mm	650 mm	670 mm
VSI	0	0	0

*Data of M60 SCC (G) Mod obtained from earlier study

Table 9. Durability properties of modified SCC mixes prepared from granite-type aggregates

Tested properties	*M60 SCC (G) Mod	M50 SCC (G) Mod	M70 SCC (G)Mod
RCPT test results			
RCPT @ 28 days	796 Coulombs	1252 Coulombs	629 Coulombs
RCPT @ 56 days	329 Coulombs	644 Coulombs	301 Coulombs
Gain in strength and reduction in RCPT			
7/28 days compressive	80%	71%	79%
7/28 Flexural strength ratio	72%	71%	75%
RCPT reduction (28 to 56	58.67%	48.56%	52.15%
Water penetration test results			
Water penetration (28 days)	3.3 mm	7 mm	3 mm
Water absorption test results			
Water Absorption (28 days)	4.4%	5%	3.9%

*Data of M60 SCC (G) Mod obtained from earlier study

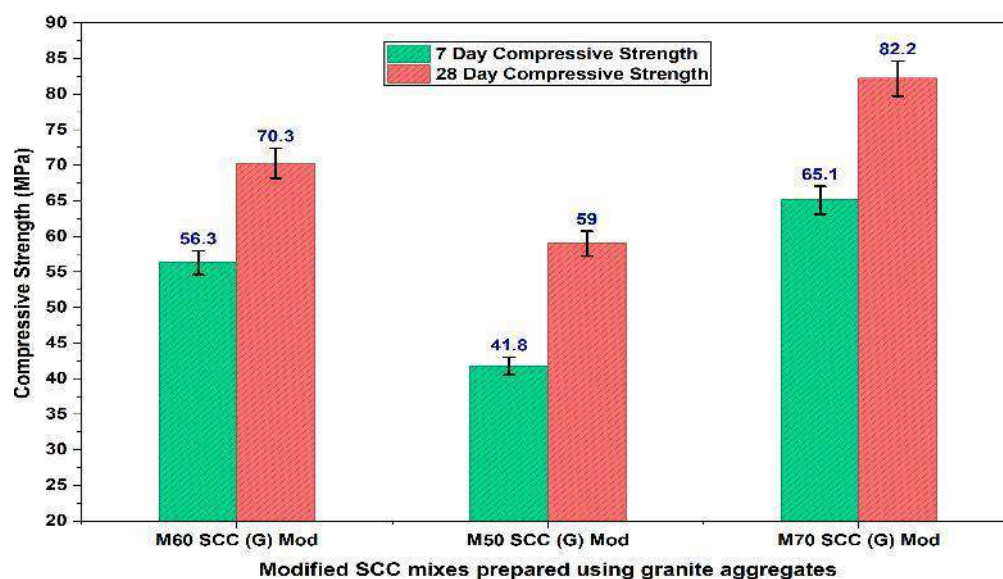


Fig. 9. Cube compressive strength results of modified SCC mixes prepared from granite-type aggregates

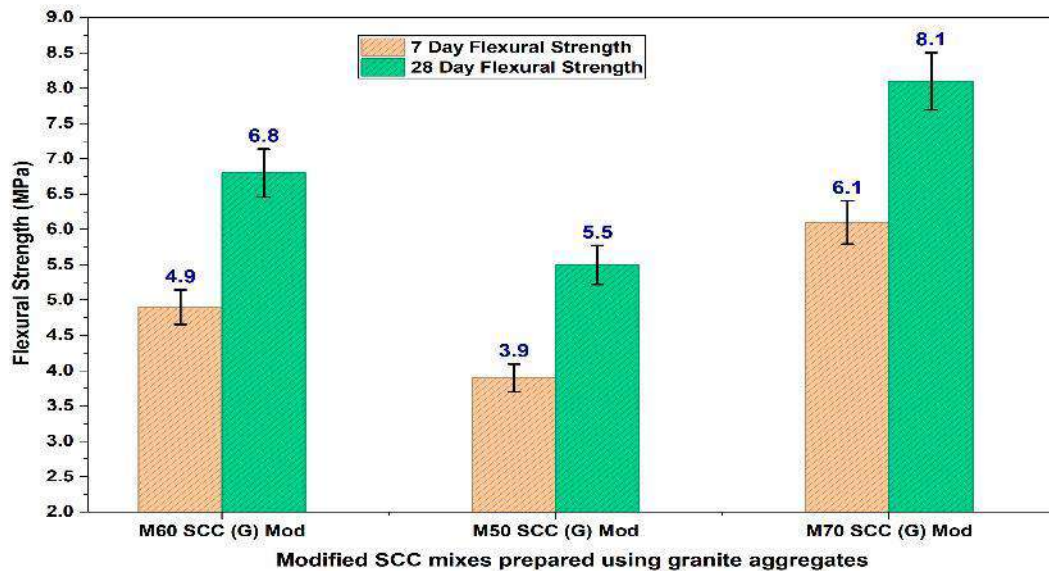


Fig. 10. Beam flexural strength results of modified SCC mixes prepared from granite-type aggregates

4.3 Experimental Investigations on the Modified M50, M60, and M70 SCC Mixes Using Basalt-Type Aggregates

In this phase of the experimental study, the modified approach incorporating UFBFS and NS was adopted to design M50, M60, and M70 grade SCC mixes using basalt-type aggregates. These mixes are identified as M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod, in which 'B' denotes basalt-type aggregates. Higher relative density values of basalt-type aggregates increased the design density of these SCC mixes compared with SCC mixes designed with granite-type aggregates. With increased aggregate content, which increases the aggregate surface area, these SCC mixes are more likely to segregate when the binder content is optimized.

Table 10. M50, M60, and M70 modified SCC mix designs with basalt-type aggregates

Type of materials	M50 SCC(B) Mod	M60 SCC(B) Mod	M70 SCC(B) Mod
OPC	330	370	410
Fly ash	170	170	170
UFBFS	20	25	30
12.5 mm	896	883	876
CS Sand	431	424	421
Water	170	165	155
Total	2432	2446	2468
CONXL PCE 2635	4.70	5.20	5.60
Nano silica	1.60	1.40	1.10
UFBFS / OPC	6.10%	7.30%	7.30%
W/B ratio	0.33	0.29	0.25
Total binder content by weight, kg	520	565	610
Paste content by volume, liters	362.70	372.10	376.60
NS/OPC	0.48%	0.38%	0.27%

The experimental study showed that it was possible to produce an M50 grade SCC mix with the lowest binder content among the three validated grades (M50, M60, and M70), and that the addition of NS made it uniform and cohesive. Hence, similar performance could be expected in SCC mixes with basalt-type aggregates. While designing M50 SCC (B) Mod, M60 SCC (B) Mod, and M70 SCC (B) Mod mixes using the modified approach, the binder and free water contents were kept identical to those of modified SCC mixes of similar grades with granite-type aggregates. The volumetric proportion of coarse to fine aggregates was maintained at 50:50 in these mixes as well.

Table 10 presents the mix-proportion details for the M50, M60 and M70 modified SCC mixes prepared with basalt-type aggregates. Figs. 11 to 13 illustrate visual representations of fresh property measurements for the M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod mixes, respectively. Tables 11 and 12 present the fresh and durability properties of modified SCC mixes containing basalt-type aggregates. Figs. 14 and 15 illustrate the progression of compressive strength and flexural strength results of modified SCC mixes (M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod) at the age of 7 and 28 days, prepared using basalt-type aggregates.

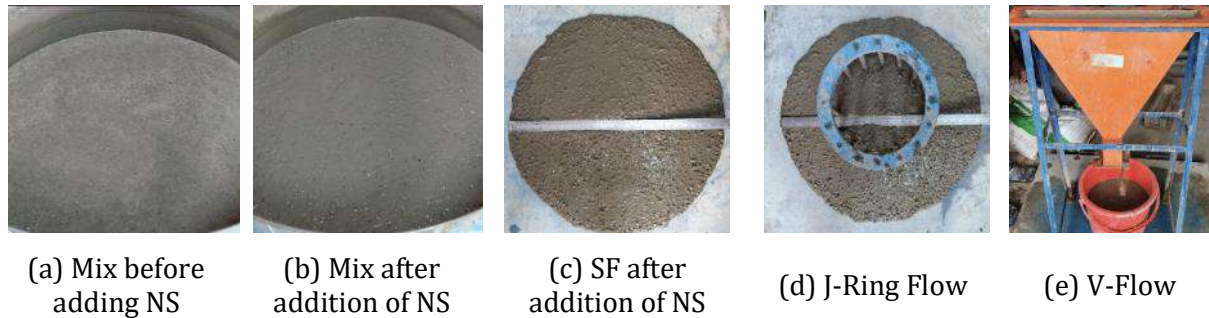


Fig. 11. Fresh concrete properties Tests done on M50 SCC(B) Mod

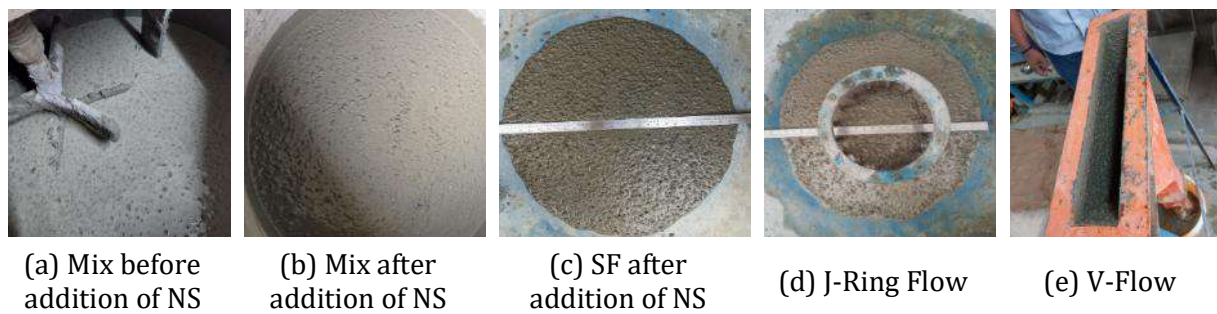


Fig. 12. Fresh concrete properties testing on M60 SCC (B) Mod SCC mix

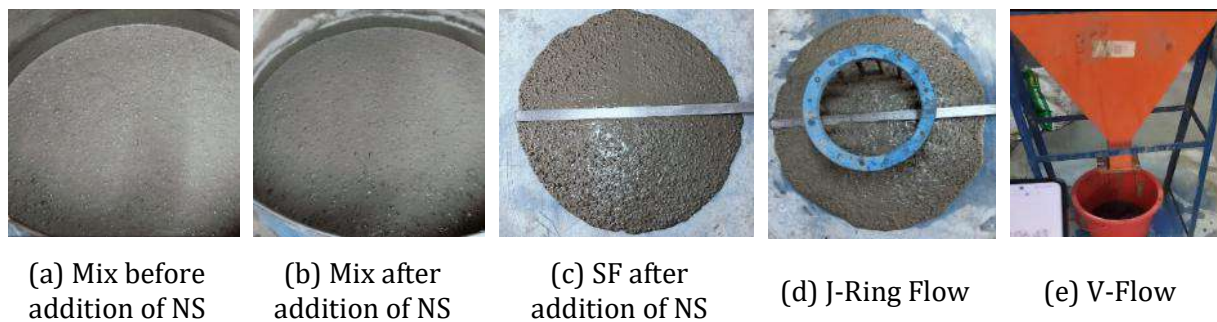


Fig. 13. Fresh concrete properties testing on M70 SCC (B) Mod SCC mix

Table 11. Fresh properties of modified SCC mixes containing basalt-type aggregates

Tested properties	M50 SCC (B) Mod	M60 SCC (B) Mod	M70 SCC (B)
Fresh concrete properties			
Slump flow	670 mm	680 mm	680 mm
T500	3.5 seconds	4.5 seconds	5.0 seconds
V-Flow	6.0 seconds	6.0 seconds	8.0 seconds
J-Ring flow	650 mm	660 mm	640 mm
VSI	0	0	0

Table 12. Durability properties of modified SCC mixes containing basalt-type aggregates

Tested properties	M50 SCC (B) Mod	M60 SCC (B) Mod	M70 SCC (B)
RCPT test results			
RCPT @ 28 days	1546 Coulombs	1438 Coulombs	927 Coulombs
RCPT @ 56 days	743 Coulombs	586 Coulombs	445 Coulombs
Water absorption test			
Water absorption @ 28 days	5.40%	5.0%	4.80%
Water penetration test results			
Water penetration (28 days)	9 mm	8 mm	5 mm
Gain in strength and reduction in RCPT			
7/28 days compressive	76%	75%	75%
7/28 days Flexural strength	72%	80%	73%
RCPT reduction (28 to 56 days)	51.90%	59.20%	52.0%

In the trial mixes conducted to validate M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod mixes, the quantity of NS required to be added to make the mixes uniform is found to be higher in comparison with similar grades of SCC mixes having granite-type aggregates. The increase in density of M50, M60, and M70 modified SCC mixes with basalt-type aggregates is 81 kg/m³, 80 kg/m³, and 79 kg/m³, respectively, compared with similar grades of modified SCC mixes with granite-type aggregates.

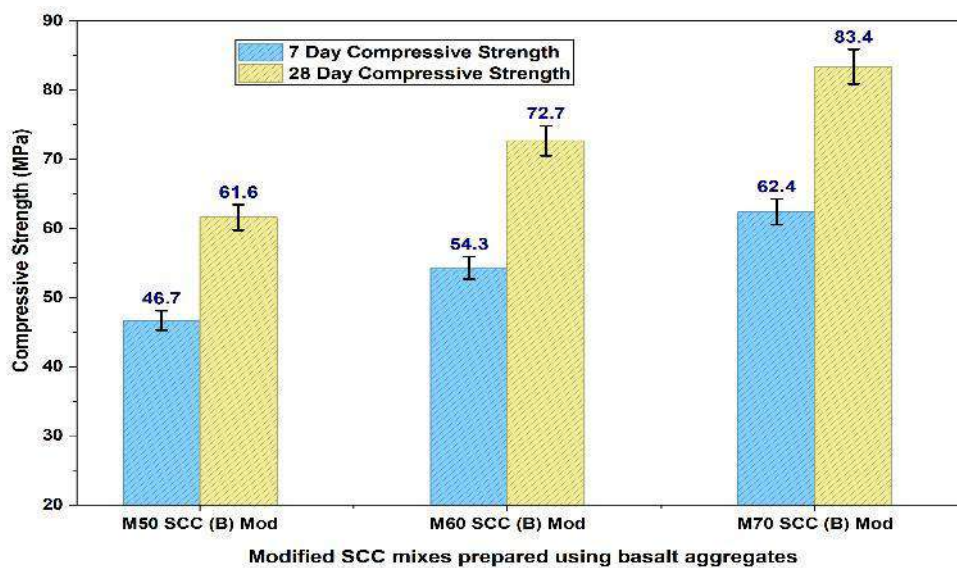


Fig. 14. Cube compressive strength results of modified SCC mixes prepared from basalt-type aggregates

This is due to the higher relative density of basalt-type aggregates compared to granite-type aggregates, i.e., 2.70 & 2.64 vs 2.88, 2.77 & 2.67. According to Okamura and his team, reducing coarse aggregate content and increasing paste content are required to reduce internal stress caused by coarse aggregates in the deformable mix, thereby preventing segregation and blocking issues [9, 11]. Hence, the conventional approach would be to offset an increase in aggregate quantity, partially or fully, by maintaining a higher slurry or paste medium to ensure a uniform, free-from-segregation, and flowable mix. However, in this experiment, a slight increase in the NS dosage is found to be sufficient to make the slurry or paste medium viscous and robust enough to make SCC mixes with basalt-type aggregates cohesive and to meet the flow requirements. It is more evident that the modified approach replaces the conventional approach of relying on higher binder content. Compressive strength increment in the three grades of SCC mixes with basalt-type aggregates from 7 day to 28 day is around 75%, and flexural strength increment is between 70% and 80% over the same period. The reduction in RCPT coulomb values from 28 days to 56 days is

between 50% and 60% for these three mixes, indicating enhanced resistance to chloride penetration with age. The 28 day age tests show that water penetration is less than 10 mm and water absorption is less than 5.5% in all three mixes. The durability properties of the three mixes of M50, M60 and M70 grades are found to be excellent.

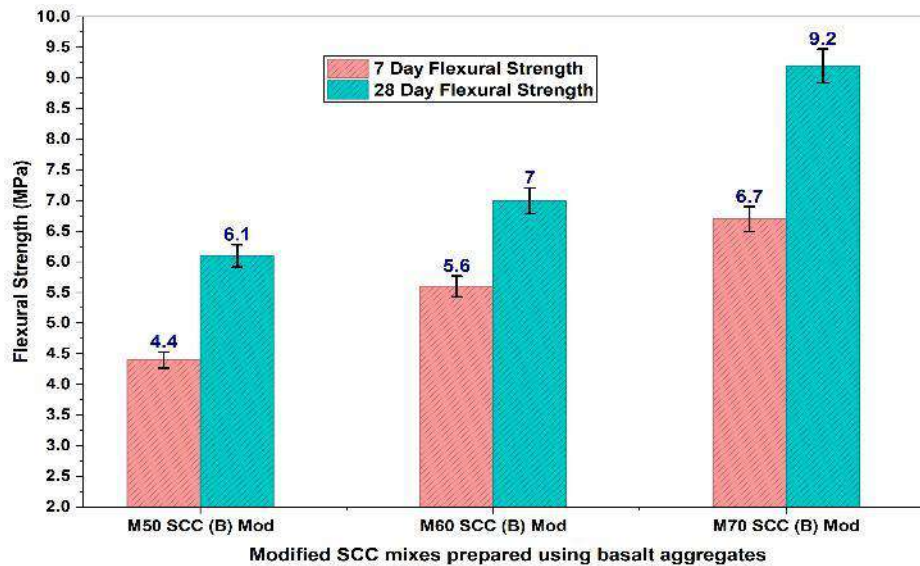


Fig. 15. Beam flexural strength results of modified SCC mixes prepared from basalt-type aggregates

5. Proposed Simplified Mix Design Methodology

Designing an SCC mix usually requires conducting a series of trial mixes and repeatedly fine-tuning the mix ingredients, especially the binder content, to meet the required flow properties. Previous experimental research by the authors on M60-grade SCC mix with granite-type aggregates, comparing conventional and modified approaches, demonstrated the combined role of UFBFS and NS in reducing the binder content while meeting fresh concrete properties satisfactorily [41]. The modified M60 SCC mix exhibited early-age cracking resistance and reduced drying shrinkage behavior, thereby addressing the cracking issue associated with a typical conventional SCC mix [41]. In the present study, a similar approach has been adopted to design and validate M50 and M70 SCC mixes with granite-type aggregates. These two modified mixes, apart from meeting the flow requirement, also meet the strength and durability requirements. In the next stage, M50, M60 and M70 mixes with basalt-type aggregates are designed using a modified approach incorporating UFBFS and NS. When designing SCC mixes with both granite-type and basalt-type aggregates, the same modified mix design approach is adopted. The only difference in SCC mixes with basalt-type aggregates is an increase in aggregate content, attributable to their higher relative density.

It is found that a marginal increase in NS dosage enhanced the viscosity of the slurry medium, thereby keeping the relatively heavier coarse aggregate particles as an integral part of the mix and ensuring uniformity and flowability of the SCC mixes. Many studies on the addition of NS to concrete mixes, including SCC, have reported its role in enhancing mechanical and durability properties [45–47]. While reporting the influence of NS on the rheology of the mixes, the research works stated that the addition of NS caused workability and flow losses, and an increase in thixotropy and yield stress. Such an effect on the rheology of concrete mixes holds true when the NS dosage exceeds 0.50%, as evidenced by the authors' Marsh-Cone test in their previous study [41]. However, in the previous study and in the current study conducted by the authors, the NS dosages used to enhance the uniformity and cohesiveness of M50, M60 and M70 grade SCC mixes were less than 0.5%. At such a low dosage, NS was found to make the mixes sufficiently viscous to prevent segregation while maintaining their fresh-concrete properties and meeting the required SCC flow criteria. This novel approach also helped in “simplifying the SCC mix design methods,” as illustrated in the flowchart in Fig. 16.

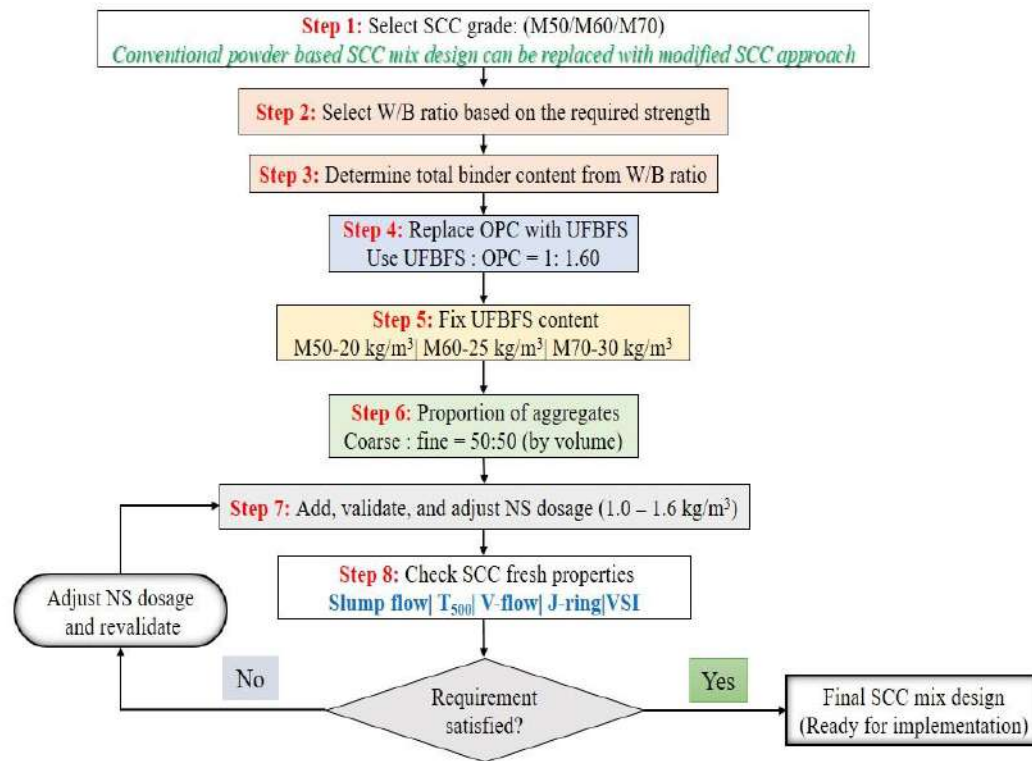


Fig. 16. Step-by-step procedure involved in developing a simplified SCC mix design approach

Validation of the performance of all six mix designs in both fresh and hardened states showed that the modified approach simplified the SCC mix design. The resultant simplification is presented as follows;

- Conventional powder-based SCC mix designs can be replaced with modified SCC mix designs.
- Binder contents are determined solely by the water-to-binder ratio needed to achieve the specified strength.
- The preliminary study conducted by the authors on M60 mixes showed that 25 kg/m³ UFBFS could replace 40kg/m³ OPC while achieving a 4.5% higher compressive strength. The pozzolanic efficiency of UFBFS relative to OPC was found to be 1:1.6 [41]. Incorporating UFBFS in the current experimental study helps to maintain optimal OPC content.
- UFBFS are added at dosages of 20 kg, 25 kg, and 30 kg per cubic meter in M50, M60, and M70 SCC mixes, respectively, to ensure effective incorporation during manufacturing.
- Coarse and fine aggregates are proportioned at a 50:50 volume ratio.
- NS is the last ingredient added to the fresh mix to convert the slurry medium robust enough to carry aggregates, ensuring a uniform mix with SCC flow properties.
- The NS dosage observed during validation of SCC mixes ranges from 1 kg/m³ to 1.6 kg/m³. Adjusting NS to meet the requirements of flowable SCC mixes is a simpler, more effective step and the final step in SCC mix design formulation and validation.
- As far as NS addition in the batching plant is concerned, it can be added either in the central mixer or in the transit mixer as the final ingredient, as is usually done for fibers or additives such as waterproofing compounds. With the enhanced batching control mechanism in the modern-day batching plants, an auto-dispensing arrangement can also be implemented, as is done for UFBFS.

6. Results and Discussions

6.1 Effect of NS Incorporation on Setting Time and Early Strength of Cement Paste

An earlier experimental study conducted by the authors demonstrated that a modified mix design approach for M60 SCC mix, incorporating UFBFS and NS, was effective in mitigating early-age shrinkage and cracking behavior. In that study, the role of NS in contributing to early strength gain required to resist cracking was inferred from the research findings [39, 40]. A reduction in total binder content and the role of NS in providing stiffness at an early age could have helped to mitigate early-age shrinkage. In the present experimental work, a preliminary study was conducted on OPC paste containing 0% and 0.25% NS. The paste with 0.25% NS achieved an earlier setting time by 65 minutes and a higher 12-hour strength by 35.60%. As concluded by other researchers, while a reduction in total binder content is essential to mitigate the probability of shrinkage in SCC mixes, the benefit of NS in promoting hydration and early strength gain is an additional factor that offers potential resistance to early-age plastic/autogenous cracking by accelerating structural buildup [43, 44].

6.2 Fresh, Mechanical and Durability Assessment of Modified SCC Mixes with Granite-Type Aggregates

Building on the earlier experimental study comparing the conventional and modified SCC mix design approaches, the current study has expanded its scope to include M50 and M70 SCC mix grades. Similar to M60 SCC (G) Mod, M50 and M70 SCC mixes are also designed with granite-type aggregates. The total binder content for M50 SCC (G) Mod is 520 kg/m³, and for M70 SCC (G) Mod it is 610 kg/m³, considered based on the strength requirements. In the trial mixes conducted to validate both these mixes, segregation is noticed before the addition of NS. The addition of NS at 1.4 kg/m³ (0.42 NS/OPC) to M50 SCC (G) Mod and at 1.0 kg/m³ (0.24% NS/OPC) to M70 SCC (G) Mod makes both mixes uniform and cohesive. Flow tests conducted are found to meet the SCC criteria. The 28-day compressive strengths of M50 SCC (G) Mod and M70 SCC (G) Mod mixes are measured as 59.0 MPa and 82.2 MPa, respectively, indicating their potential to meet the characteristic strength requirement. Water penetration depth of M50 SCC (G) Mod and M70 SCC (G) Mod is limited to 7.0 mm and 3.0 mm, respectively, at the 28-day age test, and the rapid chloride penetration test results at 56 days of age are 644 coulombs and 301 coulombs, respectively. Water absorption values obtained at 28 days of age for M50 SCC (G) Mod and M70 SCC (G) Mod are 5.0% and 3.9%, respectively.

6.3 Fresh, Mechanical and Durability Assessment of Modified SCC Mixes with Basalt-Type Aggregates

The modified approach to designing SCC mixes is extended to mixes with basalt-type aggregates, which have a higher relative density than granite-type aggregates. M50, M60, and M70 SCC mixes are designed for experimental validation. The total binder contents are kept the same as that of SCC mixes with granite-type aggregates, which means that M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod mixes with basalt-type aggregates have total aggregate content increased by 81 kg/m³, 80 kg/m³, and 79 kg/m³, respectively. The trial mixes showed that adding NS as the final ingredient at dosages of 1.6 kg/m³ (0.48% NS/OPC), 1.4 kg/m³ (0.38% NS/OPC), and 1.1 kg/m³ (0.27% NS/OPC), respectively, allowed all three SCC mixes to achieve uniformity.

Flow tests conducted on these three mixes indicate that they meet the requirements of fresh concrete properties of SCC. The 28-day compressive strength of M50 SCC (B) Mod, M60 SCC (B) Mod and M70 SCC (B) Mod SCC mixes are found to be 61.6 MPa, 72.7 MPa and 83.40 MPa, respectively, exceeding target mean strength requirements. Water penetration resistance values for these three mixes at 28 days of age are 9.0 mm, 8.0 mm, and 5.0 mm, respectively, and rapid chloride penetration test results at 56 days of age are 743, 586, and 445 coulombs, respectively. Water absorption test results at 28 days of age are 5.4%, 5.0%, and 4.8%, respectively.

Various studies have reported the role of fly ash, UFBFS and NS in pore refinement and in improving impermeability through the pozzolanic reaction, which converts calcium hydroxide (CH) into

secondary C-S-H. Fly ash and NS also eliminate bleeding and make the interfacial transition zone (ITZ) strongly bound to the aggregate phase, enhancing the strength potential of concrete. While the secondary reaction of NS is observed at an early age due to its much higher fineness and reactivity, the reaction of fly ash is said to continue for a prolonged period beyond 28 days. Thus, all three of these ingredients contribute to the enhanced durability of concrete. This is evident in the increased RCPT values from 28 to 56 days of age.

In M50, M60 & M70 modified SCC mixes with granite-type aggregates, the NS/OPC dosages used are 0.42%, 0.32% and 0.24%, respectively. In mixes with basalt-type aggregates, the dosages in terms of NS/OPC for M50, M60 & M70 are 0.48%, 0.38% and 0.27%, respectively. Although an increase in the relative density of aggregates commonly requires a higher binder content to reduce friction between relatively high quantities of aggregates, a marginal increase in NS can nullify that dependence on a higher binder content. As reported by other researchers, NS, due to its large surface area and adsorption characteristic enhances plastic viscosity and yield stress (moderately) even at lower dosage [36-38]. The observation in this study also indicates that NS plays a crucial role in making the paste/slurry medium viscous and in contributing to an SCC mix that is uniform, cohesive, and free of segregation, with an optimal binder content.

It is also observed that the demand for NS, as measured by the NS-to-OPC ratio, decreases as the concrete grade increases. Increased binder content (610 kg/m³), and paste content (376.6 litres) in M70 SCC, increase the matrix's plastic viscosity and intrinsically lower the additional NS requirement for segregation resistance. With the water-to-binder ratio maintained at 0.33, 0.29, and 0.25 for the M50, M60, and M70 mixes, respectively, both strength and durability test results are found to be satisfactory. Reductions in OPC and total binder can be achieved by modifying the SCC mix design with the incorporation of UFBFS and NS. The modified approach to SCC mix designs undertaken in this experimental research has also simplified the process. With binder content kept optimal in the modified approach to meet the water-to-binder ratio requirements, as in normal concrete mix design, the challenge of making SCC free from segregation and achieving its flow properties is addressed by adding NS. Incorporation of NS as the final ingredient is an important factor in simplifying SCC mix designs.

7. Conclusions

- The main objective of the experimental study was to design SCC mixes using a modified approach, as an alternative to powder-based SCC mix design. The modified method adopted in this research work incorporated UFBFS and NS to design M50, M60, and M70 SCC mixes with optimal binder content to meet the water-to-binder ratio requirements, rather than increasing binder content to enhance the uniformity of the SCC mixes when segregation is encountered. Two sets of experiments were considered: one with granite-type aggregates, which possessed lower relative density values, and another with basalt-type aggregates, which possessed higher relative density values. Binder content and its individual components, namely OPC, UFBFS and fly ash, were kept constant for both sets of SCC mixes.
- The experimental work conducted on M50, M60 and M70 grade modified SCC mixes with both granite-type and basalt-type aggregates included testing of fresh concrete properties, namely slump flow, T_{500} , V-flow, J-ring flow, and VSI, to validate the performance requirements.
- The 28-day cube compressive strength results for granite-type aggregate-based mixes were measured at 59.0 MPa for M50 SCC, 70.3 MPa for M60 SCC and 82.2 MPa for M70 SCC. For M50, M60 and M70 SCC mixes with basalt-type aggregates, the cube compressive strength results were 61.6 MPa, 72.7 MPa and 83.4 MPa, respectively. These results indicate that the designed mixes with optimal binder content met the intended strength requirements.
- In durability tests conducted, RCPT values of all six mixes at 56 days were less than 1000 coulombs, thus falling in the category of "very low chloride penetrability" as per ASTM C1202. Water penetration at 28 days was limited to less than 10mm in all six mixes, with the M50 basalt-type-based SCC mix exhibiting a maximum value of 9mm. Water absorption values at 28 days of age were found to fall within the ranges of 3.9% to 5.0% for granite-type aggregate-based SCC mixes and 4.8% to 5.4% for basalt-type aggregate-based SCC mixes.

- Fresh concrete densities of SCC mixes with basalt-type aggregates were comparatively higher due to increased aggregate content. In experimental work aimed at achieving a robust, uniform and cohesive mix to obtain a VSI index of “0”, it was found that mixes with basalt-type aggregates required slightly more NS than similar mixes with granite-type aggregates. In M50, M60 & M70 modified SCC mixes with granite-type aggregates, the NS dosages were 1.4 kg/m³, 1.2 kg/m³ and 1.0 kg/m³, corresponding to 0.42%, 0.32% and 0.24% NS/OPC, respectively. Similarly, for mixes with basalt-type aggregates, the corresponding NS dosages were 1.6 kg/m³, 1.4 kg/m³ and 1.1 kg/m³, corresponding to 0.48%, 0.38% and 0.27% NS/OPC, respectively.
- All the trial mixes exhibited segregation before addition of NS and became cohesive achieving VSI index of “0” after the addition of NS. In the modified approach, NS must be added as the final ingredient to achieve a homogeneous, robust SCC mix and, more importantly, to determine an appropriate dosage. This experimental study recommends a novel approach of varying NS dosage for SCC mixes with varying grades, aggregate content, or aggregate density, rather than fixing the NS dosage and assessing its impact on SCC properties. The objective of this experimental research was to provide a modified approach, as an alternative to the conventional binder/powder-based SCC mix design, with an optimal binder content to minimize the potential risk of shrinkage and to fulfil the requirements for fresh and hardened SCC parameters. This objective was achieved.
- The modified approach, validated in the experimental studies, was also shown to be a simplified design method for SCC mixes containing either granite or basalt-type aggregates. Thus, this approach could be found useful in a broader area across India.

7.1 Future Scope of the Study

SEM/EDS and TGA microstructural studies will be carried out to gain an understanding of the development of the cementitious matrix. Drying shrinkage on different grades of modified SCC mixes will be conducted. Future studies on SCC mixes having coarse aggregates higher than 50% in solid volume and in M40 grade will be taken up for experimental investigation using the modified SCC mix design approach. To evaluate the feasibility of applying the proposed approach in practice, casting into site-scale structural elements will also be carried out.

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Seismic performance optimization of reinforced concrete shells generated by the force density method: A nonlinear multilayered approach

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Article Info	Abstract
<p>Article History:</p> <p>Received 04 Aug 2026</p> <p>Accepted 01 Oct 2026</p> <p>Keywords:</p> <p>Concrete shells; Geometric optimization; Non-linear analysis; Time-history analysis; Form-finding; Earthquake design; Buckling; Stiffness degradation; Force density method; Nonlinear multi-layered model; Geometric imperfections; Seismic reintroduction</p>	<p>The construction of concrete shell roofs in Mexico has declined owing to seismic vulnerability and slenderness effects. Traditional form-finding guarantees static equilibrium under self-weight alone and remains inadequate under seismic demand, where stiffness degradation and stress reversal govern the response. Three form-found anticlastic shells with four, six and eight supports were generated by the force density method, which yields compression-dominant configurations before reinforcement is defined. Reinforcement was first designed with the Eurocode 2 three-layer sandwich model, which neglects the compressive contribution of the reinforcement; a nonlinear multilayered formulation was then adopted to resolve the stress state through the shell thickness. Design actions followed NTC-2023; the analyses incorporated initial geometric imperfections, nonlinear buckling, the Takeda hysteretic model and synthetic accelerograms for Mexico City. The shells developed controlled, low-level nonlinear behavior, with compression governing the response and the reinforcement carrying a significant fraction of the compressive demand under stress reversal. Eigenmode-affine imperfections reduced the ultimate capacity by 45.2% to 22.1%, the reduction diminishing with the number of supports: more petals stiffen the shell and raise its capacity, at the cost of ductility. Since imperfections consume so large a share of that capacity, recovering design margin becomes decisive: accounting for the compressive contribution of the reinforcement reduces the required area by 10%, making the multilayered representation a design requirement rather than a refinement. The numerical cracking patterns coincide spatially with the damage documented after the 2017 earthquake in Félix Candela's Los Manantiales restaurant, supporting a framework transferable to other earthquake-prone regions.</p>

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1. Introduction

Over the past eighty years, concrete shell structures have played a fundamental role in the development of industrial activities, primarily serving as containment systems for materials. Their main advantage lies in their ability to cover large areas with high span-to-thickness ratio, efficiently distributing stress while requiring very small quantities of concrete and reinforcing steel. Concrete shell roofs depart from traditional rectilinear forms of conventional construction. Today, they are widely employed in contemporary architecture for iconic global projects where designers develop

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progressively more complex and challenging free-form structures with high aesthetic and sculptural value. This evolution is the result of significant advances in computational modeling, geometric optimization and numerical analysis techniques. Recent research has extended form-finding beyond the purely static domain: [1] implemented the natural force density method within an efficient parametric computational framework, and [2] established the conditions for the existence and uniqueness of force density solutions, together with the singularities that constrain the method. In parallel, the seismic behavior of thin concrete shells has received renewed attention: [3] assessed the performance of Félix Candela's shells during the 1985 Mexico City earthquake, [4] quantified damage and stiffness degradation in a doubly curved concrete shell under spatially variable seismic excitation, and [5] characterized the influence of initial geometric imperfections on the buckling capacity of single and double curvature concrete shells.

In Mexico, the construction of concrete shell structures experienced a period of significant expansion between 1960 and 1980, driven by the work of renowned architects such as Fernando López Carmona, Pedro Ramírez Vázquez, and Félix Candela [6,7]. During that period, even national construction codes from the 1980s included specific sections dedicated to the design and verification of shell concrete structures. However, over time and given that Mexico is located in a high-seismicity zone, many of these shells suffered significant irreversible damage during seismic events, leading to costly repairs and, in some cases, demolition due to their inability to comply with modern safety standards. Consequently, although interest in shell structures among architects and engineers remains high, their construction has declined substantially. The main reasons are the inadequacy of traditional methods in predicting nonlinear behavior and stiffness degradation under seismic loads; stability, buckling, and cracking issues inherent to slender shells, as well as time-dependent effects such as creep and the overall complexity of construction processes.

The objective of this article is to apply contemporary geometric optimization to the nonlinear time-history analysis of a set of form-found anticlastic concrete shells subjected to the static and dynamic load conditions established by the Mexican construction codes (NTC 2023) [8], incorporating geometric nonlinearity and initial geometric imperfections. Among the available form-finding strategies, the force density method was selected because it transforms the nonlinear equilibrium problem into a single system of linear equations, avoiding the iterative convergence difficulties inherent to dynamic relaxation and transient stiffness approaches [9], and providing both computational efficiency and numerical stability in the definition of the reference geometry a decisive advantage when that geometry must subsequently serve as the starting configuration for nonlinear seismic analyses. The goal is to propose an analytical methodology for the design of concrete shell structures that contributes to the development of technical guidelines and design criteria. Although the methodology is calibrated here against the seismic hazard and soil conditions of Mexico City, it constitutes a general framework applicable to thin-shell structures in any region of high seismic risk.

The originality of this work does not lie in applying existing optimization techniques to a standard seismic analysis. Its contribution is the identification and resolution of a specific limitation of current design practice: the three-layer sandwich model prescribed by Eurocode 2 (EN 1992-2, Informative Annex LL) [10] assumes that the reinforcement resists tension exclusively and assigns the entire compressive capacity to the concrete core. Under bilateral seismic loading with stress reversal, that assumption does not represent the actual state of the section, and it leads systematically to conservative and uneconomical shells. The present study replaces it with a nonlinear multilayered formulation that resolves the stress state through the shell thickness and explicitly captures the compressive contribution of the reinforcement, and quantifies, through nonlinear time-history analyses, both the magnitude of the discrepancy and the design efficiency that can be recovered.

2. Concrete Shells and Form Finding

A shell is defined as a three-dimensional curved structure whose thickness is significantly smaller than its other two dimensions. Its primary advantage lies in its ability to transfer external loads

into internal membrane stresses of tension and compression, while neglecting bending and shear stresses. Depending on their geometry, shells can be classified into [11]:

- Free-form shells: Geometries generated without any simple mathematical constraints, primarily designed according to aesthetic principles. They can be digitally described using high-degree complex polynomials, such as NURBS surfaces.
- Mathematical shells: Structures generated from analytical mathematical functions. These forms were the main types used for concrete shell construction due to their practicality in closed-form solution calculations and their symmetry, which facilitated construction.
- Form-finding shells: Geometries whose primary purpose is to achieve a static equilibrium state of the structure. This category includes scaled physical models as well as advanced computational simulation models.

Among these three categories, the present study focuses on form-found shells. The reason is structural rather than aesthetic. In seismic regions, a form-found geometry is one configured to carry its loads predominantly through membrane action, minimizing the parasitic bending moments that arise when the surface departs from a funicular configuration. A moment-free or nearly moment-free stress state maximizes the exploitation of the compressive capacity of concrete, which is its strongest response, and reduces the tensile demand that governs cracking and reinforcement requirements. This has a direct dynamic consequence: the high geometric stiffness and low mass resulting from such configurations place the fundamental frequencies of the shell above the dominant frequencies of seismic excitation, a mechanism identified for small and mid-sized shells in [12]. Geometry is therefore treated here as the first line of seismic design, defined before any reinforcement is considered.

2.1 Form-Finding Methods

From their origins, concrete shells have been remarkable for their aesthetic value, often resembling natural forms, while requiring rigorous analysis and calculations to perform safely and efficiently under both permanent and accidental loads. The origins of shell-type structures can be traced back to the 18th and 19th centuries with the construction of iconic buildings worldwide, including churches, auditoriums, temples, and theaters.



Fig. 1. The idea behind La Sagrada Familia: Gaudí's hanging chain model. Available from: <https://parametric-architecture.com/la-sagrada-familia-gaudis-chain-model/>

The period of significant expansion of shell structures began primarily in Europe, especially in Italy, Germany, and Spain. In Spain, the experimental development of the “form-finding” concept was advanced by architect Antonio Gaudí, who employed analog scale models consisting of suspended chains and threads under gravity, generating catenaries and inverted arches subjected to tensile forces. These models had then materialized into architectural constructions that relied primarily on compressive forces, enhancing structural stability without sacrificing sculptural and aesthetic

value. Continuing with research on slender shell structures and innovations in materials and construction techniques, it is essential to highlight the work of German architect Frei Otto, who introduced the concepts of “minimal surface” and “minimal work” using soap films and spider webs to create experimental analog models of geometric optimization. These studies promoted structural efficiency and material optimization in the geometric design of shell and tensile structures, laying theoretical and practical foundations for spatial, branched, pneumatic, and other advanced structural systems.

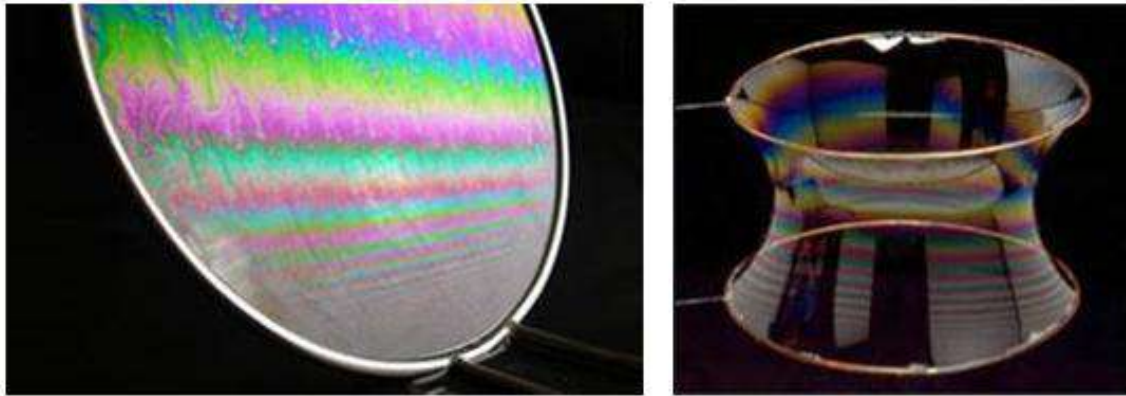


Fig. 2. Soap films, minimal surfaces and beyond, Hong Kong Laureate Forum. Available from: <https://www.hklaureateforum.org/en/soap-films-minimal-surfaces-and-beyond>

Over time, with the advent of computer-aided design (CAD) and finite element numerical analysis from the 1950s to the present, increasingly complex free-form structures have been modeled, analyzed, and understood using optimization and parametrization techniques based on algorithms and geometric rules. These methods allow arbitrary surfaces to be numerically optimized and described under predefined loading conditions, typically considering self-weight only, while accounting for the following parameters [13]:

- Model topology.
- Boundary conditions and supports.
- External loads applied.
- Relationship between internal stress and geometry.

By satisfying these conditions, the geometric design of shell structures becomes an efficient and controlled iterative parametric process. Principal methods employed in this context include the Transient Stiffness Method, the Force Density Method, and the Dynamic Relaxation Method [9].

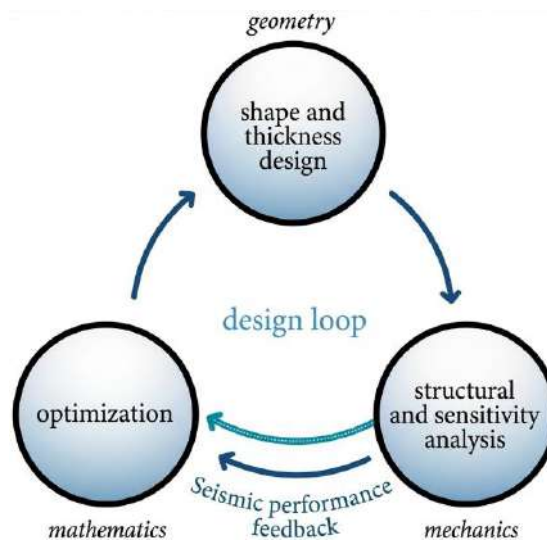


Fig. 3. Steps of form-finding optimization [13]

2.2 Force Density Method

The Force Density Method enables the determination of the equilibrium geometry of a series of points connected through a cable network or a compression structure, where the loads are discretized into nodal point loads. Based on the formulations of K. Linkwitz and H.J. Schek [14,15] in the 1970s, this optimization aims to minimize the potential strain energy of a surface by satisfying static equilibrium through stress redistributions, solving the algebraic equilibrium systems using linear approximations, which makes the calculations simple, efficient, and robust.

The approach applied to concrete shells is based on the creation of a grid model that emulates membrane behavior. This grid achieves its static equilibrium configuration under the influence of external loads acting at discrete joints. Subsequently, these concentrated actions are transformed into distributed loads along the linear elements. It is imperative that the following conditions are satisfied for the grid system to achieve equilibrium in three-dimensional space:

$$s_a \cos(a, x) + s_b \cos(b, x) + s_c \cos(c, x) + s_d \cos(d, x) = p_x \quad (1)$$

$$\text{direction cos} = \frac{\text{coordinate difference}}{\text{distance in space}} \quad (2)$$

$$\frac{s_a}{a}(x_m - x_i) + \frac{s_b}{b}(x_j - x_i) + \frac{s_c}{c}(x_k - x_i) + \frac{s_d}{d}(x_l - x_i) = p_x \quad (3)$$

By incorporating the force density concept also referred to as tension coefficients into the equations to address the nonlinear characteristics of the equilibrium problem, it can be expressed three-dimensionally as:

$$\text{force density} = \frac{\text{force in a bar}}{\text{stressed length of the bar}} \quad (4)$$

$$q_a(x_m - x_i) + q_b(x_j - x_i) + q_c(x_k - x_i) + q_d(x_l - x_i) = p_x \quad (5)$$

$$q_a(y_m - y_i) + q_b(y_j - y_i) + q_c(y_k - y_i) + q_d(y_l - y_i) = p_y \quad (6)$$

$$q_a(z_m - z_i) + q_b(z_j - z_i) + q_c(z_k - z_i) + q_d(z_l - z_i) = p_z \quad (7)$$

The analog models of Gaudí and Frei Otto and the force density method are not merely historically related: they address the same equilibrium problem in two different representations. A soap film carries a uniform, isotropic surface tension and therefore adopts the configuration of minimum area compatible with its boundary, a minimal surface of zero mean curvature. In the force density method, the ratio $q = s/l$ between the force in a branch and its length plays the role of that uniform tension, and the linear system of Eqs. (5) to (7) is the stationarity condition of the functional $\sum q l^2/2$ under the prescribed nodal loads and boundary conditions. Assigning a constant force density to every branch therefore minimizes the sum of squared branch lengths: the network converges to the discrete surface of least Dirichlet energy, which coincides with the minimal surface when the branch lengths are uniform and closely approximates it otherwise. The exact correspondence with the soap-film geometry is recovered by updating the force densities in inverse proportion to the branch lengths, as formalized in the natural force density method [1].

In this study a uniform force density is assigned within each model. The departure from the minimal surface is therefore introduced by the applied load and by the prescribed crown height, not by the force density field. This departure modifies the rise and the curvature of the shell and, consequently, its dynamic properties, which is the mechanism exploited here for seismic purposes (Section 3).

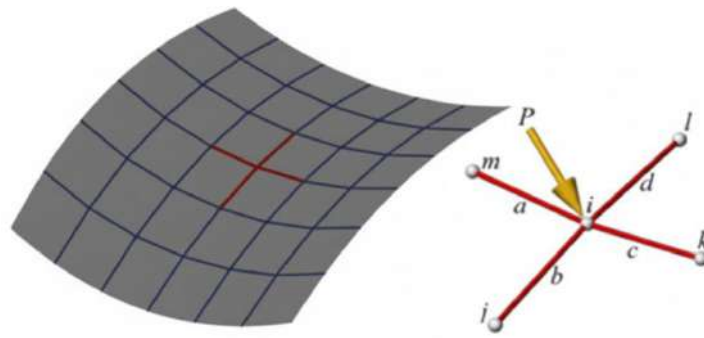


Fig. 4. Generic mesh and discretization of elements and loads at a generic node using Rhinoceros 3D- Grasshopper software

2.2.1 Matrix Formulations

Following the determination of the general equilibrium solution for an individual three-dimensional node, the mathematical framework must be extended to determine the static equilibrium of surfaces characterized by arbitrary geometries and an indeterminate number of free and fixed nodes. This problem is addressed through the synergistic application of matrix algebra and graph-theoretic principles.

The methodology relies on the application of branch-node incidence matrices of size $(m \times n)$, with m branches and n nodes) to define the system's topology and the concurrent evaluation of partial derivatives via the Jacobian matrix. Within this approach, the connectivity between the nodes and branches of the system is formally described by a matrix, denoted as C , which is governed by the following principles:

$$C_{ij} = \begin{cases} +1 = & \text{if branch } j \text{ ends in node } i \\ -1 = & \text{if branch } j \text{ begins in node } i \\ 0 = & \text{otherwise} \end{cases} \quad (8)$$

The matrix C is strictly a branch-node incidence matrix that captures only the topological relationship of the mesh, being completely independent of the system's geometric or metric properties.

$$\begin{matrix} i & j & k & l & m \\ \begin{bmatrix} 1 & -1 & 0 & 0 & 0 \\ 1 & 0 & -1 & 0 & 0 \\ 1 & 0 & 0 & -1 & 0 \\ 1 & 0 & 0 & 0 & -1 \end{bmatrix} & \begin{matrix} a \\ b \\ c \\ d \end{matrix} \end{matrix} \quad (9)$$

The branch-node incidence matrix, C , is subsequently partitioned into two constituent submatrices: one corresponding to the unconstrained (free) nodes and the other to the constrained (fixed) nodes (representing known boundary conditions). Concurrently, the vector of nodal coordinates is similarly subdivided according to the same partitioning scheme.

$$C = [C_N \quad C_F] \quad (10)$$

$$x = [x_N \quad x_F] \quad (11)$$

By resolving the problem along the x , y , and z directions, the following expressions are obtained:

$$u = C_N x_N + C_F x_F \quad (12)$$

$$v = C_N y_N + C_F y_F \quad (13)$$

$$w = C_N z_N + C_F z_F \quad (14)$$

By defining U , V , and W as the diagonal matrices corresponding to u , v , and w , respectively, along with the lengths of the bars in the mesh, the following formulation can be established:

$$L = (U^2 + V^2 + W^2)^{\frac{1}{2}} \quad (15)$$

By formulating the equilibrium equations of the Force Density Method in matrix form within Euclidean space and incorporating the transpose of the Jacobian operator, the following results are obtained:

$$\left(\frac{\partial f(x_i)}{\partial x_i} \right)^T = \begin{bmatrix} \frac{\partial l_a}{\partial x_i} & \frac{\partial l_b}{\partial x_i} & \frac{\partial l_c}{\partial x_i} & \frac{\partial l_d}{\partial x_i} \end{bmatrix} = \begin{bmatrix} \frac{-(x_j - x_i)}{l_a} \\ \frac{-(x_k - x_i)}{l_b} \\ \frac{-(x_l - x_i)}{l_c} \\ \frac{-(x_m - x_i)}{l_d} \end{bmatrix}^T \quad (16)$$

$$\left(\frac{\partial f(x)}{\partial x} \right)^T = C_N^T U L^{-1} \quad (17)$$

Through the incorporation of the force vectors f and the load component vector p , along with the force density in each bar $Uq = QCx$, the objective is to derive the system of linear equations in the following form:

$$C_N^T Q C_N x_N + C_N^T Q C_F x_F + p = 0 \quad (18)$$

$$D_N = C_N^T Q C_N \quad (19)$$

$$D_F = C_N^T Q C_F \quad (20)$$

In this formulation, the problem is represented as a system of linear equations in the form $Ax=b$ where the three-dimensional static equilibrium can be expressed for as:

$$x_N = D_N^{-1}(p_X - D_F x_F) \quad (21)$$

$$y_N = D_N^{-1}(p_Y - D_F y_F) \quad (22)$$

$$z_N = D_N^{-1}(p_Z - D_F z_F) \quad (23)$$

The coordinate vectors obtained from Eqs. (21) to (23) define the reference configuration of the shell. This configuration is not merely a geometric output: it is the geometry on which the finite element discretization is built, and therefore the geometry that determines both the linear stiffness matrix K_L and the geometric stiffness matrix K_G of Eq. (57), which governs the nonlinear response. The partition of the incidence matrix introduced in Eq. (10) is what carries the boundary conditions into that configuration: the constrained nodes x_F are the supports of the shell, and their number and position condition the modal properties, the buckling capacity of the resulting structure, as shown in Sections 5.1.2, 5.2 and 5.5. The force density method is therefore not only a form-finding tool but the first stage of the seismic analysis itself.

3. Concrete Shell Roof Models

The design of the concrete shell structures was conducted using visual programming within Rhinoceros 3D, employing the Grasshopper plugin and implementing the Force Density Method [1] on a quadrilateral mesh anchored at the base of the structures. The resulting surfaces are anticlastic, i.e., of negative Gaussian curvature, and their petals resemble hyperbolic paraboloids; however, they are form-found rather than analytical hypars, and the term anticlastic is used throughout this paper to denote them.

The form-finding network is a radial quadrilateral net whose supports and central crown are fixed nodes, with the crown held at a prescribed height. A unit load is applied at every free node, and a single force density q is assigned to every branch, both quantities being dimensionless. This uniform assignment is deliberate. It leaves the plan layout of the network determined solely by topology and boundary conditions, so that q scales only the load induced elevation. It yields branch forces proportional to branch lengths, i.e. the most uniform funicular membrane state, and a symmetric positive-definite system with a unique, singularity free solution [2]. Finally, it defines each geometry by a single scalar, which keeps the comparison between models reproducible.

With supports and crown fixed, q is the only shape parameter, and its admissible range is bounded on both sides. Low values make the load-induced component dominant: the free edges rise excessively, and the crown becomes a local depression. High values flatten the petals, increasing their radius of curvature and reducing buckling capacity, while the horizontal thrust at the supports grows in direct proportion to q . The adopted values (0.50, 0.60 and 0.70) were selected within this range, and their adequacy was verified a posteriori through the modal and nonlinear buckling analyses of Section 5. Since q depends on the size of each network, the values are model-specific and are not compared across models.

The fixed crown is a boundary condition of the form-finding only; in the finite element models it is a free node. Any residual force carried by the crown in the network is therefore not available in the built shell and must be resisted locally by bending. This is consistent with the concentration of the cracking index around the crown (Section 5.4.1). It is also the reason why the structural response is assessed with a full nonlinear shell model rather than inferred from the form-finding equilibrium.

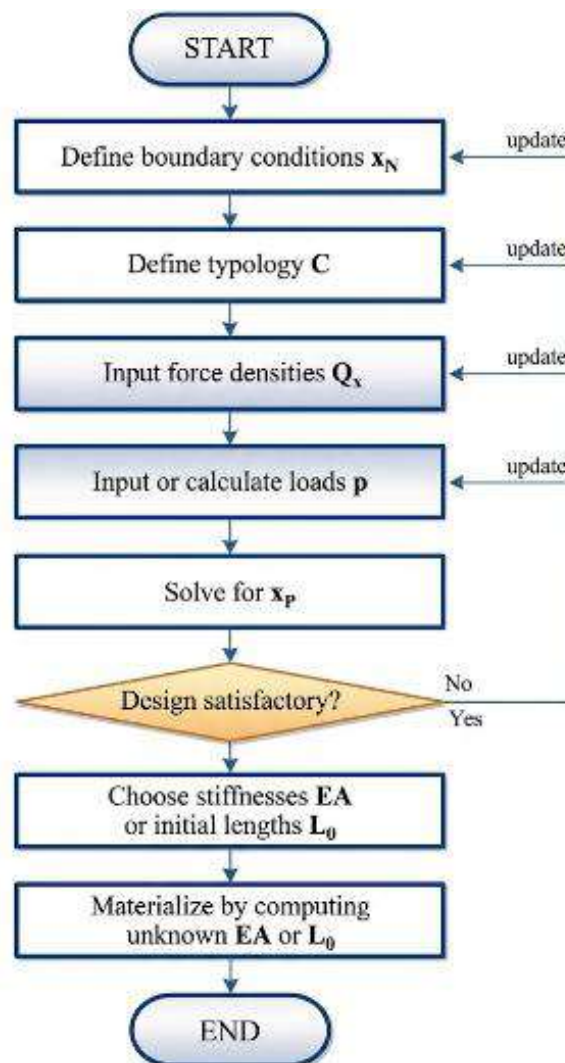


Fig. 5. Force density method flowchart [11]

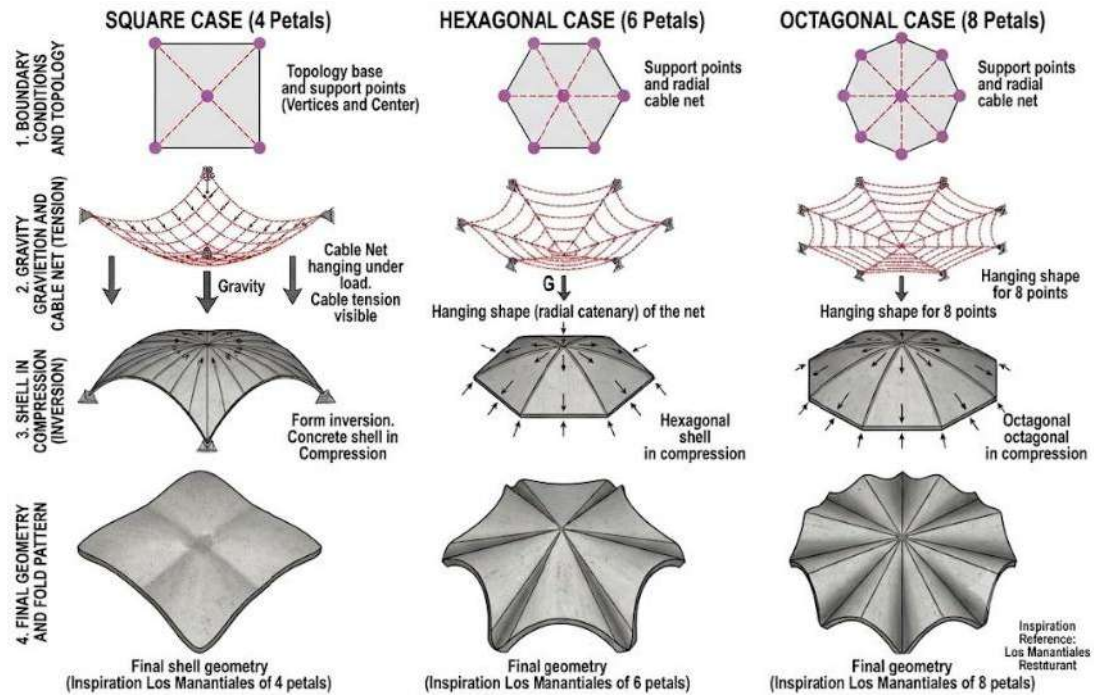


Fig. 6. Analysis model generation process using the force density method

3.1 Geometrical Characterization of The Models

The geometric and discretization parameters of the three models are summarized in Table 1. All shells share a thickness of 18 cm and differ in the number of perimeter supports and in the reference force density assigned during form-finding. Measured from the support plane, the rise-to-span ratios are 0.13, 0.15 and 0.15 for PH-04, PH-06 and PH-08, respectively, within the range usually associated with shallow shells. No shallow shell simplification is adopted: all analyses use the full three-dimensional geometry with a geometrically nonlinear formulation. Table 1 reports, for each model, the force density, the size of the form-finding network, the crown height above the supports (3.76, 5.32 and 4.74 m) and the max. height above the supports (5.16, 6.89 and 6.13 m).

Table 1. Geometric, form-finding and finite element properties of the shell models

Property	Unit	PH-04	PH-06	PH-08
Number of supports	–	4	6	8
Plan dimensions	m	39.90 × 39.90	44.97 × 42.22	39.60 × 39.60
Span between adjacent supports	m	25.25	20.03	13.45
Maximum height above supports ^a	m	5.16	6.89	6.13
Rise-to-span ratio ^a	–	0.13	0.15	0.15
Shell thickness	mm	180	180	180
Force density, q ^b	–	0.50	0.60	0.70
Crown height above supports ^c	m	3.76	5.32	4.74
Element type	–	Layered, Quad 4	Layered, Quad 4	Layered, Quad 4
Boundary conditions	–	Fixed	Fixed	Fixed
Target element size	mm	500	450	450
Nodes / elements ^d	–	2,197 / 1,941	4,881 / 4,400	6,169 / 5,872

^a Measured from the support plane; rise-to-span ratio = maximum height / maximum plan dimension; ^b Uniform in every branch of the form-finding network, with a unit load at every free node (dimensionless); ^c Prescribed in the form-finding only; the crown is a free node in the finite element models; ^d Analysis mesh before automatic meshing.

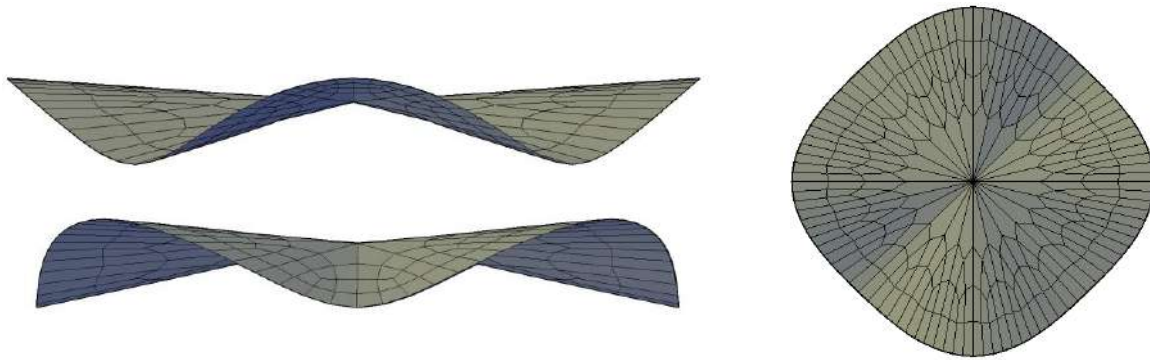


Fig. 7. Model PH-04: four anticlastic petals, uniform force density $q = 0.50$ (dimensionless, unit nodal load), crown fixed 3.76 m above the support plane

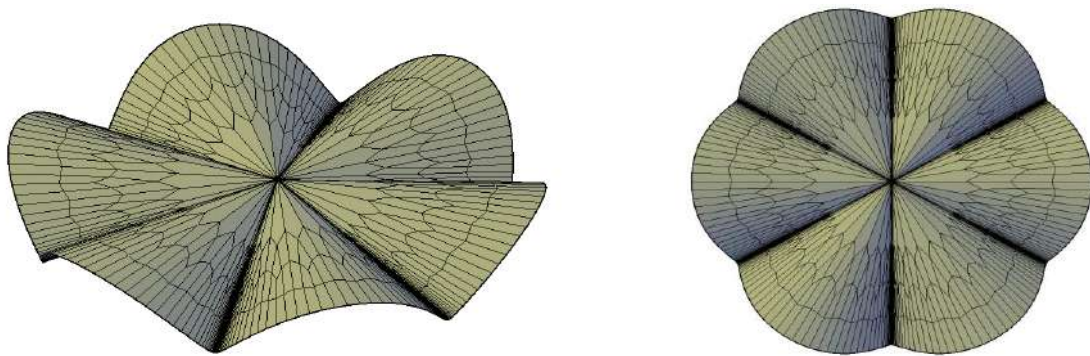


Fig. 8. Model PH-06: six anticlastic petals, uniform force density $q = 0.60$ (dimensionless, unit nodal load), crown fixed 5.32 m above the support plane

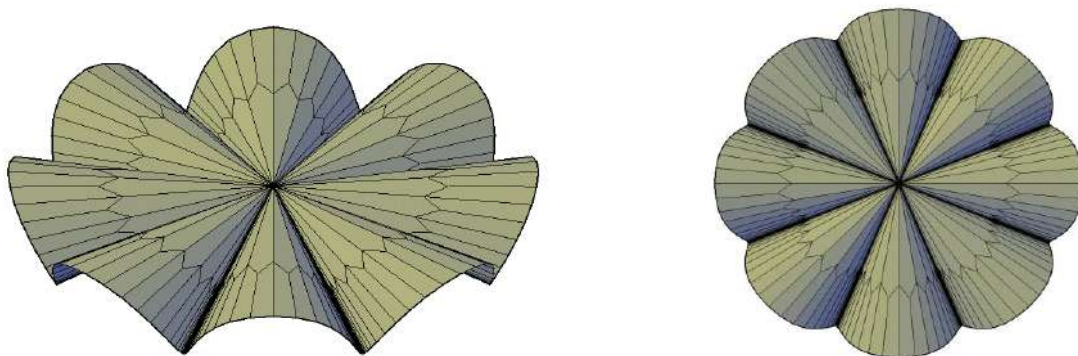


Fig. 9. Model PH-08: eight anticlastic petals, uniform force density $q = 0.70$ (dimensionless, unit nodal load), crown fixed 4.74 m above the support plane

3.2 Nonlinear Models for Concrete and Steel

The importance of accounting for the constitutive behavior of nonlinear materials stems from the need to accurately predict the structural response and the onset of damage when loads surpass the elastic limits of the material. Understanding this behavior allows engineers to capture inelastic deformations, cracking, and other forms of structural deterioration that occur beyond the linear-elastic range, which is critical for safe and efficient design. Moreover, incorporating the nonlinear properties is essential to assess the enhanced structural capacity provided by reinforcing steel, as its contribution becomes significant once the concrete begins to yield or crack.

The nonlinear model proposed by Mander et al. [16] is adopted for concrete. It reproduces strain hardening, post-peak softening and, in particular, the effect of confinement on the stress-strain response. This last feature is decisive in the sections considered here: in a 180 mm shell the

reinforcement layers occupy a substantial proportion of the thickness, so that the degree of confinement provided to the concrete core directly governs the descending branch of the response and the deformation capacity of the section. An elasto-plastic constitutive model [17] is adopted for reinforcement, reproducing the initial elastic response, yielding and plastic deformation; it is compatible with the layered formulation described in Section 5.5.2. It should be noted that the compressive softening reproduced by the material model operates at the constitutive level: the cracking criterion of EN 1992-2 Annex LL adopted in Section 4.2 for the sectional verification does not evaluate concrete crushing, a distinction discussed in Section 6.

Table 2. Properties of the concrete employed in the analyses

Compressive strength (MPa)	Tensile strength (MPa)	Modulus of elasticity (MPa)	Shear modulus (MPa)	Poisson's ratio	Weight per unit volume (kN/m ³)
24.52	3.08	21,707	9,044.79	0.20	23.54



Fig. 10. Constitutive model of the nonlinear behavior of concrete in analyses (1 Mpa=1N/mm²)

Table 3. Properties of the steel in rebars employed in the analyses

Minimum Yield Stress (MPa)	Minimum tensile stress (MPa)	Modulus of elasticity (MPa)	Shear modulus (MPa)	Poisson's ratio	Weight per unit volume (kN/m ³)
413.68	620.53	199,948	76,903.07	0.30	76.88



Fig. 11. Constitutive model of the nonlinear behavior of steel in analyses (1 Mpa=1N/mm²)

3.3 Imperfection Theory in Concrete Shells

As noted above, the efficiency of concrete shells lies in their ability to cover large spans with a high span-to-thickness ratio, a performance that depends critically on their geometry [11]. This same efficiency, however, makes them extremely sensitive to deviations from the idealized analysis model. Construction processes are subject to measurable and controllable imperfections, so that variations in formwork, material quality and element thickness may reduce the load-carrying capacity of the structure below the idealized theoretical values assumed in design [5,18,19]. In thin concrete shells these perturbations trigger geometric instability (buckling) and bending stresses that are not anticipated when the analysis relies solely on membrane theory, which in turn causes premature cracking of the concrete, stress redistribution, geometric and material nonlinear effects, and time-dependent effects such as creep [20]. The seismic damage sustained by Félix Candela's concrete shells in Mexico City during the 1985 earthquake, documented in the assessment of the Church of Our Lady of the Miraculous Medal [3], illustrates the need to account for such deviations from the structural design stage onwards.

3.3.1 Koiter's Formulations

The formal study of geometric imperfections in thin shells originates in the post-buckling theory published by Koiter in 1945 [21], which introduced the concept of imperfection sensitivity. Koiter demonstrated that the sensitivity of the buckling load to initial imperfections is governed by the structure of the expansion of the total potential energy about the critical equilibrium state, written as a Taylor series in the amplitude of the buckling modes, Eq. (24):

$$V = V_0 + \frac{1}{2} V_{ii} \xi_i^2 + \frac{1}{6} V_{ijk} \xi_i \xi_j \xi_k + \frac{1}{24} V_{ijkl} \xi_i \xi_j \xi_k \xi_l + \dots \quad (24)$$

Where V_{ijk} denotes the imperfection amplitude parameter V_{ijkl} and the post-buckling coefficient.

Defining a reduction factor that relates the actual critical loads (P_{crit}) affected by imperfections, to the theoretical critical loads (P_{ideal}) according to Eq. (25):

$$(1 - \rho) \rho^2 = \beta \cdot \delta \quad (25)$$

Where ρ is the load reduction, δ the dimensionless imperfection amplitude and β the imperfection-sensitivity exponent.

Within the framework of geometric imperfection theory, the critical thickness is obtained by adopting Zoelly's formulation for the critical buckling pressure [22], Eq. (26):

$$p_{cr} = \frac{2E}{\sqrt{3(1 - \nu^2)}} \left(\frac{t}{R} \right)^2 \quad (26)$$

Where E is the modulus of elasticity (Young's modulus) of the material, ν the Poisson's ratio of the material, t the shell thickness and R the radius of curvature of the shell.

Equation (27) is accordingly proposed as a preliminary design expression for the minimum thickness required to ensure the stability of concrete shells under design loads in the presence of geometric imperfections:

$$t \geq \sqrt{\frac{P_{design} \cdot R}{f'_c \cdot \lambda}} \quad (27)$$

Where P_{design} denotes the applied load and λ the reduction factor incorporating Koiter's imperfection-sensitivity parameter β .

The buckling capacity of thin shells is known to be extremely sensitive to deviations from the design geometry, but the magnitude of that sensitivity is not a general property of shells: it depends on the geometry considered. The classical imperfection-sensitivity formulations, and the knock-down factors derived from them, were established for cylindrical shells under axial compression and for spherical shells under external pressure, for which experimental capacities as low as 20% of the

classical analytical prediction have been reported [23,24]. In both geometries the critical load is associated with many buckling modes clustered within a narrow range of load factors, and it is this near simultaneity of modes, through the modal interaction it induces, that produces such severe reductions. Doubly curved shells of negative Gaussian curvature behave differently: their critical mode is generally well separated from the subsequent ones, modal interaction is correspondingly weaker, and the reductions in critical load reported for these geometries are substantially smaller [5]. Transferring a knock-down factor calibrated for cylindrical or spherical shells to an anticlastic surface is therefore not justified and would lead to an unnecessarily conservative design. Initial geometric imperfections nevertheless remain one of the variables with the greatest influence on the limit states of thin concrete shells [18,20], and characterizing them for the specific geometry analyzed is what allows the thickness of a concrete shell to be calibrated against the execution tolerances of its construction process rather than against idealized theoretical values. For this reason, the knock-down factor adopted in this study is not taken from the literature but determined for the shells analyzed here, through the imperfection-sensitivity series reported in Section 5.1.3.

4. Reinforced Concrete Shell Design

The primary objective of an optimal design for reinforced concrete shell structures is to determine the required amount of reinforcing steel within their thickness to resist normal stresses, bending moments, and shear forces. Due to the thin-shell configuration of these structures, it is essential to evaluate potential cracking caused by tensile stresses. While the previously employed geometric optimization models are intended to ensure that the overall behavior of the shells is primarily compressive, their limitation lies in considering only self-weight effects. This implies that, although the static equilibrium is satisfied under compression, it does not fully represent the range of loading conditions that these structures will experience throughout their service life, which includes service loads, seismic actions, time-dependent effects, and geometric nonlinearities where tensile stresses may arise.

Consequently, the design of the employed models was conducted following a combination of the updated 2023 Mexican National Building Codes [8,25], from which the applied loads and load combinations for a linear static analysis phase were obtained, yielding the design envelopes for the previously described analysis models. Subsequently, Eurocode 2: Design of Concrete Structures Part 1-1: General Rules and Rules for Buildings (EN 1992-1-1, 2004) and Eurocode 2: Design of Concrete Structures Part 2: Concrete Bridges Design and Detailing Rules (EN 1992-2, 2005) [10,26] were applied to determine the required steel reinforcement ratios for layer reinforcement in the shell elements (three layered design model). The designs obtained from this combined regulatory approach were further evaluated through a nonlinear time-history analysis, using synthetic accelerograms developed for Mexico City. This assessment aimed to validate the structural efficiency of geometric optimization and form-finding models and to demonstrate the significant reduction of tensile stresses in anticlastic shell structures, thereby confirming that their construction in high seismic risk areas can be carried out safely.

The design procedure adopted in this study constitutes a hybrid regulatory approach, and it is declared as such. Seismic hazard characterization, load combinations and displacement verification follow the Mexican Complementary Technical Standards [8], whereas the layer-by-layer reinforcement design follows Eurocode 2 [10,26]. This combination was adopted because the Mexican standards do not provide an explicit procedure for the design of shell elements subjected to the eight stress resultants, whereas Informative Annex LL of EN 1992-2 does. The two frameworks are combined at a level at which they remain consistent: the Mexican standards supply the demand, through site-specific spectra and load combinations, while the European standards supply the sectional resistance model. Partial safety factors are applied within each framework and are not mixed within a single verification. The effect of this choice is confined to the reinforcement design stage: the nonlinear time-history analyses reported in Section 5 are governed by the constitutive models, the geometry and the ground motions, and are therefore independent of the code used to size the reinforcement, which enters only through the reinforcement layout adopted in the layered sections.

4.1 Design Load Combinations Per The 2023 Mexican Building Code (NTC)

The NTC-2023 stipulates that, to verify the safety of a structure, the effects of various actions (loads) must be considered in combination. For seismic design, the fundamental combinations include permanent actions (Dead Load), variable actions (Live Load), and accidental actions (Seismic Load).

Load Combination for Gravitational Effects:

$$P = 1.3 \text{ Dead Load} + 1.5 \text{ Live Load} \quad (28)$$

4.1.1. Response Spectrum for Mexico City

The elastic design spectrum for a specific site in Mexico City, considering the seismic zone and soil type, defines the maximum spectral acceleration and the plateau period, which are critical for modal dynamic analysis. This spectrum represents the seismic forces per unit weight of the structure and is adjusted by the seismic behavior factor in the final design.

Using the SASID application, the response spectra for the analyses were generated, along with synthetic accelerograms for a region in Mexico City, which were subsequently employed in the nonlinear time-history analyses.

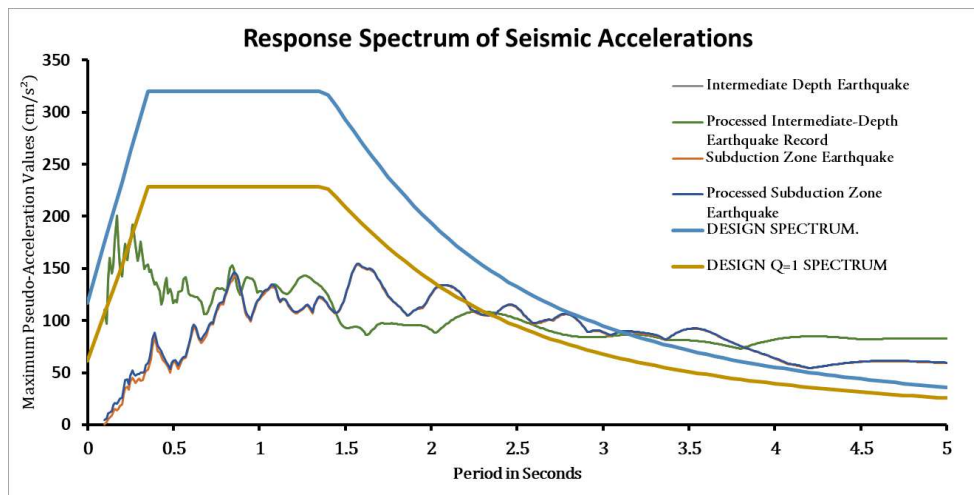


Fig. 12. Elastic design spectrum based on the Mexico City Building Code (NTC-2023), generated with SASID software [27]

A design seismic response spectrum is proposed for Mexico City at a site with coordinates 19.4023243, -99.1889867. The analyzed shell structures are assumed to be located on firm ground to avoid differential settlements and potential geotechnical or foundation-related issues that could affect structural performance.

4.1.2. Design Load Envelope in Accordance with Mexican Regulations

The combination of dead loads and seismic actions is defined as follows:

$$P1 = 1.1 \text{ Dead Load} + 1.1 \text{ Live Load} + (Sx + 0.30 Sy) \quad (29)$$

$$P2 = 1.1 \text{ Dead Load} + 1.1 \text{ Live Load} + (Sx - 0.30 Sy) \quad (30)$$

$$P3 = 1.1 \text{ Dead Load} + 1.1 \text{ Live Load} + (Sy + 0.30 Sx) \quad (31)$$

$$P4 = 1.1 \text{ Dead Load} + 1.1 \text{ Live Load} + (Sy - 0.30 Sx) \quad (32)$$

With Sx and Sy denoting the seismic forces along the x and y axes, respectively.

4.2 Cracking Evaluation of Shell-Type Structures

According to the Eurocode provisions, this type of structure is evaluated for cracking at three different locations along its cross section (upper, middle, and lower zones) as follows:

$$\text{Top} = \begin{bmatrix} S_{11}^t & S_{12}^t & 0 \\ S_{21}^t & S_{22}^t & 0 \\ 0 & 0 & 0 \end{bmatrix} \rightarrow \begin{cases} \sigma_1^t \\ \sigma_2^t \\ \sigma_3^t = 0 \end{cases} \quad (33)$$

$$\text{Center} = \begin{bmatrix} S_{11}^c & S_{12}^c & S_{13}^c \\ S_{21}^c & S_{22}^c & S_{23}^c \\ S_{31}^c & S_{32}^c & 0 \end{bmatrix} \rightarrow \begin{cases} \sigma_1^c \\ \sigma_2^c \\ \sigma_3^c \end{cases} \quad (34)$$

$$\text{Bottom} = \begin{bmatrix} S_{11}^b & S_{12}^b & 0 \\ S_{21}^b & S_{22}^b & 0 \\ 0 & 0 & 0 \end{bmatrix} \rightarrow \begin{cases} \sigma_1^b \\ \sigma_2^b \\ \sigma_3^b = 0 \end{cases} \quad (35)$$

The shear stresses at the midsection are calculated using the Jourawski method [28], according to:

$$S_{13}^c = 1.5 \frac{V1}{h} \quad (36)$$

$$S_{23}^c = 1.5 \frac{V2}{h} \quad (37)$$

Where $V1$ and $V2$ are the resultant shear forces in their respective planar directions.

For the assessment and design of shell-type concrete structures, the function Φ defines a multiaxial strength criterion for concrete, essentially representing a failure surface. Its origin lies in advanced plasticity/fracture models (such as the Drucker-Prager and modified Rankine criteria), calibrated experimentally.

The function employs stress invariants $I1$ (mean stress) and $J2$ (shear stress) to assess whether a point is subjected to critical stress levels. If $\Phi \leq 0$, the concrete remains uncracked; if $\Phi > 0$, it fails or cracks. This criterion is primarily used in nonlinear finite element analyses (FEA) and is not part of the direct design formulas in Eurocode 2 2005 Annex LL

$$\Phi = \alpha \frac{J_2}{f_{cd}^2} + \lambda \frac{\sqrt{J_2}}{f_{cd}} + \beta \frac{I_1}{f_{cd}} - 1 \begin{cases} \leq 0 & \text{Uncracked} \\ > 0 & \text{Cracked} \end{cases} \quad (38)$$

$$J_2 = \frac{1}{6} [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2] \quad (39)$$

$$J_3 = (\sigma_1 - \sigma_m)(\sigma_2 - \sigma_m)(\sigma_3 - \sigma_m) \quad (40)$$

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 \quad (41)$$

$$\sigma_m = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) \quad (42)$$

For reasons of brevity, only the main formulations of the concrete cracking assessment method proposed by European standards are presented in this article. For a complete review of the equations and the methodology employed in this research, the reader is referred to Informative Annex LL, "Concrete Shell Elements," of Eurocode 1992-2:2005 [10].

It should be noted that the application of this method exclusively considers phenomena associated with concrete cracking, thereby neglecting the effects of concrete crushing. As a result, the design

of shell-type elements is limited to tensile reinforcement bars, while compressive steel reinforcement is disregarded. Under accidental seismic loads with bilateral loading and unloading cycles, this simplification does not fully reflect the structural response.

Consequently, in this study, shell-type structures are designed using an optimization-based form-finding model, promoting structural performance primarily under compressive stresses. This approach assumes that the concrete core of the cross section absorbs compressive forces, while the reinforcing steel counteracts tensile stresses (Concrete Three-Layered Model Design).

It is well established that, under accidental loads, internal stress combinations occur within the cross section (tension in some fibers and compression in others). Accordingly, this article subsequently presents a transparent nonlinear time-history and buckling analysis under a nonlinear multi-layered model. The aim is to evaluate the contribution of each material within the cross section, assess the necessity of compressive steel reinforcement, and verify the accuracy and efficiency of current design methods, including the validity of their simplifying assumptions.

4.3 Three-Layer Design Model for Reinforced Concrete Shells

Shell elements are subjected to eight stress resultants: three membrane forces (N_{11} , N_{22} , N_{12}), two bending moments (M_{11} , M_{22}), a twisting moment (M_{12}), and two transverse shear forces (V_1 , V_2). For design, the shell is idealized as a sandwich model with two outer layers carrying membrane and bending forces, and an uncracked core carrying transverse shear [29,30].

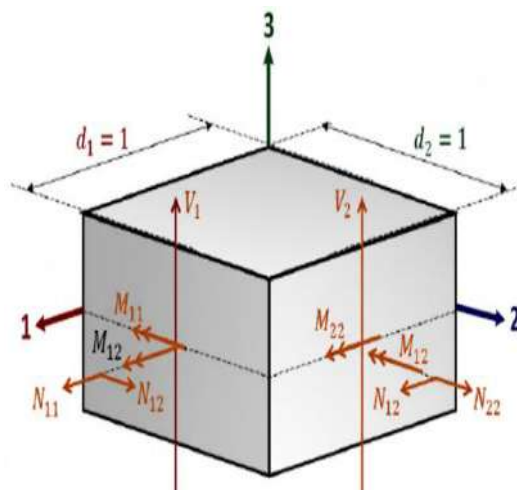


Fig. 13. Unit shell element and stress resultants

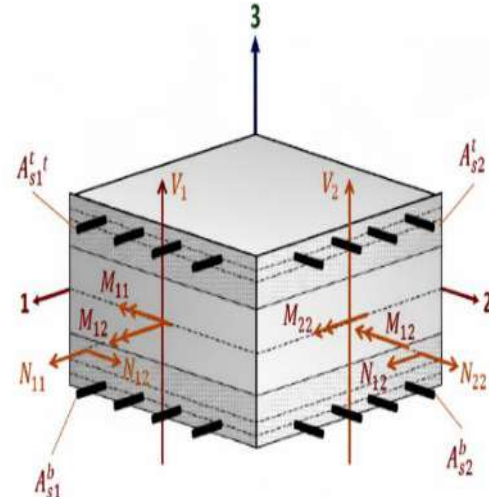


Fig. 14. Idealized three layered model

The membrane stresses associated with the upper and lower regions of the cross section of the shell-type structure, as well as the forces required to resist the reinforcing steel (top and bottom) and the concrete in the central region, are defined as follows:

$$\sum F_1 = 0 \rightarrow N_{s11}^t = N_{11}^t + N_{12}^t \tan \theta \quad (43)$$

$$\sum F_2 = 0 \rightarrow N_{s22}^t = N_{22}^t + N_{12}^t \cot \theta \quad (44)$$

$$\sum F_3 = 0 \rightarrow N_c^t = -\frac{N_{12}^t}{\sin \theta \cos \theta} \quad (45)$$

The moments associated with the membrane stresses at the mid-plane of the layers are established according to:

$$\sum M_{\text{center of the bottom layer}} = 0 \rightarrow N_{11}^t = \frac{N_{11} h^b - M_{11}}{h^t + h^b} \quad (46)$$

$$\sum M_{\text{center of the bottom layer}} = 0 \rightarrow N_{22}^t = \frac{N_{22}h^b - M_{22}}{h^t + h^b} \quad (47)$$

$$\sum M_{\text{center of the bottom layer}} = 0 \rightarrow N_{12}^t = \frac{N_{12}h^b - M_{12}}{h^t + h^b} \quad (48)$$

The same formulas are assumed for the upper layer.

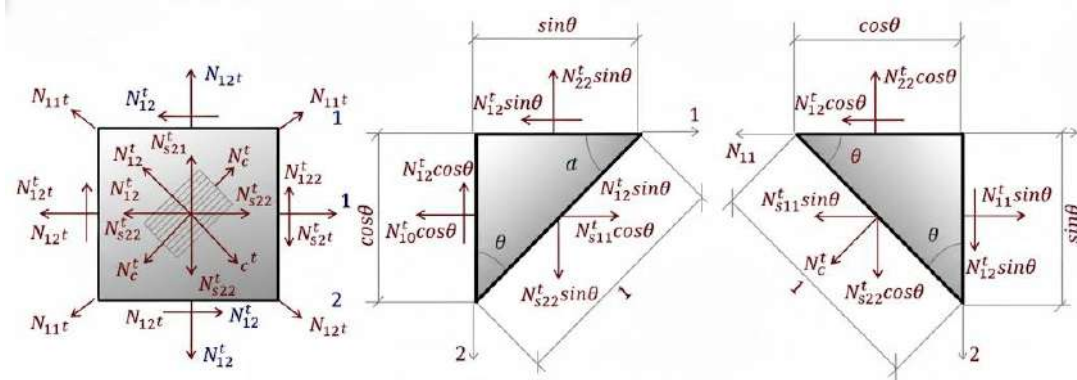


Fig. 15. Top layer forces(left); section parallel to the crack (center); section perpendicular to the crack (right)

For the case of this study, the reinforcing steel in the concrete shells is assumed to be present in both layers, upper and lower. Consequently, the forces that must be resisted by the reinforcing steel in both layers are defined as follows:

$$N_{s11}^t = N_{11}^t + |N_{12}^t| \cot \theta \quad (49)$$

$$N_{s22}^t = N_{22}^t + |N_{12}^t| \tan \theta \quad (50)$$

$$N_c^t = \frac{|N_{12}^t|}{\sin \theta \cos \theta} \quad (51)$$

The angle θ denotes the orientation of the principal compressive stress field within the concrete with respect to the reference axis of the element.

The stresses in the reinforcing steel are calculated when such reinforcement is required in both layers, according to:

$$\sum M_{\text{bottom reinforcement}} = 0 \rightarrow N_{d11}^t = \frac{N_{s11}^t(h^t + h_{s1}^b) + N_{s11}^b(h_{s1}^t - h^b)}{(h_{s1}^t + h_{s1}^b)} \quad (52)$$

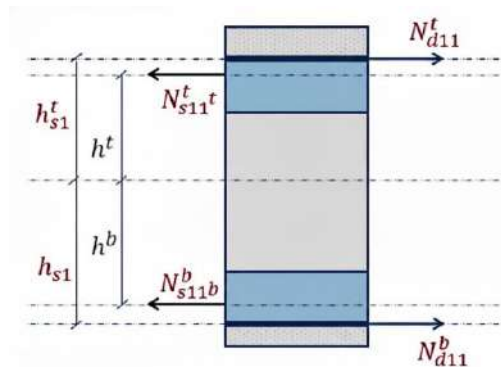


Fig. 16. Loads on the reinforcement for both layers in sandwich element

The required reinforcement area per unit length and the design thicknesses of the bottom and top layers are determined as:

$$A_{s1}^t = \frac{N_{d11}^t}{f_{yd}} \quad (53)$$

$$A_{s2}^t = \frac{N_{d22}^t}{f_{yd}} \quad (54)$$

$$a^t = \frac{N_c^t}{f_c} \quad (55)$$

The same calculations are performed for the lower layer of the cross section. Here f_{yd} is the design yield strength of the reinforcement and f_{cd} the design compressive strength of the concrete, both derived from the characteristic values reported in Table 2 and 3 through the partial safety factors $\gamma_s = 1.15$ and $\gamma_c = 1.5$ prescribed by [10,26]. Only the main formulations applicable to the models of this study are presented; the reader is referred to [31] for a comprehensive treatment.

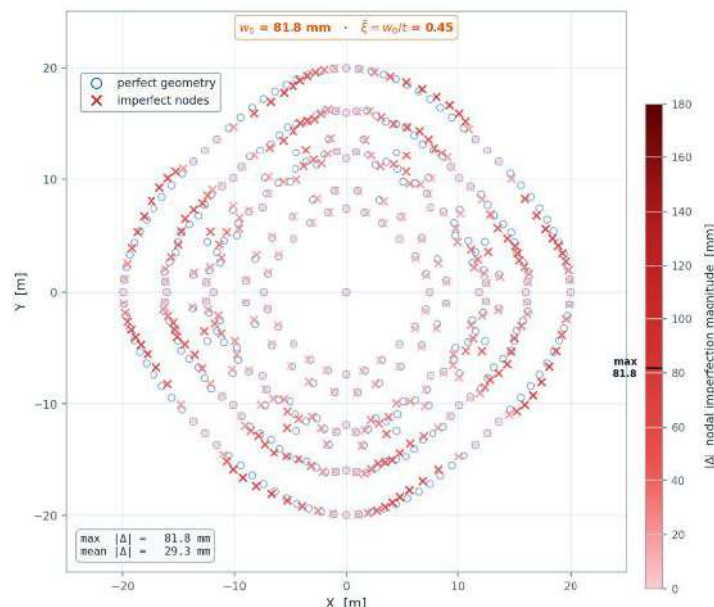
5. Nonlinear Numerical Analysis of Shell-Type Concrete Models

All results reported in this section correspond to the imperfect configuration of the three shells. The perfect geometry is retained solely as a reference for the fundamental periods and the buckling capacity, in order to quantify the loss of stiffness and of load-carrying capacity introduced by the geometric imperfections.

5.1 Initial Geometric Imperfections and Imperfection Sensitivity

5.1.1 Generation of the Imperfect Geometry

The formulations of Section 3.3.1 are applied here to the three shells. For each model the first eigenmode of a linear buckling analysis under gravity loading was extracted, normalized and superimposed on the nodal coordinates [32] according to Eq. (24). Following Koiter's principle, the mode-affine field is the most degrading imperfection shape when the critical mode is isolated, so that the configurations analyzed represent the most unfavorable case for the amplitude adopted. Since the mode vanishes at the supports, the boundary conditions of the perfect and imperfect models are identical, and the comparison isolates the effect of the geometric deviation alone. Figures 17 to 19 show both geometries; Section 5.1.2 quantifies the resulting loss of capacity.



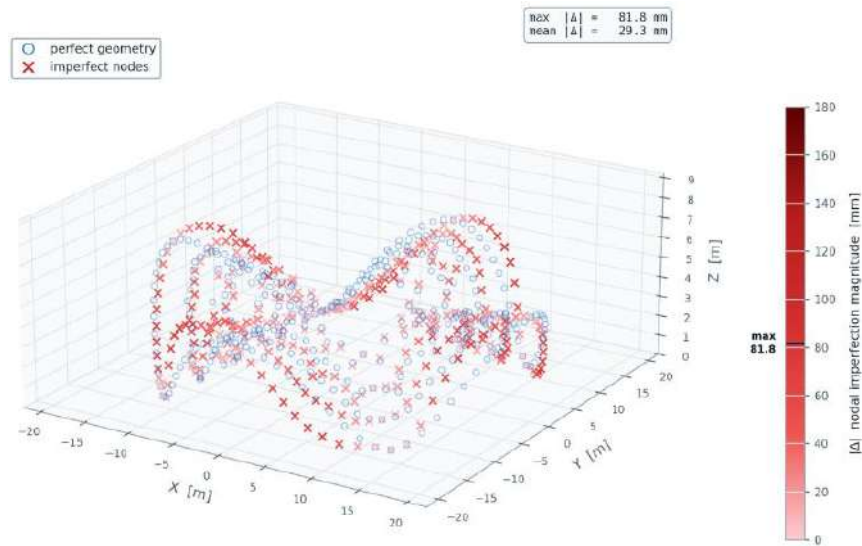


Fig. 17. Perfect and imperfect nodal geometries of the PH-04 model for $w_0 = 81.8$ mm ($\xi = 0.45$, dimensionless): (a) plan view, (b) 3D view; color: $|\Delta|$ (mm)

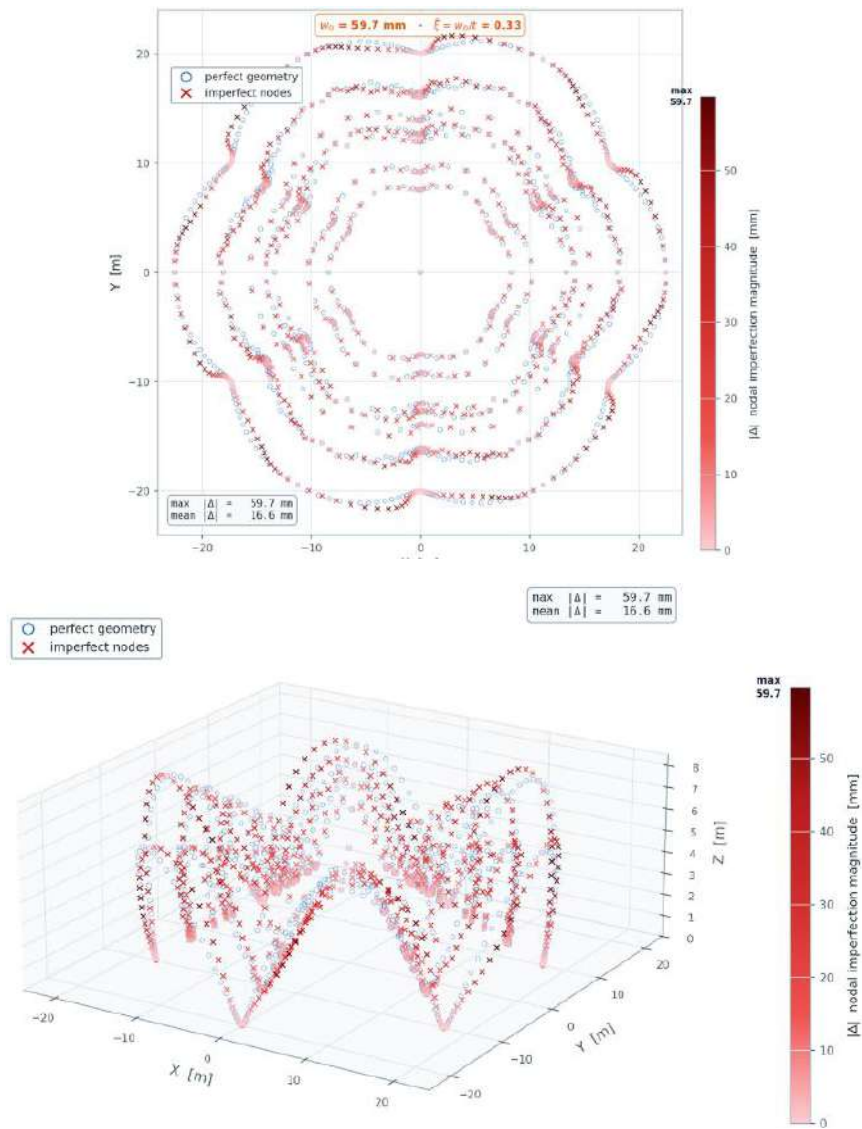


Fig. 18. Perfect and imperfect nodal geometries of the PH-06 model for $w_0 = 59.7$ mm ($\xi = 0.33$, dimensionless): (a) plan view, (b) 3D view; color: $|\Delta|$ (mm)

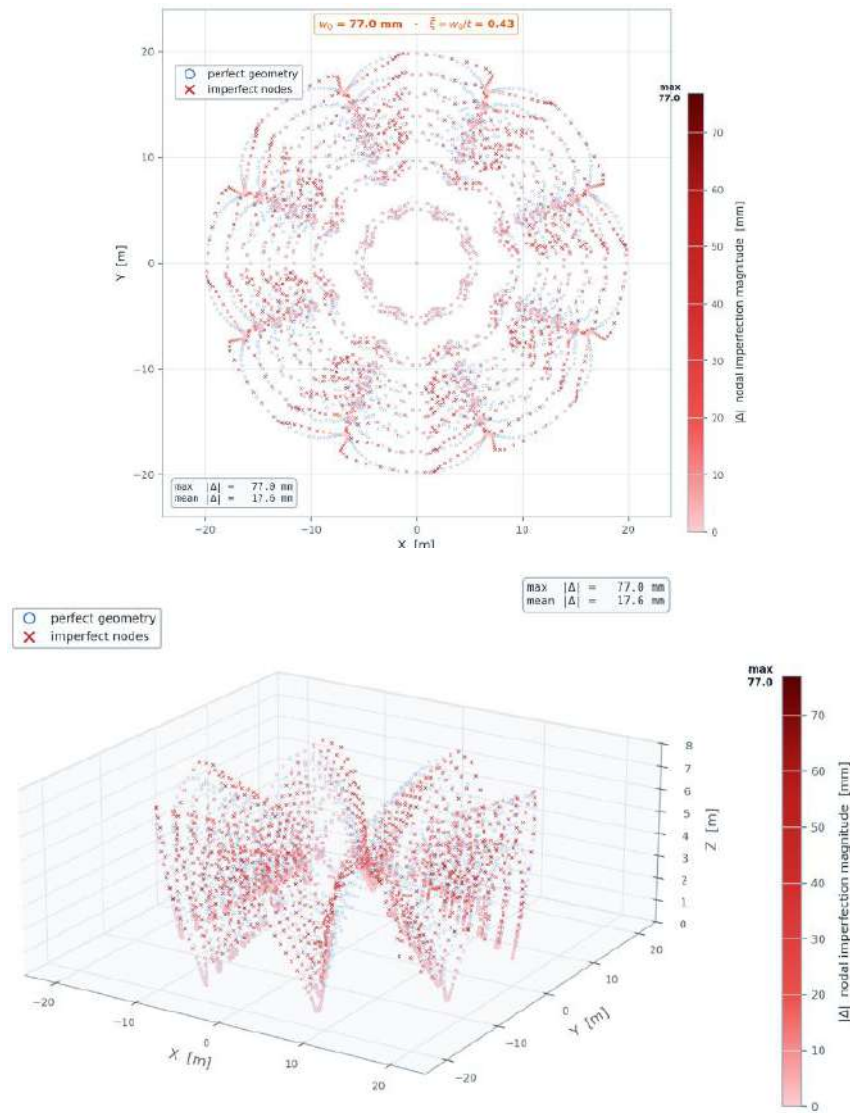


Fig. 19. Perfect and imperfect nodal geometries of the PH-08 model for $w_0 = 77.0$ mm ($\xi = 0.43$, dimensionless): (a) plan view, (b) 3D view; color: $|\Delta|$ (mm)

5.1.2 Nonlinear Buckling Analysis

Due to their slenderness, shell-type structures are prone to buckling problems, making it fundamentally important to conduct nonlinear buckling studies incorporating large-deformation theory [20]. It is well known that, for complex structures, closed-form solutions to analytically determine critical buckling loads or their associated configurations do not currently exist. Therefore, finite element simulation approaches are employed. In this study, these simulations are carried out following formulations such as:

$$K_{total}(u)\Delta u = F_{ext} - F_{int}(u) \quad (56)$$

The total nonlinear system stiffness is given by:

$$K_{total}(u) = K_L - K_G(u) + K_{NL}(u) \quad (57)$$

Where K_L represents linear stiffness, K_G the geometric stiffness due to initial stresses, K_{NL} the material nonlinear stiffness, u the displacements and F the internal and external forces. Using SAP2000, this equation is solved iteratively by employing methods such as Newton-Raphson, analyzed models produced the following results:

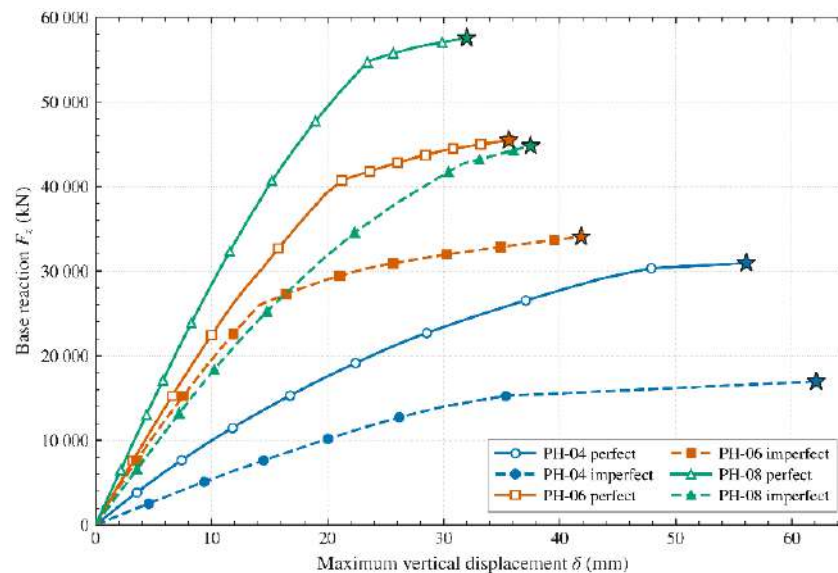


Fig. 20. Buckling in analyzed models under critical load

5.1.3 Knock-Down Factors

The imperfection sensitivity of each shell is quantified by the knock-down factor $\rho = F_{u,imp} / F_{u,per}$, obtained from the curves of Fig. 20. The resulting values, 0.548, 0.638 and 0.779 for PH-04, PH-06 and PH-08 (Fig. 21), lie well above the values of about 0.2 reported for cylindrical and spherical shells [23,24], confirming the lower sensitivity of anticlastic geometries [5]. Since PH-04 and PH-08 share nearly the same amplitude ($\xi = 0.45$ and 0.43), the higher ρ of PH-08 is geometric: shorter spans between supports confine the critical mode. These factors are adopted in this study as the stability reduction for each shell.

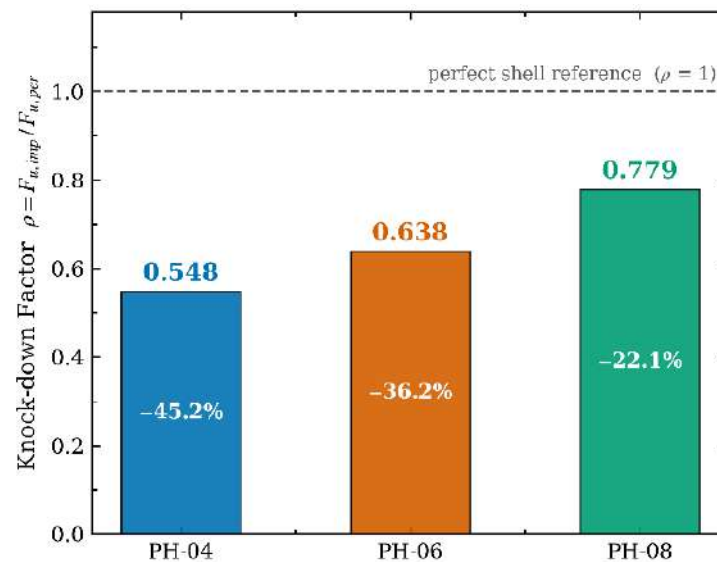


Fig. 21. Imperfection sensitivity on the shells

5.2 Modal Analysis and Vibration Characteristics

Given that the analysis is nonlinear and conducted in the time domain under accidental seismic excitations, it is essential to understand the vibration characteristics of shell-type structures. This understanding is achieved by solving the modal problem in terms of eigenvectors [33]:

$$\det[K - \Omega^2 M] = 0 \quad (58)$$

Where K represents the stiffness matrix, $\mathcal{D}$ is the diagonal matrix of eigenvalues, and M is the mass matrix. The modal shapes and fundamental periods of the studied models are defined accordingly:

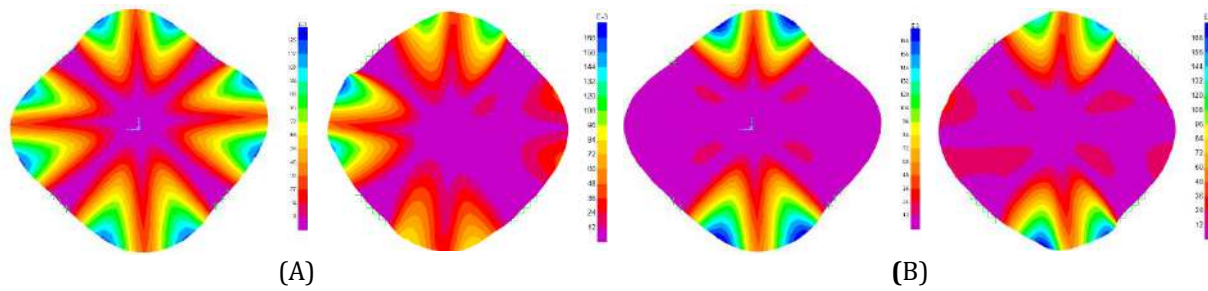


Fig. 22. Comparative modal shapes of the PH-04 model in mode 1(A) and 2 (B) (perfect geometry left, imperfect geometry right)

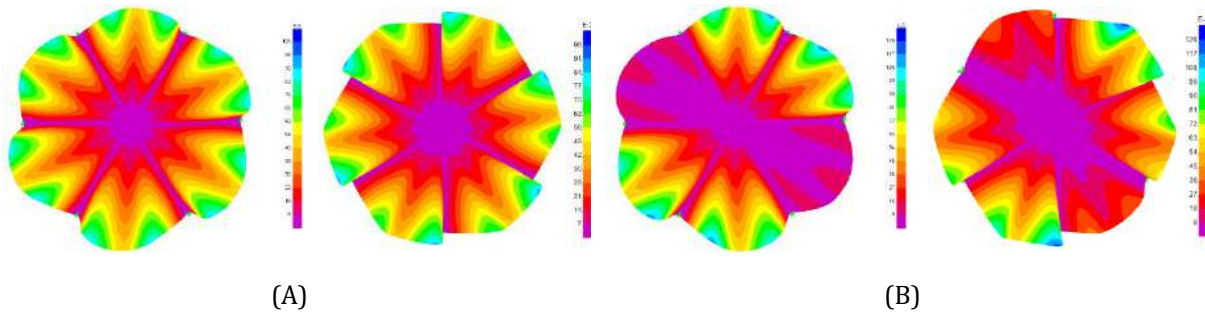


Fig. 23. Comparative modal shapes of the PH-06 model in mode 1(A) and 2 (B) (perfect geometry left, imperfect geometry right)

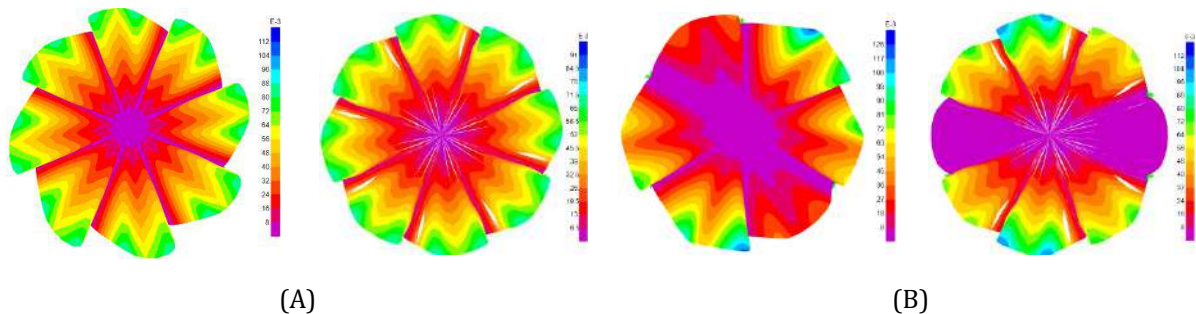


Fig. 24. Comparative modal shapes of the PH-08 model in mode 1(A) and 2 (B) (perfect geometry left, imperfect geometry right)

Table 4. Fundamental periods of the three shell models, perfect and imperfect geometry

MODE	Model PH-04		Model PH-06		Model PH-08	
	perfect	imperfect	perfect	imperfect	perfect	imperfect
1	0.59 sec	0.67 sec	0.49 sec	0.56 sec	0.26 sec	0.31 sec
2	0.56 sec	0.63 sec	0.46 sec	0.51 sec	0.25 sec	0.29 sec
3	0.56 sec	0.62 sec	0.46 sec	0.49 sec	0.25 sec	0.28 sec

The modal analysis of anticlastic shells is complex. Their fundamental vibration modes are not purely translational or rotational, due to their intrinsic geometric stiffness derived from the double curvature. This characteristic directly influences their modal configurations, leading to combined stress states involving bending moments, tension, and compression distributed throughout the entire surface. Determining these modal parameters is crucial because they provide insight into the dynamic behavior of the structure, including the distribution of deformations, resonance frequencies, and potential vulnerability under transient or extreme loading conditions.

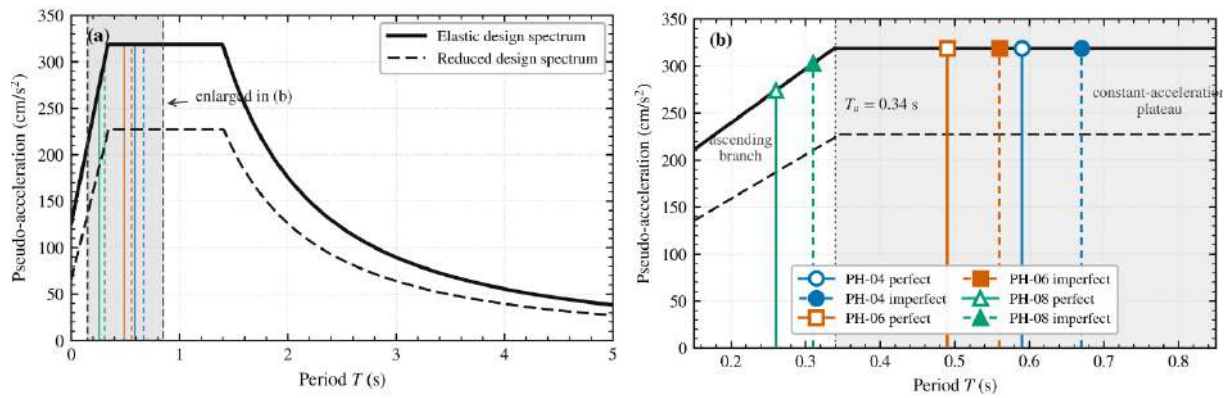


Fig. 25. Fundamental periods of the three shell models, in their perfect and imperfect configurations, placed on the design spectrum

For shell structures, which typically exhibit complex curvature and coupling between bending and membrane actions, accurate identification of the fundamental modes ensures that nonlinear time-history analyses under seismic events correctly capture the interplay between material nonlinearity, geometric effects, and dynamic response.

5.3 Stress State Under Design Load Combinations

The resultant stresses in the analyzed models incorporate the previously defined load combinations and are formulated within the framework of finite strain theory, leading to the following outcomes:

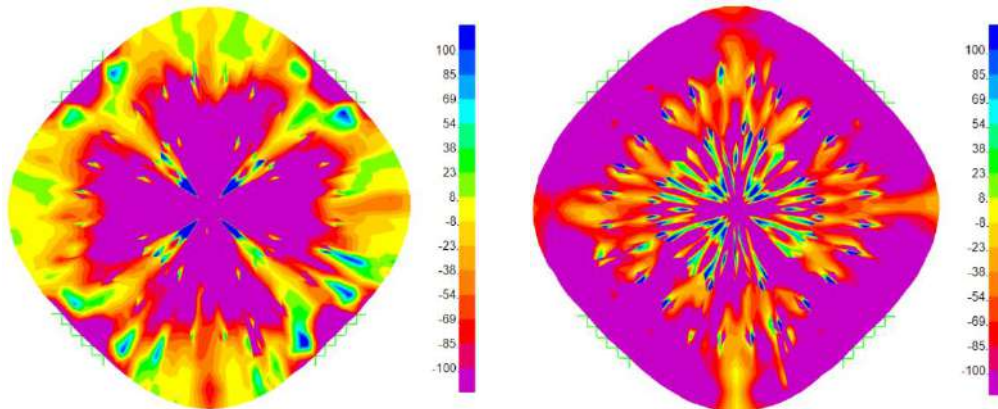


Fig. 26. Resultant forces of the PH-04 Model longitudinal (left) transversal (right) in N/mm²

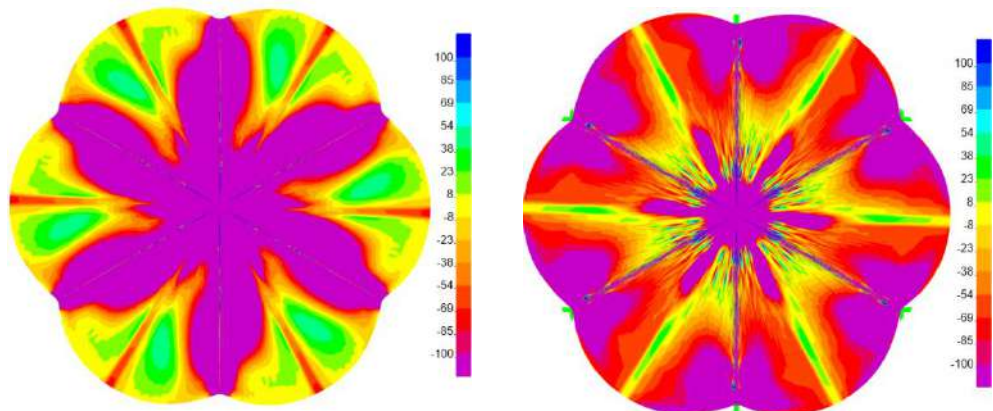


Fig. 27. Resultant forces of the PH-06 Model longitudinal (left) transversal (right) in N/mm²

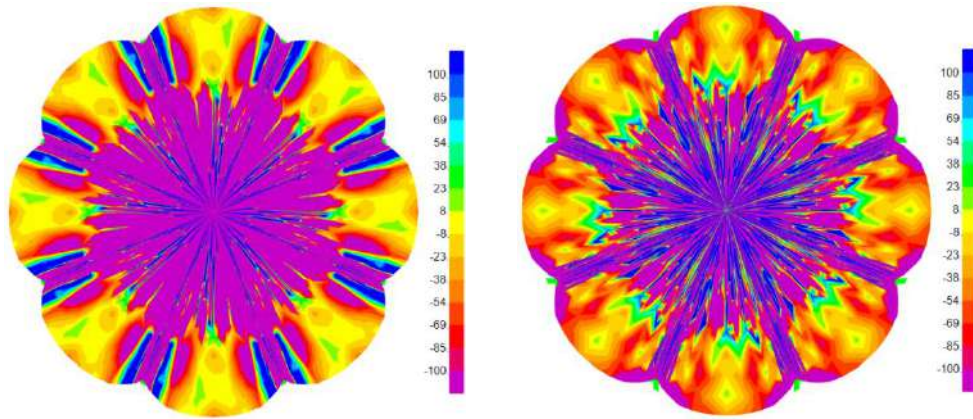


Fig. 28. Resultant forces of the PH-08 Model longitudinal (left) transversal (right) in N/mm^2

5.4 Reinforcement Design

The design of the shell structures was carried out in accordance with the Eurocode provisions previously described in Chapter 4 of this study. The numerical analysis is conducted in three fundamental stages after the design forces have been determined. The first stage involves verifying the cracking zones and intensity levels in the cross section of the concrete shells. Subsequently, the required reinforcement ratios are obtained for both the upper and lower layers in both directions. Finally, the reinforcement design using steel bars is established, which provides the basis for transparent nonlinear time-history analyses and subsequent verification using the multi-layered method.

5.4.1 Cracking Assessment

The cracking zones and intensity levels in the study models are shown, using a color scale as a reference. Blue indicates areas where the shell remains uncracked, while red represents regions experiencing the highest cracking intensity.

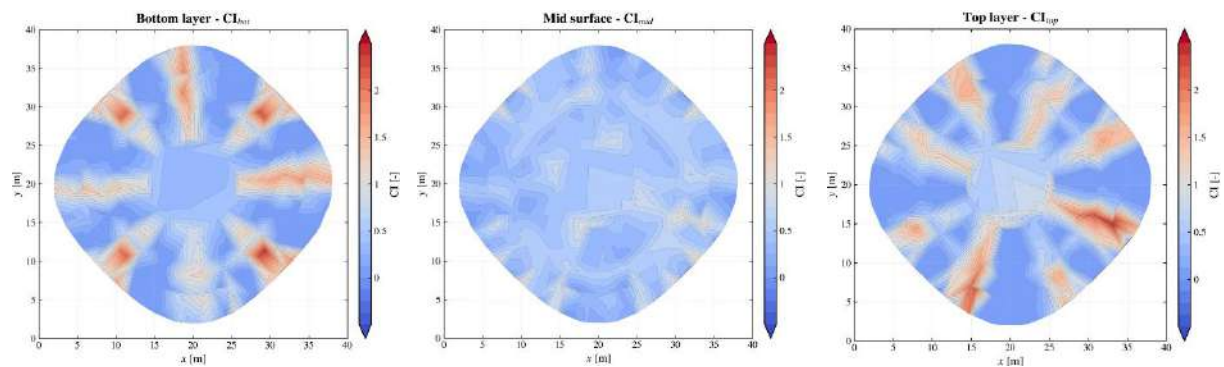


Fig. 29. Cracking of the PH-04 model (if $CI > 0 = \text{cracked zone}$)

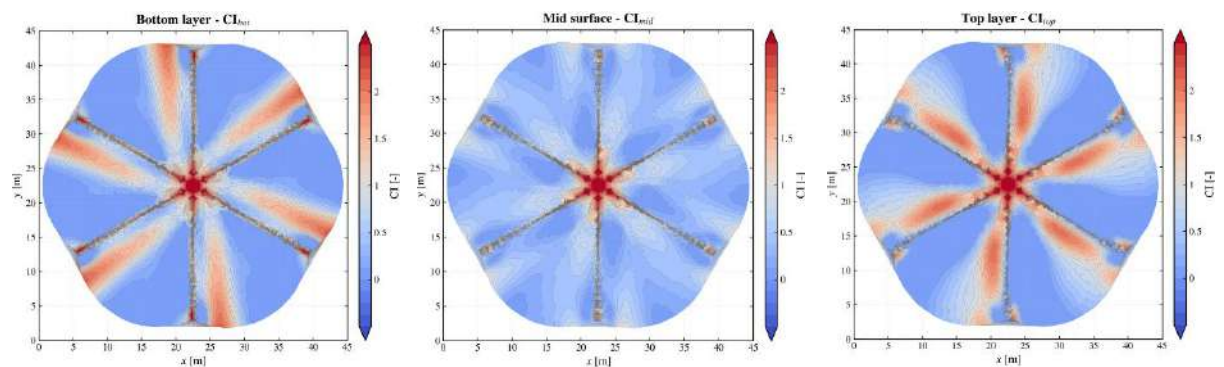


Fig. 30. Cracking of the PH-06 model (if $CI > 0 = \text{cracked zone}$)

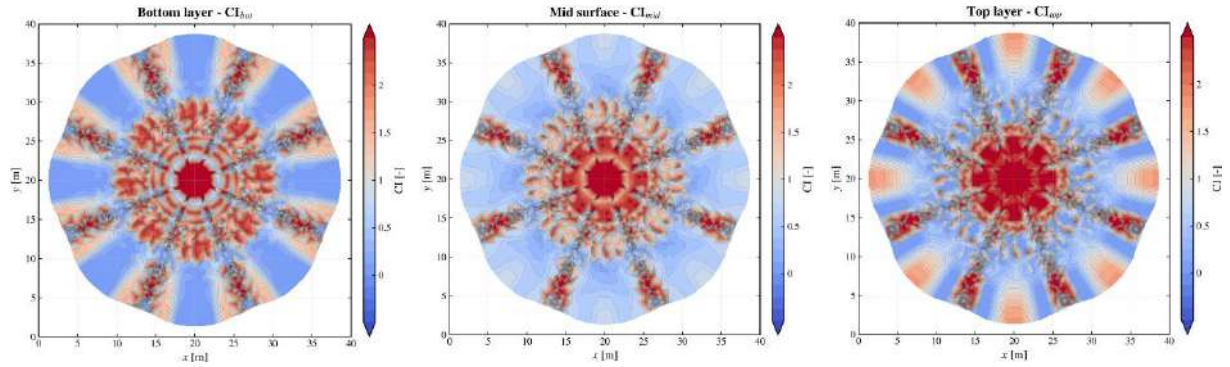


Fig. 31. Cracking of the PH-08 model (if $CI > 0 = \text{cracked zone}$)

The cracking maps of Figs. 29 to 31 must be interpreted in the light of the assumption stated in Section 4.2: the criterion of Eq. (38) evaluates concrete cracking only and does not assess crushing. The results are therefore conservative with respect to cracking every region in which the criterion is exceeded is reported as cracked but non-conservative with respect to the compressive regime, since the loss of capacity that follows the peak compressive strain is not detected. Under the bilateral loading and unloading cycles imposed by the seismic records, this asymmetry matters most where the compressive demand is highest, namely at the supports and at the peak points of the surface, and it implies that the ductility available in those regions is overestimated. The extent of this limitation is discussed in Section 6.

5.4.2 Required Reinforcement Area

Under the hybrid framework of Section 4, the ratio between the required and the minimum steel area, $A_{s,req}/A_{s,min}$, is mapped in Fig. 32 for the governing bottom layer of each model:

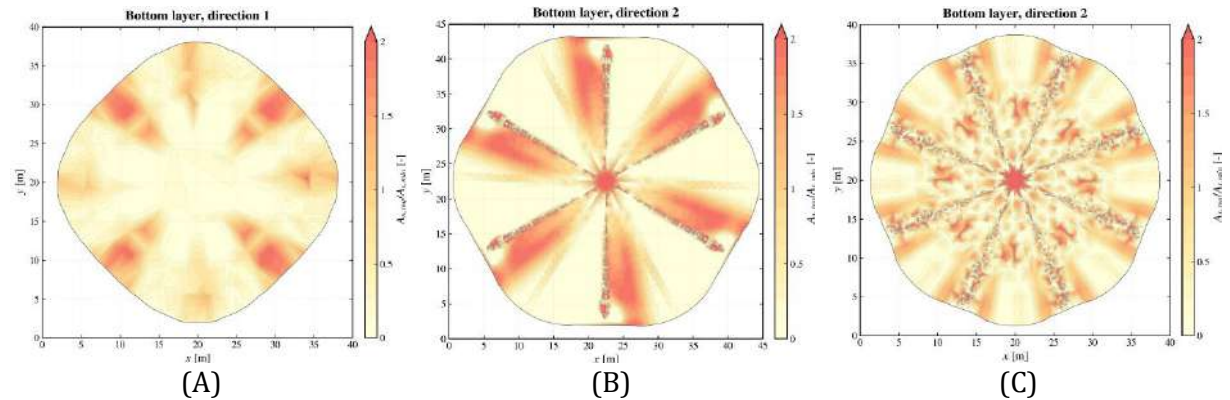


Fig. 32. Maximum $A_{s,req}/A_{s,min}$ ratio (dimensionless) in the governing bottom layer: (A) PH-04, direction 1; (B) PH-06 and (C) PH-08, direction 2

The reinforcement design, conducted under a hybrid framework (NTC-2023 actions and minimum steel; Eurocode 2 with Wood–Armer correction [34] for the conversion of shell forces into steel areas), demonstrates that form-finding by the force density method governs the reinforcement demand. The code minimum ($A_{s,min} = 337 \text{ mm}^2/\text{m}$) [35] controls 74.0%, 49.7% and 59.1% of the surface of PH-04, PH-06 and PH-08, respectively. This provides quantitative evidence that the optimized geometry preserves a near-membrane stress state under seismic combinations. The residual demand is not randomly distributed: it concentrates systematically in the bottom layer, along the meridian bands and at the crown, the same regions identified as critical in the cracking analysis. A uniform two-layer mesh of #4 bars at 125 mm (317.8 n/m^2), supplemented locally along the meridian bands, satisfies 100%, 98.2% and 99.1% of the demand. The residual peaks are confined to mesh singularities and supports. Because the multilayered formulation explicitly accounts for the compressive contribution of the reinforcement and the actual lever arm, it requires [6 - 12%] less steel than the Eurocode 2 sandwich model. This result shows that the sandwich idealization is systematically conservative for seismically loaded thin shells.

Table 5. Max required steel area and adopted two-layer reinforcement of the three shell models

LAYER	Model PH-04		Model PH-06		Model PH-08	
	As,req,max (mm ² /m)	reinforcement	As,req,max (mm ² /m)	reinforcement	As,req,max (mm ² /m)	reinforcement
Top, direction x	260	#4 @ 125 mm	1 069	#4 @ 125 mm	2 442	#4 @ 125 mm
Top, direction y	354	#4 @ 125 mm	1 666	#4 @ 125 mm	2 141	#4 @ 125 mm
Bottom, direction x	1 055	#4 @ 125 mm	3 929	#4 @ 125 mm	4 022	#4 @ 125 mm
Bottom, direction y	686	#4 @ 125 mm	4 326	#4 @ 125 mm + local*	4 424	#4 @ 125 mm + local*
Area governed by As,min (%)	74.0		49.7		59.1	
Area covered by reinforcement (%)	100		98.2		99.1	

5.5 Nonlinear Seismic Time-History Analysis

This type of analysis is commonly employed to evaluate the nonlinear dynamic response of structures under seismic loading in an incremental manner, by solving the equations of motion as follows:

$$Kx(t) + C\dot{x}(t) + M\ddot{x}(t) = f(t) \quad (59)$$

Where K is the stiffness matrix, C is the damping matrix, M represents the diagonal mass matrix, and x , $\dot{x}$, and $\ddot{x}$ are the displacement, velocity, and acceleration vectors of the structure, respectively, with f denoting the vector of applied forces.

The seismic signals utilized in this study were synthetically generated in accordance with the provisions of Mexican seismic regulations, representing a stiff soil site condition in Mexico City, and are defined as follows:

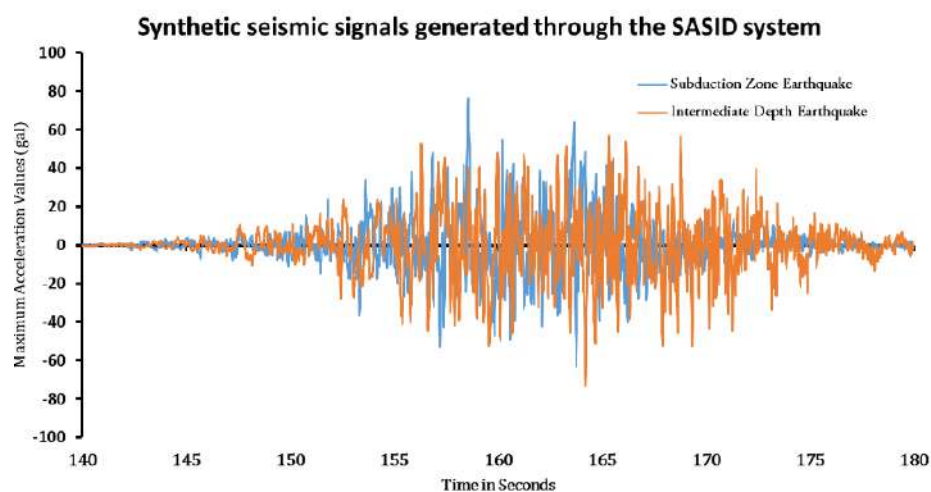


Fig. 33. Synthetic Seismic Records used in Time-History Analysis

The nonlinear time-history analyses were not started from an unstressed configuration. A nonlinear static case comprising dead load and service live load was solved first, and the seismic cases were defined to continue from its final state, so that the stresses and deformations already present under operating conditions are inherited as the initial condition of the seismic response

rather than superimposed afterwards. Consistently, the mass source adopted for both the modal and the time-history analyses comprised 100% of the dead load and 100% of the instantaneous live load $W_a = 0.69 \text{ kN/m}^2$, prescribed for roofs in Table 6.1.2.2 of the Mexican Complementary Technical Standards on Criteria and Actions for Structural Design, whose Section 6.1.2.2 requires the instantaneous live load to be adopted for accidental actions such as earthquake [8]. Time-dependent material effects were not included in this initial state: creep and shrinkage are excluded from the model, and their omission is discussed in Section 6.

5.5.1 Hysteretic Model Used in The Analysis

A hysteretic model based on the formulation proposed by Takeda et al. (1970) [36] is employed to simulate the nonlinear behavior of reinforced concrete structures and of elements with degradable stiffness. This model allows reproducing the bilinear post-yield hardening and the progressive degradation of material stiffness as a function of cyclic displacements.

The Takeda hysteretic model effectively reproduces the experimentally observed degradation of secant stiffness in reinforced concrete under cyclic loading. Its formulation employs a modified unloading rule toward the origin, governed by secant stiffness and the amplitude of prior maximum displacements, thus capturing stiffness degradation and the pinching effect typical of cracking materials. Consequently, its application in the nonlinear analyses of this study is justified, as it allows realistic simulation of damage progression and seismic response in concrete shells, supporting the evaluation and validation of geometrically optimized structural models.

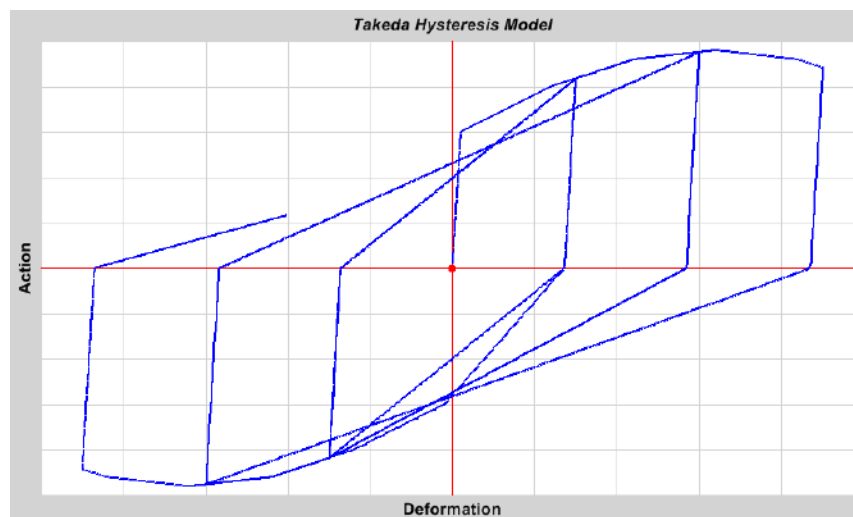


Fig. 34. Takeda's Hysteretic Model [36]

5.5.2 Nonlinear Multi-Layered Model

The implementation of a multi-layer model in nonlinear time-history analyses aims to ensure compatibility with the nonlinear material models, large-deformation theory, and hysteretic behavior models, in order to accurately identify the damage zones and intensity levels experienced by shell-type structures under seismic loading. The multi-layer approach enables a detailed representation of the stress and strain distribution through the shell thickness, considering a linear variation of deformations across the depth. This improves accuracy compared to the three-layer sandwich model [37,38], which assumes a uniform distribution.

The main rationale for employing a multilayered model in the nonlinear time-history analyses conducted in this research lies in the need to evaluate the hypotheses and assumptions prescribed in the Eurocode, specifically in Annex LL, where it is assumed that the reinforcement in concrete shell structures only resists tensile stresses, while the central concrete layer of the three-layered element is responsible for carrying compressive stresses. Unlike the simplified sandwich model, the multilayered approach enables an accurate representation of compressive reinforcement behavior, which the former cannot adequately capture. This limitation often leads to over-dimensioned and excessively heavy concrete shells.

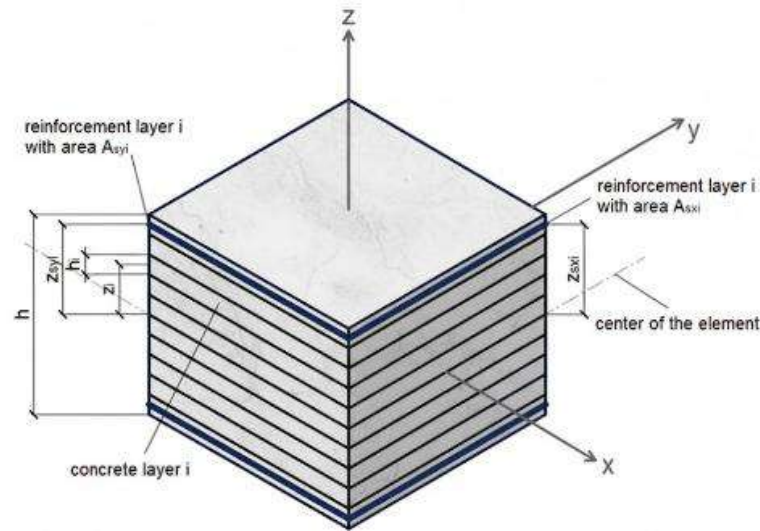


Fig. 35. Reinforced concrete shell element idealized by Kollegger et al. [37]

Therefore, the employment of advanced nonlinear modeling techniques, such as the multilayered model, allows for a more accurate and reliable structural analysis, fostering the safe slenderness optimization of shell-type elements, which represents one of the core design principles of this investigation.

The compressive contribution of the reinforcement reported here does not imply that compression reinforcement is required as such in a thin shell. It follows from the reversal of the stress state under bilateral seismic excitation: a fiber that carries tension in one half-cycle carries compression in the next, so that reinforcement placed to resist the tensile demand is subsequently compressed [30,39]. What the multilayered analyses show is that this contribution is not negligible: at the sections of highest demand, the reinforcement layers carry a measurable share of the peak compressive force, which the three-layer model assigns entirely to the concrete core. The three-layer sandwich model of Annex LL assigns that force entirely to the concrete core, and the consequence is systematic: the concrete layer is sized for a demand it does not carry alone, and the resulting section is larger than the response requires.

The principal equation of the multilayer method is defined as:

$$K_{multilayer}(u)\Delta u = F_{ext} - F_{int}(u) \quad (60)$$

$$K_{multilayer} = \sum_{i=1}^n \int_A B_i^T C_i B_i dA \quad (61)$$

Where B represents the strain matrix for each layer, C the stiffness matrix for each layer, and A the finite element of the shell-type surface.

Only the main simplified formulations of the multilayered method are presented herein; readers are encouraged to consult [40] for a comprehensive understanding.

5.5.3 Nonlinear Time-History Results in Models

The three shells were subjected to the synthetic records EQ ST 01 and EQ SM 02 ($\Delta t = 0.02$ s, 45 s), applied to the gravity-deformed state of the imperfect geometry. The lateral displacement of three control joints and the base shear V_x were recorded for each model. The plots show the total displacement u_x with the initial state u_0 as reference, whereas the reported quantities are excursions from u_0 , that is, the displacement component induced by the earthquake. The response was evaluated through the peak displacement, which quantifies the instantaneous demand; the residual displacement at the end of the record, which quantifies the irreversible deformation; and the linearity of the $V_x - u_x$ relationship, expressed by the correlation coefficient R .

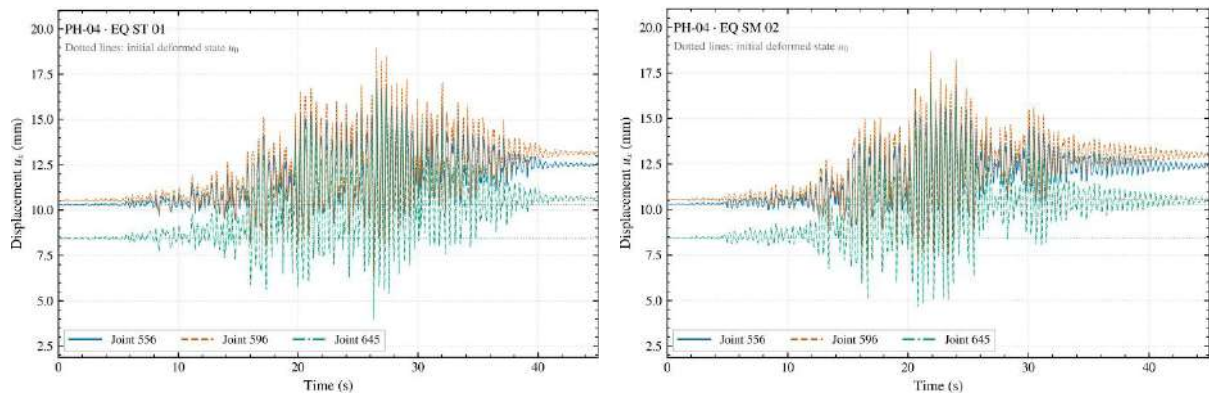


Fig. 36. Maximum lateral displacements in PH-04

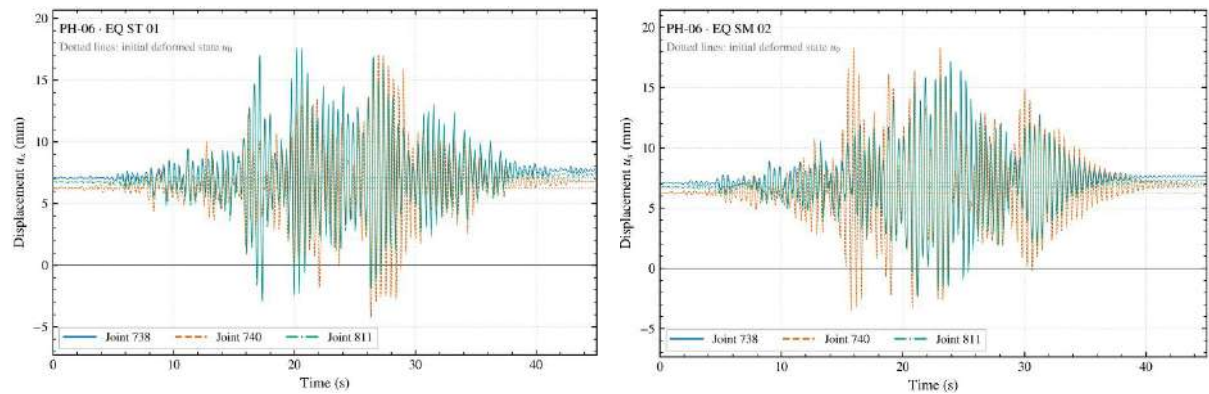


Fig. 37. Maximum lateral displacements in PH-06

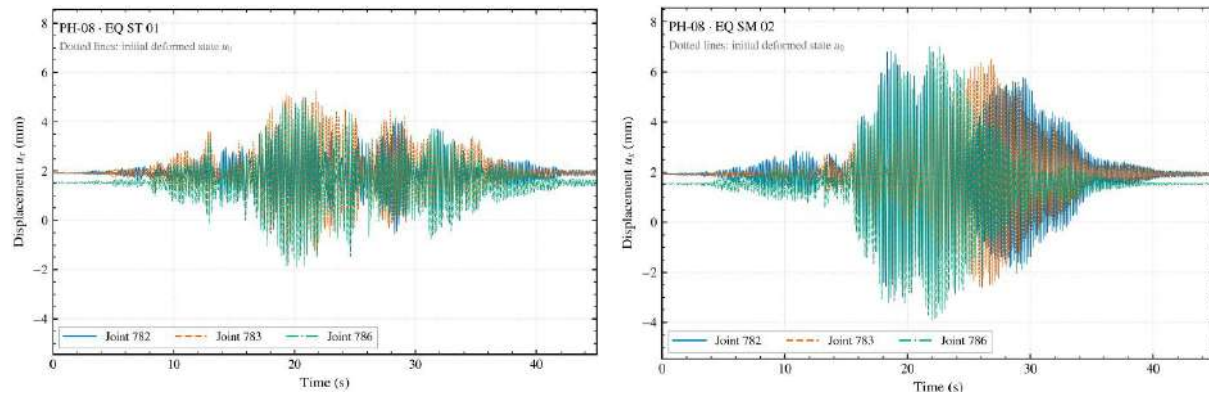


Fig. 38. Maximum lateral displacements in PH-08

Figs. 36 to 38 show the displacement histories of the three control joints of each shell under both records. The joints of each model respond consistently with one another and between records. In PH-04, the three joints end the record displaced above u_0 , whereas in PH-06 and PH-08 they return practically to their initial position.

The peak excursions were 8.2 to 8.4 mm in PH-04, 10.9 to 12.0 mm in PH-06, and 3.5 to 5.5 mm in PH-08, with no monotonic trend with the number of supports. PH-06 reached the largest peak despite a lower base shear than PH-08 because of its lower lateral stiffness. The residual displacement, in contrast, ranked the models monotonically: 2.4 to 2.6 mm in PH-04 (29% to 31% of the peak), 0.6 to 0.8 mm in PH-06 (5% to 6%), and 0.06 mm or less in PH-08 (below 2%). Because it does not depend on the instantaneous amplitude, this is the indicator that reflects accumulated damage. To compare the models on a common scale, Fig. 39 superimposes the response of the homologous joints 596, 740, and 786. Figs. 40 to 42 present the relationship between base shear and displacement at the control joints of each shell.

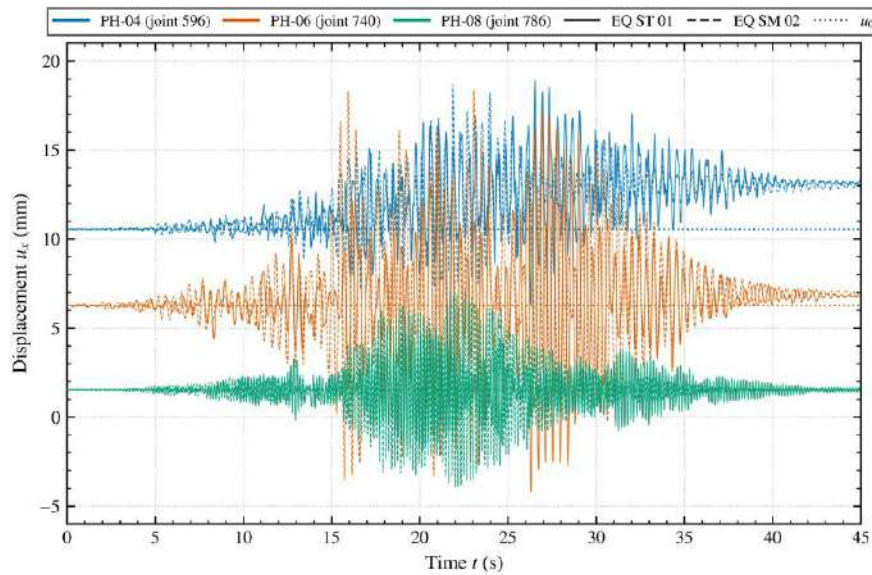


Fig. 39. Lateral displacement histories of the homologous control joints of PH-04 (596), PH-06 (740), and PH-08 (786) under both records

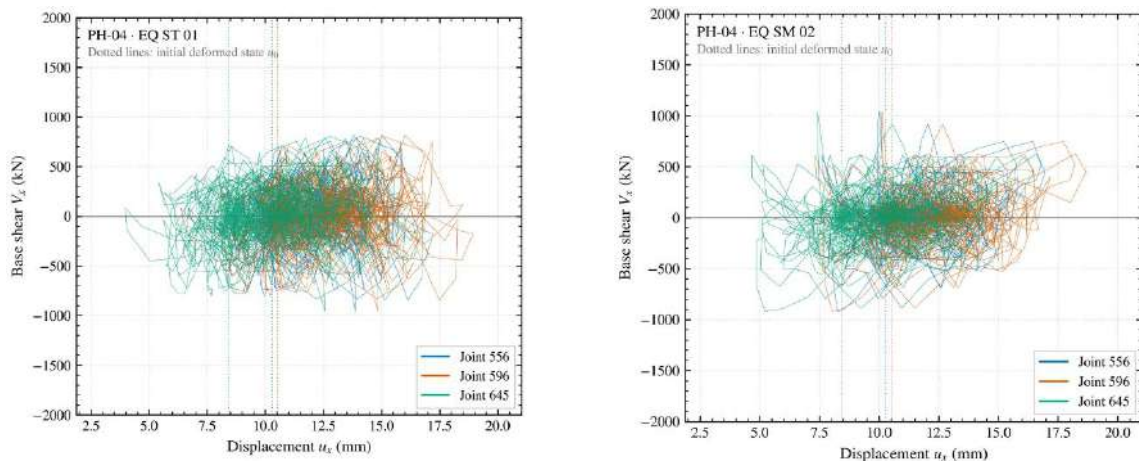


Fig. 40. Base shear V_x versus lateral displacement u_x of the control joints of PH-04 under EQ ST 01 (left) and EQ SM 02 (right)

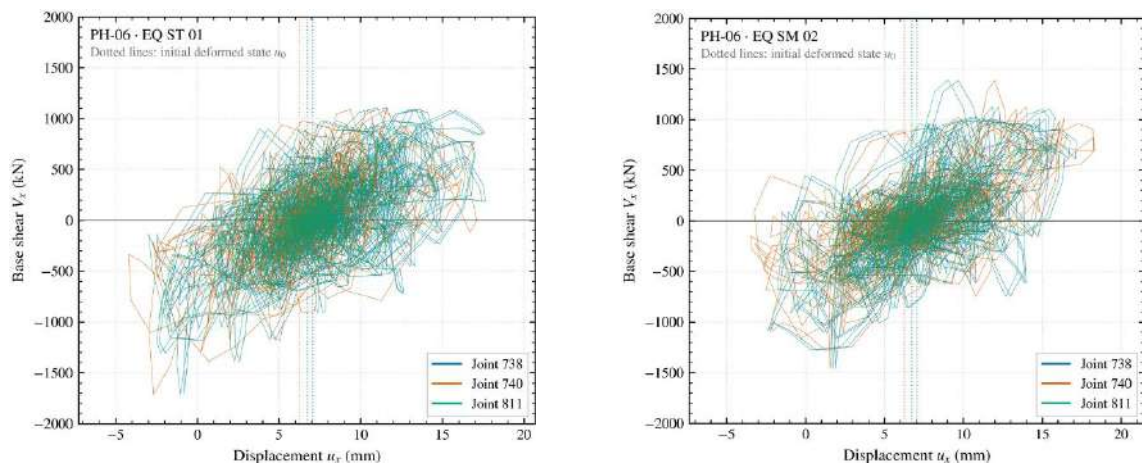


Fig. 41. Base shear V_x versus lateral displacement u_x of the control joints of PH-06 under EQ ST 01 (left) and EQ SM 02 (right)

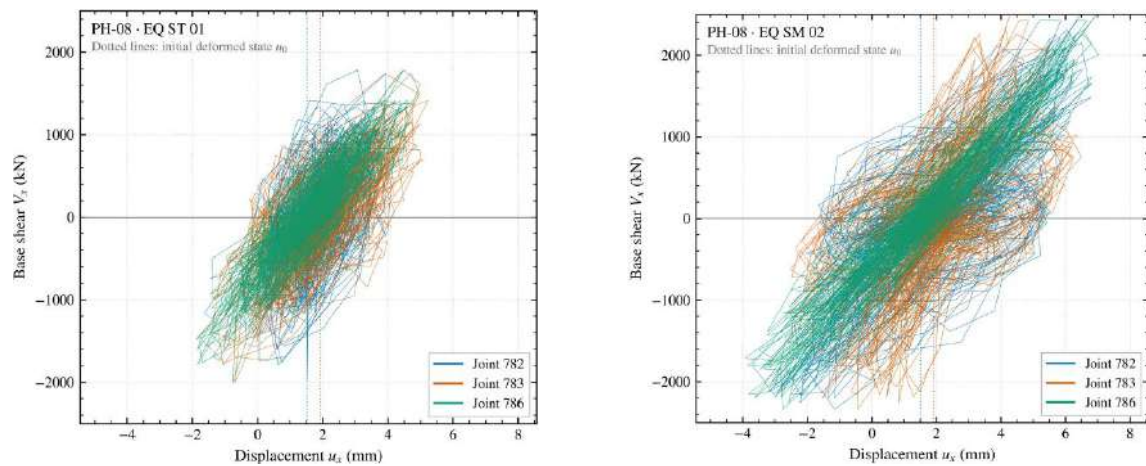


Fig. 42. Base shear V_x versus lateral displacement u_x of the control joints of PH-08 under EQ ST 01 (left) and EQ SM 02 (right)

The shape of the response changes with the number of supports. In PH-08 it is a narrow band aligned with a straight line ($R = 0.88$ to 0.96); in PH-06, an inclined and more scattered cloud ($R = 0.55$ to 0.59); and in PH-04, a practically horizontal cloud, shifted toward positive values and with no correlation between force and displacement ($R = 0.13$ to 0.20). Fig. 43 combines the loops of the homologous joints with the stiffness obtained by linear regression, which allows the mechanism to be quantified.

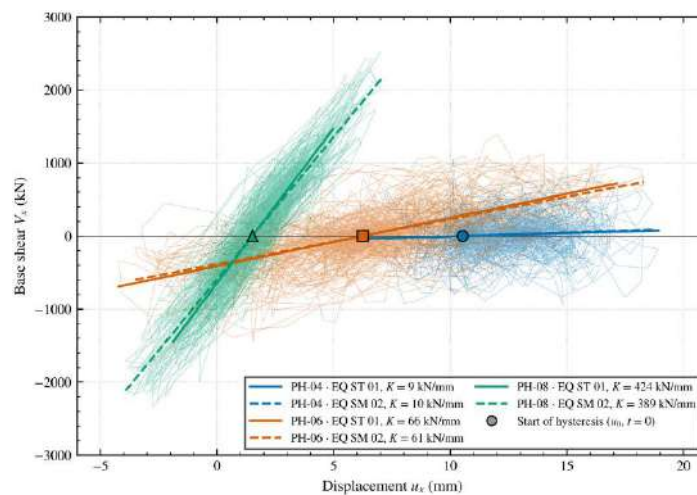


Fig. 43. Base shear–displacement loops of the homologous control joints of the three shells, with the stiffness K obtained by linear regression; markers indicate the start of each loop ($u_0, t = 0$)

5.6 Design Efficiency of The Multilayer Formulation Relative to The Three-Layer Sandwich Model

The three shells were designed with both procedures under the design load combination. The three-layer model (EN 1992-2, Annex LL) is a local, lower-bound check based on a single linear elastic stress field: the reinforcement resists tension only, and the most highly stressed element governs the thickness of the shell. The multilayer model integrates nonlinear concrete and steel layers through the thickness, allowing the reinforcement to act in both tension and compression, and determines thickness from the global capacity, with no reinforcement yielding under the design load, thereby accounting for load redistribution.

For $h = 180$ mm, the multilayer model allows 2.3, 1.3, and 3.7 times the live load permitted by the three-layer check for PH-04, PH-06, and PH-08, respectively. The three-layer thickness increases more rapidly with w_L , largely because of the elastic bending peaks at the supports, which add up to 70 mm in PH-06 (Fig. 44) and are redistributed by cracking in the nonlinear analysis.

At $w_L = 6 \text{ kN/m}^2$, the multilayer design reduces the concrete volume and self-weight by 33%, 26%, and 40% (Figs. 45 and 46), and by 24%, 7%, and 38% when the support elements are excluded; the PH-08 value is a lower bound because its thickness remains at the 150 mm minimum.

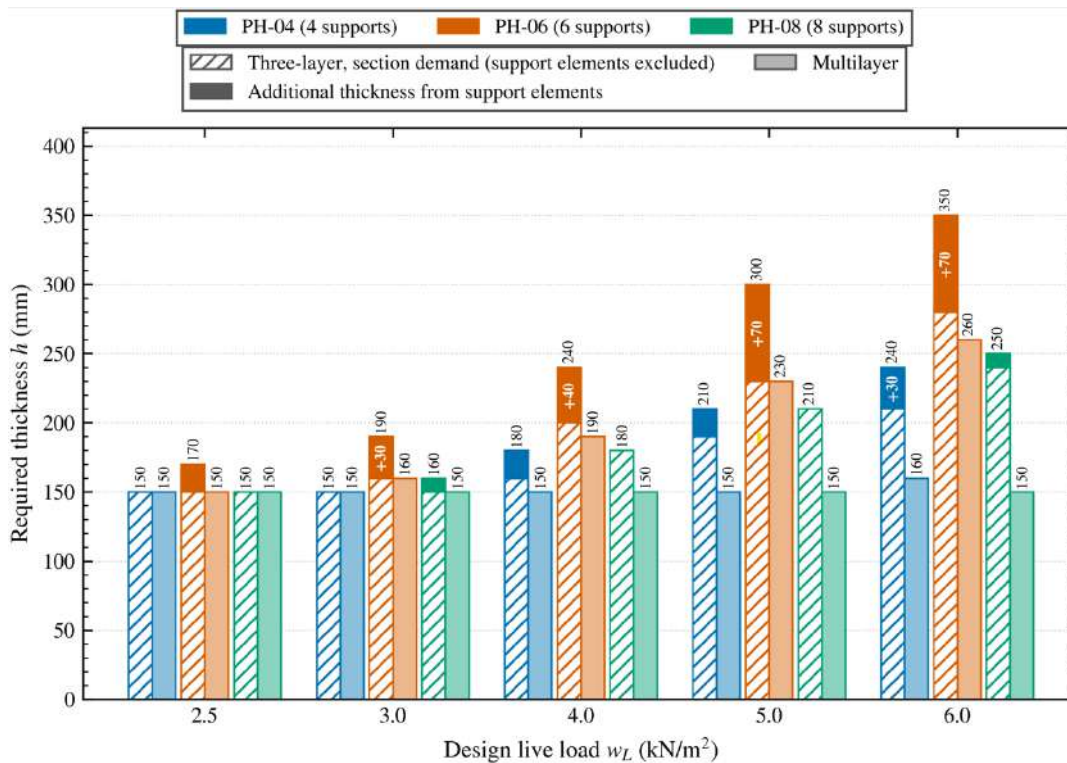


Fig. 44. Required thickness versus design live load, hatched three-layer without supports, light multilayer and solid additional thickness for support elements

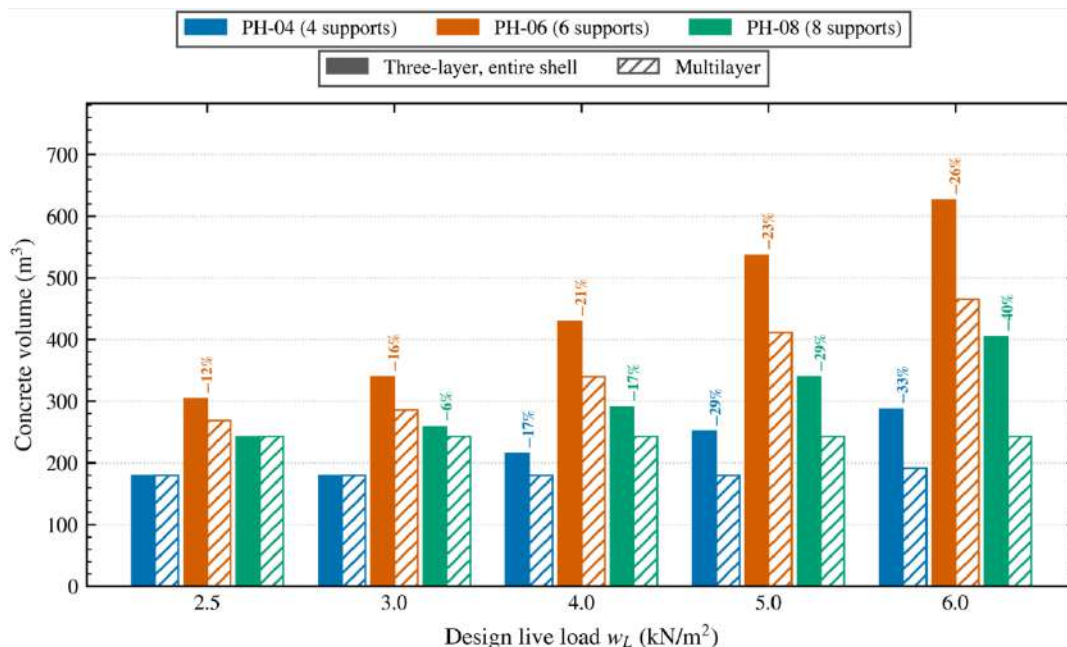


Fig. 45. Concrete volume versus design live load, solid: three-layer; hatched: multilayer

Fig. 45. compares the concrete volume required by the two design procedures for the three shells over the range of design live loads considered. The corresponding self-weight, which governs both the permanent gravity load and the seismic mass of the shell, is shown in Fig. 46.

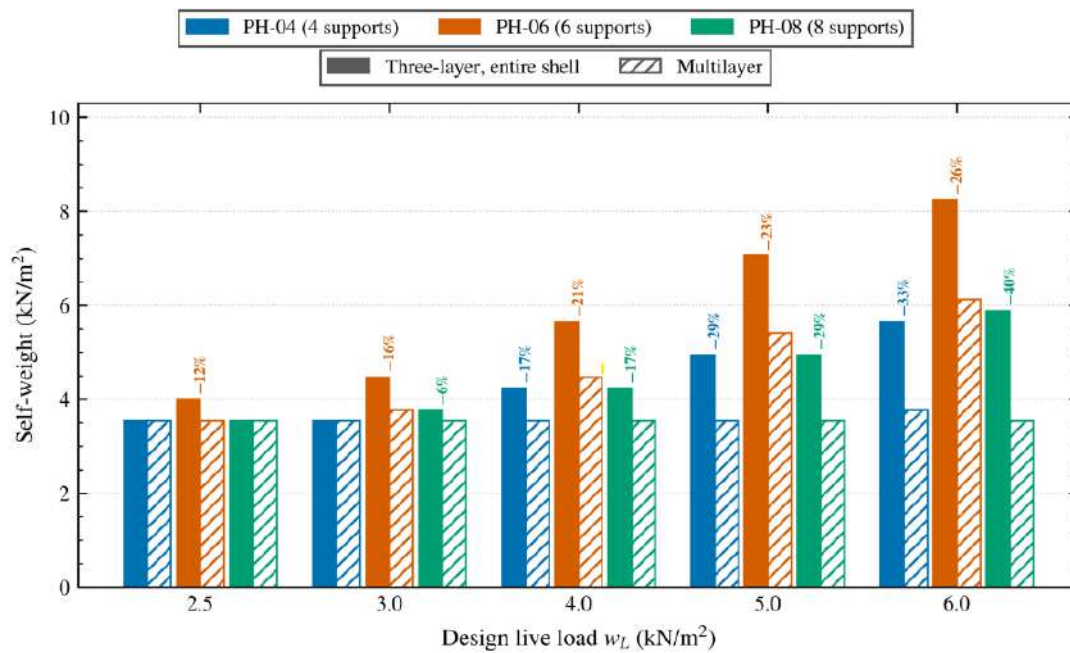


Fig. 46. Concrete self-weight versus design live load, solid: three-layer; hatched: multilayer

6. Limitations and Future Research

The methodology presented provides a robust assessment of the seismic performance and nonlinear behavior of concrete shell roofs, but its analytical scope must be delimited. Declaring these computational and phenomenological boundaries is necessary both for the correct interpretation of the results and for establishing the framework within which the findings apply. The main limitations are the following.

- Concrete crushing. The constitutive model adopted captures tensile cracking and global buckling instability but does not explicitly simulate concrete crushing. Localized concentrations of compressive stress, which may precipitate brittle failure in regions of high curvature and in the vicinity of the supports, are therefore not evaluated. The available ductility is consequently overestimated, and the overestimation is not uniform across the models: it is most significant in the eight-support shell, which is the stiffest configuration and the one that develops the least ductility.
- Exclusion of soil–structure interaction. The model assumes an infinitely rigid base [43]. Neglecting foundation flexibility and the kinematic and inertial interaction between soil and structure prevents any assessment of how radiation damping and the lengthening of the fundamental period would modify the stress distribution in the shell. This is particularly relevant for the Valley of Mexico, where the soft lacustrine deposits amplify long-period demand, whereas the fundamental periods obtained here lie in the short-period range.
- Time-dependent effects. Creep and shrinkage were not included in the model [44]. In thin-walled shells these phenomena induce progressive long-term deflections that redistribute the internal stress field and reduce the effective stiffness and may therefore aggravate the sensitivity of the structure to instability. The interaction between time-dependent behavior and seismic demand cannot be assessed from the analysis reported here.

Finally, the treatment of initial defects was restricted to purely geometric imperfections. In construction practice the limit performance of a shell is also governed by material deviations: the spatial variability of the cast thickness and the stochastic reduction of the initial modulus of elasticity are both relevant to a realistic assessment of risk.

Addressing these limitations defines the authors' research agenda. The development of a comprehensive probabilistic framework for shell structures, incorporating uncertainty in thickness and material stiffness, is the objective of the next phase of this work. In parallel,

subsequent studies will incorporate recorded ground motions, explicit modeling of the supporting half-space, and the assessment of the long-term rheological deterioration of the concrete.

7. Conclusions

This study showed that the geometry obtained with the force density method (FDM) constitutes the first line of seismic design for thin reinforced concrete shells. Three anticlastic shells with four, six, and eight supports were evaluated within a hybrid framework that combined the actions and minimum reinforcement of NTC-2023, the sectional design of Eurocode 2 with the Wood–Armer correction, and nonlinear time-history verification using a multilayer shell formulation.

Under synthetic ground motions for Mexico City, the three shells exhibited a controlled nonlinear response, dominated by compression and far from collapse. The response showed a clear dependence on the number of supports: from PH-04 to PH-08, the fundamental period decreased by 55.9%, the gravity deflection by 86%, and the residual displacement by 98%, so that stiffness increased and accumulated damage decreased monotonically. Only the peak seismic displacement departed from this trend, because PH-06 remained essentially elastic and developed the full seismic demand.

The geometric efficiency was reflected in the reinforcement: the minimum steel ($337 \text{ mm}^2/\text{m}$ per layer and direction) governed 74.0%, 49.7%, and 59.1% of the surface of PH-04, PH-06, and PH-08, respectively, indicating a predominantly membrane stress state that was satisfied by a uniform mesh of two layers of #4 bars at 125 mm, with local reinforcement along the meridian bands. In contrast, geometric imperfections affine to the first buckling mode reduced the ultimate capacity by 45.2%, 36.2%, and 22.1% and lengthened the fundamental periods by 13.6% to 19.2%. This sensitivity decreased with the number of supports, as the shorter spans confined the critical buckling mode, and remained below that reported for cylindrical and spherical shells.

This loss of capacity makes the margin recovered by a more rigorous analysis decisive. The three-layer model of Eurocode 2 proved conservative and uneconomical, although not unsafe: for a thickness of 180 mm, the multilayer model admitted 2.3, 1.3, and 3.7 times the live load allowed by the three-layer check, and at 6 kN/m^2 it reduced the concrete volume and self-weight by 33%, 26%, and 40%. The excess arises because the linear sectional criterion admits neither redistribution through cracking nor the actual internal lever arm of the layered section, which reduces the required reinforcement area by 10%. The multilayer representation is therefore a requirement, not a refinement, for the design of slender shells in seismic regions. The computed cracking zones coincided in location with the damage documented in Los Manantiales after the 2017 earthquake [41,42]: supports, valley lines, and free edges. Although this is a coincidence rather than a validation, it has scientific value: it indicates that, in shells whose efficiency derives from their geometry, the location of damage is governed by form rather than by the particular case, since the membrane state degrades in the same critical regions. This result is the starting point for a future study, because the models do not reproduce the actual geometry, thickness, or materials; its validation requires surveying the as-built structure, calibrating the model with ambient vibration measurements, and analyzing it with the 2017 records on lacustrine soil.

The main contribution is an integrated and reproducible framework that combines form-finding, affine imperfections, hybrid code-based design, and nonlinear multilayer verification, and that is transferable to other seismic regions by replacing the local codes and ground-motion records. Future work includes the multilayer comparison under seismic combinations and smaller thicknesses; a probabilistic framework based on FORM/SORM and Monte Carlo simulation; imperfections with multiple amplitudes, measured shapes, and variable thickness; and the inclusion of recorded ground motions, soil–structure interaction, creep, and shrinkage. Together, these developments will contribute to the safe and efficient reintroduction of thin concrete shells in seismic regions worldwide.

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Fresh, physical, and mechanical performance of concrete incorporating pre-soaked recycled coarse aggregate derived from construction and demolition waste: Experimental evaluation and linear regression analysis

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Abstract

Construction and demolition waste can partially substitute natural aggregate when processed as recycled coarse aggregate (RCA), although adhered mortar is associated with higher porosity and may reduce concrete performance. This study evaluates RCA produced from demolition waste collected in Tikrit, Iraq, at 0%, 20%, 40%, 60%, and 80% replacement of natural coarse aggregate by mass. All RCA mixtures used the same 48 h water-conditioning protocol to obtain a saturated surface-dry state. At a constant w/c ratio of 0.48, slump, 28-day water absorption and density, and compressive, flexural, and splitting tensile strengths at 7 and 28 days were measured. Slump decreased from 83 to 53 mm, water absorption increased from $3.97 \pm 0.45\%$ to $7.53 \pm 0.49\%$, and density decreased from 2.394 ± 0.070 to 2.268 ± 0.020 g/cm³. At 28 days, compressive strength decreased from 31.13 ± 0.35 to 22.80 ± 0.40 MPa; RCA20 and RCA40 retained 96.5% and 91.3% of control strength, respectively. At 7 days, compressive strength decreased from 22.40 ± 0.56 to 13.93 ± 2.05 MPa as RCA replacement increased from 0% to 80%, with the 7-day/28-day strength ratio declining from 72.0% to 61.1%. Inferential analysis confirmed significant replacement-level effects for water absorption and several strength responses, while regressions incorporating within-mixture variability remained statistically significant. Within the investigated mixtures, 20–40% RCA provided the most favorable measured balance between reducing natural coarse-aggregate use and retaining 91–97% of the control 28-day compressive strength; however, structural and durability applications require additional verification.

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1. Introduction

Urban development increases both the demand for natural aggregate and the quantity of construction and demolition waste (CDW). Recycling demolished concrete as aggregate can divert waste from disposal and conserve primary resources, thereby supporting circular use of construction materials [1,2].

Recycled concrete aggregate differs from natural aggregate because particles commonly retain old cement mortar. The adhered mortar produces a rougher surface, lower density, and higher porosity and water absorption, which can alter workability and mechanical response [3–5]. Recycled aggregate concrete also contains multiple interfacial regions: the original interface between old mortar and the parent natural aggregate and the new interface formed between the recycled particle and new cement paste. These dual interfacial transition zones (ITZs), together with pre-existing microcracks in the attached mortar, are frequently identified in the literature as potential

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weak paths under loading; however, direct attribution of the present results to these mechanisms requires microstructural evidence. Recent reviews consistently show that RCA performance depends on source-concrete quality, adhered-mortar content, processing, replacement level, conditioning, and curing [6,7]. Improvement strategies reported in the literature include water pre-saturation, mechanical or thermal removal of old mortar, acid treatment, pozzolanic or cementitious slurry coating, polymer or mineral surface treatments, and accelerated carbonation [7-10]. These methods act differently: pre-saturation primarily controls short-term moisture exchange during mixing, whereas coating and carbonation treatments can densify accessible pores or modify the new/old ITZ. Comparative studies therefore caution against treating a single conditioning method as universally beneficial, because effectiveness depends on RCA quality and treatment conditions [8-10].

In the present work, 48 h water pre-soaking was used only as a fixed aggregate-conditioning protocol to obtain a saturated surface-dry (SSD) condition before batching; it was not an independent treatment variable because no untreated-RCA control was included. Region-specific evidence remains relevant because demolition sources and aggregate quality vary. Iraqi studies have demonstrated the feasibility of using CDW-derived aggregates in concrete and masonry [13,14], but published characterization specific to Tikrit remains limited. This study therefore reports measured physical and chemical indices of locally collected RCA together with concrete performance across a broad partial-replacement range.

It was hypothesized that moderate RCA replacement would preserve most of the control concrete strength while higher replacement levels would increasingly reflect the lower density, higher absorption, and weaker mechanical indices of the recycled particles. The objectives were to: (i) quantify changes in fresh, physical, and 7/28-day mechanical properties; (ii) evaluate whether replacement level produces statistically significant differences using one-way ANOVA and Tukey HSD; (iii) reassess linear trends while incorporating within-mixture variability; and (iv) identify a practical replacement range that reduces natural-coarse-aggregate demand without overstating structural applicability. The contribution is therefore a combined engineering and statistical assessment of locally sourced RCA under a controlled conditioning and curing regime, together with explicit definition of the study's experimental and methodological limitations.

2. Materials and Methods

2.1 Materials

Ordinary Portland cement (Type I) complying with Iraqi Specification No. 5 (1984) was used. The reported cement properties were a specific gravity of 3.15, Blaine fineness of 3280 cm²/g, initial setting time of 110 min, final setting time of 255 min, and 7-day compressive strength of 33.8 MPa. The chemical composition of the Type I Portland cement used in the concrete mixtures is summarized in Table 1. Natural river sand and crushed natural coarse aggregate complied with Iraqi Specification No. 45 (1984). The natural coarse aggregate had a nominal maximum size of 25 mm. For the natural fine aggregate, the measured bulk, SSD, and apparent specific gravities were 2.63, 2.67, and 2.73, respectively, and water absorption was 1.52%. Potable tap water was used for mixing and curing.

RCA was produced from concrete debris collected from demolished buildings in Tikrit. Reinforcing steel, wood, plaster, soil, and other visible contaminants were removed manually. The concrete debris was crushed with a laboratory jaw crusher, washed, and sieved to the required coarse-aggregate size range. The parent-concrete strength and quantitative adhered-mortar content were not recorded. Before batching, RCA was immersed in clean water for 48 h, then drained and surface-dried until no visible free water remained. The 3.21% absorption value reported in Table 2 is the aggregate absorption determined by the ASTM C127 test procedure and is distinct from the separate 48 h conditioning step. Initial RCA moisture content immediately before pre-soaking was not preserved in the archived records. Figure 1 documents the principal preparation, casting, and testing stages.

Table 1. Chemical composition of type I Portland cement

Oxide composition	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	Na ₂ O + K ₂ O	LOI	Insoluble material
content (%)	20.32	4.06	3.67	60.12	2.04	2.03	0.24	2.95	1.18



Figure 1. Experimental program: (a) crushing concrete debris; (b) processed RCA; (c) slump measurement; (d) cast specimens; (e) compression testing; and (f) flexural testing.
Photographs supplied by the author

Table 2. Physical properties of natural fine, natural *coarse*, and recycled coarse aggregates used in this study

Property	Natural Fine Aggregate	Natural Coarse Aggregate	Recycled Coarse Aggregate	Relevant Specification
Bulk specific gravity	2.63	2.55	2.18	ASTM C128/ASTM C127
SSD specific gravity	2.67	2.56	2.25	ASTM C128/ASTM C127
Apparent specific gravity	2.73	2.58	2.34	ASTM C128/ASTM C127
Water absorption (%)	1.52	0.39	3.21	ASTM C128/ASTM C127
Aggregate impact value (%)	—	13.66	16.58	BS 812
Aggregate crushing value (%)	—	14.53	16.2	BS 812
Los Angeles abrasion (%)	—	17.5	20.85	ASTM C131

Note: ASTM C128 was used for the natural fine aggregate, while ASTM C127 was used for the natural and recycled coarse aggregates.

The physical properties of the natural fine aggregate, natural coarse aggregate, and recycled coarse aggregate are presented in Table 2. The natural fine aggregate data complete the basic material characterization, while the RCA exhibited lower specific gravity and higher water absorption than the natural coarse aggregate. The higher impact, crushing, and Los Angeles abrasion indices of RCA indicate comparatively lower resistance to the corresponding aggregate tests. These aggregate-scale indicators are used here as descriptive material characteristics rather than as direct proof of any particular microstructural failure mechanism. The chemical composition of the recycled concrete aggregate (RCA) used in this study is presented in Table 3.

Table 3. Chemical composition of recycled concrete aggregate (RCA)

Chemical oxide	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	Na ₂ O + K ₂ O	LOI
RCA (%)	60.78	4.55	3.15	16.34	1.25	0.95	0.24	11.03

2.2 Mixture Proportions and Variables

Five mixtures were prepared by replacing natural coarse aggregate with RCA at 0%, 20%, 40%, 60%, and 80% by mass. Cement, fine aggregate, nominal total coarse-aggregate mass, mixing water, and the water-to-cement ratio were held constant (Table 4). Because RCA had a lower SSD specific gravity (2.25) than natural coarse aggregate (2.56), equal-mass substitution increased the absolute coarse-aggregate volume as RCA content increased; consequently, the mortar-to-aggregate volume ratio was not strictly constant. This mass-based approach reflects the original experimental design and is now explicitly recognized when interpreting the results. The program was designed to examine progressive partial replacement, with 80% selected as the upper bound; 100% replacement was outside the predefined experimental matrix. All RCA mixtures used the same 48 h water-conditioning protocol, so the study cannot isolate the benefit of pre-soaking relative to untreated RCA. The reported mixture record contains no chemical admixture; therefore, no superplasticizer is included in Table 4. The nominal mixing-water content was 144 kg/m³ (w/c = 0.48), and the 48-h water-conditioning step was intended to limit immediate water exchange, although effective free-water changes during mixing were not independently measured.

Table 4. Concrete mixture proportions (kg/m³)

Mix	RCA (%)	Cement	Fine agg.	Natural coarse agg.	RCA	Water	w/c
RCA0	0	300	890	1150	0	144	0.48
RCA20	20	300	890	920	230	144	0.48
RCA40	40	300	890	690	460	144	0.48
RCA60	60	300	890	460	690	144	0.48
RCA80	80	300	890	230	920	144	0.48

2.3 Specimen Preparation and Testing

Dry constituents were blended in a laboratory rotary mixer for approximately 2–3 min before water was added gradually. After water addition, mixing continued for approximately 3–4 min. Fresh concrete was then placed in steel molds in three layers and compacted on a vibrating table. Specimens were demolded after 24 h and water-cured under laboratory conditions at approximately 22.5°C until testing. For every replacement level, three specimens were tested for each applicable property and age. Six cubes, six prisms, and six cylinders were produced per mixture, giving 90 hardened specimens in total. In addition, three separate 150-mm cubes were prepared for each mixture at 28 days for water absorption and density measurements in accordance with ASTM C642. Thus, the total number of hardened specimens prepared for the experimental program was 105. A summary of specimen types, dimensions, ages, and test

standards is provided in Table 5. For the ASTM C293 flexural test on 100 × 100 × 400 mm prisms, a center-point loading configuration was used with a 300 mm clear span and a single load applied at the mid-span of the prism specimens. Splitting tensile strength was determined on 100 × 200 mm cylinders (L/D = 2) using the ASTM C496 expression based on the actual specimen length and diameter; no separate size-effect coefficient was applied.

Table 5. Specimens and test methods reported in the experimental program

Property	Specimen	Dimensions	Age (days)	Reported method
Slump	Fresh concrete	—	—	ASTM C143
Water absorption	Cube	150 × 150 × 150 mm	28	ASTM C642
Density	Cube	150 × 150 × 150 mm	28	Gravimetric measurement
Compressive strength	Cube	150 × 150 × 150 mm	7, 28	BS EN 12390-3
Flexural strength	Prism	100 × 100 × 400 mm	7, 28	ASTM C293
Splitting tensile strength	Cylinder	100 × 200 mm	7, 28	ASTM C496

2.4 Statistical Treatment

For each hardened property, the arithmetic mean, sample standard deviation (SD), and coefficient of variation ($CV = 100SD/\text{mean}$) were calculated from three replicate measurements. One-way ANOVA was performed for the five replacement levels using three replicate measurements per mixture; when the omnibus test was significant ($p < 0.05$), Tukey HSD comparisons were computed using the pooled within-group mean square. Slump was reported as one fresh-concrete measurement per mixture, so no SD or inferential group test is claimed for slump. Ordinary least-squares regression was also recalculated using all 15 individual replicate observations for each hardened property: the fitted slope and intercept are identical to those obtained from equal-size group-mean regression, but residual variance and R^2 incorporate the reported within-group variation. Slope p-values and 95% confidence intervals are reported. These inferential results should still be interpreted cautiously because $n = 3$ per mixture provides limited power for assessing distributional assumptions.

3. Results

3.1 Fresh and Physical Properties

Slump decreased monotonically from 83 mm for RCA0 to 53 mm for RCA80, a 36.1% reduction; because slump was recorded once per mixture, this trend is descriptive only. Water absorption increased from $3.97 \pm 0.45\%$ to $7.53 \pm 0.49\%$, corresponding to an 89.7% increase relative to the control. One-way ANOVA indicated a significant replacement-level effect on water absorption ($F(4,10) = 30.92$, $p < 0.001$), with several Tukey HSD pairwise differences becoming significant from 40% RCA upward (Table 8). Density decreased by 5.3%, from 2.394 ± 0.070 to 2.268 ± 0.020 g/cm³; the five-group ANOVA was not significant at $\alpha = 0.05$ ($F(4,10) = 2.66$, $p = 0.096$), although the prespecified linear trend remained significant. These fresh and physical trends are summarized in Figure 2 and Table 6, while regression metrics are reported in Table 7.

3.2 Mechanical Properties

All mixtures gained strength between 7 and 28 days. At 28 days, compressive strength decreased from 31.13 ± 0.35 MPa for RCA0 to 22.80 ± 0.40 MPa for RCA80, a reduction of 26.8%; RCA20 and RCA40 retained 96.5% and 91.3% of the control value. One-way ANOVA confirmed a significant replacement-level effect on 28-day compressive strength ($F(4,10) = 30.86$, $p < 0.001$), and Tukey HSD showed significant differences mainly in comparisons involving the 60% and 80% RCA mixtures (Table 8). The 7-day/28-day compressive-strength ratio declined progressively from

72.0% (RCA0) to 69.3%, 66.5%, 62.3%, and 61.1% for RCA20, RCA40, RCA60, and RCA80, respectively. This pattern indicates slower early-to-later strength development as RCA content increased; within the limits of the dataset, this may reflect greater heterogeneity and moisture redistribution associated with adhered mortar rather than a directly measured change in hydration kinetics. Figure 3 illustrates the compressive-strength development at 7 and 28 days, and the complete mechanical results are listed in Table 6.

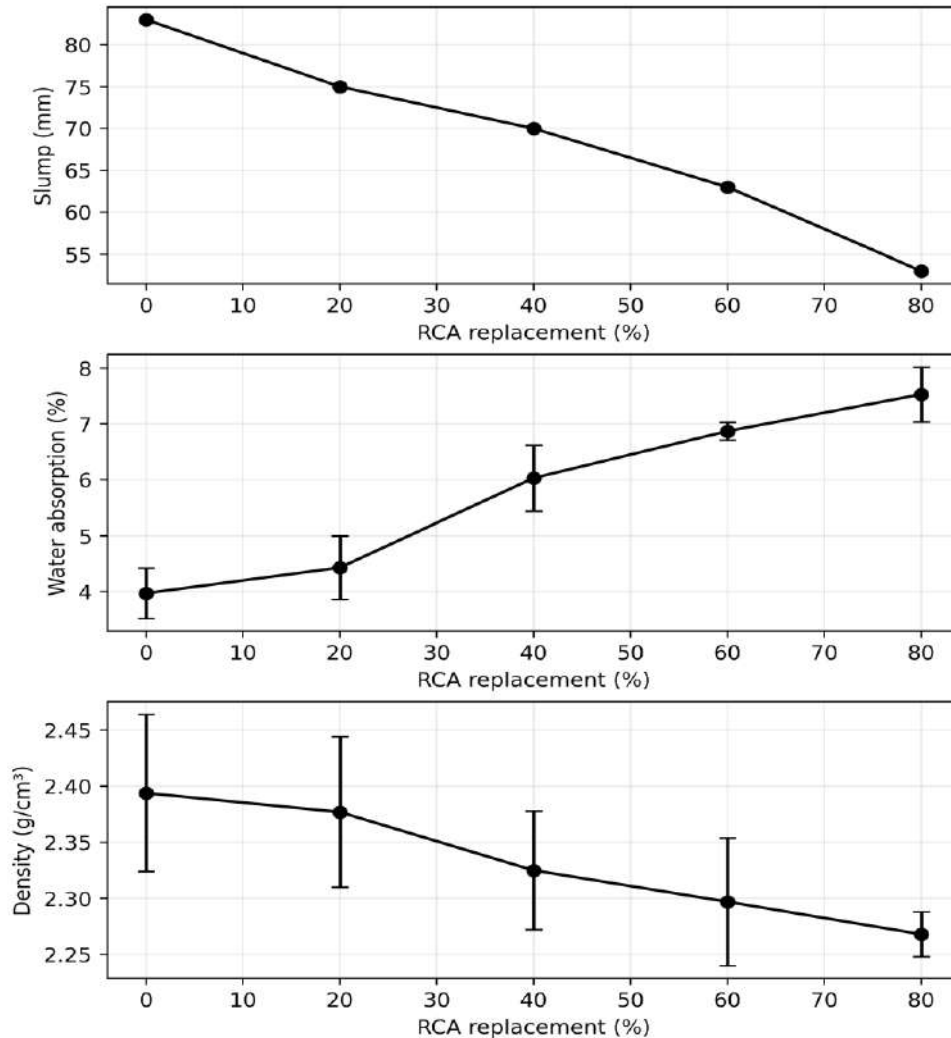


Figure 2. Effect of RCA replacement on slump, 28-day water absorption, and 28-day density. Error bars show ± 1 sample SD where replicate measurements were available ($n = 3$)

At 28 days, flexural strength decreased from 4.60 ± 0.26 to 3.45 ± 0.22 MPa (25.0%), while splitting tensile strength decreased from 3.30 ± 0.20 to 2.20 ± 0.10 MPa (33.3%). The five-group ANOVA was significant for 28-day flexural strength ($F(4,10) = 5.25$, $p = 0.015$) and 28-day splitting tensile strength ($F(4,10) = 26.72$, $p < 0.001$), but not for the corresponding 7-day flexural ($p = 0.066$) or splitting ($p = 0.318$) datasets. Figures 4 and 5 show the flexural and splitting-tensile trends. The 28-day splitting-to-compressive-strength ratio declined from 0.106 for RCA0 to 0.096 for RCA80, supporting greater proportional sensitivity of tensile behavior at high RCA contents. No outlier-removal procedure was applied to the reported data. The high 7-day CVs for RCA0 splitting strength (36.8%) and RCA60 flexural strength (31.7%) are retained transparently and interpreted as evidence of limited early-age repeatability rather than selectively excluding observations. Table 7 reports regressions recalculated with within-mixture variance, and Table 8 summarizes ANOVA/Tukey results. Representative post-test fracture patterns are provided in Figure 6. The visible cracks and fractured surfaces confirm macroscopic failure across the tested specimen types; however, they do not by themselves distinguish cracking through old mortar, new paste, RCA particles, or the individual interfacial transition zones.

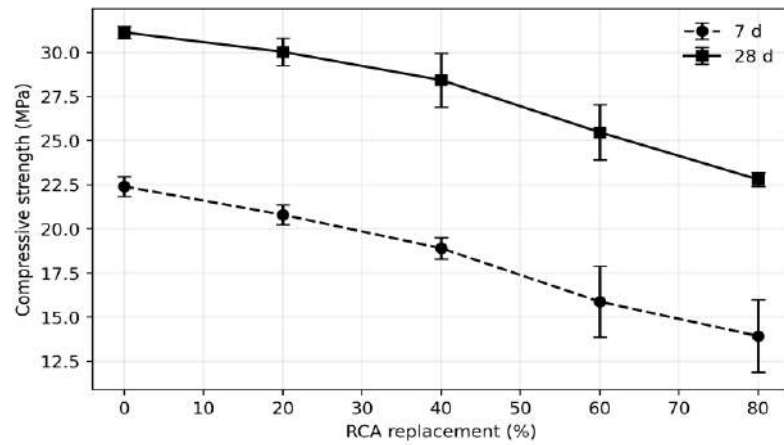


Figure 3. Compressive strength at 7 and 28 days. Error bars show ± 1 sample SD ($n = 3$)

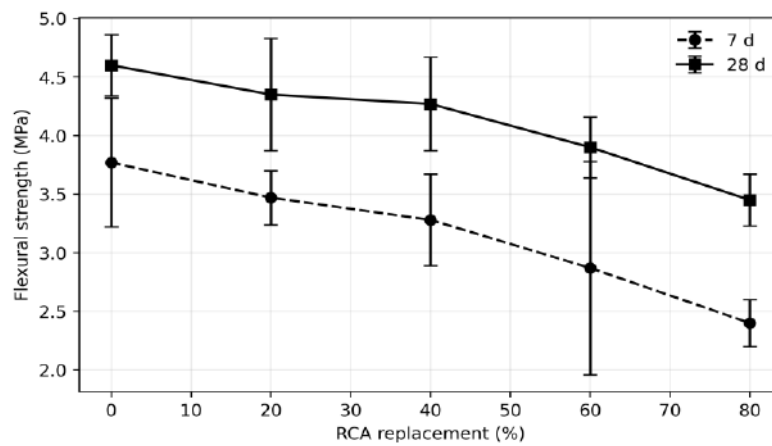


Figure 4. Flexural strength at 7 and 28 days. Error bars show ± 1 sample SD ($n = 3$)

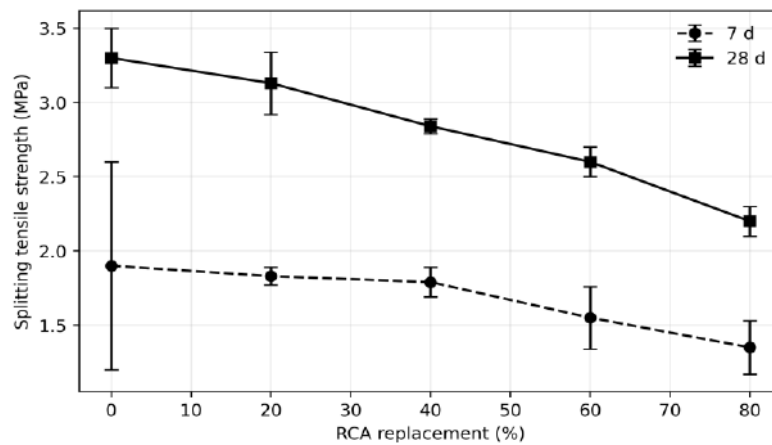


Figure 5. Splitting tensile strength at 7 and 28 days. Error bars show ± 1 sample SD ($n = 3$)



(a) Flexural failure of prisms (b) Representative fractured cubes (c) Splitting-tensile cracking, RCA 0-80%

Figure 6. Representative post-test failure patterns: (a) flexural cracking of prism specimens, (b) fractured cube specimens, and (c) splitting-tensile cracking of cylinders containing 0-80% RCA. The photographs document macroscopic failure only and are not used as direct microstructural evidence of ITZ behavior

Table 6. Experimental results (mean $\pm$ sample SD; $n = 3$ for hardened properties; slump = one measurement per mixture)

Mix	Slump (mm)	Abs. 28 d (%)	Density 28 d (g/cm ³)	Comp. 7 d (MPa)	Comp. 28 d (MPa)	Flex. 7 d (MPa)	Flex. 28 d (MPa)	Split. 7 d (MPa)	Split. 28 d (MPa)
RCA0	83	3.97 $\pm$ 0.45	2.394 $\pm$ 0.070	22.40 $\pm$ 0.56	31.13 $\pm$ 0.35	3.77 $\pm$ 0.55	4.60 $\pm$ 0.26	1.90 $\pm$ 0.70	3.30 $\pm$ 0.20
RCA20	75	4.43 $\pm$ 0.57	2.377 $\pm$ 0.067	20.80 $\pm$ 0.56	30.03 $\pm$ 0.78	3.47 $\pm$ 0.23	4.35 $\pm$ 0.48	1.83 $\pm$ 0.06	3.13 $\pm$ 0.21
RCA40	70	6.03 $\pm$ 0.59	2.325 $\pm$ 0.053	18.90 $\pm$ 0.61	28.43 $\pm$ 1.53	3.28 $\pm$ 0.39	4.27 $\pm$ 0.40	1.79 $\pm$ 0.10	2.84 $\pm$ 0.05
RCA60	63	6.87 $\pm$ 0.16	2.297 $\pm$ 0.057	15.87 $\pm$ 2.01	25.47 $\pm$ 1.56	2.87 $\pm$ 0.91	3.90 $\pm$ 0.26	1.55 $\pm$ 0.21	2.60 $\pm$ 0.10
RCA80	53	7.53 $\pm$ 0.49	2.268 $\pm$ 0.020	13.93 $\pm$ 2.05	22.80 $\pm$ 0.40	2.40 $\pm$ 0.20	3.45 $\pm$ 0.22	1.35 $\pm$ 0.18	2.20 $\pm$ 0.10

Table 7. Linear regression of hardened-property responses on RCA replacement percentage, incorporating within-mixture variance ($N = 15$)

Response	Intercept	Slope per 1% RCA	R ²	Slope p-value	95% CI low	95% CI high
Water absorption, 28 d (%)	3.8540	0.04780	0.899	<0.001	0.03818	0.05742
Density, 28 d (g/cm ³)	2.3986	-0.00166	0.506	0.003	-0.00264	-0.00068
Compressive strength, 7 d (MPa)	22.7540	-0.10935	0.877	<0.001	-0.13391	-0.08479
Compressive strength, 28 d (MPa)	31.8160	-0.10610	0.893	<0.001	-0.12807	-0.08413
Flexural strength, 7 d (MPa)	3.8260	-0.01670	0.541	0.002	-0.02591	-0.00749
Flexural strength, 28 d (MPa)	4.6640	-0.01375	0.638	<0.001	-0.01996	-0.00754
Splitting strength, 7 d (MPa)	1.9600	-0.00690	0.320	0.028	-0.01293	-0.00087
Splitting strength, 28 d (MPa)	3.3600	-0.01365	0.897	<0.001	-0.01642	-0.01088

Table 8. One-way ANOVA and significant Tukey HSD pairwise comparisons for hardened properties (n = 3 per replacement level)

Response	F(4,10)	ANOVA p	Significant Tukey pairs (p < 0.05)
Water absorption, 28 d (%)	30.92	<0.001	0-40%, 0-60%, 0-80%, 20-40%, 20-60%, 20-80%, 40-80%
Density, 28 d (g/cm ³)	2.66	0.096	None
Compressive strength, 7 d (MPa)	19.63	<0.001	0-60%, 0-80%, 20-60%, 20-80%, 40-80%
Compressive strength, 28 d (MPa)	30.86	<0.001	0-60%, 0-80%, 20-60%, 20-80%, 40-60%, 40-80%
Flexural strength, 7 d (MPa)	3.12	0.066	None
Flexural strength, 28 d (MPa)	5.25	0.015	0-80%
Splitting strength, 7 d (MPa)	1.35	0.318	None
Splitting strength, 28 d (MPa)	26.72	<0.001	0-40%, 0-60%, 0-80%, 20-60%, 20-80%, 40-80%, 60-80%

4. Discussion

The continuous slump loss with increasing RCA content is consistent with the rough texture, angularity, and attached mortar of recycled particles reported in the literature [1,2,7]. Because the aggregates were conditioned to SSD, immediate absorption of mixing water was reduced as a confounding factor; nevertheless, SSD conditioning does not remove surface friction or particle-shape effects. The present results therefore do not demonstrate that pre-soaking improved workability relative to dry RCA. At higher replacement levels, a slump of 53 mm may also be restrictive for some placement or pumping conditions. Future mixture optimization should evaluate high-range water-reducing admixtures or rheology-based adjustments while maintaining the target effective w/c ratio. The macroscopic fracture photographs in Figure 6 are compatible with heterogeneous crack paths in the RCA-containing specimens, but no SEM, XRD, MIP, or other microstructural method was performed; accordingly, dual-ITZ weakness and crack-path selection are discussed only as literature-supported possible mechanisms rather than directly observed causes.

The concurrent rise in absorption and decrease in density are consistent with a progressively more porous concrete system as RCA content increased, in agreement with prior RCA studies [5,15]. However, water absorption should not be interpreted as a direct cause of strength loss. Instead, the higher RCA crushing, impact, and abrasion indices in Table 2, attached old mortar, and associated particle heterogeneity are parallel material characteristics that can contribute to both higher absorption and lower strength. The RCA chemical composition in Table 3 provides source characterization: SiO₂ is associated mainly with mineral aggregate, whereas CaO and the measured loss on ignition are compatible with cementitious/carbonated constituents and mineral phases. Without XRD, TGA, or microstructural imaging, these chemical results cannot identify the dominant strength-loss mechanism.

The limited compressive-strength loss at 20-40% RCA and the larger reductions at 60-80% agree with the general replacement-level trend reported for recycled aggregate concrete [16]. Tensile-related properties were proportionally more sensitive at high replacement. A plausible explanation from the literature is that the old mortar-natural aggregate interface and the newly formed paste-RCA interface create dual ITZs where local heterogeneity may facilitate crack initiation [3,8]. In this study, however, SEM, XRD, MIP, and systematic post-failure fracture-plane imaging were not performed; ITZ weakening and microcrack propagation are therefore presented only as literature-supported possible mechanisms, not as directly observed causes.

The high early-age CV values in two groups further indicate that local heterogeneity or casting/testing variability may have influenced the 7-day response, and no observations were removed as outliers. Among the tested mixtures, 20-40% RCA provided the most favorable balance between reducing natural coarse-aggregate consumption and retaining 91-97% of control 28-day compressive strength. This is an engineering performance observation, not a blanket recommendation for structural members. Modulus of elasticity, reinforcement-concrete bond, shrinkage, creep, chloride transport, carbonation resistance, and other durability indicators were not measured. Accordingly, structural use must be verified against the applicable design and exposure requirements using project-specific testing. In the Tikrit/Iraq context, the results nevertheless support further development of controlled CDW sorting and RCA production for applications in which the measured strength and exposure requirements can be demonstrated. The significant linear trends in Table 7 may serve as preliminary within-range prediction tools for the investigated material source, but they should not be extrapolated to other RCA sources or beyond 0-80% replacement. Natural fine-aggregate specific gravity and absorption were available and are reported in Table 2; however, unit weight and fineness modulus were not included in the supplied quantitative dataset and therefore were not invented or inferred.

4.1 Limitations and Future Work

The study used one RCA source, one water-to-cement ratio, and three replicate specimens per hardened-property group. Basic RCA absorption and specific-gravity indices were reported in Table 2, but parent-concrete strength, quantitative adhered-mortar content, unit weight and fineness modulus of the fine aggregate, full aggregate gradation curves, initial RCA moisture content, records of effective free-water changes during mixing were not preserved in the archived dataset. The absence of an untreated-RCA series prevents direct quantification of any benefit from 48 h pre-soaking. Slump was measured once per mixture, limiting uncertainty analysis for fresh workability. Systematic post-failure photographs and microstructural analyses (SEM/XRD/MIP) were not available, so proposed ITZ and crack-path mechanisms remain literature-based interpretations. Long-term durability indicators, modulus of elasticity, bond strength, shrinkage, and creep were not measured. Future work should use larger replication, explicitly separated physical-test specimens, 100% RCA as a boundary condition, untreated and alternative-treatment controls, and treatments such as acid washing, cementitious/pozzolanic slurry coating, or accelerated carbonation [8-10]. These extensions should be combined with durability and microstructural characterization before broad structural recommendations are made.

5. Conclusions

This study demonstrates that progressively replacing natural coarse aggregate with locally sourced recycled coarse aggregate (RCA) changes fresh, physical, and mechanical behavior in a systematic but property-dependent manner. The decline in slump with increasing RCA content confirms that bringing RCA to an SSD condition does not eliminate the workability consequences associated with particle roughness, angularity, and attached mortar. At the same time, the increase in water absorption and reduction in density show that the recycled aggregate introduces a more porous and heterogeneous constituent into the concrete. These physical changes occurred alongside reductions in compressive, flexural, and splitting tensile strengths, with tensile-related behavior showing greater proportional sensitivity at high replacement levels. The decreasing 7-day/28-day compressive-strength ratio from 72.0% in the control to 61.1% at 80% RCA further indicates that high RCA contents were associated with slower early-to-later strength development under the investigated curing regime. The statistical analysis strengthens these observations while also qualifying them. One-way ANOVA identified significant replacement-level effects for water absorption, 7- and 28-day compressive strength, 28-day flexural strength, and 28-day splitting tensile strength, whereas density and some 7-day tensile-related responses did not reach the 0.05 level with only three replicates per group. The regression models should be viewed as source-specific descriptive or preliminary predictive relationships rather than universal design equations.

Within the tested range, 20-40% RCA provided the most favorable measured compromise: these mixtures replaced a meaningful fraction of natural coarse aggregate while retaining approximately 91-97% of the control 28-day compressive strength. This finding has practical relevance for construction and demolition waste management in Tikrit and similar regional contexts, where controlled recovery of concrete debris could reduce landfill disposal and demand for virgin aggregate. However, the study does not establish that 20-40% RCA is automatically suitable for all structural exposure classes. Modulus of elasticity, bond, creep, shrinkage, chloride transport, carbonation resistance, and other durability indicators were not evaluated, and no untreated-RCA control was available to isolate the effect of the 48-h pre-soaking protocol. Consequently, field or structural adoption should be supported by project-specific performance and durability verification. Overall, the results support moderate RCA incorporation as a technically promising pathway for resource conservation while clearly defining the additional evidence needed before broader structural implementation.

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Performance trade-offs and microstructural mechanisms of cement mortars modified with high-dosage hydroxypropyl methylcellulose (HPMC) for thermal insulation and high adhesion

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Abstract

Hydroxypropyl methylcellulose (HPMC) is typically restricted to low dosages ($\leq 0.3\%$) to maintain workability and bulk compressive strength. This study investigates the functional trade-offs and microstructural mechanisms of high HPMC dosages (0.5%, 0.75%, or 1.0% by weight of cement) to develop energy-efficient, high-adhesion mortar matrices. The cement mortar incorporates a 1:3 cement-to-sand ratio by weight and a water-to-cement ratio of 0.5. Experimental evaluations demonstrate that high HPMC dosages lower bulk density and compressive strength (reduction between 17% and 30%); however, the use of high HPMC dosages significantly enhances interfacial and thermal performance. Notably, incorporating 0.75% HPMC yields a 64% reduction in thermal conductivity to 0.79 W/m. $^{\circ}$ C versus 2.18 W/m. $^{\circ}$ C for control mortar, while 1.0% HPMC increases pull-off strength by 176% (2.15 MPa) and shear strength by 167% (0.4 MPa). Additionally, the use of HPMC caused an increase in the initial setting time and the formation of a thick layer on the surface; the absorption of cement mortar increased with an increasing dosage of HPMC, with a 71% increase in the absorption for cement mortar with 1% HPMC compared to the control mortar. Scanning Electron Microscopy (SEM) observations revealed progressive changes in the mortar matrix morphology with increasing HPMC dosage, including changes in the appearance and distribution of pores. These observations are qualitatively consistent with the measured changes in density, water absorption, mechanical properties, and thermal conductivity. These findings confirm that high-dosage HPMC mortars, especially with 0.75%, provide an optimal balance for energy-efficient exterior renders and high-adhesion repair coatings where thermal resistance and bonding performance take priority over bulk compressive capacity.

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1. Introduction

Buildings account for a major share of global energy consumption [1], largely due to thermal inefficiencies in the building envelope [2]. Issues such as substantial heat loss during winter, excessive heat gain in summer, and localized thermal bridging not only increase heating and cooling loads [3] but also compromise indoor thermal comfort and elevate operational carbon emissions [4]. Consequently, cement mortar is commonly used as an exterior render worldwide; however, modern building demands require thermal-insulating render materials to mitigate envelope heat losses and reduce energy consumption. Therefore, researchers continue to improve its properties. Using additives can play a significant role in improving the fresh and hardened properties of mortar and concrete. Hydroxypropyl methylcellulose (HPMC) is one of these additives that can be incorporated into cement mortar or concrete. In practical application, HPMC

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serves as a crucial functional additive in cementitious renders and tile adhesive mortars [5]. When incorporated into tile adhesives, HPMC acts as a rheology modifier and thickener, significantly improving slip resistance (anti-sagging) on vertical surfaces while extending open time, which enhances application workability and long-term adhesive bond strength [5].

The incorporation of HPMC in concrete mixtures provided an adequate air-void system and enhanced freeze-thaw durability; however, scaling resistance remains a challenge requiring further investigation and potential mitigation strategies [6]. Furthermore, the incorporation of HPMC improved crack resistance [7]. In addition, the use of HPMC decreases the migration of chloride [8]. Regarding the effects of HPMC on the hydration of cement, it was found that the use of HPMC usually retards the hydration of cement, lowers the heat release, and increases the setting time [8,9,10,11,12,13]. HPMC's viscosity has a significant impact on hydration delay and setting time [8]. The presence of HPMC caused a delay in cement hydration and an effective reduction in bleeding, segregation, and permeability [12,14]. Additionally, the effect of using HPMC alongside a polycarboxylate-based superplasticizer was investigated. It was revealed that the use of HPMC had a negative impact on the dispersion capacity of the superplasticizer by obstructing its adsorption on the surface of cement particles [15].

The incorporation of HPMC in the cement paste matrix caused an increase in the air voids, which caused a reduction in the compressive strength [10]. X-ray computed tomography (X-CT) and Mercury intrusion porosimeter (MIP) tests found that the increase in the HPMC dosage caused an increase in the total cumulative pore volume of the mortar [8]. A noticeable decrease in the mechanical properties of cement mortars incorporating low dosages of HPMC was observed; the presence of HPMC caused a reduction in compressive strength (29.09% to 57.79%), flexural tensile strength (20.81% to 49.48%), and dynamic modulus of elasticity (16.7% to 47.22%) depending on the percentage of HPMC [16]. A slight decrease in the compressive strength was also revealed in concrete mixtures incorporating iron tailings [7]. Exceptionally, the incorporation of HPMC into chloroaluminate cement for printable mortar caused an increase in the compressive strength of printed mortar [9]. Both flexural and shear bond strengths decreased with increasing the dosage of HPMC, especially for mortars with steel fibers, and extending the curing period from 7 to 28 days does not have a clear impact on the mechanical properties of mortars [11]. Lower dosages of HPMC (0.05%, 0.1%, or 0.15% by weight of cement) caused a reduction in both compressive and bond strength [17].

Increasing the dosage of HPMC caused an increase in the air voids and porosity [10,11]. Due to the increase in porosity, an 11.76% reduction in weight occurred in cement mortar with HPMC; this increase in porosity also led to higher water absorption and water loss [16]. The increase in porosity and change in the microstructure of cement mortar with HPMC caused an improvement in thermal resistance by 32.6% compared to the control mixture without HPMC [16]. The use of 0.1% HPMC by weight of cement caused a reduction in the thermal conductivity of cement mortar by 12% compared to cement mortar without HPMC [17]. In addition, it was revealed that both water retention and plastic viscosity increased proportionally with HPMC dosage [7,17,18].

Microstructural characterization via Scanning Electron Microscope-Backscattered Electron-Image Analysis (SEM-BSE-IA) showed that after hardening, the HPMC formed bridges, which prevent microcracks between aggregate interfaces [7]. The SEM analysis revealed that the use of HPMC in cement mortars forms a three-dimensional colloidal film that fundamentally changes the behavior of water migration within the mortar matrix. HPMC adsorbed onto the surfaces of both cement and sand, which created a grid similar to the structure of hydration products [18].

After reviewing the previous studies, which focused on using HPMC in cement mortar, it can be seen that key parameters remain inconsistent across the literature, particularly regarding bonding and mechanical behavior. For example, while some studies report that HPMC improves long-term adhesive bond strength through increased water retention [5], others observe that higher HPMC dosages decrease shear and flexural bond strength due to elevated air-void formation [11,17]. Moreover, the reported compressive strength reduction varies considerably in the literature—ranging from marked drops of 29% to 58% [16] to minor decreases [7] or even strength increases in specialized cements [9]. However, the effect of relatively high HPMC dosages on the combined

physical, mechanical, thermal, interfacial, and microstructural properties of cement mortar is still not fully understood. Especially, an understanding of the simultaneous/cross mechanisms between the reduction in thermal conductivity and interfacial bond strengths (pull-off and shear strength) at high dosages (0.5%–1.0%), as well as matrix porosity, remains lacking in the literature. Therefore, the current study investigates the effect of relatively high HPMC dosages of 0.5%, 0.75%, and 1% by weight of cement. A comprehensive analysis of fresh-state properties, physical/mechanical performance, thermal insulation, and microstructural mechanisms was performed to investigate the comprehensive behavior of cement mortar with HPMC.

The main contribution of the current study is to provide a comprehensive evaluation of relatively high HPMC dosages in cement mortar by considering physical, mechanical, thermal, interfacial, and microstructural properties together. In particular, the study evaluates the relationship between the reduction in thermal conductivity and the improvement in shear and pull-off strengths, specialty rendering and repair mortars where adhesion and thermal resistance take priority over compressive strength.

2. Materials and Methods

2.1 Materials

Ordinary Portland Cement (OPC) Type I, manufactured at the Badosh plant complying with ASTM C150 [19], was used in this study. Chemical compositions and physical properties of cement were presented in Tables 1 and 2, respectively. The fine aggregate (sand) used in this study was obtained from the Kanhash River.

Table 1. Chemical composition (percentage) and specific gravity of ordinary Portland cement

Component	Portland cement	ASTM C150 Limits [19]
SiO ₂	19.8	-----
Al ₂ O ₃	4.13	6.0
Fe ₂ O ₃	3.85	6.0
CaO	61.87	-----
MgO	3.52	6.0
SO ₃	1.79	3.0
Cl ⁻	0.01	-----
LOI	2.35	3.5
Specific Gravity	3.15	
C ₃ S	63.04	-----
C ₂ S	9.21	-----
C ₃ A	4.43	8.0
C ₄ AF	11.72	-----

Table 2. Physical properties of ordinary Portland cement

Properties	Value	ASTM C150 Limit [19]
Initial setting time (Minutes)	125	Not less than 45
Final setting time (Minutes)	300	Not more than 375
Blaine (m ² /kg)	300	Not less than 260
Expansion (%)	0.04	Not more than 0.8
3 days compressive strength (MPa)	11	Not less than 7.0
7 days compressive strength (MPa)	18	Not less than 12.0
28 days compressive strength (MPa)	29	-----

The fine aggregate had an absorption of 1.51%, a specific gravity (saturated-surface dry, SSD) of 2.61 in accordance with ASTM C128 [20], and a fineness modulus of 3.1. Figure 1 illustrates the gradation of fine aggregates. The HPMC powder used in this study is an industrial-grade product, manufactured by Shijiazhuang Bairuisite Cellulose Company; the HPMC has a nominal viscosity of

200,000 mPa·s; methoxy content: 18%–32%; hydroxypropyl content: 8%–16%; fineness: 80–100 mesh; gel temp: 54–75°C; moisture: $\leq 5\%$; ash content: $\leq 2\%$; pH 7–8. Table 3 shows the mixture proportions per unit of kg/m³. For simplicity, the cement-to-sand ratio (oven-dry) is 1:3 by weight, and the water-to-cement ratio is 0.5, where tap water was used. The HPMC dosages are 0.5%, 0.75%, or 1% by weight of cement.

Table 3. Mixture proportions (kg/m³)

Mixtures*	w/c	Cement kg/m ³	Water kg/m ³	Fine Aggregate kg/m ³ (oven dry)	HPMC kg/m ³
M-0	0.5	509	254	1525	0
M-0.5	0.5	509	254	1525	2.54
M-0.75	0.5	509	254	1525	3.82
M-1	0.5	509	254	1525	5.09

* Mixture designation: M-X; X = Dosage of HPMC by weight of cement; Note: 1 kg/m³ = 0.06243 lb/yd³

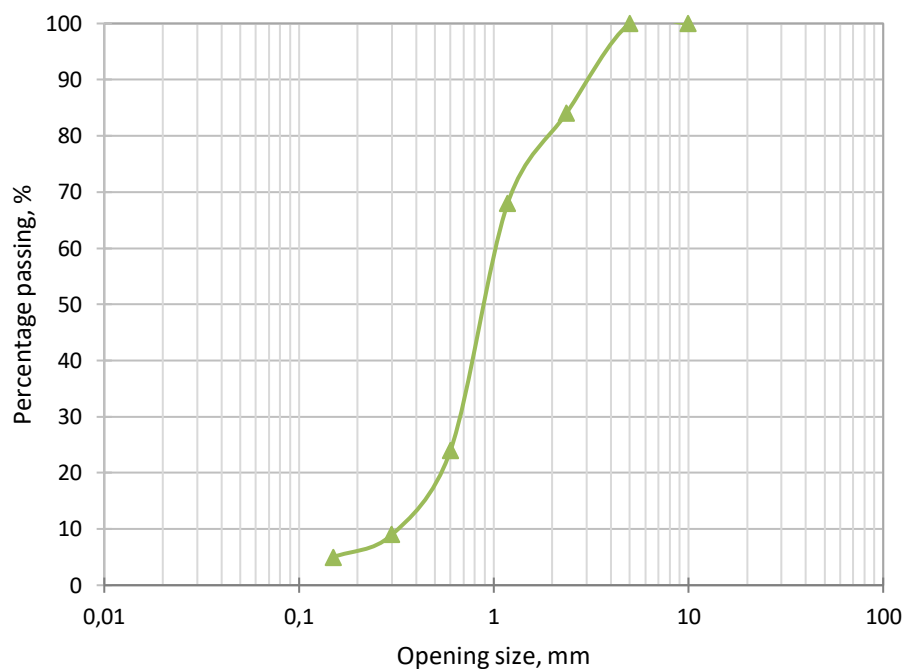


Figure 1. Grading curve for fine aggregate

2.2 Methods

The mixing procedure and the preparation of cement mortar were executed in accordance with BS EN 196-1 [21]. The HPMC powder was mixed with the whole amount of water for 2 minutes with manual stirring to prepare the HPMC solution; cement and sand (oven-dry at 105°C) were mixed together for 1 minute, then the HPMC solution was added gradually and evenly to the mixture of cement and dry sand. The whole mixture was mixed manually for 4 minutes. The testing of the flowability of cement mortar was conducted in accordance with BS EN 1015-3 [22]. The scanning electron microscope (SEM) (Zeiss EVO-10) was used to investigate the microstructural characterization of cement mortars; the tested specimens were at an age of 28 days without stopping the hydration process; also, the surface of the tested specimens was not coated. The SEM was operated at an accelerating voltage of 10 kV with working distances ranging from 14 to 20 mm, using the secondary electron (SE1) detector. The tests of initial and final setting time of cement paste with or without HPMC were conducted in accordance with ASTM C-191 [23]. The bleeding test was utilized in accordance with ASTM C940-22 [24]. The testing of compressive and flexural strengths was conducted in accordance with BS EN 1015-11 [25]. For each mixture, 3 prisms

(40×40×160 mm) were cast, demolded after 24 hours, and cured for 28 days in a lime-saturated water tank. The average compressive strength was obtained from 3 specimens.



Figure 2. Shear strength test (Triplet test)



Figure 3. Pull-off strength test



Figure 4. Thermal conductivity test



The wet and dry densities were measured in accordance with BS EN 1015-6 [26] and BS EN 1015-10 [27], respectively. The absorption of cement mortar was measured in accordance with BS EN 1015-18 [28]. The shear strength of cement mortars (triplet test) was utilized in accordance with BS EN 1052-3 [29], clay bricks were used in the test, as shown in Figure 2. The pull-off strength of cement mortars was determined in accordance with BS EN 1015-12 [30]; the cement mortar was placed on the back side surface of terrazzo tiles, where the back side surface is rough and made of cement mortar, as illustrated in Figure 3. For the thermal conductivity test, cement mortar was cast in a plastic mold with a diameter of 100 mm and a thickness of 10 mm. After 24 hours, the specimens were demolded and cured for 28 days in a lime-saturated water tank. Then, the specimens were dried in an oven at 60°C until reaching a constant weight. The thermal conductivity test was performed using a custom steady-state thermal conductivity apparatus, as shown in Figure 4. More details about the apparatus can be explained in the previous study [31]. The developed device operates in accordance with ASTM C177 [32]/ASTM C518 [33].

3. Experimental Results and Discussion

Four cement mortar mixtures and a series of experimental tests were utilized to investigate the physical and mechanical properties of cement mortars incorporating HPMC. The one-way analysis of variance (ANOVA) was utilized in the current study to test the differences between results. In the ANOVA test, if the p -value is less than 0.05, Tukey's HSD test was followed to identify which groups differed from each other. In Tukey's HSD test, if the p -value is ≤ 0.05 , this means that the difference is statistically significant.

3.1 Flowability of Cement Mortar

Table 4 and Figure 5 show the results of the flow test for the cement mortars. It was found that the presence of HPMC caused a significant decrease in the flowability of cement mortar. Specifically, relative to the control mixture (160.7 mm), increasing the HPMC dosage reduced the fresh mortar flow values to 150.3 mm, 135.0 mm, and 120.0 mm, representing percentage decreases of 6.5%, 16.0%, and 25.3%, respectively.

Table 4. Mortar properties

Mixtures*	Flow (mm)*	Temp., °C†	Wet density (g/cm ³)*	Dry density (g/cm ³)*	Bleeding %*	Initial setting (min)*	Surface skinning time (min)*	Absorption %*
M-0	160.7	23.0	2.17	2.16	2.37	90	380	2.11
M-0.5	150.3	22.5	1.95	1.90	0.00	95	350	3.17
M-0.75	135.0	24.0	1.90	1.86	0.00	110	280	3.31
M-1	120.0	23.0	1.86	1.84	0.00	120	240	3.60

* Mixture designation: See Table 3; * Each value is an average of three tested specimens; † Temperature for fresh mortar during mixing; Note: 1 in. = 25.40 mm

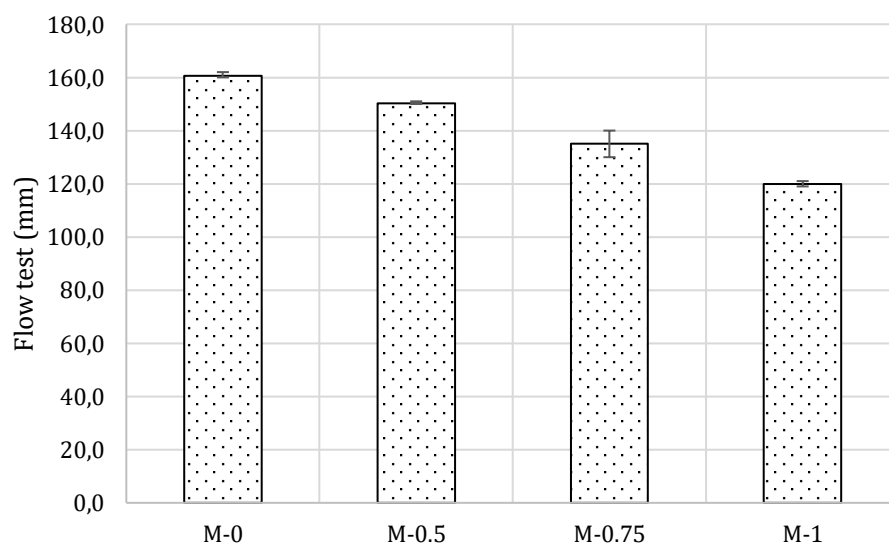


Figure 5. Flow vs. mortar mixtures

The reduction in the flowability of cement mortar by increasing the dosage of HPMC means that the plastic viscosity is increased. This decrease in flowability is primarily governed by HPMC's high water retention capacity, its ability to bind free water, and the formation of a polymeric network among cement particles [7]. Although HPMC also entrains micro-air bubbles, which can reduce inter-particle friction, the governing mechanism reducing flow value is the immobilization of free water within the polymeric network. Additionally, Table 4 reports fresh mortar temperatures ranging from 22.5 °C to 24.0 °C; however, these minor temperature variations exert a negligible influence on viscosity or setting kinetics, confirming that workability changes are governed by HPMC dosage rather than thermal effects. Also, it is important to mention that the fixed water-to-cement ratio of 0.5 used for all cement mortars had a direct relationship to the reduction in

flowability by increasing the dosage of HPMC. In the current study, the ANOVA and Tukey's HSD test showed that the reduction in flowability is statistically significant, with p -values ranging from 0.0010 to 0.0058. This finding is in line with the studies [9,18].

3.2 Microstructural Characterization / SEM Analysis

The SEM analysis, as shown in Figure 6, illustrates that the incorporation of HPMC induced progressive microstructural changes. The control cement mortar (0% HPMC, Figure 6a) exhibited a relatively heterogeneous morphology with irregular hydration-product features and visible voids.

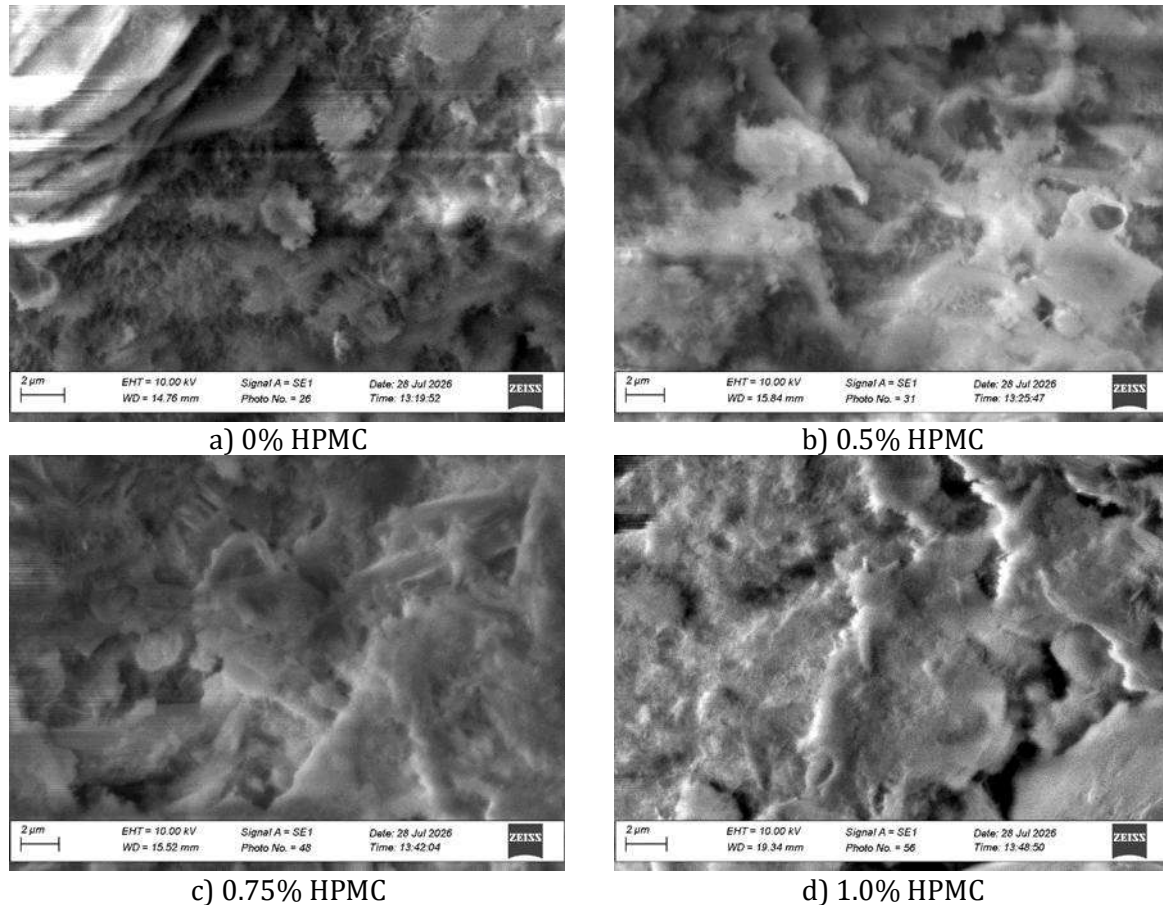


Figure 6. Qualitative SEM micrographs illustrating the surface morphology and microstructural evolution of cement mortars with varying dosages of HPMC (2 μ m scale)

On the other hand, the incorporation of 0.5% HPMC caused a change in the matrix, as shown in Figure 6b compared to Figure 6a. Since early hydration is bound to use up the available water, the dissolved cellulose ether creates fine film networks that may bridge the micro-voids, connecting C-S-H aggregates. The observed morphology may be influenced by the film-forming and water-retention behavior of HPMC. Increasing the HPMC dosage to 0.75%, the SEM analysis (Figure 6c) shows a visually denser and more continuous matrix compared with the control and 0.5% HPMC mixtures. Some hydration-product features appeared less distinct, while fewer large visible voids were observed in the examined regions. These observations suggest modification of the matrix morphology with increasing HPMC dosage. Finally, at an HPMC dosage of 1.0% (Figure 6d), the examined SEM region appeared more continuous and compact; however, this localized morphology should not be interpreted as an overall reduction in porosity, particularly because the measured density decreased with increasing HPMC dosage. The addition of HPMC to the mixture led to a decrease in wet and dry densities and an increase in absorption, but thermal conductivity decreased.

The changes in matrix structure, including the presence of pores, may provide a rationale for these results. At the same time, the modification of the matrix with the help of HPMC may have

contributed to the growth of shear and pull-off strengths. However, as a methodological limitation, because SEM observations are primarily morphological and obtained from selected microscopic regions, they may not fully represent the pore structure of the bulk mortar, confirm the precise spatial distribution of HPMC, or conclusively establish the presence of continuous polymer films or polymer–C–S–H bonding. Therefore, the SEM results are interpreted qualitatively and in conjunction with the macroscopic test results as supporting evidence rather than direct proof of the mechanisms responsible for these macroscopic changes.

3.3 Density of Cement Mortar

The results of wet and dry density of cement mortar, as presented in Table 4 and Figure 7, revealed that both wet and dry densities decreased with increasing the dosage of HPMC. The reduction in wet and dry densities were 14.3% and 14.8%, respectively, for cement mortar with 1% HPMC (M-1) compared to the control mortar (M-0). In the case of the wet density, the reduction compared to the control cement mortar (M-0) and between M-1 and M-0.5 were statistically significant, with a p -value of 0.0010, while the differences among other cement mortars (M-0.5 versus M-0.75, and M-0.75 versus M-1) were not statistically significant.

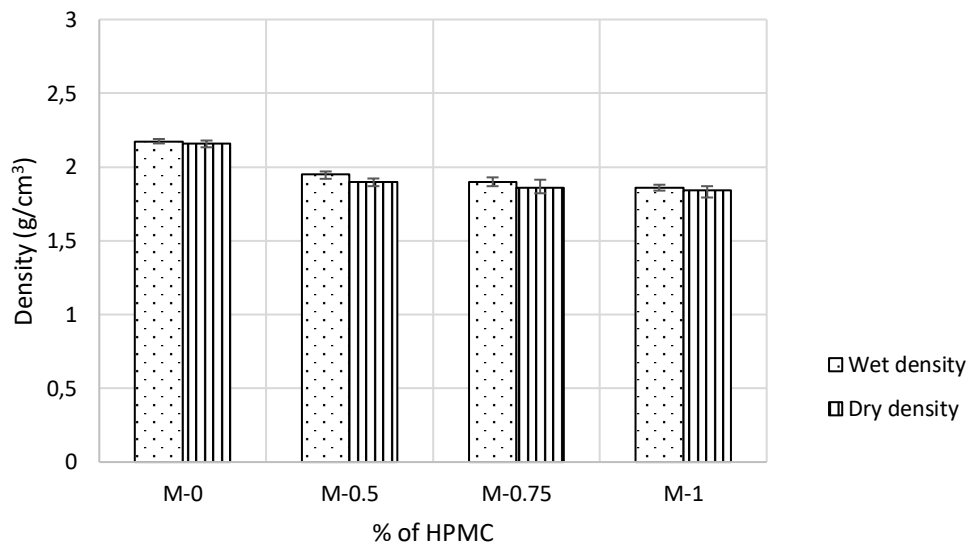


Figure 7. Density vs. mortar mixtures

In addition, the reduction in the dry density compared to the control cement mortar (M-0) was statistically significant, with a p -value of 0.0010, while the differences among other cement mortars (M-0.5, M-0.75, and M-1) were not statistically significant. The SEM observations provide morphological evidence that is consistent with the reduction in density. HPMC works as a surfactant in fresh mortar, leading to the trapping of air bubbles and micro-cavities during mixing. For cement mortar incorporating 1% of HPMC (M-1, Figure 3d), the SEM micrograph shows a modified matrix structure that reflects these entrained air features. Although the specific gravity of HPMC is lower than that of hydrated cement, the addition of 0.5% or 1% of HPMC by weight of cement cannot explain the noticeable reduction in density. Instead, trapped air bubbles and the resulting increase in porosity are the main causes of the reduced bulk density. This finding is consistent with previous studies [7]. A distinct pattern also appears in the wet-to-dry density gap. While M-0 changes by only 0.01 g/cm³ upon drying (2.17 to 2.16 g/cm³), HPMC mortars show a 0.04–0.05 g/cm³ difference. This expanded delta reflects HPMC's moisture retention: ether functional groups lock in pore water via hydrogen bonding, and developing polymer membranes block capillary channels. Retaining this extra bound water retards internal drying and reduces capillary tension associated with drying shrinkage.

3.4 Initial Setting Time and Surface Skinning Time

Table 4 and Figure 8 show the results of the initial setting time and surface skinning time of cement pastes with or without HPMC. For the initial setting time, it was found that increasing the dosage

of HPMC caused an increase in the setting time. The increase in the initial setting time for M-0.5, M-0.75, and M-1 was approximately 6%, 22%, and 33%, respectively, compared to M-0. The statistical analysis reveals that the increase in the initial setting time was statistically significant between the following: M-0 versus M-0.75 (p -value is 0.0012), M-0 versus M-1 (p -value is 0.0010), M-0.5 versus M-0.75 (p -value is 0.0073), and M-0.5 versus M-1 (p -value is 0.0010). However, it was not statistically significant between the following: M-0 versus M-0.5 and M-0.75 versus M-1, where the p -value was higher than 0.05. This finding was also revealed by another study [12]. The increase in initial setting time may be associated with the interaction of HPMC with cement particles and the modification of water mobility and cement hydration kinetics. The increased viscosity and water-retention capacity of HPMC may additionally influence the transport of dissolved species and the development of early hydration products. For the surface skinning time, it was found that by increasing the dosage of HPMC, the surface skinning time was decreased. However, this result does not represent the actual situation of the bulk mortar. It was found that there is a thick layer on the top of the Vicat mold. The formation of this thick superficial layer can be attributed to localized water retention, air entrainment, and rapid polymer film formation at the air–mortar interface [7]. After breaking that superficial layer, plastic and unhydrated mortar was still present beneath it. Therefore, the reported surface skinning time does not represent the final setting time.

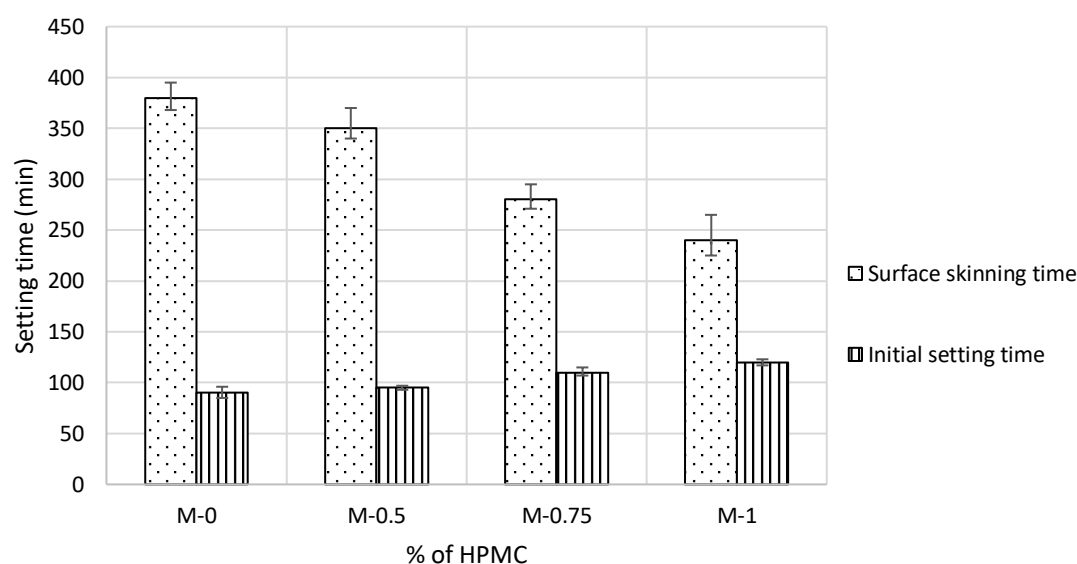


Figure 8. Initial setting and surface skinning times vs. mortar mixtures

3.5 Absorption of Cement Mortar

The results of water absorption capacity in percentage of hardened cement mortars, as illustrated in Table 4 and Figure 9, revealed that increasing the dosage of HPMC caused an increase in absorption; the absorption increased by approximately 50%, 57%, and 71% for cement mortars M-0.5, M-0.75, and M-1, respectively, compared to the control cement mortar. The presence of HPMC in the fresh cement mortar caused changes in the pore structure and may have promoted the formation and retention of air voids and microcavities, as suggested by the SEM observations in Figure 6. This polymer matrix has the hydrophilic nature of HPMC, which increased the absorption capacity of the cement mortar. In the current study, the increase in absorption is statistically significant compared to the control cement mortar (M-0), where the p -value was 0.0010; also, the increase in absorption between M-0.5 and M-1 was statistically significant, where the p -value was 0.0472. Whereas, the increase in absorption was not statistically significant between M-0.5 and M-0.75, or between M-0.75 and M-1. This finding is supported by a previous study [16].

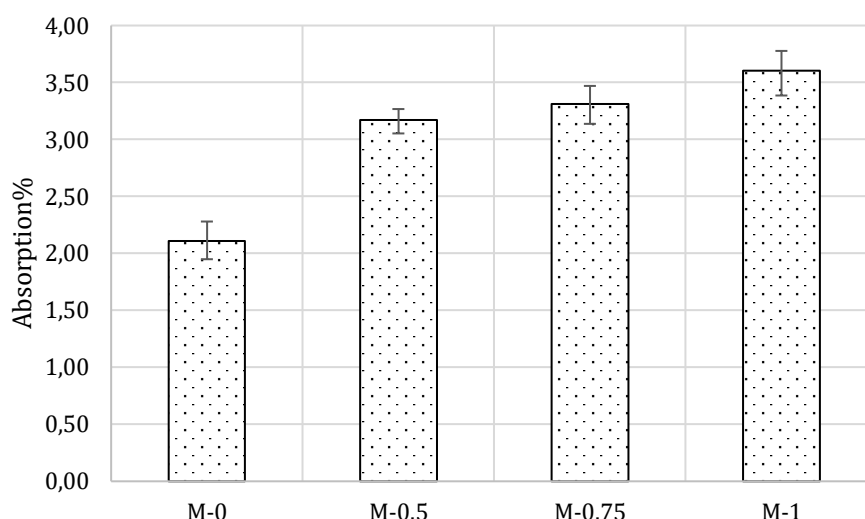


Figure 9. Absorption vs. mortar mixtures

3.6 Bleeding of Cement Mortar

The bleeding of cement mortars was observed for cement mortar mixtures, as shown in Table 4. Based on the results, it is obvious that the use of HPMC, regardless of its dosage, prevented bleeding. This finding is also found in a previous study [12]. The reason behind this could be the adsorption of HPMC to cement and sand particles, which caused a reduction in the migration of water [12]. In addition, the use of HPMC caused an increase in water retention, which led to keeping water and reducing its migration [18]. In job-site applications like exterior renders and plasters, this complete lack of bleeding brings clear practical advantages. Halting internal water movement stops heavy sand grains from settling out, which prevents aggregate segregation during pumping and troweling. The higher paste viscosity also boosts matrix cohesion, giving the wet render enough sag resistance so workers can apply thicker coats to vertical walls without slumping.

3.7 Mechanical Properties of Cement Mortar

3.7.1 Compressive Strength

The results of compressive strength of cement mortar specimens after 28 days of curing are illustrated in Table 5 and Figure 10. The results showed that the incorporation of HPMC caused a reduction in the compressive strength. This finding is in the same line with the findings of prior studies [7,16]. Specimens of mixture M-0.75 experienced a 17% reduction in compressive strength compared to the control mixture M-0, and had the lowest reduction compared to other mixtures. This reduction in compressive strength can be attributed to the surfactant behavior of HPMC, which entrains air bubbles and promotes micro-cavity formation within the paste matrix, leading to lower bulk density, as explained earlier. The statistical analysis showed that the reduction in the compressive strength for mixtures M-0.5 and M-1 compared to the control mixture (M-0) is statistically significant, with p -values of 0.0269 and 0.0443, respectively.

Whereas the reduction among other mixtures (M-0.75 versus M-0, M-0.75 versus M-0.5, M-0.5 versus M-1, and M-0.75 versus M-1) is not statistically significant, with p -values greater than 0.05. The reason behind the lower strength reduction in mixture M-0.75 compared to M-0.5 and M-1 could be related to the formation of polymer films. In mixture M-0.5, the HPMC dosage is low, which causes trapping of air bubbles, while the polymer film is not sufficient to support the matrix. In mixture M-0.75, increasing the dosage allows HPMC to form continuous polymer films that fill the micro-cavities and bridge small cracks, which leads to reducing the negative impact of air bubbles and improving the matrix structure. However, at a higher dosage in mixture M-1, the excessive use of HPMC could entrain more air bubbles and coat the unhydrated cement particles, which delays the hydration process and causes a further reduction in compressive strength.

Table 5. Results of mechanical strengths and thermal conductivity

Mixtures*	Compressive strength (MPa)*	Flexural strength (MPa)*	Shear strength (MPa)*	Pull-off strength (MPa)*	Thermal Conductivity (W/m. °C)*
M-0	22.7	5.77	0.15	0.78	2.18
M-0.5	15.93	5.10	0.23	1.43	1.33
M-0.75	18.80	5.00	0.23	1.95	0.79
M-1	16.60	4.73	0.40	2.15	0.88

* Mixture designation: See Table 3

* Each value is an average of three tested specimens (at age of 28 days)

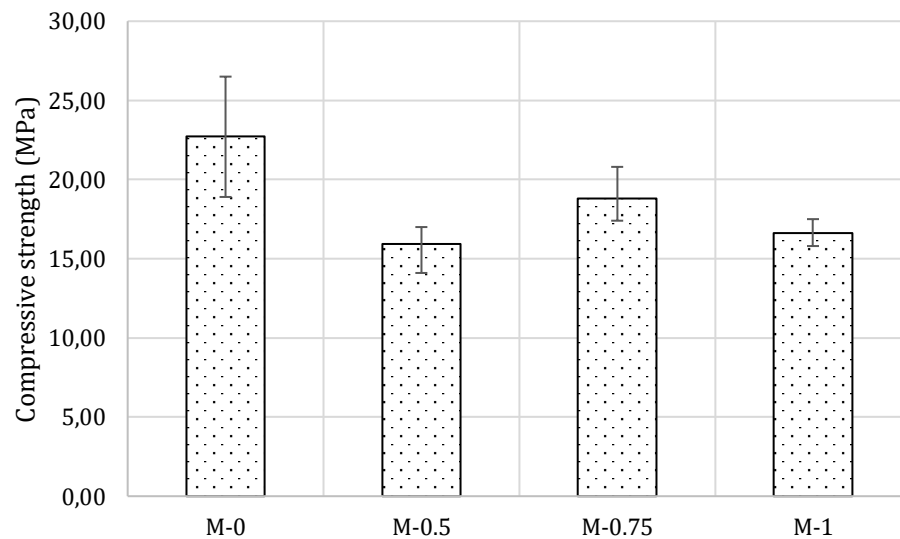


Figure 10. Compressive strength vs. mortar mixtures

3.7.2 Flexural Strength

The results for the flexural strength of cement mortar prisms, as shown in Table 5 and Figure 11, illustrate that the use of HPMC caused a moderate reduction in the flexural strength. This finding is in line with prior studies [11,16]. The reduction values range from 12% to 18% compared to the control cement mortar (M-0). This decrease is linked to the increased porosity caused by the air-entraining effect of HPMC. However, the flexural strength loss was smaller than expected relative to the density loss, as the polymer film formation helps transfer tensile stresses across micro-voids. The statistical analysis showed that the reduction in flexural strength is not statistically significant, except that the reduction for mixture (M-1) compared to the control mixture (M-0) is statistically significant, with a p -value of 0.0262.

Figure 12 illustrates the relationship between the flexural-to-compressive strength ratio (F/C) versus mixtures; the main finding is that the ratio (F/C) is increased for all mixtures compared to the control one. That increase in the ratio (F/C) confirms an enhancement in matrix ductility and toughness under stress. This finding is supported by another prior study [18]. Although complete load-deflection curves were not measured to calculate total energy absorption, the higher F/C ratio indicates an improvement in relative flexibility and ductility, which leads to a less brittle matrix. The SEM analysis, as shown in Figure 6, revealed morphological differences between the control mortar and mortars including different dosages of HPMC. These changes, as illustrated in the SEM analysis, may be associated with the water-retention and rheological effects of HPMC and may contribute to the observed changes in mechanical and interfacial properties. However, the SEM images alone cannot confirm the formation of continuous polymer films or polymer bridges within the cementitious matrix.

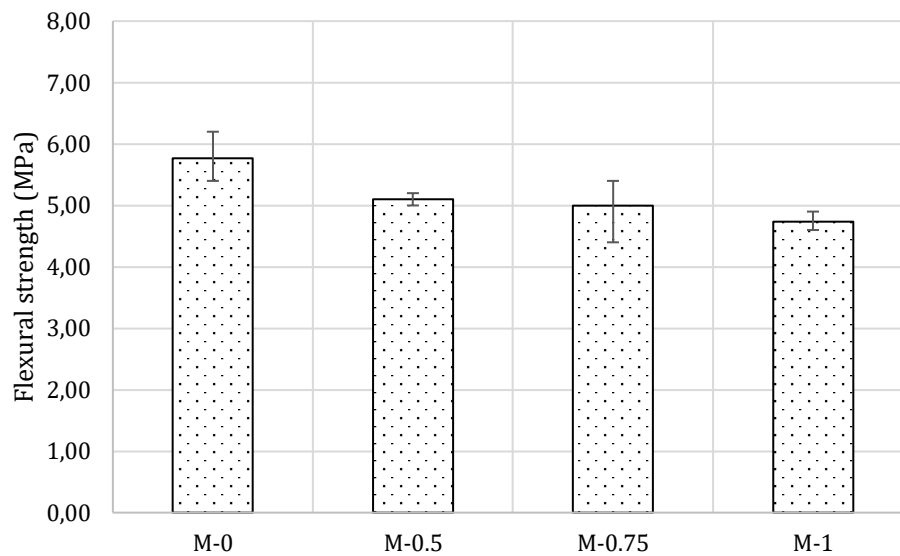


Figure 11. Flexural strength vs. mortar mixtures

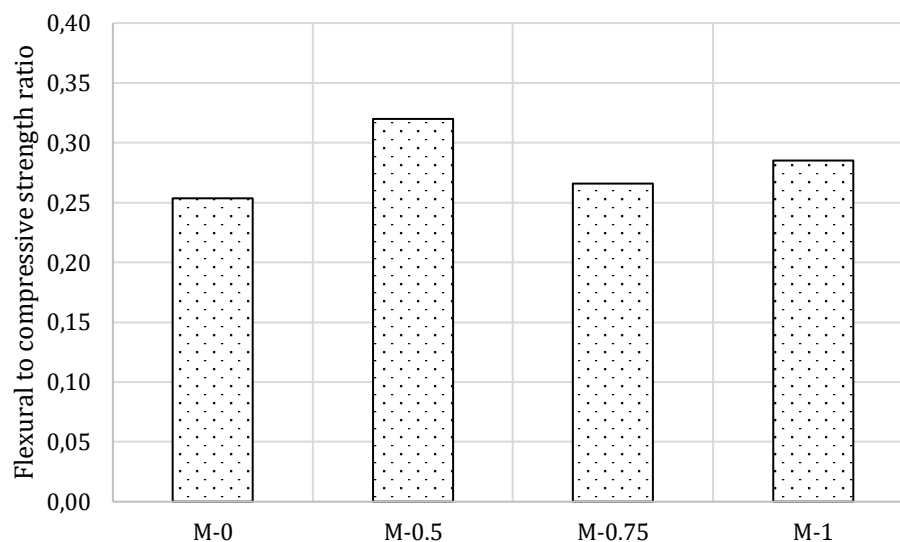


Figure 12. Flexural to compressive strength ratio vs. mortar mixtures

3.7.3 Shear Strength

Table 5 and Figure 13 illustrate the shear strength of cement mortars. The results show that mixtures M-0.5 and M-0.75 had an identical increase in shear strength of 53%, while mixture M-1 had a noticeable increase of 167% compared to the control mixture M-0. That increase in shear strength was statistically significant when comparing the results for (M-1 versus M-0, M-1 versus M-0.5, and M-1 versus M-0.75), where p -values are 0.0010, 0.0022, and 0.0022, respectively. whereas, the increase was not significant for (M-0.5 versus M-0, M-0.75 versus M-0, and M-0.75 versus M-0.5). The observed morphological changes, as shown in Figure 3, may be associated with the increase in shear strength; however, SEM observations alone cannot confirm the underlying mechanism. This increase could be attributed to an improvement in interfacial adherence and friction coefficient. The presence of HPMC helps retain water near the brick interface. This allows full C-S-H gel hydration and the formation of polymer films. Also, HPMC increases the cohesion and packing density of the mortar, which increases the frictional resistance along the joint. All tested specimens exhibited shear failure exclusively within the mortar layer, corresponding to the “*shear failure only in the mortar*” failure mode defined in BS EN 1052-3.

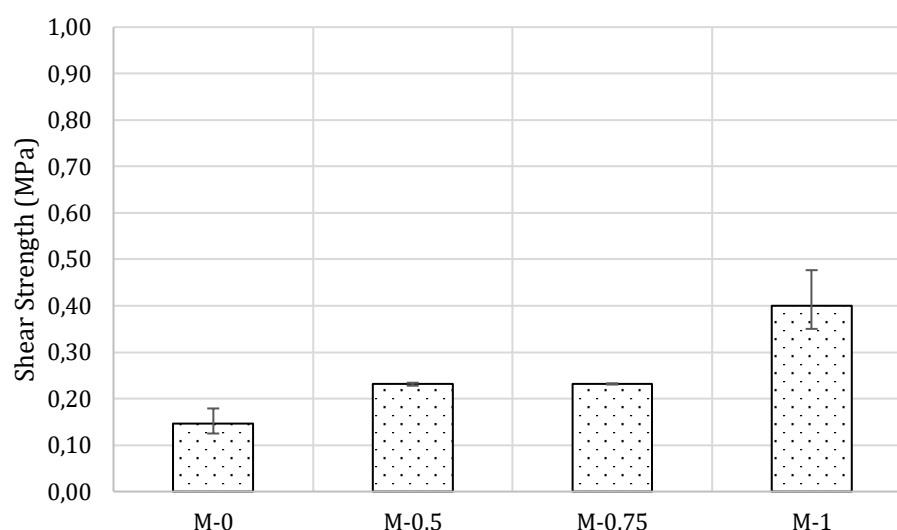


Figure 13. Shear strength vs. mortar mixtures

3.7.4 Pull-off Strength

Table 5 and Figure 14 show the pull-off strength of cement mortars. The results show that mixtures M-0.5, M-0.75, and M-1, compared to the control mixture M-0, had an increase of 83%, 150%, and 176%, respectively. The statistical analysis showed that the increase in the pull-off strength was statistically significant, with p -values ranging from 0.0010 to 0.0205, except for the comparison of M-1 versus M-0, where the statistical analysis was not significant. As illustrated in Figure 6, adding HPMC was associated with morphological changes in the examined matrix regions.

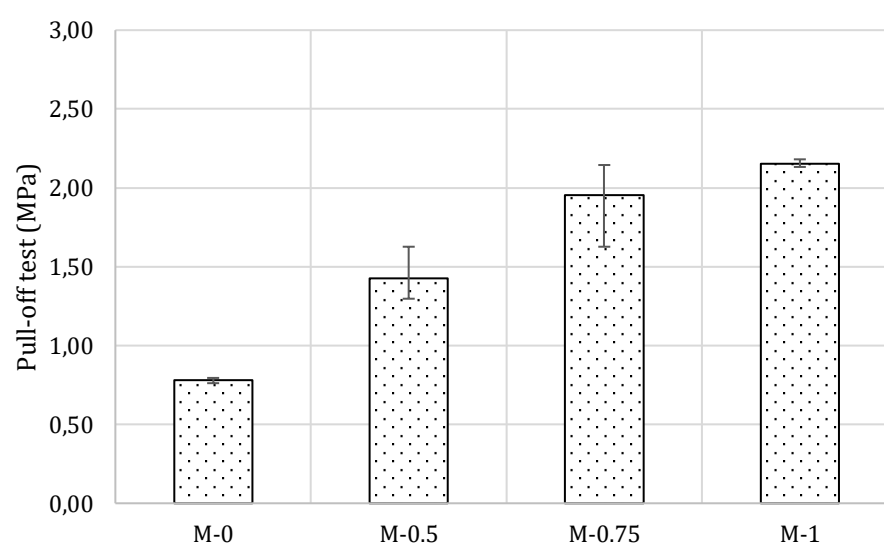


Figure 14. Pull-off strength vs. mortar mixtures

The failure mode in all tested specimens was a cohesion fracture within the mortar, similar to type B, as indicated in BS EN 1015-12. These changes, together with the measured increase in pull-off strength, may indicate modification of the matrix and crack-bridging behavior; however, the formation of flexible polymer bridges cannot be confirmed from SEM morphology alone. The improved pull-off strength may therefore be associated with changes in matrix cohesion, water retention, and the interaction of HPMC with the cementitious constituents.

3.8 Thermal Conductivity of Cement Mortar

Table 5 and Figure 15 illustrate the positive impact of using HPMC on the thermal conductivity of cement mortars. The results reveal that the incorporation of HPMC caused a clear reduction in the thermal conductivity of cement mortars. Specifically, thermal conductivity dropped from 2.18

W/m·°C in M-0 to 1.33 W/m·°C in M-0.5, 0.79 W/m·°C in M-0.75, and 0.88 W/m·°C in M-1, representing reductions of 39%, 64%, and 60%, respectively. Among these mixtures, the mixture with 0.75% HPMC (M-0.75) had the highest reduction in thermal conductivity, 64% compared to the control mixture with 0% HPMC (M-0). The statistical analysis showed that the reduction in thermal conductivity was statistically significant, with p -values ranging from 0.0010 to 0.0229, except for the comparison between M-0.75 and M-1.0, where the statistical analysis was not significant. This finding is in the same line with the findings of other studies [16,17]. The reduction in thermal conductivity may be associated with the reduction in density and changes in the pore structure caused by HPMC. However, mixture M-1 showed a slightly higher thermal conductivity than M-0.75, despite having a dry density of 1.84 g/cm³ and water absorption (3.60%) compared to 1.86 g/cm³ and water absorption (3.31%). The possible explanation could be attributed to pore coalescence at 1.0% HPMC dosage, where isolated micro-voids merge into connected capillary channels. Also, higher HPMC content increases moisture retention within the pore structure, and water conducts heat more effectively than trapped air. In future work, the micro-pore characterization tests, like Mercury Intrusion Porosimetry (MIP), are essential to explain the obtained results. The SEM images provide qualitative evidence of morphological changes in the examined regions, including differences in the appearance and distribution of voids. However, SEM alone cannot quantify the total pore volume or establish the specific mechanism responsible for the reduction in thermal conductivity. HPMC acts as a surfactant that entrains closed micro-cavities within the paste matrix, which act as thermal insulating air barriers. Furthermore, HPMC may modify the connectivity and distribution of capillary pores through its water-retention and film-forming behavior. Together with the increased presence of entrained air voids, these changes may contribute to the observed reduction in thermal conductivity. Based on the experimental findings and limitations in the current study, the following future research areas are suggested. First, studying the air content in either plastic or hardened cement mortar, the volume of air bubbles, and their distribution.

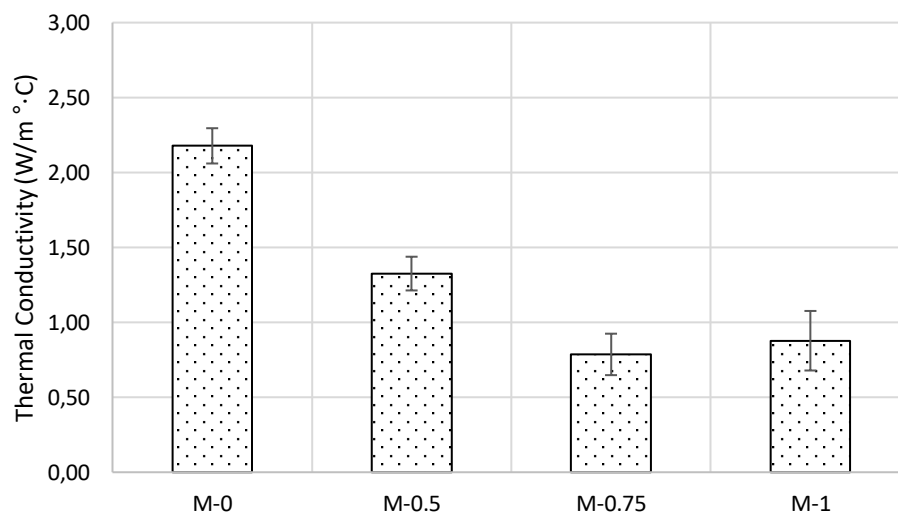


Figure 15. Thermal conductivity vs. mortar mixtures

Micro-pore characterization tests such as Mercury Intrusion Porosimeter (MIP) are highly recommended for future work. Second, performing durability tests, such as freezing and thawing and wetting and drying, should be performed before using the HPMC cement mortar as an exterior render material.

4. Conclusions

Four cement mortar mixtures were cast, and a series of experimental tests were utilized to investigate comprehensively the physical and mechanical properties of cement mortar incorporating HPMC. Based on statistical evaluations using ANOVA and Tukey's HSD test and experimental findings, it can be concluded:

- The use of HPMC caused a reduction in the flowability of cement mortars compared to the control mixture, and the flowability is reduced by increasing the dosage of HPMC, decreasing by 25.3% from 160.7 mm in M-0 to 120.0 mm in M-1.
- The use of HPMC caused a reduction in both wet and dry densities compared to the control cement mortar mixture, where dry density decreased from 2.16 g/cm³ in M-0 to 1.84 g/cm³ in M-1.
- The use of HPMC caused a delay in the setting process, where the initial setting time increased with an increasing dosage of HPMC, rising from 90 min in M-0 to 120 min in M-1. However, regarding the apparent reduction in final setting time, it was found that there is a thick layer on the top of the Vicat mold, and that layer gives an indication of the surface skinning time rather than reflecting the true bulk matrix setting.
- The use of HPMC caused an increase in absorption, where absorption increased with an increasing dosage of HPMC. The absorption increased by 71% for cement mortar with 1% HPMC compared to the control cement mortar, rising from 2.11% in M-0 to 3.60% in M-1.
- The use of HPMC, regardless of its dosage, prevented bleeding of cement mortars, dropping the bleeding percentage from 2.37% in M-0 to 0.00% across all HPMC mixtures.
- The use of HPMC caused a reduction in compressive strength. The M-0.75 experienced a 17% reduction in compressive strength compared to the control mixture M-0, and had the lowest reduction compared to other mixtures.
- The use of HPMC caused a reduction in flexural strength; the reduction values range from 12% to 18% compared to the control cement mortar (M-0).
- The use of HPMC caused an increase in shear strength and pull-off strength, showing maximum increases of up to 167% (0.40 MPa) and 176% (2.15 MPa) at 1.0% HPMC dosage, respectively.
- The use of HPMC caused a reduction in thermal conductivity, where the 0.75% HPMC dosage achieved a maximum reduction of 64% (0.79 W/m·°C compared to 2.18 W/m·°C for M-0).
- The SEM observations revealed progressive changes in the mortar matrix morphology with increasing HPMC dosage, including changes in the appearance and distribution of pores. These observations are qualitatively consistent with the measured changes in density, water absorption, mechanical properties, and thermal conductivity; however, SEM alone cannot confirm the chemical distribution of HPMC or the formation of continuous polymer films.
- The results revealed that HPMC-modified mortars exhibited enhanced thermal resistance and interfacial bond strength, making them highly suitable for energy-efficient exterior renders and high-adhesion repair mortars, where a 0.75% HPMC dosage is optimal for thermal insulation renders, while a 1.0% HPMC dosage is recommended for high-adhesion repair coatings.

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Height-dependent seismic performance of reinforced concrete buildings in Portoviejo: A comparative study on the influence of masonry infill walls and staircases

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Abstract

The seismic response of reinforced concrete buildings can be significantly affected by masonry infill walls and staircases, which are often not explicitly considered in analytical models. This study evaluates the effect of these components on typical buildings in Portoviejo, Ecuador, to identify their influence on structural capacity and seismic performance. Three representative building typologies of 3, 6, and 10 stories were analyzed through nonlinear static pushover analysis, performed in SeismoStruct using inelastic force-based frame elements with fiber sections, considering four structural configurations: frame (P), frame with masonry infill walls (P+M), frame with staircases (P+S), and frame with masonry infill walls and staircases (P+M+S). Based on the capacity curves, performance points, and damage distribution, the results indicate that the influence of these components depends on building height. In low-rise buildings, masonry infill walls increased the initial stiffness by approximately 90–110% and the lateral strength relative to the bare frame, although they reduced global ductility. In mid-rise buildings, the most critical response was observed, associated with vertical irregularity, damage concentration in the lower stories, and a tendency toward soft-story mechanisms, consistent with field observations following the 2016 Portoviejo earthquake. In high-rise buildings, the global effect was smaller, although local effects on the distribution of internal forces and damage remained. These findings indicate that neglecting masonry infill walls and staircases may underestimate relevant seismic demands and misrepresent stiffness, deformation patterns, and damage concentration; therefore, their explicit consideration is recommended in the seismic assessment of existing reinforced concrete buildings with similar configurations in Portoviejo.

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1. Introduction

Performance-based seismic assessment of buildings has become particularly relevant in regions of high seismic hazard, as it allows not only the structural safety against severe seismic events to be estimated, but also the expected deformation, damage level, and post-earthquake functionality at different levels of seismic demand. For the existing building stock, this framework is a necessary basis for assessment rather than a general analytical tool, because structural safety depends on the capacity to sustain inelastic deformations and on how damage concentrates as seismic demand increases [1-3]. In framed buildings, this assessment is especially important for irregular reinforced concrete (RC) buildings, where nonuniform distributions of stiffness and strength may concentrate inelastic demands in specific stories or structural members.

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In the Ecuadorian context, and particularly in Portoviejo, this issue is especially relevant because the city is located in a zone of high seismic hazard [4,5]. The damage observed during the April 16, 2016 earthquake, revealed the vulnerability of a significant portion of the reinforced concrete building stock, particularly in buildings with open ground stories, irregularities in story height, and a substantial reduction of masonry infill walls at the first level [6,7]. Approximately 40 RC buildings collapsed in downtown Portoviejo, with contributing factors that included seismic demands exceeding the code spectra at periods of about 0.5 s, the addition of stories without structural strengthening, highly flexible structural systems, and typologies with tall ground stories and mezzanines [7]. Field observations and three-dimensional pushover analysis of a representative RC building further highlighted the significant influence of masonry infill walls on the observed seismic response [8]. The combination of a taller ground story and a reduced density of masonry infill walls at this level can generate a pronounced vertical stiffness irregularity, favoring deformation concentration in the first story and the development of soft-story mechanisms [6, 7, 9]. Numerical analyses of corner buildings that collapsed in Portoviejo have further shown that irregular distributions of masonry infill walls in plan can induce a predominantly torsional response that significantly influences the collapse response [10]. These observations indicate that the seismic response of the local building stock cannot be interpreted exclusively from the properties of the bare RC frame.

Masonry infill walls and staircases are among the components that can significantly modify the response of RC frames, although they are commonly omitted from the explicit representation of the main lateral-load-resisting system. In the case of masonry infill walls, their interaction with the frame may increase the initial stiffness and strength of the system, but may also reduce its ductility and induce more brittle responses and damage concentrations in columns and joints [11-14]. Staircases can likewise modify lateral stiffness, internal-force distribution, and the lateral load path because stair flights and landings provide additional connections between adjacent stories [15-17]. When intermediate landings are connected to frame columns, the effective deformable length of these members may be reduced, increasing local shear demand and promoting a short-column-type response [17]. Recent field observations following the 2023 Kahramanmaraş earthquakes have also documented significant staircase damage, including cracking, shear-related failures, and local collapse, emphasizing the importance of explicitly considering staircase-frame interaction and connection detailing in seismic assessment [18]. Consequently, neglecting staircases in numerical models may conceal both changes in the global structural response and local damage mechanisms in members adjacent to the staircase [17].

A previous study by Ortega and Cagua [19] examined the combined influence of masonry infill walls and staircases on representative low-, mid-, and high-rise RC building typologies in Portoviejo. That study focused on changes in dynamic properties, including vibration periods and modes, and identified potential adverse effects such as short-column behavior, increased base shear, stress redistribution, and soft-story response. It also showed that the relative influence of these components decreases with building height and recommended nonlinear modeling for a more detailed assessment of seismic vulnerability. The present study addresses this need by extending the assessment of these typologies into the nonlinear range. Specifically, four configurations, namely bare frame (P), frame with staircases (P+S), frame with masonry infill walls (P+M), and frame with both components (P+M+S), are compared in terms of capacity curves, performance points for seismic demands associated with return periods of 475 and 2475 years, interstory drift profiles, and the localization and severity of damage in RC members and masonry infill panels.

In this context, the objective of this study is to comparatively evaluate how masonry infill walls and staircases modify the capacity and seismic performance of representative 3-, 6-, and 10-story RC framed buildings in Portoviejo through nonlinear static analysis. By characterizing the variation of global capacity, interstory drift, and local damage distribution among building-height typologies, the study seeks to support more informed assessment and strengthening decisions for existing buildings with similar configurations. The findings also provide technical evidence that may contribute to future refinements of local modeling and seismic-assessment practices within the NEC-SE-DS framework [5], particularly regarding components that modify the lateral response but are frequently omitted from conventional analytical models.

2. Material and Method

2.1 Structural Typologies Analyzed

The study considered three reinforced concrete framed-building typologies representative of the building stock in Portoviejo, Ecuador, corresponding to low-, mid-, and high-rise categories with 3, 6, and 10 stories, respectively. These typologies were defined based on recurring geometric, architectural, and structural characteristics documented through post-earthquake field observations and previous studies of buildings in Portoviejo following the April 16, 2016 earthquake [7,19].

The three typologies reproduce a common architectural layout in the city: a taller ground story with a lower density of masonry infill walls, generally associated with commercial use, and upper stories with a greater density of masonry infill panels associated with residential use. This condition generates a vertical discontinuity in stiffness and strength that may favor the development of soft-story mechanisms and the concentration of deformations at the first level [5], [7]. Uniform spans of $l_n = 4.0$ m were adopted in both principal directions as a controlled geometric assumption to reduce the number of variables in the comparative analysis, allowing the influence of masonry infill walls and staircases to be evaluated under consistent plan geometry. Consequently, plan irregularities, variable spans, and their associated torsional effects are not represented by the adopted archetypes and are outside the scope of the present comparative assessment. The ground-story height was $h_1 = 3.60$ m, whereas the upper stories had a constant height of $h_n = 3.00$ m. The staircase layout was established based on common construction configurations: in the low- and mid-rise buildings, the staircases were located near the interior façade, whereas in the high-rise typology they were placed closer to the structural core. The general plan configuration, masonry infill distribution, staircase location, and three-dimensional representation of each typology are shown in Fig. 1.

The main geometric, material, and reinforcement properties of the three structural typologies are summarized in Tables 1 and 2. Table 1 presents the building height, concrete compressive strength, column dimensions (b_c and h_c), and longitudinal reinforcement (A_s), whereas Table 2 presents the beam dimensions (b_b and h_b) and longitudinal reinforcement adopted for the corresponding story groups.

Table 1. Geometric, material, and column properties of the analyzed structural typologies

Typology	Height (m)	f'_c (MPa)	b_c (m)	h_c (m)	Columns	
					Longitudinal reinforcement (A_s)	
Low-rise	9.60	21	0.40	0.40	8 ϕ 16 mm	
Mid-rise	18.60	24	0.60	0.60	12 ϕ 20 mm	
High-rise	30.60	28	0.80	0.80	Corners	4 ϕ 22 mm
					Remaining bars	16 ϕ 20 mm

Table 2. Beam dimensions and longitudinal reinforcement of the analyzed structural typologies

Typology	b_b (m)	h_b (m)	Beams		
				Lower-story group	Upper-story group
Low-rise	0.30	0.40	Top	4 ϕ 14 mm	-
			Bottom	3 ϕ 14 mm	-
Mid-rise	0.35	0.45	Top	6 ϕ 16 mm	5 ϕ 16 mm
			Bottom	5 ϕ 16 mm	4 ϕ 16 mm
High-rise	0.45	0.60	Top	4 ϕ 20 mm	3 ϕ 20 mm
			Bottom	4 ϕ 20 mm	3 ϕ 20 mm

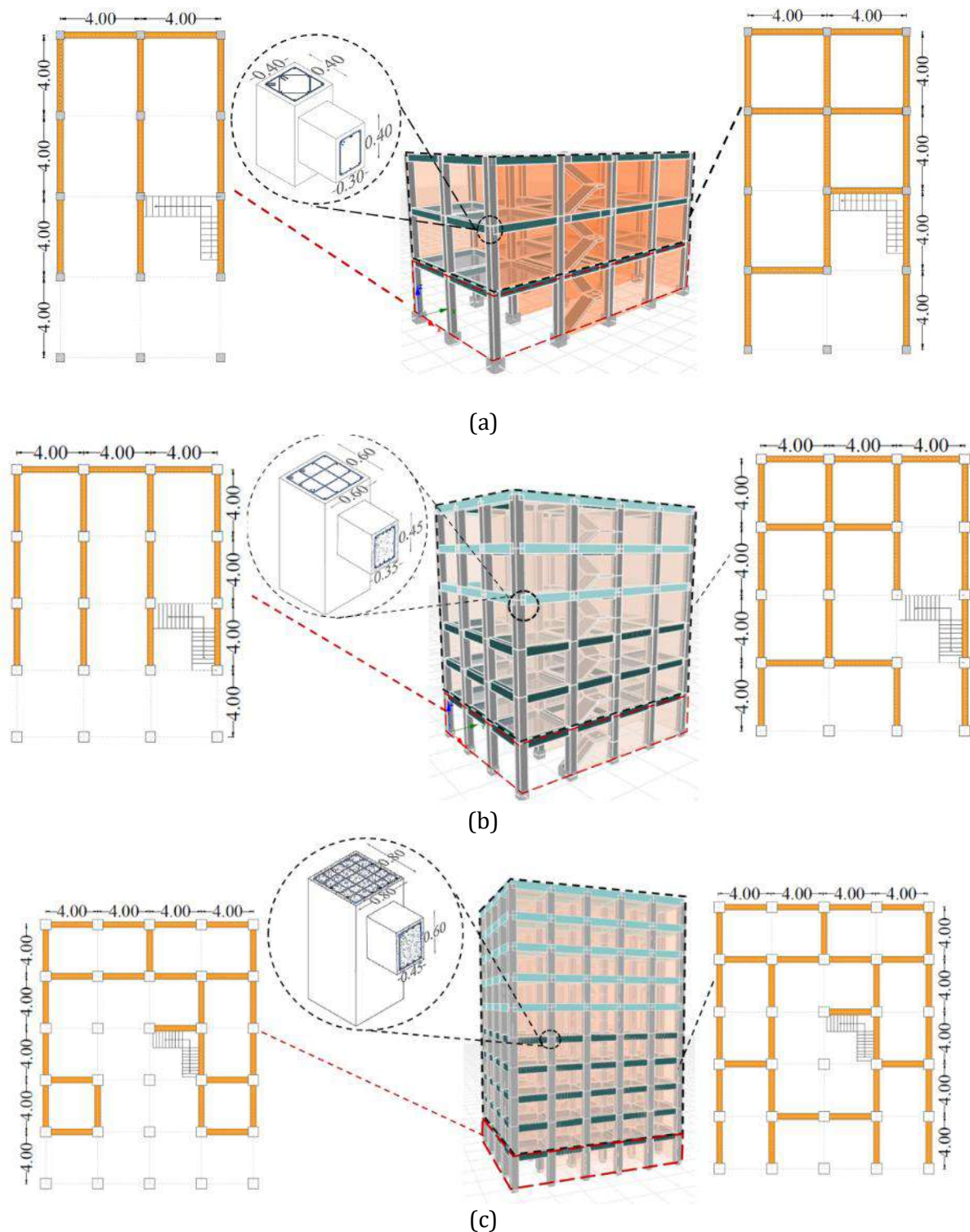


Fig. 1. Representative structural typologies adopted in the study: (a) low-rise building, (b) mid-rise building, and (c) high-rise building

A single beam reinforcement arrangement was adopted throughout the 3-story typology, whereas the 6- and 10-story typologies were divided into lower and upper story groups to represent the variation in reinforcement along the building height. A yield strength of 420 MPa was adopted for the reinforcing steel in all models, and transverse reinforcement in beams and columns consisted of 10 mm diameter stirrups spaced at 10 cm in the critical regions. The adopted story heights, vertical distribution of beam reinforcement groups, and reinforcement detailing of beams and columns are illustrated schematically in Fig. 2.

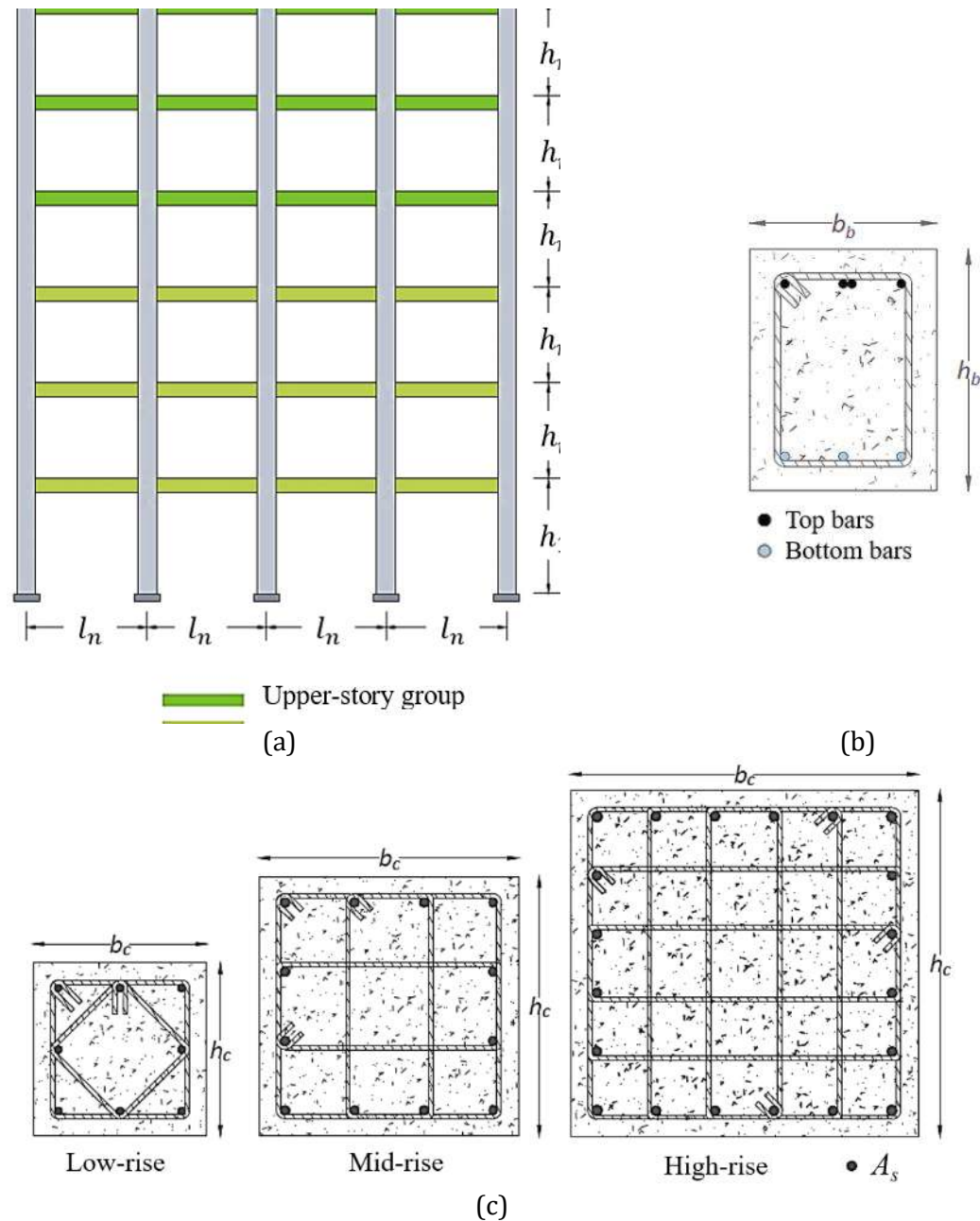


Fig. 2. Structural dimensions and reinforcement detailing adopted in the numerical models: (a) story heights and vertical distribution of beam reinforcement groups; (b) beam cross-section notation and longitudinal and transverse reinforcement; and (c) column cross-section notation and longitudinal reinforcement arrangements for the low-, mid-, and high-rise typologies

2.2 Structural Configurations and Numerical Modeling

For each typology, four structural configurations were evaluated, as shown in Fig. 3: frame (P), frame with staircases (P+S), frame with masonry infill walls (P+M), and frame with masonry infill walls and staircases (P+M+S). This comparison allowed the individual and combined effects of both components on lateral capacity, stiffness, ductility, and damage distribution to be identified.

Numerical modeling was performed in SeismoStruct. The reinforced concrete elements were represented using inelastic force-based beam-column elements with fiber sections. The constitutive model proposed by Mander et al. [20] was used for concrete, whereas the Menegotto and Pinto model [21] was adopted for reinforcing steel. Masonry infill walls were simulated using the nonlinear double-diagonal macromodel proposed by Crisafulli [22], which is widely used to represent the interaction between frames and infill panels [12,13].

To maintain consistency with the previously studied Portoviejo building typologies, the masonry input parameters were adopted from Ortega and Cagua [19]. The main properties considered were a panel thickness of $t = 0.15$ m, a masonry compressive strength of $f_m = 1.469$ MPa, and a diagonal-compression shear strength of $f_{ws} = 0.44$ MPa.

Staircases were explicitly incorporated into the structural model using inelastic force-based plastic-hinge frame elements (infrmFBPH), rather than shell elements or rigid plates. In this formulation, inelasticity is concentrated within prescribed plastic-hinge regions at the member ends, where the sectional response is evaluated through fiber integration, following the plastic-hinge integration approach proposed by Scott and Fenves [23]. The staircase elements were assigned the same concrete and reinforcing-steel constitutive models adopted for the reinforced-concrete frame members. Consequently, the nonlinear fiber-section formulation was also used to represent the inelastic response of the staircase members. When masonry infill walls or staircases were not explicitly modeled, their weight was included through equivalent loads to preserve the gravitational mass of the building and allow a consistent comparison among configurations.

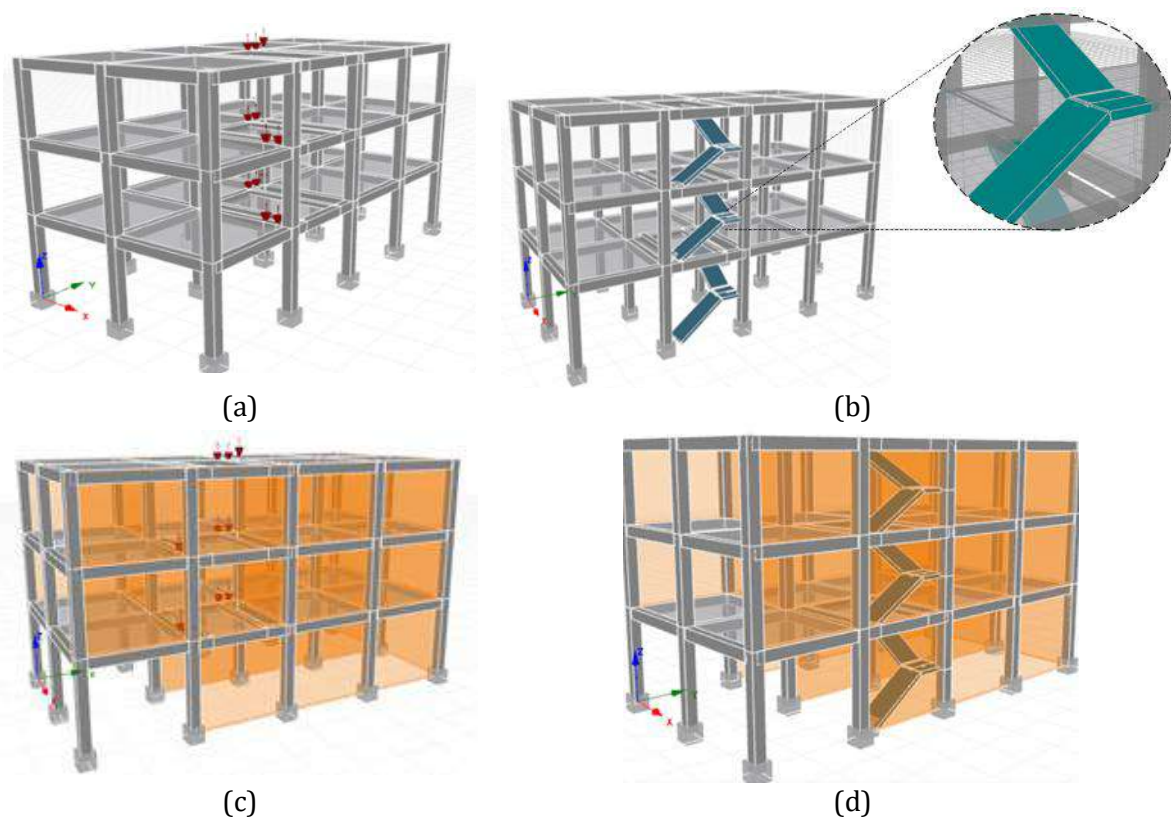


Fig. 3. Structural configurations analyzed: (a) P; (b) P+S; (c) P+M; (d) P+M+S

2.3 Seismic Demand

The seismic demand was defined for the city of Portoviejo according to NEC-SE-DS 2015, considering a high seismic hazard zone (VI), a zone factor $Z = 0.50$, an importance factor $I = 1.0$, and soil type D [5]. For the assessment, an elastic acceleration spectrum corresponding to a return period Tr of 475 years was used, together with a reference spectrum for $Tr = 2475$ years, obtained by scaling the design spectrum based on the site hazard curves, following the procedure applied by Ilbay (2025) [24]. Based on these spectra, the seismic demands associated with the three analyzed structural typologies were identified (Fig. 4). The seismic capacity of the buildings was evaluated through nonlinear static pushover analysis, applying an incremental lateral load pattern based on the first vibration mode. From the capacity curves, global parameters such as initial stiffness, base shear, ultimate displacement, ductility, and performance point were determined, following criteria commonly used in performance-based assessment [1-3].

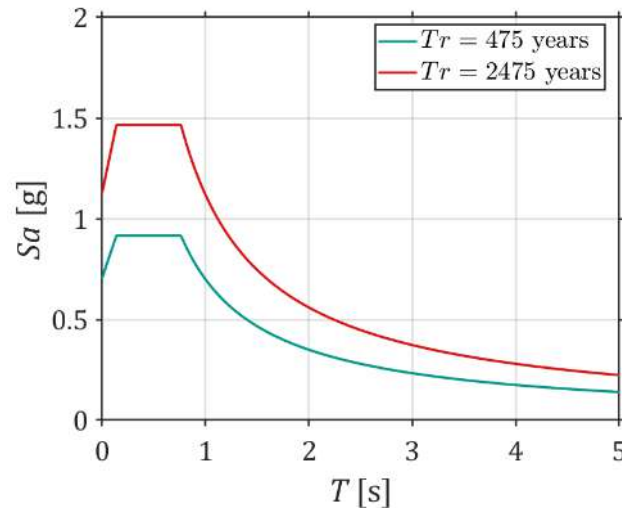
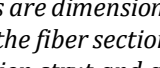
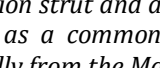


Fig. 4. Acceleration spectra considering soil type D, seismic zone VI, $Z = 0.50$, and $I = 1.0$, for Portoviejo

2.4 Performance and Damage Evaluation Criteria

The performance evaluation was carried out based on the performance point obtained for each typology and configuration, as well as on the global response parameters. In addition, the local damage distribution in the reinforced concrete elements and masonry infill panels were examined to identify differences in the response mechanisms according to building height and the presence of nonstructural elements. For the reinforced concrete elements, material strain limits associated with the behavior of concrete and steel were used, allowing damage in the main frame and in the structural elements of the staircase to be evaluated. The confined-concrete crushing limit of $\varepsilon_c = 0.008$ was adopted as a common damage-classification threshold for all typologies and was not calculated individually from the Mander model. This classification threshold is distinct from the confinement-dependent nonlinear concrete response represented by the constitutive model. For the masonry infill walls, dimensionless axial-strain thresholds of the equivalent compression strut were adopted to classify the equivalent damage states (Table 3). Together, these criteria made it possible to distinguish different damage levels in both the main lateral-load-resisting system and the infill panels, based on damage criteria reported in the literature.

Table 3. Criteria used for damage evaluation

Material	Legend	Description	Monitored quantity	Strain limit, ε (dimensionless)
Concrete		Unconfined concrete crushing	Concrete material strain	0.0035
		Confined concrete crushing		0.008
Steel		Steel yielding	Reinforcing-steel material strain	0.0025
		Steel fracture		0.1
Masonry		Slight damage	Equivalent-strut axial strain ε (-)	≤ 0.0015
		Moderate damage		0.0015 – 0.0030
		Extensive damage		0.0030 – 0.0060
		Complete damage		> 0.0060

Note: All strain limits are dimensionless. For concrete and reinforcing steel, the values correspond to material strains evaluated in the fiber sections. For masonry infill walls, the limits correspond to the axial strain of the equivalent compression strut and are distinct from interstory drift ratios. The confined-concrete limit of $\varepsilon_c = 0.008$ was adopted as a common damage-classification threshold for all typologies rather than being calculated individually from the Mander model.

3. Results and Discussion

Fig. 5 presents the capacity curves of the three analyzed typologies for the four structural configurations considered. In general terms, the inclusion of masonry infill walls and staircases modifies the global response, although their effect varies depending on the height of the structure. The seismic performance point was evaluated from the intersection between the structural capacity and the seismic demand corresponding to the design-level earthquake and the maximum considered earthquake, with return periods Tr of 475 and 2475 years, respectively. This allowed the variation in the structural response of each typology and configuration to be compared under different hazard levels.

Table 4 summarizes the main quantitative parameters derived from the capacity curves for all structural typologies and configurations. The values include the maximum base shear V_{max} , yield displacement Δ_y , performance-point displacement Δ_{pp} , and the corresponding displacement ductility demand μ_{pp} for $Tr = 475$ and 2475 years, together with the conservative interpretation limit Δ_{lim} . These parameters complement the graphical comparison presented in Fig. 5 and provide a direct basis for evaluating the influence of masonry infill walls and staircases on the global structural response.

In the low-rise building, the presence of masonry infill walls produced the greatest modification in the capacity curve, reflected in a marked increase in initial stiffness and lateral strength. As summarized in Table 4, V_{max} increased from 1104.1 kN for the bare frame (P) to 1247.6 and 1351.5 kN for P+M and P+M+S, corresponding to increases of approximately 13.0% and 22.4%, respectively. In parallel, Δ_y decreased from 0.040 m for P to 0.018 m for P+M and 0.022 m for P+M+S. This response was observed in both the P+M and P+M+S configurations, indicating that, in this type of building, masonry infills actively participate in the lateral behavior of the system. For $Tr = 475$ years, the corresponding displacement ductility demand, μ_{pp} , decreased from 2.46 for P to 1.87 for P+M and 1.51 for P+M+S, indicating a lower deformation demand relative to yielding in the infilled configurations at the design-level performance point. The exclusive incorporation of staircases (P+S) also produced an increase in stiffness with respect to the frame configuration (P), although to a lesser extent than that produced by the masonry infill walls.

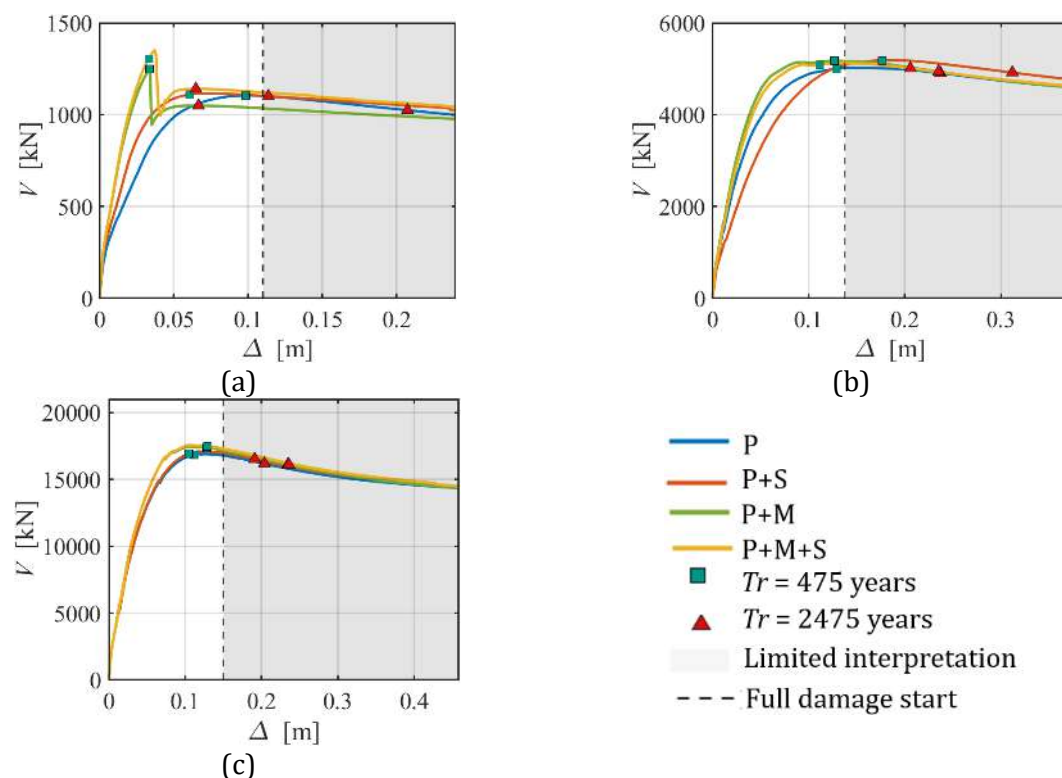


Fig. 5. Capacity curves in the X direction for the analyzed typologies: (a) low-rise building, (b) mid-rise building, and (c) high-rise building

In the mid-rise typology, the effect of masonry infill walls and staircases remained appreciable, although it was less pronounced than in the 3-story building. The configurations with masonry infill walls (P+M and P+M+S) exhibited steeper initial slopes and higher strength values than the frame model (P), but with more moderate differences. This behavior suggests that, as building height increases, the relative contribution of nonstructural elements to the global response decreases, although they still modify the lateral capacity and damage evolution. In the high-rise typology, the capacity curves of the different configurations showed smaller differences among them, indicating a lower global influence of masonry infill walls and staircases on the building response. In this case, the behavior was mainly governed by the flexibility of the resistance frame. However, although the global effect was smaller, this does not imply that the presence of these elements is irrelevant from a local perspective, since their influence may be reflected in the redistribution of internal forces and the concentration of damage in specific regions.

Table 4. Summary of capacity-curve parameters and performance-point demands

Typology	Config.	$V_{\max}$ (kN)	Δ_y (m)	$\Delta_{pp,475}$ (m)	$\mu_{pp,475}$	$\Delta_{pp,2475}$ (m)	$\mu_{pp,2475}$	Δ_{lim} (m)
Low-rise	P	1104.116	0.040	0.098	2.46	0.207†	5.18†	0.110
	P+S	1115.749	0.028	0.061	2.17	0.114†	4.06†	0.110
	P+M	1247.556	0.018	0.034	1.87	0.067	3.70	0.110
	P+M+S	1351.509	0.022	0.033	1.51	0.065	2.95	0.110
Mid-rise	P	5024.919	0.051	0.129	2.53	0.237†	4.65†	0.138
	P+S	5185.819	0.076	0.177†	2.32†	0.312†	4.10†	0.138
	P+M	5175.364	0.043	0.127	2.95	0.235†	5.47†	0.138
	P+M+S	5120.986	0.046	0.112	2.43	0.206†	4.47†	0.138
High-rise	P	16909.013	0.051	0.112	2.20	0.204†	4.00†	0.150
	P+S	17104.651	0.055	0.105	1.91	0.191†	3.47†	0.150
	P+M	17477.018	0.050	0.128	2.56	0.234†	4.69†	0.150
	P+M+S	17577.855	0.052	0.129	2.48	0.236†	4.54†	0.150

Note: Δ_y denotes the yield displacement; Δ_{pp} denotes the roof displacement at the performance point; and $\mu_{pp} = \Delta_{pp}/\Delta_y$ denotes the displacement ductility demand at the corresponding performance point. Δ_{lim} denotes the conservative roof-displacement limit adopted to delimit the range of physical interpretation of the capacity curves and is not interpreted as an ultimate or collapse displacement. † Performance-point values beyond Δ_{lim} should be interpreted with caution.

Overall, masonry infill walls showed a more significant influence than staircases on initial stiffness and strength, especially in the low- and mid-rise typologies. Nevertheless, staircases also modify the structural response and should be considered in the numerical model. Although the capacity curves are presented up to the maximum displacement reached by the numerical procedure the response beyond the conservative global roof-displacement limits adopted for each typology should be interpreted with caution. In this study, the global collapse of the structure was not explicitly modeled; therefore, the final branch of the curve mainly reflects the numerical continuity of the analysis and does not necessarily represent a physically reliable structural capacity from a performance-based perspective. For this reason, conservative roof-displacement limits, denoted Δ_{lim} , were adopted to delimit the range of physical interpretation of the capacity curves. Values of 0.110, 0.138, and 0.150 m were adopted for the low-, mid-, and high-rise typologies, respectively, corresponding to global roof drift ratios of approximately 1.15%, 0.74%, and 0.49%. These limits do not represent ultimate or collapse displacements; beyond Δ_{lim} , the response is retained only as a numerical reference and should be interpreted with caution. This interpretation is consistent with Fig. 6, corresponding to the design-level earthquake ($Tr = 475$ years), where local damage is mainly concentrated in beams and columns of the lower stories, with a predominance of steel yielding and unconfined concrete crushing. Fig. 6 shows that, in the low-rise typology, the configurations with masonry infill walls reduce the overall extent of deformation, although critical zones remain at the base. In the mid-rise typology, the most marked concentration of damage is identified at the first level, whereas in the high-rise typology, damage is distributed over a larger portion of the height, with less pronounced differences among configurations.

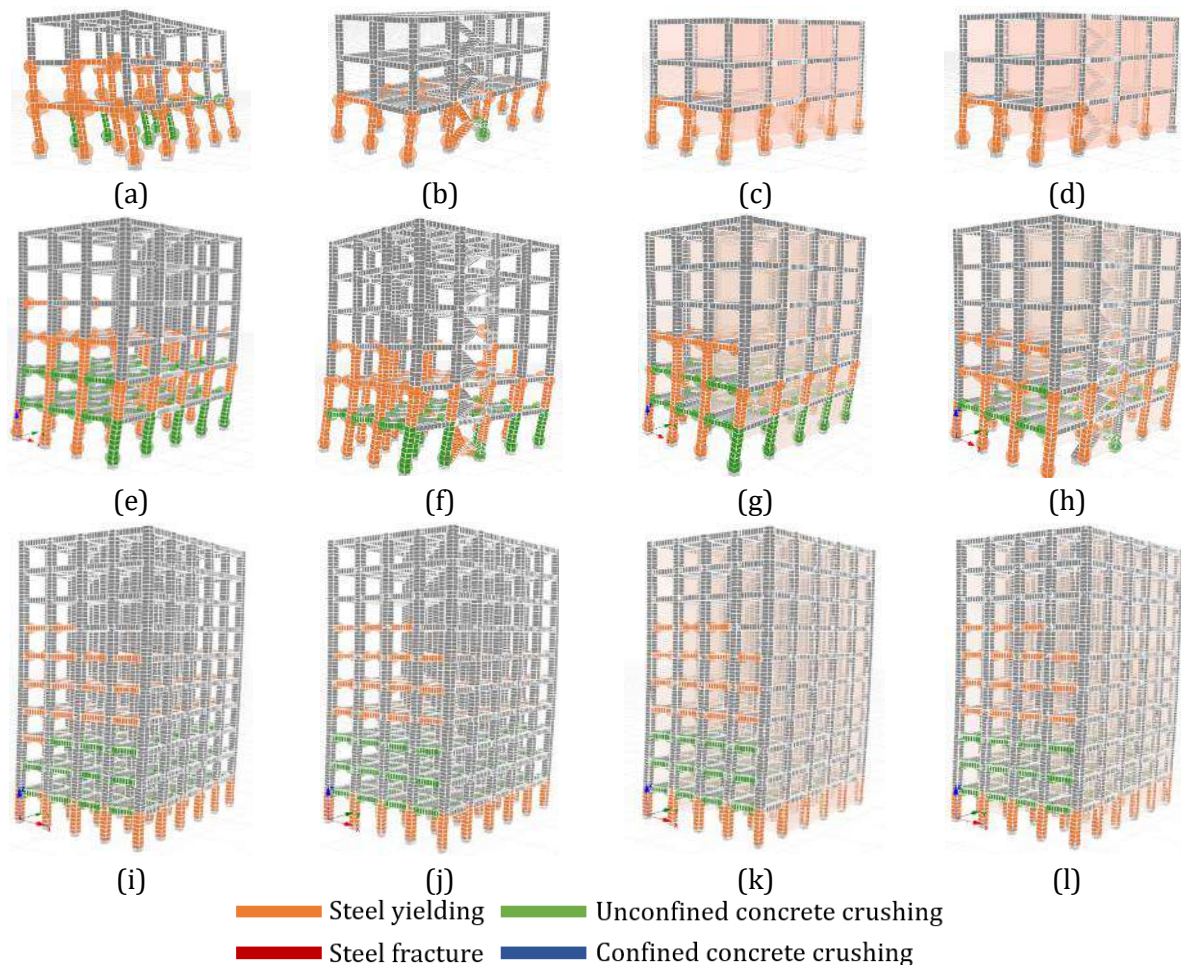


Fig. 6. Local damage distribution in beams and columns ($T_r = 475$ years and pushover analysis in the X direction). Low-rise typology: (a) P, (b) P+S, (c) P+M, and (d) P+M+S. Mid-rise typology: (e) P, (f) P+S, (g) P+M, and (h) P+M+S. High-rise typology: (i) P, (j) P+S, (k) P+M, and (l) P+M+S

Fig. 7 shows the spatial distribution of local damage in reinforced concrete structural elements for the extreme demand scenario ($T_r = 2475$ years). Compared with the design-level earthquake, an increase is observed in both the extent and severity of damage, particularly in the mid- and high-rise typologies, where damage states associated with confined concrete crushing appear more clearly in the lower stories. In the low-rise typology, although damage continues to be mainly concentrated at the first level, the configurations with masonry infill walls maintain a lower overall dispersion of deformation compared with the bare frame. In the mid-rise typology, damage accumulation in columns and beams at the first level becomes more severe, indicating a pronounced concentration of seismic demand. This response is further examined through the interstory drift profiles presented in Fig. 8. In the high-rise typology, the global differences among configurations remain more moderate; however, the increase in damage in the lower half of the building shows that, even when the global influence of masonry infills and staircases decreases with height, their local effects remain relevant. To complement the local damage distributions, Fig. 8 presents the interstory drift ratio (IDR) profiles along the building height for the four structural configurations and both seismic demand levels. The results show a clear concentration of deformation demand in the lower stories of the low- and mid-rise typologies. In the mid-rise building, the largest IDRs occur within the first two stories and decrease markedly toward the upper levels, particularly for $T_r = 2475$ years. This response is consistent with the concentration of beam and column damage observed in Fig. 6 and Fig. 7 and supports the identified tendency toward soft-story-type behavior. In contrast, the high-rise typology exhibits a more distributed drift profile over the building height, although the largest demand remains concentrated in its lower portion.

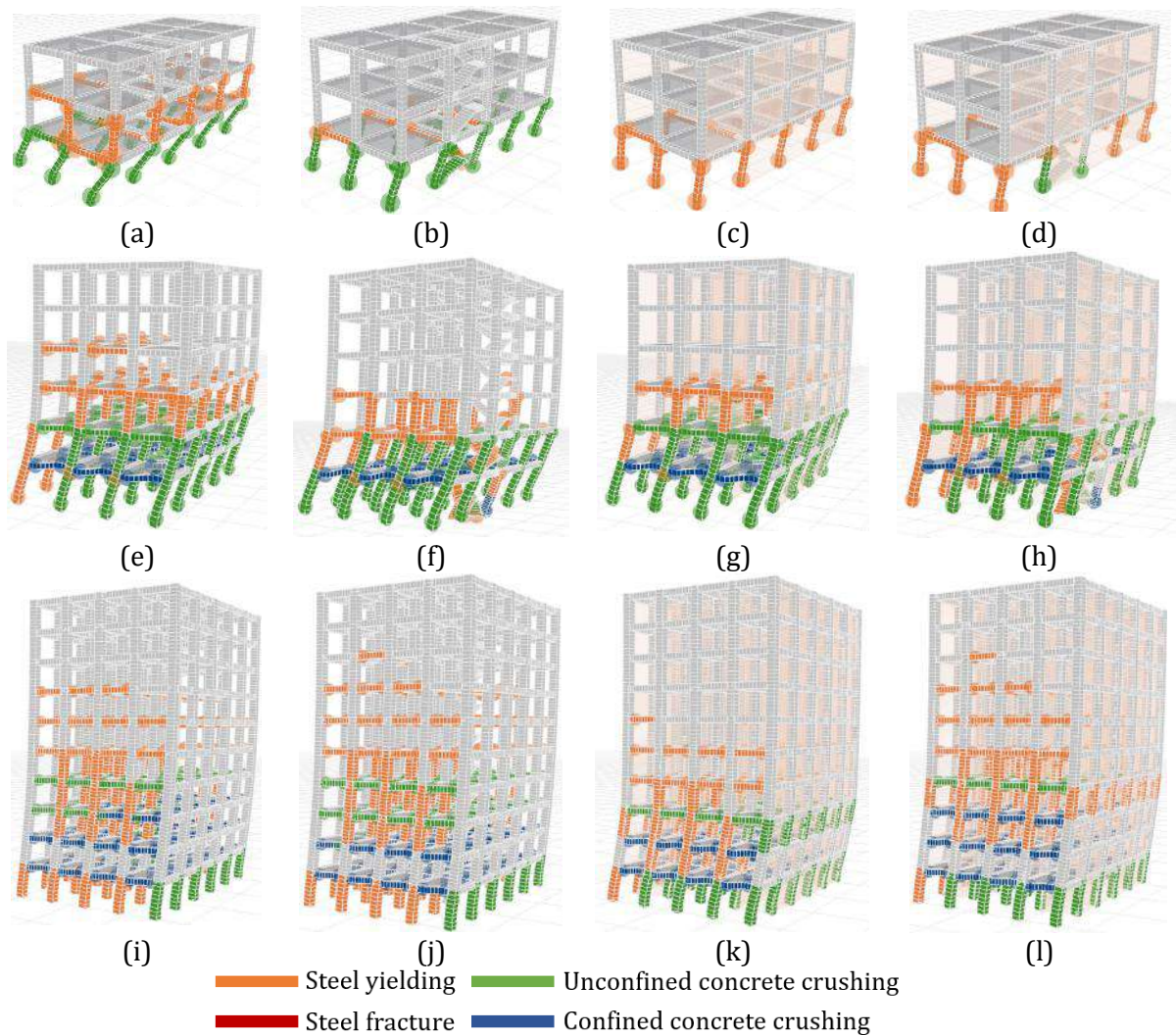


Fig. 7. Local damage distribution in beams and columns ($T_r = 2475$ years and pushover analysis in the X direction). Low-rise typology: (a) P, (b) P+S, (c) P+M, and (d) P+M+S. Mid-rise typology: (e) P, (f) P+S, (g) P+M, and (h) P+M+S. High-rise typology: (i) P, (j) P+S, (k) P+M, and (l) P+M+S

These IDR profiles therefore provide a quantitative counterpart to the spatial damage patterns and clarify the height-dependent distribution of seismic deformation demand. Fig. 9 shows that damage in the masonry infill walls is mainly concentrated in the lower stories, where the largest interstory deformations also develop. When moving from the design-level earthquake ($T_r = 475$ years) to the extreme earthquake ($T_r = 2475$ years), both the extent and severity of damage increase, with a transition from predominantly slight and moderate damage states toward a greater presence of extensive and complete damage. This pattern is consistent with the vertical stiffness irregularity of the analyzed typologies, which are characterized by a more flexible ground story and greater continuity of panels in the upper stories. This interpretation is also supported by the IDR profiles in Fig. 8, which show lower interstory deformation demands in the upper stories than in the lower portion of the low- and mid-rise buildings. This analytical response is consistent with post-earthquake observations reported for Portoviejo after the 2016 Ecuador earthquake, where reinforced-concrete buildings with taller and more open ground floors associated with commercial use, together with a reduced presence of masonry infill walls at the first level, exhibited increased vulnerability to deformation and damage concentration in the lower stories [7]. The damage distribution shown in Fig. 9 indicates that, in the low-rise typology, masonry infill walls contribute significantly to global stiffness, limiting the overall deformation of the building; however, the most severe damage is concentrated in the panels at the first level. For $T_r = 475$ years, slight and moderate damage predominates in the upper stories, whereas extensive and complete damage appears at the base; for $T_r = 2475$ years, this concentration intensifies.

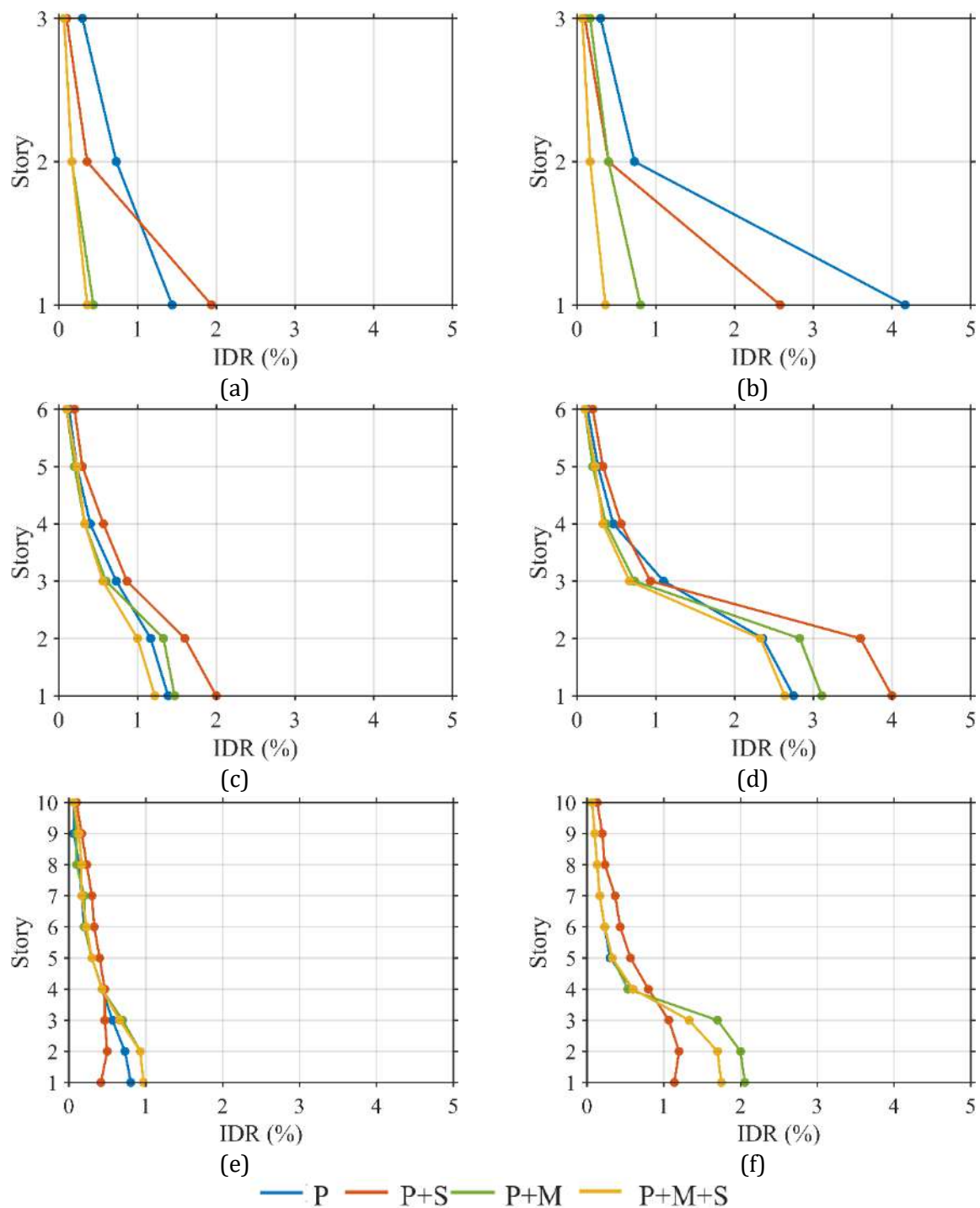


Fig. 8. IDR profiles in the X direction for the four structural configurations under the design-level and maximum-considered seismic demands: low-rise building at (a) $Tr = 475$ years and (b) $Tr = 2475$ years; mid-rise building at (c) $Tr = 475$ years and (d) $Tr = 2475$ years; and high-rise building at (e) $Tr = 475$ years and (f) $Tr = 2475$ years

In the mid-rise typology, the most critical response is observed: even for $Tr = 475$ years, a continuous band of severe damage appears in the lower stories, which extends and becomes more severe for $Tr = 2475$ years, confirming the pronounced concentration of deformation demand in the lower stories and supporting the identified soft-story-type response. In the high-rise typology, the relative contribution of masonry infill walls to global behavior is lower, but their local effect remains relevant, since damage is concentrated in the lower part of the building and becomes more continuous under the extreme demand. To complement the spatial damage distribution, Fig. 10 quantifies the masonry response as the percentage of infill panels reaching or exceeding each damage-state axial-strain threshold.

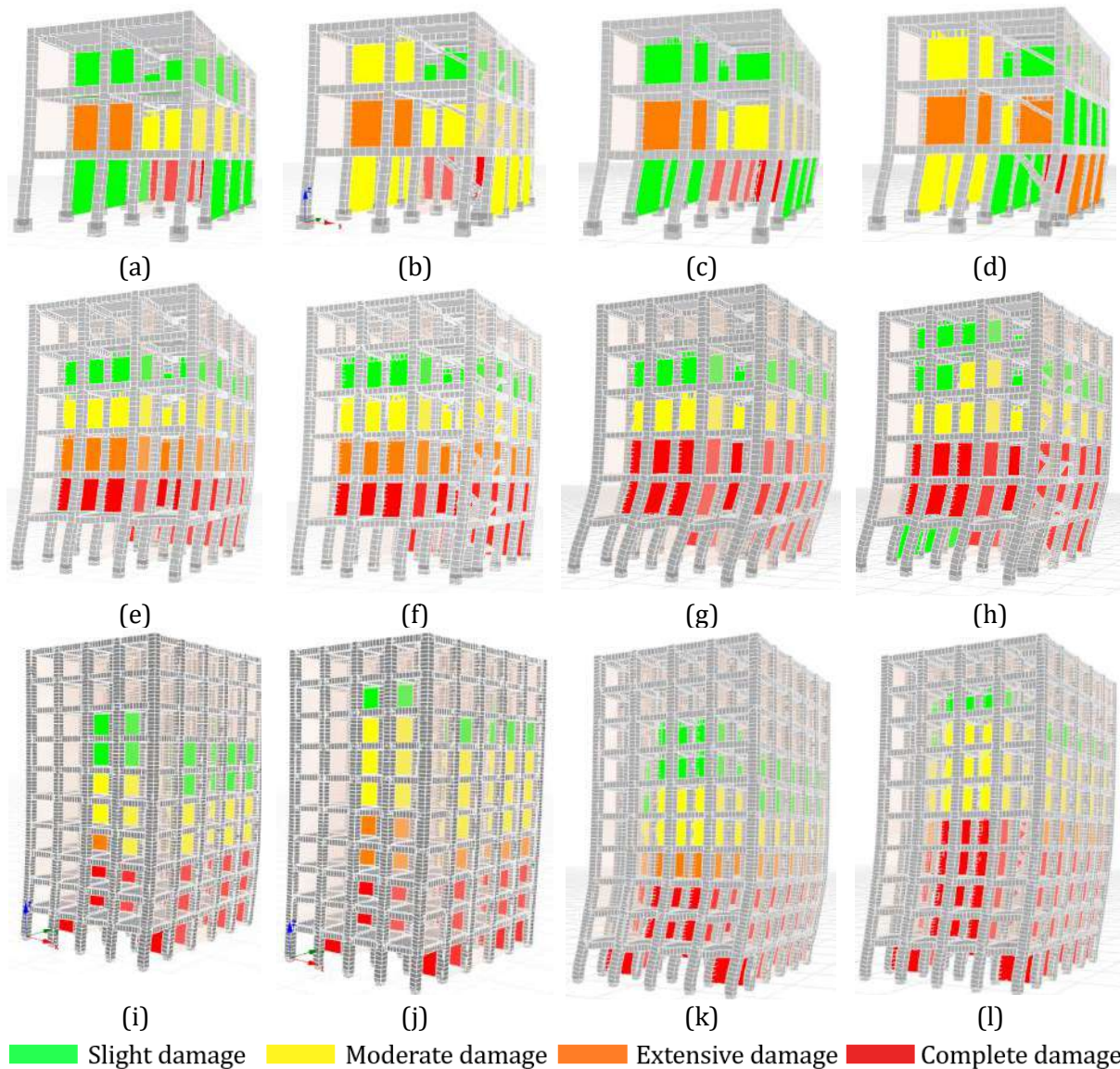


Fig. 9. Damage localization and severity in masonry infill walls. Low-rise typology: (a) P+M and (b) P+M+S for $T_r = 475$ years; (c) P+M and (d) P+M+S for $T_r = 2475$ years. Mid-rise typology: (e) P+M and (f) P+M+S for $T_r = 475$ years; (g) P+M and (h) P+M+S for $T_r = 2475$ years. High-rise typology: (i) P+M and (j) P+M+S for $T_r = 475$ years; (k) P+M and (l) P+M+S for $T_r = 2475$ years

The percentages were calculated with respect to the total number of infill panels in each typology: 39 panels for the low-rise building, 120 for the mid-rise building, and 314 for the high-rise building. The comparison was performed for both $T_r = 475$ and 2475 years. The results provide a quantitative counterpart to Fig. 9 and show how the proportion of panels reaching progressively more severe damage thresholds changes with building height, structural configuration, and seismic demand.

The comparison among configurations indicates that the incorporation of staircases does not eliminate the concentration of damage at the base, although it modifies its spatial distribution, in some cases making it more dispersed or increasing the proportion of panels with moderate and extensive damage. In the P+S and P+M+S configurations, the staircase-frame interaction modifies the local response of the members adjacent to the intermediate landings. The staircase flights and landings restrain the deformation of the connected column segments, effectively reducing their deformable length and promoting a localized concentration of demand. This behavior is consistent with a short-column-type response, expressed in the present models through changes in the local damage distribution around the staircase region. Accordingly, the influence of the staircase is interpreted primarily as a local response mechanism rather than as a dominant contributor to the global capacity of the building.

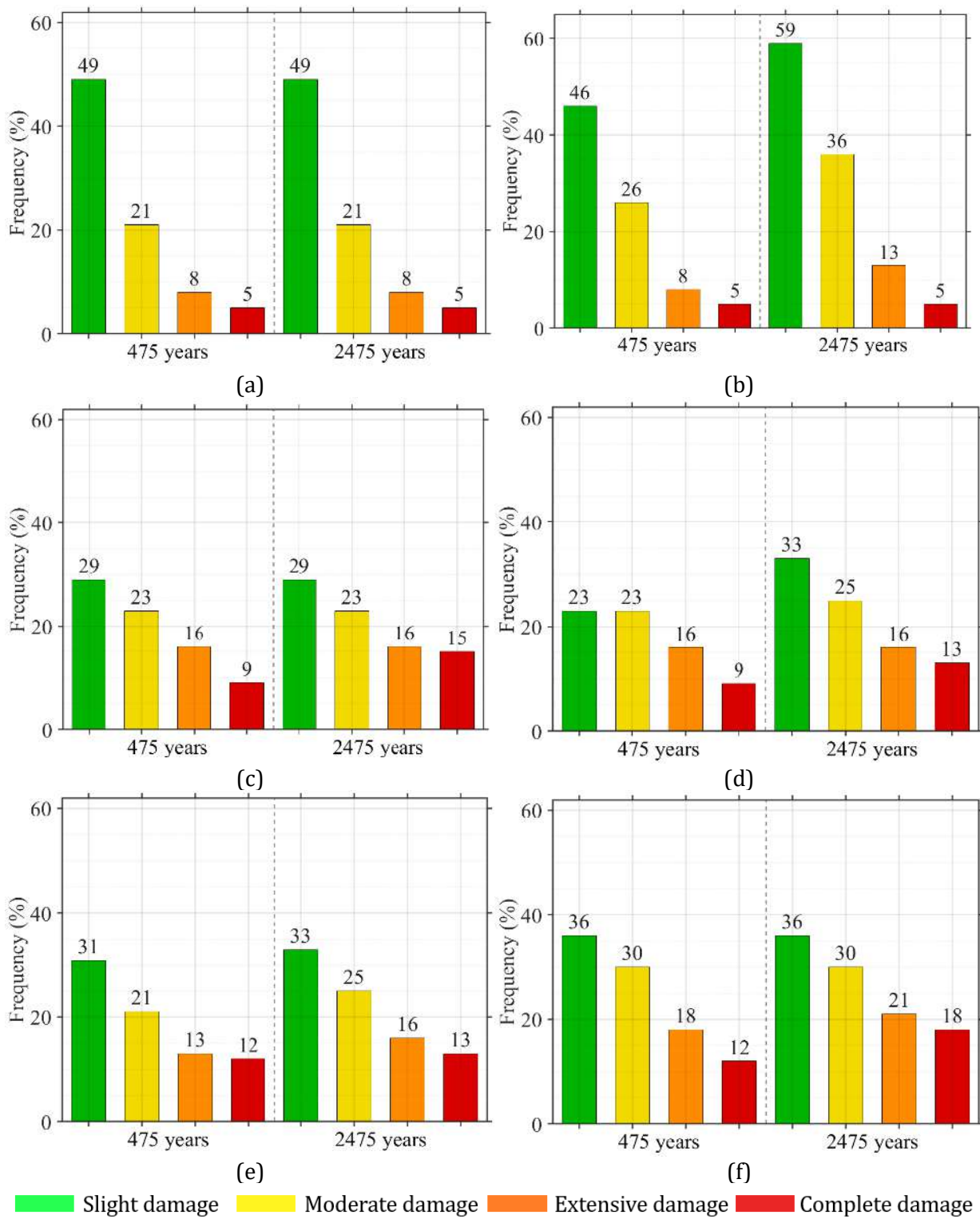


Fig. 10. Percentage of masonry infill panels reaching or exceeding each damage-state axial-strain threshold for $Tr = 475$ and 2475 years: (a) low-rise P+M; (b) low-rise P+M+S; (c) mid-rise P+M; (d) mid-rise P+M+S; (e) high-rise P+M; and (f) high-rise P+M+S. Percentages are referred to the total number of infill panels in each typology (39, 120, and 314 panels for the low-, mid-, and high-rise buildings, respectively)

4. Conclusions

The inclusion of masonry infill walls and staircases significantly modified the seismic response of the analyzed buildings, although their influence depended on building height. The results indicate that masonry infill walls constitute a non-negligible component in the seismic performance

assessment of the analyzed buildings, particularly because their contribution modifies the initial stiffness and lateral strength of the system. Their influence is more pronounced than that of the staircases at the global response level, whereas staircases, despite their smaller global contribution, remain relevant to the local damage distribution and frame response.

In the low-rise typology, masonry infill walls produced the greatest increase in stiffness and strength, reducing the demanded global displacements and improving the structural performance under the design-level earthquake. Quantitatively, the maximum base shear increased by approximately 13.0% for P+M and 22.4% for P+M+S relative to the bare-frame configuration. However, this benefit was accompanied by a reduction in deformation capacity and by the persistence of critical damage zones at the first level, indicating that the reduced global deformation demand does not necessarily imply lower local vulnerability, since the stiffness discontinuity and infill-frame interaction may concentrate force demands, including shear, in the ground-floor columns.

The mid-rise typology exhibited the most critical behavior with respect to the interaction between the frame, masonry infill walls, and staircases. The concentration of beam and column damage in the lower stories, together with the IDR profiles showing the largest deformation demands within the first two stories, identifies the mid-rise typology as the most susceptible among the analyzed cases to the development of a lower-story deformation mechanism under the adopted seismic demands. This response is associated with the vertical stiffness irregularity of the building and is consistent with soft-story-type behavior.

In the high-rise typology, the global differences among configurations were smaller, indicating that the response was mainly governed by the flexibility of the resisting frame. Nevertheless, under high levels of seismic demand, local effects on the damage distribution persisted, especially in the lower half of the building, confirming that the influence of masonry infilling walls and staircases does not completely disappear with height. In particular, the staircase-frame interaction locally modifies the stiffness distribution and seismic load path, leading to redistribution of force and deformation demands even when its influence on the global capacity is comparatively limited.

Overall, the results show that omitting masonry infill walls and staircases may lead to an unrealistic representation of the seismic behavior of typical building stock in Portoviejo, especially in low- and mid-rise structures. Therefore, for seismic safety assessments of existing mid-rise RC buildings in Portoviejo with architectural and structural characteristics similar to those analyzed herein, the explicit consideration of masonry infill walls and staircases in the numerical model is recommended within the NEC-SE-DS assessment framework, particularly when evaluating lower-story deformation and damage concentration.

The findings are limited to the three representative typologies, structural configurations, and nonlinear static analysis framework considered in this study. Future research may extend this assessment through nonlinear dynamic analyses and a broader range of building geometries and infill/staircase configurations to evaluate the generality of the observed response trends.

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Synergistic effects of magnesium oxide nanoparticles on B30 waste cooking oil biodiesel: Performance optimization and artificial neural network modeling

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Abstract

The growing demand for clean energy has intensified interest in alternative fuels like biodiesel. In this work, waste cooking oil (WCO) was utilized as a low-cost, sustainable feedstock for biodiesel production. The produced biodiesel was tested in a two-cylinder diesel engine operating at 1500 rpm under various load conditions, focusing on the B30 blend. To enhance performance, magnesium oxide nanoparticles (nano-MgO) were added to the B30 blend at concentrations of 10, 30, and 60 ppm. Experimental results demonstrated that the B30 blend provided balanced engine performance. The incorporation of nano-MgO enhanced combustion characteristics, leading to improved brake thermal efficiency by (3.52% increase) and lower brake-specific fuel consumption by (11% reduction) with the use of B30 + 60 ppm MgO, compared with untreated biodiesel. Emission analysis revealed a noticeable decline in major regulated pollutants (including a 30% reduction in CO, 26% reduction in HC, 24.3% reduction in NO_x, and 33% reduction in smoke opacity with B30 + 60 ppm MgO) driven by the catalytic influence of nano-MgO, which promotes oxygen availability for complete combustion. Additionally, a multi-output artificial neural network (ANN) based on Bayesian regularization was developed to predict engine performance and emissions. The model achieved high predictive accuracy, with an average test coefficient of determination (R^2) of approximately 0.9439 across all outputs. Parity and sample-based assessments confirmed its robustness under different operating conditions, validating the developed ANN as an efficient tool for performance prediction and optimization.

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1. Introduction

Modern societies rely heavily on diverse energy sources to sustain economic growth. Because fossil fuels are not renewable, their reserves are running out. Fossil fuels are also linked to environmental and ecological problems, such as global warming. Non-renewable conventional energy resources are still predominantly used to meet countries' energy demands. Therefore, finding suitable substitutes for these conventional fuels is critically important.

Renewable energy is defined as energy that mostly comes from natural sources, such as the sun, biomass, wind, geothermal, tidal energy, and hydropower. The ideal alternative to fossil fuels is biomass because it is renewable, produces fewer emissions (such as carbon dioxide, carbon monoxide, nitrogen oxides, sulfur oxides, smoke, etc.), and is less expensive than traditional fossil fuels. The usage of these alternative energies is currently being emphasized in several nations [1]. However, biomass is available in diverse forms and can be stored and used at any time of the year for energy production. The seasonality of wind and solar energy sources can be lessened by using biomass as a hybrid supplement. Fuels produced from organic materials, such as animal waste and

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plant matter, are known as biofuels. Biofuels and other alternative fuels can produce electricity, heat houses, and power vehicles. Seeds, kernels, leaves, agricultural leftovers, sewage waste, forest residue, human waste, and animal excrement are a few examples of the different types of biomass feedstocks. Biomass is therefore a sustainable and renewable source of green energy [2]. Because they can lower greenhouse gas emissions and the need for fossil fuels, biofuels are being researched extensively as an alternative energy source. However, their widespread use endangers biodiversity, land use, and food security. Among the feedstocks utilized in biofuels are intensive agricultural inputs, which have been connected to environmental deterioration and deforestation. They can also compete with food crops, potentially escalating prices and worsening food insecurity [3]. To mitigate these sustainability concerns, waste cooking oil (WCO) has emerged as a promising, non-edible alternative. Among the diverse production pathways for converting such feedstocks, transesterification stands out as the most commercially viable method. Within this process, heterogeneous catalysts are systematically preferred over their homogeneous counterparts due to their excellent recyclability and significantly mitigated corrosion risks. Biodiesel and petroleum-based diesel have many physical and chemical properties. Biodiesel is made from biodegradable, reusable, and low-toxicity biomass feedstock. It can be used in blended or raw form, with or without mechanical modifications, in diesel engines that are currently in use. Biodiesel improves engine performance because it emits less carbon monoxide and particulate matter, extends an engine's operating life due to its lubricating properties, and is less volatile due to its higher boiling point, making it easier to transport and store [4]. Its higher flash point also improves handling and storage safety. Biodiesel is often considered carbon-neutral because the CO₂ released during combustion is approximately balanced by the CO₂ absorbed during biomass growth. Despite these advantages, the use of neat biodiesel often leads to a notorious 'biodiesel NO_x penalty' due to its inherent oxygen-rich structure which promotes higher peak flame temperatures during the premixed combustion phase. This thermal trade-off necessitates the development of effective mitigation strategies, such as fuel-borne nano catalysts.

On the other hand, utilizing biodiesel in traditional diesel engines has several disadvantages, such as poor performance in cold weather, ineffective fuel atomization, piston ring sticking, relatively high density, and a drop in fuel economy. However, these drawbacks can be mitigated using high-energy nanoadditives, which are added in trace amounts to improve combustion efficiency and fuel stability [5]. Such additives offer advantages including low dosage requirements, corrosion resistance, reusability, as well as improved sustainability. Researchers looked into how nanoparticles affected the characteristics of biodiesel. Among the nanoparticles that have been evaluated for their ability to enhance fuel characteristics and engine performance are carbon nanotubes, titanium oxide, magnesium oxide, zinc oxide, graphene, cerium oxide, and aluminum oxide [6-8] as well as other hybrid carbon-based materials [9]. Specifically, zinc oxide nanoparticles have demonstrated remarkable potential in improving thermal performance and reducing hazardous smoke emissions [10]. Additionally, Nachippan, Balaji [11] demonstrated the effectiveness of nano-MgO in enhancing the performance of cottonseed biodiesel. While carbon-based and metal-oxide nanoadditives have been widely reported to enhance brake thermal efficiency (BTE) as well as reduce harmful emissions over various engine configurations [12-14], magnesium oxide (MgO) nanoparticles offer distinct physicochemical advantages over other metal oxides. Compared with cerium oxide, magnesium oxide can be produced more easily and sustainably [15-18]. Zinc oxide is another studied nanoadditive [19-21]; however, its more complicated processing requirements make MgO nanoparticles a more attractive option for biodiesel applications due to production efficiency, cost-effectiveness, and environmental compatibility. The specific catalytic mechanism of nano-MgO relies on its behavior as an effective oxygen-donating catalyst with a basic surface and high surface-area-to-volume ratio, which promotes critical interfacial fuel processes, enhances atomization, and improves overall combustion efficiency. This catalytic activity significantly lowers carbon monoxide emissions by promoting more complete oxidation of hydrocarbon molecules, converting CO to CO₂, while its coupling effect in transesterification highlights its stability as a heterogenic nano catalyst. Furthermore, nano-MgO significantly influences the heat transfer and thermal conductivity within the combustion chamber. Due to its high inherent thermal stability, the addition of MgO nanoparticles improves the in-cylinder thermal conductivity and controls the combustion

parameters. This enhanced thermal conductivity reduces peak in-cylinder temperatures and alters heat transfer characteristics, leading to a noticeable decline in thermal nitrogen oxide (NO_x) emissions. Previous literature has demonstrated the capabilities of nano-MgO in distinct configurations; for instance, Nachippan et al. [11] successfully proved the effectiveness of nano-MgO in enhancing the performance and emission characteristics of cottonseed biodiesel. Similarly, Farouk, Tayeb [22] established that synthesis via MgO nano catalysts results in enhanced engine performance and reduced exhaust emissions.

Despite these extensive studies, limited research has focused on the synergistic effects of MgO nanoparticles in B30 waste cooking oil biodiesel, particularly considering combined performance, emission, and closed-loop circular economy sustainability aspects. To contextualize these research gaps, Table 1 evaluates various carbon-based and metal-oxide nanoadditives widely reported to enhance brake thermal efficiency (BTE) and reduce harmful emissions. However, while candidates like GO, Al₂O₃, and TiO₂ improve engine parameters, they are often constrained by high synthesis costs, chemical complexity, or environmental toxicity risks. In contrast, magnesium oxide (MgO) nanoparticles offer a superior alternative, effectively addressing these limitations through excellent cost-effectiveness, environmental friendliness, and ease of production. These findings further support the strategic selection of MgO nanoparticles in the present investigation, as MgO exhibits comparable catalytic activity and thermal stability, especially when applied to B30 waste cooking oil biodiesel.

Table 1. Performance and emissions of diesel and biodiesel fuels mixed with nanoadditives based on carbon and metal oxides

Nanoadditive vs	Fuel Blend	Engine Type	Main Performance Effects	Emission Reduction	Reference
CeO ₂	B10+3% water+100 ppm CeO ₂	Single-cylinder diesel	(4.5-8%) BTE	(12 -25%) CO, (10 -20%) NO _x	[15]
GO + Butanol	B25(Nigell) + 90 ppm GO	CRDI diesel	(3-6.5%) BTE	(15 -30%) CO, (8 -18%) NO _x , (12 -25%) Smoke	[12]
CoO	B20(Waste oil)+50ppm CoO	Urea-SCR diesel	(3.5-7%) BTE	(10 -24%) CO, (12 -22%) NO _x	[17]
Al ₂ O ₃	B20(Waste Oil)+ 100 ppm Al ₂ O ₃	CRDI diesel	(4-8.5%) BTE	(12 -28%) CO, (10 -19%) NO _x	[19]
CNT	B20(Palm) + 50 ppm CNT	Single-cylinder DI diesel	(5-9.5%) BTE, improved combustion	(15 -32%) CO, (14 -28%) HC, (18 -35%) Smoke	[23]
GO	B20(Soybe) +100 ppm GO	VCRsingle-cylinder diesel	(3.5-7.2%) BTE	(15 -27%) Smoke, (14-30%)CO, (9 -20%)NO _x ,	[13]
GNP	B20 (Jatropha)+ 75 ppm GNP	Turbochargd DI engine	(4-8%) BTE, faster ignition	(18 -33%)CO, (10 -22%)NO _x	[24]
ZnO	B25(Soybe) + 50 ppm ZnO	VCRsingle-cylinder	(3.8-7.5%) BTE	(12 -26%) CO, (11 -21%) NO _x , (14 -30%) Smoke	[25]
TiO ₂	B30+75ppm TiO ₂	Single-cylinder DI	(3.2-6.8%) BTE	(16 -31%) HC, (20 -38%) Smoke	[26]

Due to the highly non-linear, coupled, and multivariable nature of combustion and exhaust emission chemistry in compression ignition engines under varying loads, traditional thermodynamic and analytical modeling approaches often fall short in real-time predictions.

Consequently, black-box modeling tools, especially artificial neural networks (ANNs), have emerged as highly efficient digital surrogates. Process models are generally classified as either linguistic or mathematical. The linguistic approaches, in particular black-box models, can play an important role when system behavior is too complex to be captured through analytical methods. These models focus on linking inputs to outputs without the necessity to explicit knowledge of underlying mechanisms. This characteristic makes them especially suitable for various systems characterized by nonlinearity, multivariable interactions, and incomplete physical understanding. Among black-box approaches, artificial neural networks (ANNs) have gained increasing attention due to their ability to learn from data, assimilate large volumes of system information as well as accurately simulate real-world responses while remaining computationally efficient. Inspired by biological neural systems, ANNs function as distributed information-processing frameworks capable of prediction, adaptation, and operation in complex environments where conventional mechanistic or kinetic models are impractical or unreliable. Although highly system-specific and not readily transferable between applications, ANN models are particularly attractive for real-time control, process optimization, and fitting complex kinetic behavior, as demonstrated in many recent applications, where traditional mathematical modeling proves extremely challenging [27-29]. It is hypothesized that the integration of MgO nanoparticles into high-viscosity B30 waste cooking oil biodiesel will physically improve fuel droplet atomization and chemically act as an oxygen donor, thereby facilitating cleaner combustion while simultaneously reducing the fuel consumption penalty. This study aims to systematically evaluate the effects of MgO nanoparticles at concentrations of 10, 30, and 60 ppm on the combustion, performance, and emission characteristics of B30 waste cooking oil biodiesel in a constant-speed diesel engine under varying loads. In addition, all process responses were simultaneously predicted and optimized using an artificial neural network (ANN) model trained on the experimental dataset. This enables data-driven evaluation without reliance on explicit mechanistic assumptions. It also provides a comprehensive evaluation of the ANN's multi-output predictive capability.

2. Materials and Methods

2.1 Production Process of Biodiesel

Waste cooking oil belongs to the triglyceride family, which is similar to virgin vegetable oils and animal fats. It is a feedstock for biodiesel that is chemically comparable to virgin oils, but it undergoes compositional changes during frying. It is a cheaper and more sustainable feedstock for biodiesel production. The materials used in this study are waste cooking oil, methanol, and sodium hydroxide (NaOH) as a catalyst.

The biodiesel production process consisted of three main steps: pretreatment, transesterification, and distillation. First, raw waste cooking oil (WCO) with a high initial free fatty acid (FFA) content of 4.12 wt.% went through a rigorous pretreatment stage. The oil was heated to 80°C for 45 minutes under continuous stirring, passed through a 5-micron stainless steel mesh filter to remove solid impurities, and left in a separating funnel for 24 hours to settle. The upper oil layer was then collected and dried at 105°C under vacuum for 60 minutes. This thermal and vacuum process successfully stripped out trace moisture and volatile components, lowering the effective FFA level down to 0.25 wt.% before the chemical reaction. Under these low-FFA conditions, the alkali-catalyzed transesterification could proceed efficiently without any significant soap formation. The transesterification reaction was performed in a three-necked flask fitted with a thermometer, a reflux condenser, and a magnetic hot-plate stirrer. The chemical formulation used an optimized 1:6 oil-to-methanol molar ratio, which combined 4 L of pretreated WCO with 1 L of high-purity methanol, and a 1.25 wt.% NaOH catalyst loading (50 g of NaOH based on the total mass of the oil). The mixture was reacted at a constant temperature of 65°C under continuous magnetic stirring at 600 rpm for 60 minutes. From a chemical kinetics perspective, this specific formulation and high-shear stirring minimized mass-transfer resistance between the immiscible liquid phases, driving the alcoholysis of triglycerides toward forward conversion and achieving a final biodiesel yield of approximately 88%, which was calculated using the mass balance framework expressed in Equation 1.

$$\text{Biodiesel Yield (\%)} = \left(\frac{\text{Mass of purified biodiesel (g)}}{\text{Mass of pretreated oil (g)}} \right) \times 100 \quad (1)$$

The reaction mechanism for converting triglyceride ester bonds into fatty acid methyl esters and glycerol byproducts is shown in Equation 2. The complete production sequence and the transesterification experimental setup are illustrated in Fig. 1 and Fig. 2, respectively, while the properties of the raw waste cooking oil are summarized in Table 2.

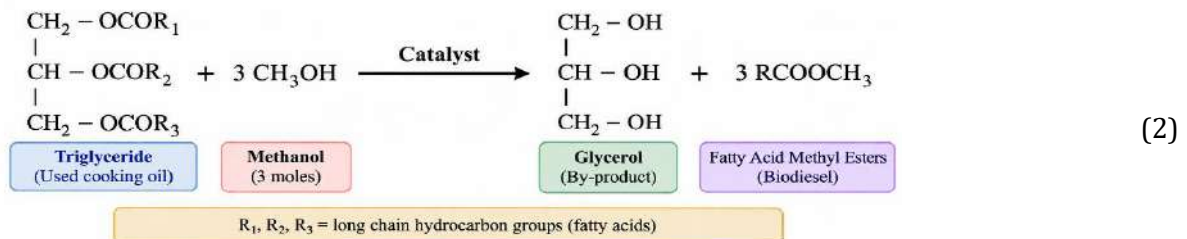
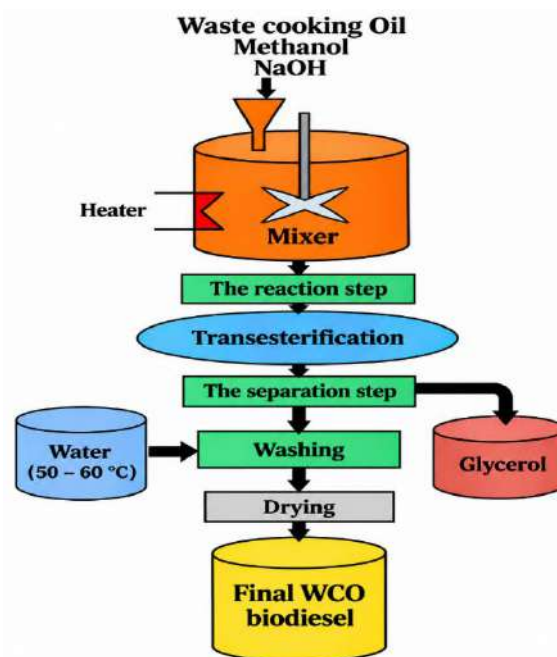


Fig. 1. Flow diagram of the biodiesel production process [9]



Fig. 2. The transesterification experimental setup

Table 2. Waste cooking oil properties

Property	ASTM Standard	Value
Free fatty acid (FFA, wt.%)	ASTM D5555	4.12
Density (kg/m ³)	ASTM D1298	918
Kinematic viscosity @ 40 °C (mm ² /s)	ASTM D445	41.3
Calorific value (MJ/kg)	ASTM D240	39.2
Flash point (°C)	ASTM D92	244
Fire point (°C)	ASTM D92	276
Cloud point (°C)	ASTM D2500	9
Pour point (°C)	ASTM D97	3
Moisture content (wt.%)	ASTM D6304	0.46

2.2 Nanoadditive (MgO)

Magnesium oxide (MgO) is an inorganic compound naturally occurring as a hygroscopic solid and commonly known as magnesia. Structurally, MgO consists of Mg²⁺ and O²⁻ ions and arranged in a cubic ionic (rock-salt) lattice. Magnesium oxide nanoparticles are non-toxic, odorless, and chemically stable. They have high melting point as well as hardness. This makes them suitable for high-temperature applications. Due to its high thermal stability and basic surface characteristics besides its catalytic activity, MgO has been widely investigated for fuel and combustion-related applications. When added into diesel-biodiesel fuel blends, MgO nanoparticles can act as combustion-enhancing additives. This is likely due to its capability to promote oxidation reactions, improve flame stability, and facilitate more complete fuel-air interaction. These properties may contribute in improvement of combustion efficiency and reduction of pollutant formation. The physicochemical properties of magnesium oxide used herein are listed in Table 3. To mix the MgO nanoparticles uniformly into the biodiesel, a Drawell probe sonicator (China, 20–25 kHz) with a 250 W nominal capacity was used. The device was fitted with a 13 mm titanium alloy probe tip and operated at a 60% amplitude for 30 minutes. To prevent heat buildup during the processing time, the unit was run in intermittent pulse cycles (5 seconds on and 2 seconds off) while keeping the mixture vessel inside an ice-chilled water medium. This successfully kept the temperature of the fuel mixture below 30°C to ensure optimal dispersion phase stability. Sodium dodecyl sulfate (SDS) was added at 0.03 wt.% to stop the nanoparticles from clumping together. Preliminary tests showed that adding 0.03 wt.% SDS alone (without MgO) had no noticeable effect on engine performance or emissions compared to pure B30. This confirms that the surfactant only serves to keep the nanoparticles suspended without affecting the combustion results over a 7-day period.

Table 3. Physicochemical properties of magnesium oxide (MgO)

Purity (%)	≥ 99
Appearance	White solid
Crystal structure	cubic
Molecular Weight (g/mol)	40.3
Density (g/cm ³)	3.58
Surface Morphology	Spherical
Average Particle Size (nm)	30
Specific Surface Area, BET m ² /g	60
Melting Point (°C)	2852
Boiling Point (°C)	3600

2.3 Properties of Waste Cooking Oil Biodiesel (WCOB)

The physicochemical properties of neat biodiesel and MgO nanoparticle-modified biodiesel fuels were determined in accordance with ASTM standard test methods. These properties were evaluated to assess their influence on fuel performance and exhaust emissions when used in a compression ignition (CI) engine. Different MgO nanoparticle concentrations were assessed to study their effects on fuel characteristics and engine behavior. The main goal of this analysis was

to identify variations in fuel properties resulting from the addition of MgO nanoparticles and to evaluate the influence of additive concentration. Engine performance obtained using various biodiesel fuel samples was compared with that of conventional diesel fuel. The measured properties of waste cooking oil biodiesel blends, with and without nanoadditives, are summarized in Table 4. Compared with diesel fuel, the kinematic viscosity of the fuel increased with increasing biodiesel content. The academic innovation of this study relies on the theoretical interaction between the basic surface sites of nano-MgO and the high free fatty acid (FFA) profile (4.12 wt.%) of waste cooking oil. At high in-cylinder temperatures, nano-MgO acts as a cracking catalyst that breaks down heavy fatty acid chains into lighter, volatile fractions, which suppresses soot formation and lowers real-time viscosity. Furthermore, the selection of the B30 blend serves a clear physical purpose; lower blends (B10, B20) do not maximize renewable substitution, while higher blends (B100) suffer from excessive viscosity (5.5 mm²/s) and lower calorific value (38.5 MJ/kg). Therefore, B30 provides the optimized baseline offering a balanced cetane number (54) and density (855 kg/m³) that match standard injection dynamics without engine modifications, while allowing nano-MgO to fully exert its oxygen-donating effects.

However, the addition of MgO nanoparticles to fuel improved its flow characteristics by reducing the viscosity of the biodiesel blends toward values closer to those of conventional diesel. The cetane number of the waste cooking oil biodiesel blends exhibited an increasing trend with higher biodiesel blending ratios, whereas their calorific value demonstrated a corresponding decline. The flash point, which shows a fuel's propensity to ignite, also increased with higher biodiesel content and nanoadditive dosage. Higher flash and fire point values indicate improved handling and storage safety. The observed 10°C rise with MgO nanoparticle addition is realistically governed by the basic porous surface sites of the nanoparticles adsorbing residual free methanol from the synthesis stage, alongside the SDS surfactant establishing an interfacial barrier that slows down the escape rate of volatile fractions [22]. MgO nanoparticle-enhanced biodiesel blends may be regarded as safer than the comparable base fuels since fuels with flash points higher than ambient temperature are safer to handle and transport.

Table 4. Properties of waste cooking oil biodiesel and MgO nanoparticle-enhanced biodiesel blends

Fuel Sample	Kinematic Viscosity @40°C (mm ² /s)	Calorific Value (MJ/kg)	Density (kg/m ³)	Cetane Number	Flash Point (°C)	Fire Point (°C)
D100 (Diesel)	2.8	43	836	48	58	70
B10	3.4	41.5	843	50	75	90
B20	3.8	40.5	848	52	95	115
B30	4.4	39.6	855	54	110	130
B30 + 10 ppm MgO	4.35	40	855.01	57	115	135
B30 + 30 ppm MgO	4.3	40.2	855.03	58	118	140
B30 + 60 ppm MgO	4.22	40.3	855.06	60	120	145
WCO Biodiesel (B100)	5.5	38.5	890	60	160	190

The kinematic viscosity, calorific value, density, cetane number, flash point, and fire point were experimentally measured in accordance with ASTM D445, ASTM D240, ASTM D1298, ASTM D613, ASTM D93, and ASTM D92, respectively. All reported fuel properties represent the definitive mean values calculated from experimental tests conducted systematically in triplicate, with the comprehensive experimental uncertainty analysis for the measurements detailed within Section (2.4).

2.4 Experimental Setup

All experiments were conducted on a two-cylinder, four-stroke, water-cooled diesel engine (Lombardini LDW 502) mounted on an engine test bench. This engine features an indirect injection (IDI) configuration with a pre-combustion chamber design, which fundamentally influences the fuel-air mixing and emission formation mechanisms (specifically NO_x and soot). The engine has a

total cylinder displacement of 505 cc, a compression ratio of 22.8:1, a cylinder bore of 72 mm, and a stroke length of 62 mm. Comprehensive technical attributes of the test engine are detailed in Table 5. The experiments were carried out at a constant engine speed of 1500 rpm. Water was used as the working fluid in the cooling system, circulating through the cylinder water jackets at a controlled temperature of 80°C. The fuels tested in this study were conventional diesel fuel (D100) and waste cooking oil biodiesel blends (B10, B20, and B30), corresponding to 10%, 20%, and 30% volumetric blending ratios, respectively. Engine loading conditions ranged from no load to full load at five intervals: 0%, 25%, 50%, 75%, and 100%. A schematic representation of the experimental engine setup is shown in Fig. 3. Exhaust emissions, including nitrogen oxides NO_x, carbon monoxide (CO), and unburned hydrocarbons (HC), were measured using an exhaust gas analyzer (HPC501, China). Smoke opacity was measured using an AVL smoke meter (415 S, Austria).

Table 5. Technical specifications of the Lombardini LDW 502 test engine

Engine Parameter	Specification and Operational Value
Engine Manufacturer / Model	Lombardini LDW 502 (FOCS Series), Italy
Engine Configuration	4-stroke, Liquid-cooled Diesel Engine
Cylinder Arrangement	2 cylinders in-line, vertical
Fuel Injection System	Indirect Injection (IDI)
Combustion Chamber Design	Pre-combustion chamber (Pre-chamber)
Cylinder Bore × Piston Stroke	72 mm × 62 mm
Total Engine Displacement	505 cc (0.505 L)
Compression Ratio	22.8:1
Intake Aspiration	Naturally aspirated
Maximum Rated Power	8.6 kW (11.7 HP) @ 3600 rpm
Maximum Rated Torque	24.5 Nm @ 2200 rpm
Engine Cooling Medium	Liquid-cooled (water jacket system)

Gaseous exhaust emissions, specifically nitrogen oxides (NO_x), carbon monoxide (CO), and unburned hydrocarbons (HC), were monitored continuously using a calibrated exhaust gas analyzer. At the same time, the smoke density and soot formation limits were recorded via a digital smoke opacity meter connected to the exhaust line. The inlet air mass flow rate was monitored by directing the airflow through a calibrated orifice plate coupled with a U-tube differential manometer. To cancel out aerodynamic pulsations and intake pressure cyclic fluctuations caused by the two-cylinder operation, a 50-liter surge tank was installed upstream of the engine intake manifold, which successfully ensured steady-state manometer fluid displacements. An institutional error analysis was performed to verify the statistical reliability and accuracy of the collected data. The exact measurement ranges, factory accuracy margins, and minimum resolution limits for each sensor and tool are summarized in Table 6.

Table 6. Measurement range, accuracy, and resolution of the experimental instruments

Apparatus / Sensor	Measurement Range	Accuracy	Resolution
Gas Analyzer (NO _x)	0–5000 ppm	± 5 ppm	1 ppm
Gas Analyzer (CO)	0–10% vol.	± 0.02% vol.	0.01% vol.
Gas Analyzer (HC)	0–9999 ppm	± 10 ppm	1 ppm
Smoke Opacity Meter	0–100%	± 1%	0.1%
Load Cell (Engine Load)	0–50 kg	± 0.1 kg	0.05 kg
K-type Thermocouples	0–1200 °C	± 1.5 °C	0.1 °C
Fuel Flow Meter	0–500 ml/min	± 0.5%	0.1 ml/min
Air Flow Manometer	0–60 mbar	± 1%	0.5 mbar

The propagation of discrete measurement errors was mathematically determined using the standard Kline and McClintock method to calculate the Total Experimental Uncertainty (U_{total}) based on the percentage uncertainty of each individual variable, according to the following Equation (3):

$$U_{total} = \sqrt{(U_{Temp})^2 + (U_{Load})^2 + (U_{Fuel})^2 + (U_{Air})^2 + (U_{CO})^2 + (U_{HC})^2 + (U_{NOx})^2 + (U_{Smoke})^2} \quad (3)$$

Where U_{Temp} , U_{Load} , U_{Fuel} , U_{Air} , U_{CO} , U_{HC} , U_{NOx} , and U_{Smoke} represent the percentage uncertainties of engine temperature, load, fuel flow rate, air flow rate, and NO_x, CO, HC, and smoke emissions, respectively. Based on the individual accuracy limits and the baseline variances of each measuring unit, the cumulative experimental uncertainty for the complete test campaign was found to be 2.84%. This result is well within the acceptable 5% error margin standard for internal combustion engine research [30].

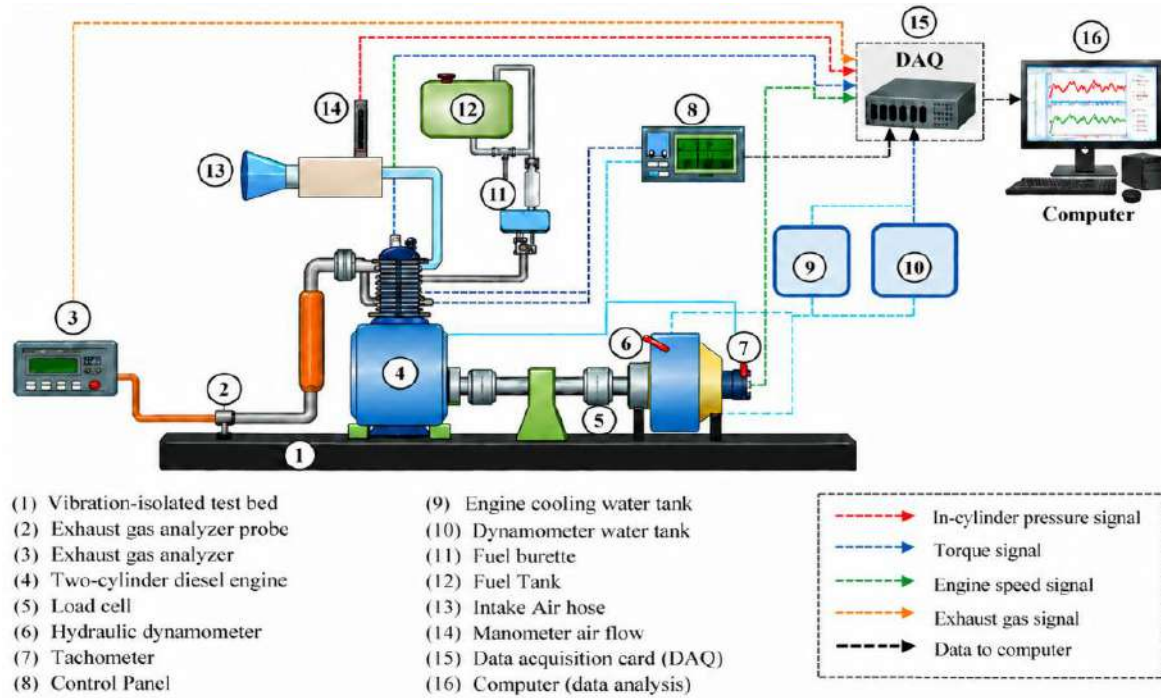


Fig. 3. Schematic diagram of the experimental engine test setup

2.5 Artificial Neural Network

The artificial neural network (ANN) is a parallel and distributed computing model modeled after the architecture of the human brain. Information within the network is processed through interconnected units called as neurons [27, 31]. These neurons are commonly arranged in a configuration with layers. These layers consist of an input layer, one or more hidden layers, and an output layer. At the input layer, data are introduced, which are then altered gradually as they pass through the network. Nonlinear activation functions in the hidden layers enable the model to capture complex patterns that cannot be represented by linear relationships alone. The processed signals will be then forwarded to the output layer. The final predictions are produced in this layer [32, 33]. This makes ANNs effective in particular for modeling highly nonlinear and multivariable systems. The input layer architecture was constructed with a detailed numerical dimensional mapping to represent the operational states and fuel chemistry accurately. Rather than using arbitrary string labels or numerical indices for the different fuel options (D100, B10, B20, B30, B100, and nano-MgO treated blends), the fuel types were numerically codified within the neural space using their distinct, experimentally determined physicochemical properties as distinct input vectors. Specifically, the dynamic input domain for each simulation layout consisted of five baseline independent variables: real-time engine load (0% to 100%), kinematic viscosity, lower calorific value, density, and cetane number. This multi-variable indexing framework allowed the input layer to translate structural chemical differences between pure petroleum diesel, unblended biodiesel, and nano-suspended fluids into quantifiable computational matrices. The main goal is to predict engine performance and emission characteristics under different operating conditions. The input space consisted of engine load (from 0% to 100% full load), and fuel type (eight different blends), which were B10, B20, B30, B100, D100, and B30 mixed with MgO nanoparticles at three

concentrations of 10, 30, and 60 ppm. The network predicted six response variables, which were specific fuel consumption (SFC), brake thermal efficiency (BTE), carbon monoxide (CO), hydrocarbon (HC) emissions, nitrogen oxides (NO_x), and smoke opacity. This multi-output structure enabled the model to learn shared dependencies among performance and emission parameters rather than treating each response independently.

To ensure stable as well as unbiased learning, all input and output variables were normalized to the range of 0-1 using map minmax scaling [29]. This would prevent variables with larger numerical ranges from dominating the training process. The network architecture was based on a multilayer perceptron (MLP) containing two hidden layers. Both linear and nonlinear interactions were effectively captured by employing Tansig activation functions in the hidden layer and a Purelin function in the output layer [34, 35]. Model training and selection were conducted using a nested cross-validation approach, allowing robust evaluation of generalization performance and avoidance of overfitting. Three advanced training algorithms including Levenberg-Marquardt (trainlm), Bayesian regularization (trainbr), and scaled conjugate gradient (trainscg) were systematically examined due to their strong convergence behavior, computational efficiency, and ability to control overfitting compared to conventional gradient-based methods [36]. Hyperparameter optimization was carried out using 5-fold cross-validation [37]. The number of neurons in each of the two hidden layers was varied from 3 to 25. This procedure resulted in 529 possible network configurations for each training algorithm. The optimal architecture was chosen based on the minimum mean squared error (MSE) obtained, which established the selected architecture with 18 and 12 hidden neurons in the first and second hidden layers, respectively.

For network optimization, the dataset was initially divided using an 80/20 partition rule, with 32 samples used for model development and 8 samples kept exclusively for final testing. The 32-sample development subset was further evaluated using 5-fold cross-validation during the network selection process. For each fold, the model was trained using the training portion and evaluated using the corresponding validation portion. The mean and standard deviation of the validation MSE were calculated across the five folds. The best network was then retrained using the complete 32-sample development subset, while the remaining 8 samples were retained as an independent final test set. Finally, the evaluation of the multi-output predictions was validated using mean squared error (MSE), root mean squared error (RMSE), mean absolute error (MAE), and the coefficient of determination (R^2) metrics.

3. Results and Discussion

3.1 Brake-Specific Fuel Consumptions (BSFC)

A crucial performance indicator for internal combustion engines is brake-specific fuel consumption (BSFC), which shows how well fuel is converted into power; a lower value denotes higher performance and vice versa. BSFC is expressed in (kg/kW.hr). Because there is a strong correlation between rising engine load and the amount of fuel supplied for combustion, the BSFC for all tested fuels decreased progressively from 25% load to full load [38].

Fig. 4 presents the variation of BSFC for all the tested fuel blends at different engine loads and a constant speed of 1500 rpm. Since diesel has a higher calorific value than waste cooking oil methyl ester blends, more biodiesel fuel is traditionally required to produce the same amount of power. However, the empirical profiles reveal a unique thermodynamic trend where the unadditived B30 blend demonstrates a BSFC curve running closer to the conventional diesel (D100) baseline compared to the lower B10 and B20 mixtures. Chemically, while pure biodiesel (B100) carries the highest intrinsic oxygen content, its operation is heavily penalized by excessive kinematic viscosity and lower energy density. Conversely, the B30 configuration achieves an optimized chemical compromise. It features a higher oxygen fraction relative to the lower blends (B10 and B20) which accelerates combustion kinetics and ensures superior air-fuel mixing completeness inside the chamber. This localized chemical availability drives a substantial expansion in the engine's brake thermal efficiency (BTE), effectively offsetting the minor calorific value deficit of the blend and compressing the specific fuel consumption rates of B30 downward toward the petroleum diesel threshold.

The engine performance in terms of brake specific fuel consumption (BSFC) across various fuel blends is depicted in Fig. 5. Typically, the higher kinematic viscosity of waste cooking oil (WCO) biodiesel deteriorates the spray and atomization characteristics inside the engine cylinder, subsequently increasing the BSFC. However, the addition of MgO nanoparticles (NPs) to the B30 blend counteracts this drawback. At full engine load, the baseline B30 blend exhibited a BSFC value of 0.28 kg/kW·h, which dropped to 0.241 kg/kW·h with the addition of 60 ppm MgO, representing a significant full-load reduction of 13.93%. Crucially, across the entire spectrum of engine operating loads, the introduction of MgO NPs at concentrations of 10, 30, and 60 ppm yielded cumulative average BSFC reductions of 5%, 10%, and 11%, respectively. This gradual improvement is attributed to the higher evaporation rate of the fuel droplets, enhanced fuel-air mixing, and the catalytic activity of the nano-doped blends. The high surface-area-to-volume ratio of the NPs promotes enhanced oxidation and improves overall combustion efficiency under varying thermal states [39].

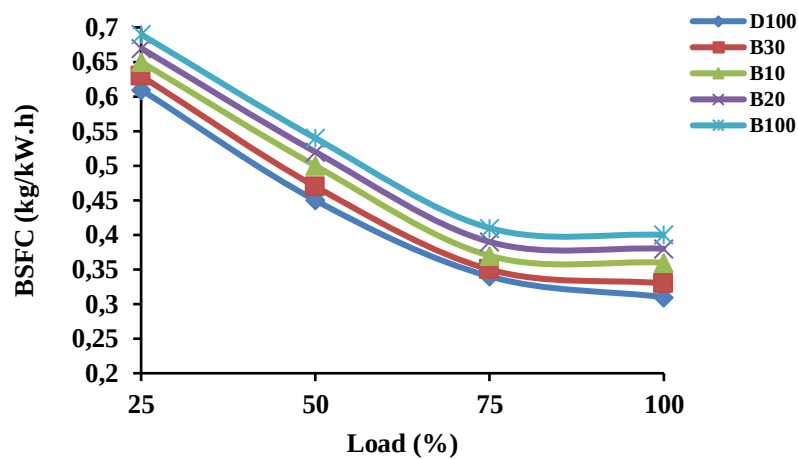


Fig. 4. Relationship between BSFC and engine load for diesel and WCO biodiesel blends

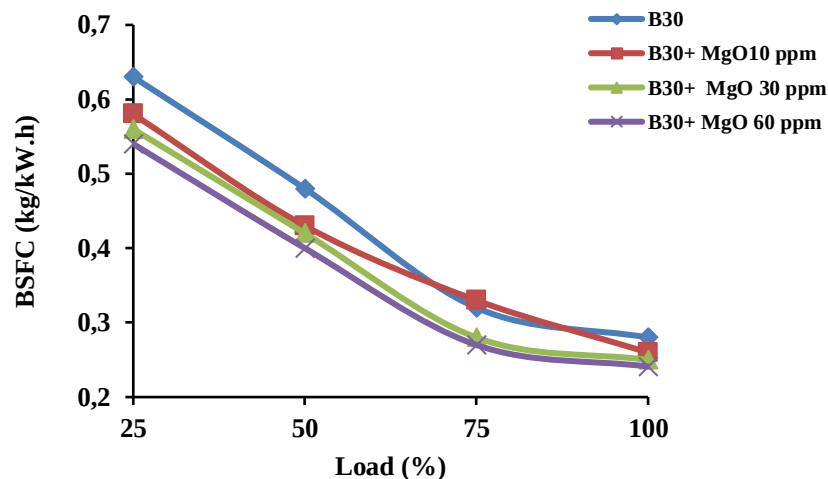


Fig. 5. Relationship between BSFC and engine load for biodiesel blends with MgO NPs

3.2 Brake Thermal Efficiency (BTE)

Brake thermal efficiency (BTE) refers to the efficiency with which an engine converts the chemical energy of fuel into useful mechanical energy. The high cetane number of biodiesels reduces the engine's ignition delay period [40]. Fig. 6 shows the brake thermal efficiency of diesel and biodiesel blends. For all tested fuels, BTE increases with increasing engine load. This behavior occurs because a higher fuel input at elevated loads enhances the combustion process and improves overall efficiency. The higher BTE of the B30 blend compared to B10 and B20, despite its higher viscosity and lower heating value, is explained by the fluid dynamics of the IDI pre-combustion chamber. During compression, air is forced through a narrow throat, creating intense micro-turbulence and swirl that breaks down the thick B30 droplets, overriding the viscosity penalty and ensuring

complete air-fuel mixing. While B100 fails because its excessive viscosity resists this shearing force, B30 strikes an ideal physicochemical balance, exhibiting better combustion efficiency than other blends [41]. Furthermore, when MgO nanoparticles are added to B30 at concentrations of 10, 30, and 60 ppm, the BTE increases further due to the additional oxygen availability, higher evaporation rate, and the larger surface-area-to-volume ratio of the nanoparticles [42].

Fig. 7 shows the effect of engine load on the BTE of biodiesel blends with MgO nanoparticles. The thermal efficiency of the B30 blend with 60 ppm MgO nanoparticles reached 26.72%. It is higher than that of the neat B30 blend (25.81%). Under full load conditions, the BTE of B30 blended with 10, 30, and 60 ppm MgO increased by 1.28%, 2.13%, and 3.52%, respectively.

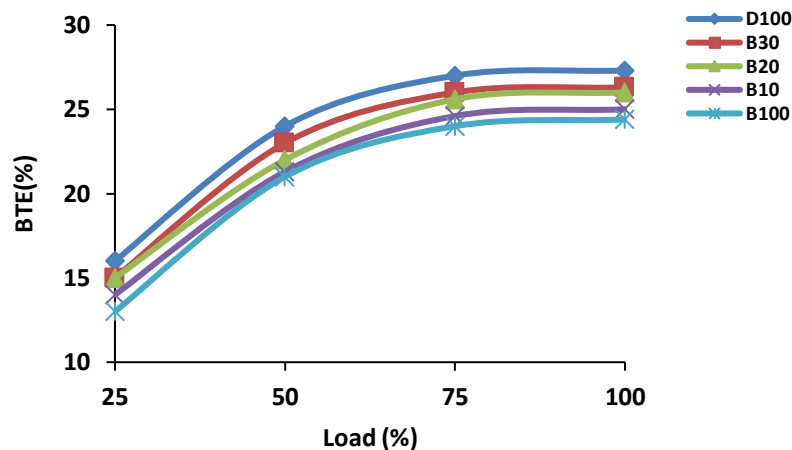


Fig. 6. Relationship between BTE and engine load for diesel and WCO biodiesel blends

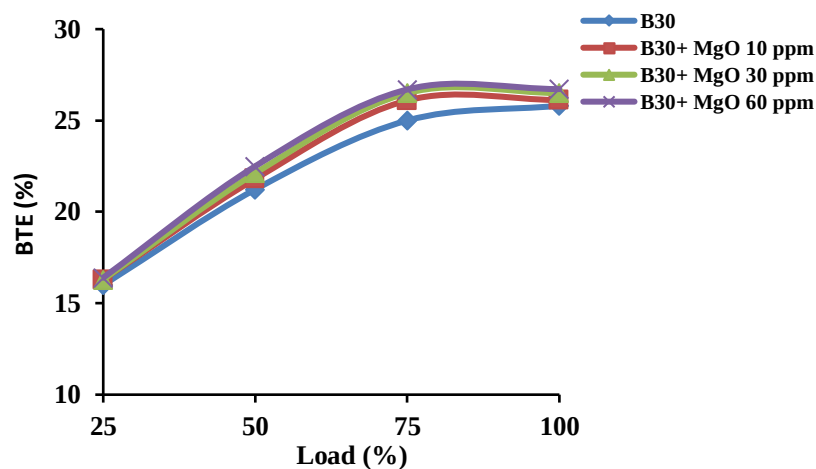


Fig. 7. Relationship between BTE and engine load for biodiesel blends with MgO NPs

3.3 Emission Analysis

Emission analysis is the practice of detecting and examining the gases and pollutants released by an engine while it is working. This is performed to assess the engine impact on the environment and to ensure the compliance with emission regulations and standards. The following contaminants are frequently evaluated in emission analysis.

3.3.1 Carbon Monoxide

Carbon monoxide (CO) is produced in the exhaust of rich-running engines because there is insufficient oxygen to convert all the carbon in the fuel into carbon dioxide. Compared to fossil fuels, biodiesel usually exhibits lower CO emissions. This is mainly attributed to its higher oxygen content, which accelerates the oxidation of CO. For all tested fuels, carbon monoxide (CO) emissions were higher at full load than at no load. When the engine operates at full load, an excessively rich

air-fuel mixture can lead to increased carbon monoxide emissions. The CO emissions of diesel and biodiesel blends are shown in Fig.8. Some of the carbon was released into the environment as carbon monoxide because not all of the carbon in the fuel was completely oxidized [43]. The lack of oxygen in the molecular structure of diesel fuel may be one of the causes of higher carbon monoxide emissions, as oxygen availability plays a critical role in combustion. Biodiesel emits less carbon monoxide than diesel. This trend continues as the biodiesel percentage in the blend increases. Fuels containing MgO nanoadditives show improved combustion owing to their higher effective oxygen availability.

The reduction in carbon monoxide emissions observed with the addition of MgO to the B30 blend at concentrations of 10, 30, and 60 ppm demonstrates the catalytic activity of the nanoparticles in biodiesel. MgO nanoparticles promote interfacial fuel processes, enhancing atomization, oxidation, and overall combustion efficiency [44]. The CO emissions of nanoparticle-doped biodiesel blends are illustrated in Fig.9. At full engine load, the baseline B30 blend exhibited a CO emission value of 0.21% vol., which significantly dropped to 0.152% vol. with the addition of 60 ppm MgO, representing a specific full-load reduction of 27.62%. Crucially, when analyzed across the entire spectrum of engine operating loads, the incorporation of MgO nanoparticles at concentrations of 10, 30, and 60 ppm yielded cumulative average CO reductions of approximately 10%, 15%, and 30%, respectively. This broader emission mitigation highlights the stable catalytic oxidation capability of the nanoadditives under varying load conditions. The addition of nano-magnesium oxide (MgO) to fuel significantly lowers carbon monoxide emissions by improving combustion efficiency.

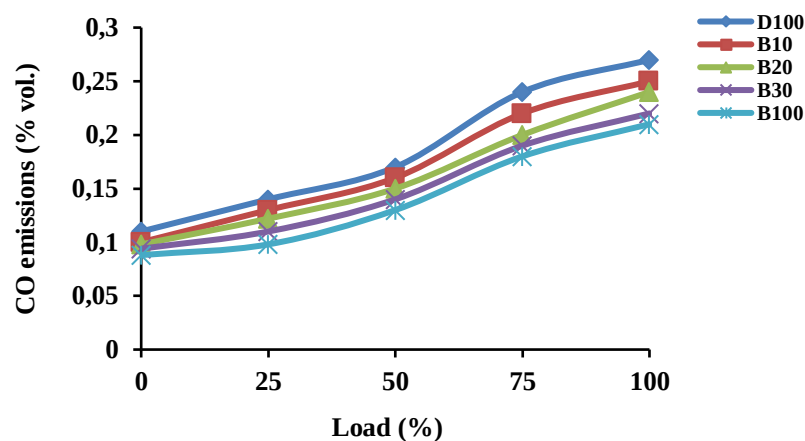


Fig. 8. Relationship between CO percentage and engine load for diesel and WCO biodiesel blends

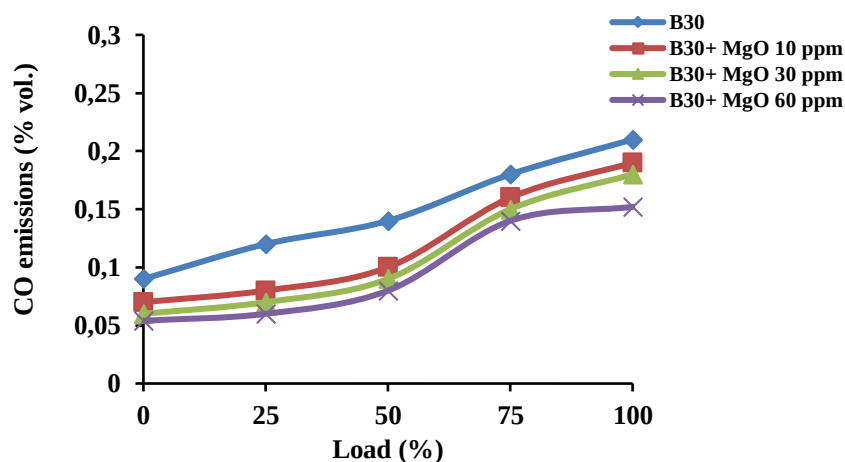


Fig. 9. Relationship between CO percentage and engine load for WCO biodiesel with MgO NPs fuel blends

Due to their large surface area and catalytic properties, MgO nanoparticles promote more complete oxidation of hydrocarbon molecules and facilitate the conversion of CO to CO₂. MgO nanoadditives also enhance fuel atomization and reduce ignition delay, improving fuel-air mixing and reducing incomplete combustion products.

3.3.2 Hydrocarbon (HC) Emission

For all fuel samples under study, HC emissions generally varied with changing engine load conditions at the constant operating speed of 1500 rpm. Biodiesel generally produces lower HC emissions due to its higher oxygen content, which promotes faster and more complete combustion. Fig.10 presents the HC emissions of diesel and biodiesel blends. Increasing the biodiesel percentage in the blend significantly reduces HC emissions compared to diesel fuel [45].

Most studies report a reduction in HC emissions when MgO nanoparticles are added to any biodiesel blends. An oxygen-donating catalyst is required to promote complete oxidation reactions, and MgO fulfills this role effectively. These improvements are primarily due to enhanced combustion processes within the combustion chamber [46]. Hydrocarbon emissions decreased with the addition of magnesium oxide nanoparticles to the B30 blend at concentrations of 10, 30, and 60 ppm. This reduction is attributed to the catalytic activity of MgO nanoparticles, which enhance the fuel-air interface and improve combustion efficiency. The HC emissions of MgO nanoparticle-doped biodiesel blends are shown in Fig.11. Compared to the B30 blend, which exhibited HC emissions of 68 ppm, the addition of 60 ppm MgO reduced HC emissions to 50 ppm. The incorporation of MgO nanoparticles at concentrations of 10, 30, and 60 ppm resulted in HC emission reductions of 18%, 22%, and 26%, respectively.

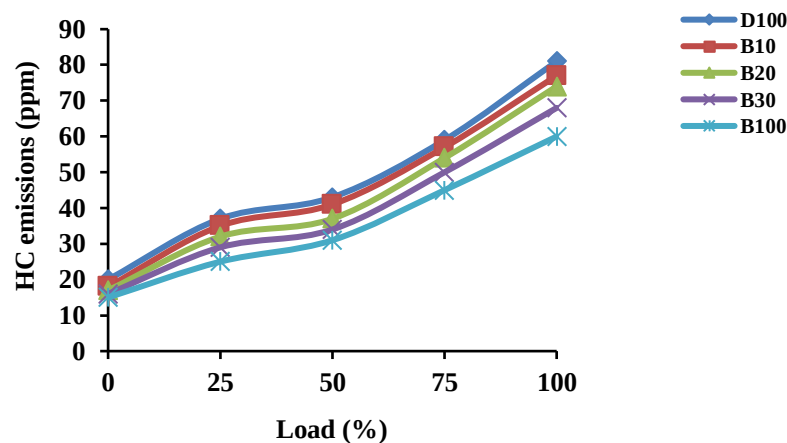


Fig. 10. Relationship between HC emissions and engine load for diesel and WCO biodiesel blends

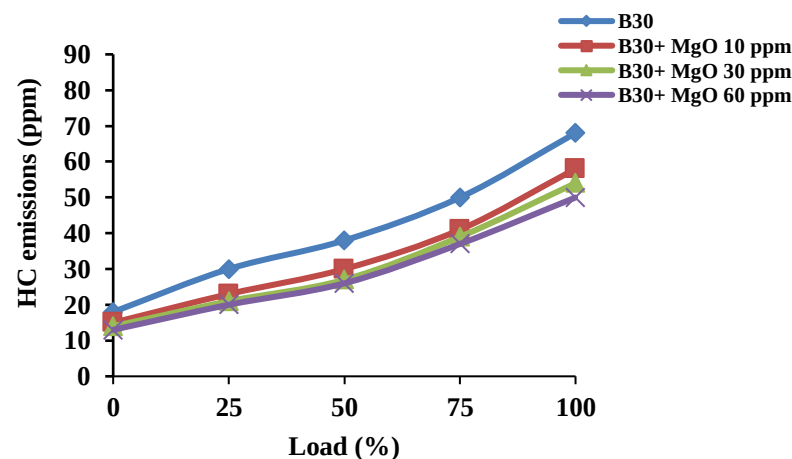


Fig. 11. Relationship between HC emissions and engine load for WCO biodiesel with MgO NPs fuel blends

3.3.3 Nitrogen Oxides (NO_x) Emissions

Nitrogen oxides (NO_x) are among the major emissions from diesel engines and are primarily formed through thermal mechanisms owing to high combustion temperatures and excess oxygen availability. For all tested fuels, NO_x emissions increased with increasing engine load. At full load, the primary cause of elevated NO_x emissions is the higher combustion temperature. Biodiesel blends exhibit in general NO_x emissions comparable to or slightly higher than diesel fuel. This increase is mainly attributed to the inherent oxygen content of biodiesel, which promotes higher combustion temperatures. As engine load increases, the rapid rise in combustion temperature leads to increased NO_x formation [47]. The NO_x emissions of diesel and biodiesel blends are shown in Fig. 12.

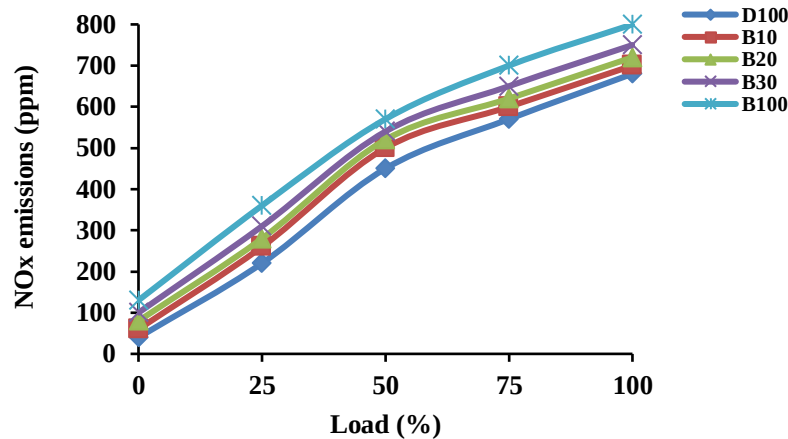


Fig. 12. Relationship between NO_x emissions and engine load for diesel and WCO biodiesel blends

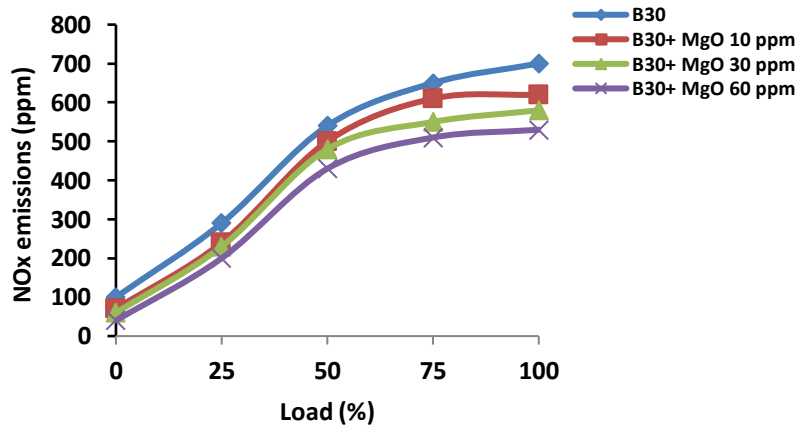


Fig. 13. Relationship between NO_x emissions and engine load for WCO biodiesel with MgO NPs fuel blends

The addition of MgO nanoparticles can enhance oxidation reactions and reduce nitrogen oxide emissions due to their catalytic influence during combustion [22]. Fig.13 shows the (NO_x) emissions of MgO nanoparticle-doped biodiesel blends. Quantitatively, at maximum engine load, the baseline B30 blend exhibited a peak (NO_x) concentration of 700 ppm, whereas the introduction of 10, 30, and 60 ppm MgO progressively minimized these emissions, with the 60ppm dosage lowering the concentration to 530 ppm, thereby demonstrating a 24.3% reduction. The simultaneous increase in BTE and reduction in (NO_x) emissions with nano-MgO addition is explained by the alteration of the combustion phases, which affects heat transfer characteristics without a simplistic drop in bulk temperature. The shorter ignition delay reduces fuel accumulation during the delay period, effectively suppressing the intense premixed combustion phase and preventing localized peak temperature spikes responsible for thermal (NO_x) formation.

Conversely, when MgO nanoparticles were added to the B30 blend, (NO_x) emissions decreased due to improve in-cylinder thermal conductivity and controlled diffusion combustion temperatures. This exceptional thermal conductivity effectively homogenizes localized high-temperature zones (hotspots) within the combustion chamber, thereby suppressing the thermal (NO_x) formation mechanism governed by the extended Zeldovich mechanism. During this subsequent diffusion phase, the catalytic oxygen-donating capability of the nano-MgO particles accelerates oxidation and ensures complete fuel conversion, enhancing thermal efficiency without generating the destructive temperature spikes that dictate NO_x generation.

3.3.4 Smoke Emissions

As the engine load intensified, a corresponding elevation in smoke opacity was documented across all evaluated fuel configurations. This upward trajectory is primarily driven by the escalated fuel injection rates required to sustain higher load thresholds, which inevitably induces localized fuel-rich pockets within the combustion chamber and restricts the oxygen available for complete chemical conversion. Moreover, the elevated thermal states inside the cylinder accelerate soot nucleation reactions, promoting the generation of carbonaceous particulates. Nonetheless, the biodiesel configurations systematically yielded lower smoke emissions than conventional diesel fuel throughout the operational spectrum. The presence of chemically bound molecular oxygen within the biodiesel matrix enhances the post-oxidation of soot precursors and elevates overall combustion efficiency [48, 49]. The experimental variations in smoke opacity for the baseline diesel and WCO biodiesel blends under different operating loads are illustrated via a continuous line chart in Fig. 14. Crucially, the incorporation of synthesized MgO nanoparticles drove an incremental suppression in smoke opacity by elevating thermal efficiency.

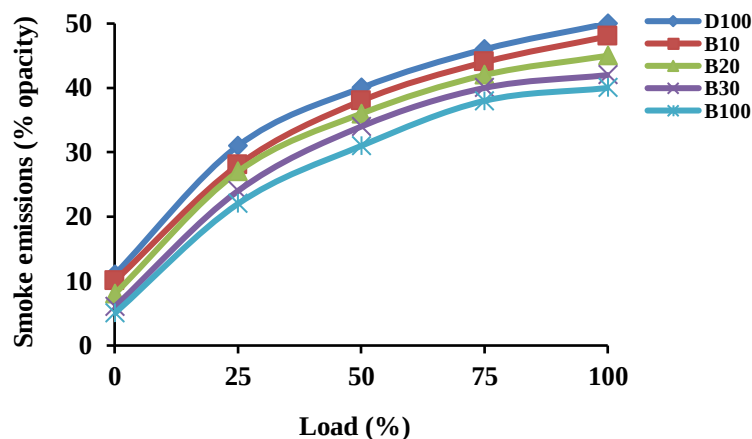


Fig. 14. Relationship between smoke emissions and engine load for diesel and WCO biodiesel blends

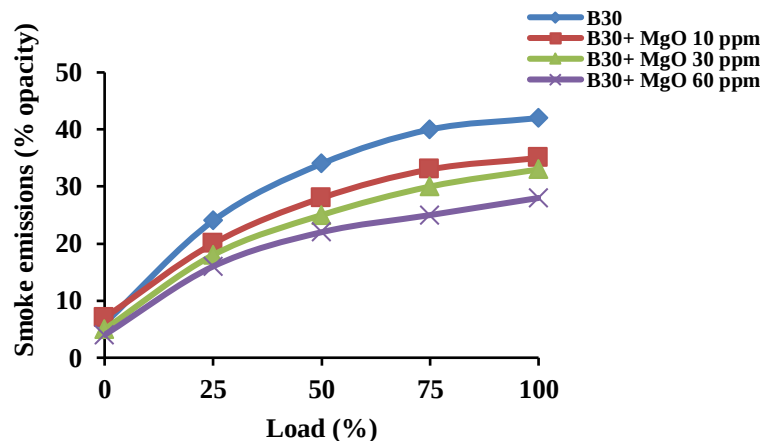


Fig. 15. Relationship between smoke emissions and engine load for WCO biodiesel with MgO NPs fuel blends

These metal-oxide nanoadditives function as effective combustion-promoting catalysts, facilitating rapid oxidation pathways and ensuring a more uniform thermal distribution within the cylinder. Consequently, elevating the MgO dosage from 10 to 60 ppm systematically lowered the smoke emission curves. The fuel mixture modified with 60 ppm MgO demonstrated the lowest smoke opacity levels among all nanoparticle-treated formulations. Furthermore, Fig. 15 displays these smoke opacity levels (%) as a function of engine load across various nanoparticle concentrations using a standardized continuous line chart, verifying the robust capability of MgO additives in mitigating exhaust smoke emissions from compression-ignition engines. Quantitatively, at 100% engine load, the baseline B30 blend exhibited a smoke opacity of 42%, which was progressively reduced to 35%, 33%, and 28% with the addition of 10, 30, and 60 ppm MgO, respectively, representing a maximum reduction of 33.3% at the highest nanoadditive dosage.

3.4 Circular Economy and Nanoadditives (MgO) in Biodiesel

A circular economy aims to eliminate waste and promote the ongoing use of resources while rebuilding natural systems. This approach contrasts sharply with the traditional linear economy model of "take, make, and dispose". Applying circular economy principles to biodiesel production using MgO nanoparticles requires several complementary strategies. The use of waste cooking oil represents an important step in designing out waste and pollution since it is a byproduct whose utilization directly reduces waste generation. Furthermore, MgO nanoparticles are employed to improve combustion efficiency and reduce emissions of harmful gases such as CO and NO_x [50-54].

In addition, advocating for the use of sustainable waste oils with positive environmental impacts is important. Biodegradable byproducts of biodiesel production, such as glycerol, can be valorized by conversion into biogas for energy generation or used as a feedstock in chemical and industrial applications. These measures jointly reduce waste and support the regeneration of natural systems. The addition of MgO nanoparticles not only enhances combustion efficiency but also lowers hazardous emissions such as CO and NO_x [46], and thereby improving overall environmental sustainability. Moreover, MgO NPs can be restored from industrial wastes. These include steel mill slag, which reduces reliance on virgin raw materials and mitigates industrial waste streams [55]. Studies have shown that the lubricating properties of biodiesel blended with nanoadditives can reduce engine wear as well as extend the service life of engine components [56, 57]. Ensuring sustainability therefore involves integrating circular economy practices such as resource recovery, recycling, and waste minimization.

By enabling biodiesel production within a closed-loop framework, this approach minimizes environmental impacts and improves resource efficiency throughout the biodiesel life cycle [58]. Compared with cerium oxide, magnesium oxide can be produced more easily and sustainably [59]. Zinc oxide is another studied nanoadditive, which has been found able to improve engine performance and reduces emissions as well. However, its more complicated processing requirements make MgO NPs a more attractive option for biodiesel applications. Overall, while both additives contribute to sustainability, magnesium oxide offers advantages in production efficiency, cost-effectiveness, and environmental compatibility. While the integration of MgO nanoadditives offers clear thermodynamic benefits, completing a holistic sustainability analysis necessitates addressing the environmental and health implications of airborne particulate emissions. During the high-temperature combustion phase, synthesized metal-oxide nanoparticles undergo rapid thermal excitation and are partially emitted within the exhaust stream. Sub-micron and nano-sized MgO particles released into the atmosphere pose potential nanotoxicity risks, as their minimal scale allows deep respiratory penetration and cellular translocation in biological systems. The integration of waste cooking oil (WCO) conversion and magnesium oxide (MgO) nanocatalysis provides a tangible empirical model that aligns perfectly with the circular economy tenets established by the Ellen MacArthur Foundation, which focus on eliminating waste, circulating products, and regenerating natural systems. Mechanistically, achieving a high transesterification yield of 88% effectively demonstrates the technical feasibility of keeping organic waste materials in continuous productive circulation rather than allowing them to degrade environmental sinks. To mitigate these risks and support absolute ecological friendliness, utilizing advanced after-treatment systems, such as commercial diesel particulate filters (DPFs), is essential

to trap ultrafine metallic fractions. Furthermore, because MgO safely transforms into magnesium carbonates when interacting with ambient CO₂ and moisture over time, its long-term environmental accumulation risks are significantly lower than those of transition or heavy metal oxide counterparts. To further demonstrate the commercial viability of the proposed method under the circular economy framework, a quantitative cost-benefit analysis was correlated with the engine performance. The cumulative additional expenses encompass the raw precursor acquisition for chemical MgO synthesis, electricity consumption during the high-shear ultrasonication stability phase 0.12\$/L, and processing overheads, totaling approximately 0.45\$ per liter of optimized nano-doped biodiesel. Conversely, the documented peak reduction in specific fuel consumption 13.93% lower (BSFC) at full load translates directly into a net fuel saving of approximately 0.18\$ per operating hour under steady-state configurations. Considering continuous institutional or industrial power generation profiles, the initial processing expenditure achieves a calculated economic break-even threshold within approximately 180 to 220 operational hours. This concrete financial balance validates that the thermal efficiency gains and fuel-saving capabilities decisively outweigh the nanomaterial preparation overheads, proving the robust economic feasibility of the proposed technique.

3.5 Artificial Neural Network (ANN) Performance

The final multi-output ANN model, which was trained using Bayesian regularization (trainbr), demonstrated strong and consistent predictive capability across all six-engine performance and emission outputs. The selected network architecture, two hidden layers with 18 and 12 neurons, provided an effective balance between learning capacity and generalization, as reflected in the test-set performance metrics. The model achieved an average coefficient of determination (R^2) of 0.9439 on the test dataset across all outputs. The individual RMSE and MAE values were evaluated separately for each output in their respective physical units. The relatively small difference in performance between training and testing ($\Delta R^2 \approx -0.0192$) indicates that overfitting was successfully controlled, and confirms the robustness of the Bayesian regularization strategy. A more detailed examination of individual outputs reveals varying degrees of prediction accuracy. It reflects the intrinsic complexity and noise sensitivity of each response variable. Brake thermal efficiency (BTE) was predicted with the highest accuracy. An R^2 of 0.9981 was achieved on the test set and a low RMSE of 0.4658%. In a similar way, specific fuel consumption (SFC) showed strong predictive agreement, with a test R^2 of 0.9599 and an RMSE of 0.03386 kg/kW·hr. These results indicate that the ANN captured the dominant thermodynamic trends governing engine efficiency with high reliability. Emission-related outputs exhibited slightly lower, yet still satisfactory, predictive performance. Hydrocarbon (HC) and nitrogen oxides (NO_x) emissions were predicted with test R^2 values of 0.9526 and 0.9573, respectively. While the absolute error metrics of them were higher, and in particular for NO_x due to its wider numerical range, the relative agreement remains strong, indicating the highly nonlinear formation mechanisms involved. Smoke opacity was predicted with a test R^2 of 0.9383 and reflected the model's ability to track particulate-related trends across varying fuel blends and operating loads. Carbon monoxide (CO) showed the lowest test-set performance ($R^2 = 0.8571$). This reduction is consistent with the known sensitivity of CO formation to small fluctuations in combustion conditions and measurement uncertainty. Even so, the model successfully captured the overall trend and values of CO emissions, as evidenced by the low absolute RMSE (0.0231% vol.). Table 7 presents the test-set prediction performance for each output parameter, including R^2 , RMSE, and MAE in their respective physical units.

Table 7. Test-set prediction performance of the developed multi-output ANN model

Output parameter	Test R^2	RMSE	MAE
SFC (kg/kW·hr)	0.9599	0.033862	0.023408
BTE (%)	0.9981	0.465815	0.395315
CO (% vol.)	0.8571	0.023125	0.021307
HC (ppm)	0.9526	4.812485	3.943458
NO _x (ppm)	0.9573	47.549845	39.095067
Smoke (% opacity)	0.9383	3.822480	3.104034

The test-set R^2 values reported above were calculated exclusively from the eight independent test samples. In contrast, the R^2 values displayed in Fig. 16, which presents parity plots comparing ANN-predicted values against experimental measurements for all six outputs were calculated using all 40 experimental and ANN-predicted observations. Therefore, these values represent the overall agreement of the model across the complete dataset. In all cases, the majority of data points, both training and testing, cluster closely around the ideal $y = x$ line.

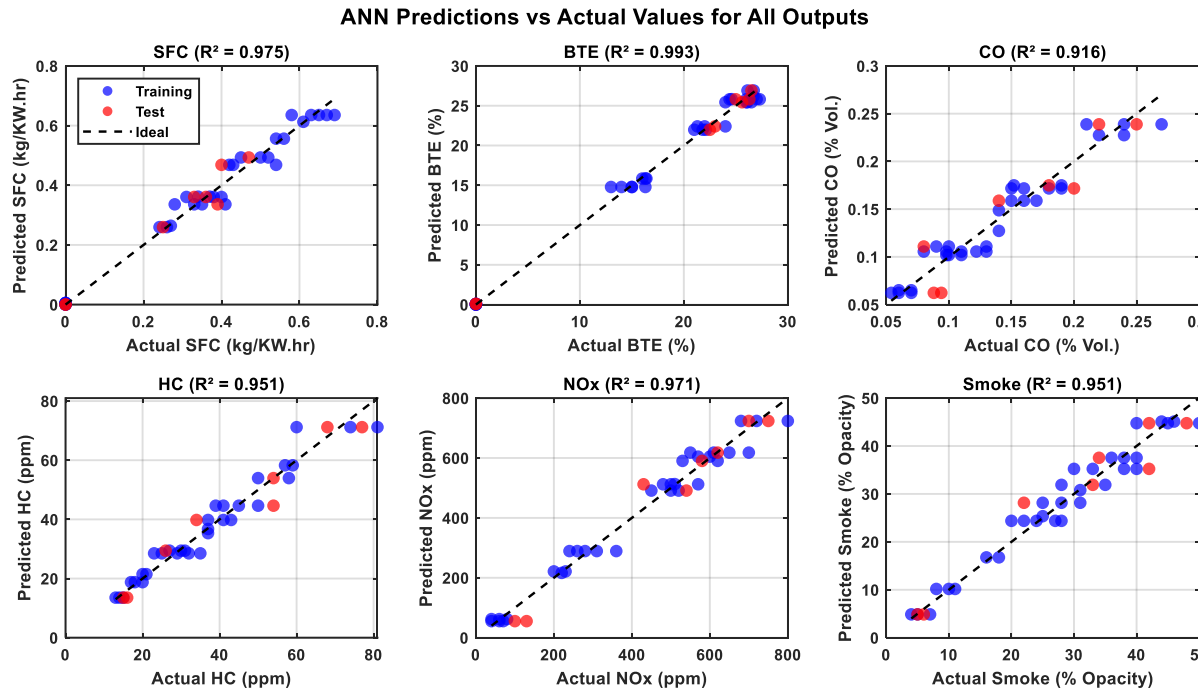


Fig. 16. Parity plots comparing experimental and ANN-predicted values for all six-engine performance and emission outputs. The R^2 values shown in the plots were calculated using all 40 observations

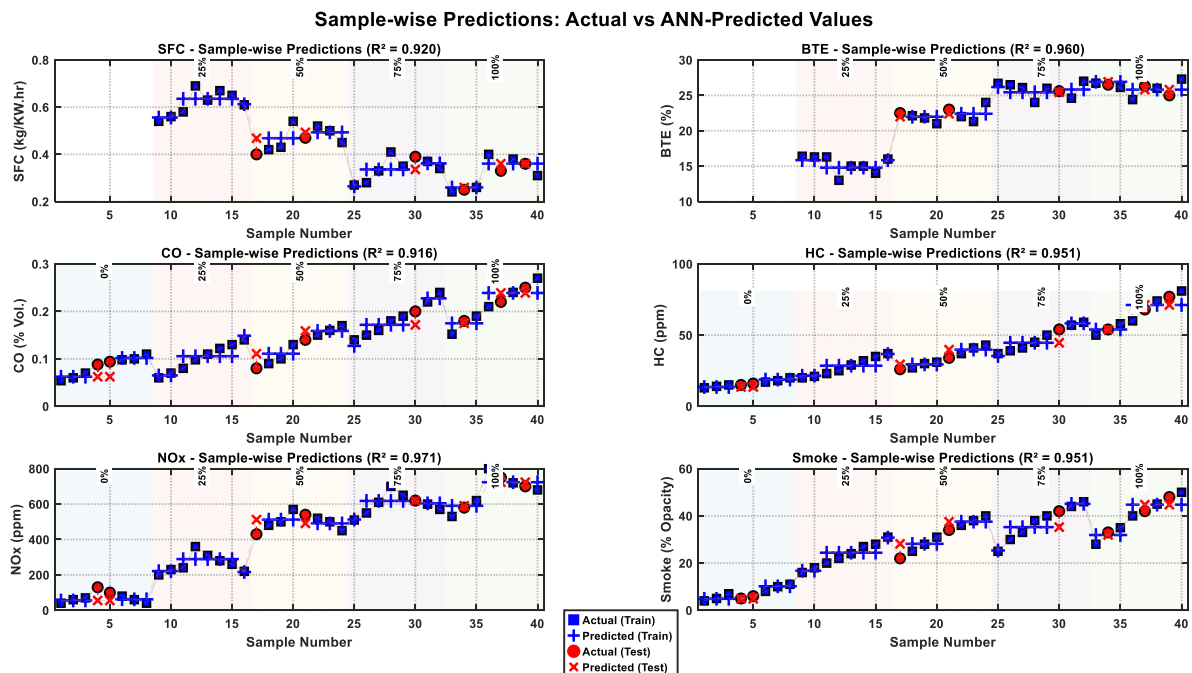


Fig. 17. Sample-wise comparison of experimental and ANN-predicted values across different fuel blends and operating conditions for all output parameters. The R^2 values shown were calculated from the plotted observations and do not represent the independent test-set R^2 values

This tight alignment confirms the high prediction accuracy obtained numerically. It also demonstrates the absence of systematic overprediction or underprediction. Notably, the densest clustering is observed for BTE and SFC, whereas slightly larger scatter appears for emission parameters and especially for CO and smoke opacity. Even so, deviations remain bounded and randomly distributed, suggesting that prediction errors arise primarily from process variability rather than structural model bias. To further assess model generalization, sample-wise comparisons between experimental and predicted values are illustrated in Fig. 17. Across different fuel blends and load conditions, the ANN regularly tracks the experimental trends for all outputs. The R^2 values associated with Fig.17 were calculated from the experimental and ANN-predicted values displayed in the sample-wise plots and therefore include the plotted training and test observations rather than the independent test set alone. Transitions between operating regimes are well captured. This indicates that the model has internalized the underlying response behavior rather than memorizing individual data points. In some test samples, there are noticeable differences, particularly for emission outputs at higher loads. These differences highlight the inherent difficulty of predicting emissions governed by coupled chemical kinetics and transient combustion phenomena. Importantly, however, the overall trend fidelity remains intact, and the deviations do not compromise the model's predictive usefulness. The difference between training and testing performance across all outputs remains modest, with ΔR^2 values ranging from -0.0741 (CO) to $+0.0064$ (BTE), based on the corresponding training and independent test-set R^2 values. This relatively small difference confirms that the ANN maintains strong generalization capability despite the limited dataset size and multi-output complexity. The consistency between numerical metrics and graphical diagnostics demonstrates that the developed ANN functions not merely as a statistical fitting tool, but as a reliable digital surrogate of the experimental engine system. Within the studied operating range, the model can therefore be confidently employed for performance prediction, comparative fuel evaluation, and preliminary optimization studies, reducing the need for extensive experimental campaigns.

4. Conclusions

In this experimental and numerical study, waste cooking oil biodiesel was synthesized via the transesterification of waste cooking oil and subsequently blended with petroleum diesel. Magnesium oxide (MgO) nanoparticles were added to the biodiesel blends at concentrations of 10, 30, and 60 ppm. From the results of this study, the following conclusions can be drawn:

- The measured physicochemical properties of the produced biodiesel and MgO-doped blends were found to be within the permissible limits specified by ASTM standards.
- Compared with conventional diesel fuel, MgO nanoparticle-enhanced biodiesel demonstrated improved engine performance; specifically, as the concentration of MgO nanoparticles increased, brake-specific fuel consumption (BSFC) decreased by up to 11%.
- Brake thermal efficiency (BTE) increased by approximately 3.52% for the B30 blended with 60 ppm MgO.
- Significant reductions in exhaust emissions were observed. Carbon monoxide (CO), hydrocarbons (HC), nitrogen oxides (NOx), and smoke opacity were all reduced by up to 30%, 26%, 24.3%, and 33.3%, respectively, compared to the base B30 blend.
- The developed multi-output artificial neural network (ANN), based on Bayesian regularization with an [18, 12] hidden-layer architecture, demonstrated high predictive accuracy for the six-engine performance and emission parameters. The model achieved an average test-set coefficient of determination (R^2) of 0.9439, with individual R^2 values ranging from 0.8571 for CO to 0.9981 for BTE. These results confirm the capability of the ANN to reliably predict engine performance and emission characteristics under the investigated fuel and operating conditions.

Although the incorporation of nanoparticles increases the initial fuel preparation cost, the long-term economic and environmental benefits stemming from improved combustion efficiency, reduced fuel consumption, lower emissions, and extended engine component life can outweigh these additional costs. The results of this study indicate that MgO nanoparticles are promising as well as effective fuel additives for biodiesel applications. It offers a viable pathway for partial

replacement of conventional diesel fuel in compression ignition engines. The findings support both industrial implementation and governmental policies. They have a goal to adopt cleaner and more sustainable fuel alternatives. However, the present study is limited by the use of a two-cylinder diesel engine, which may not fully represent real-world operating conditions. Therefore, future work should focus on multi-cylinder engines. Furthermore, future investigations should address the long-term colloidal stability, shelf-life, and potential sedimentation issues of MgO nanoparticles in biodiesel blends under various storage temperature. Also, it should perform long-duration testing to assess engine wear, durability, and long-term performance. This will provide an excellent evaluation of the commercial viability of MgO nanoparticle-enhanced biodiesel fuels. Overall, the B10, B20, and B30 waste cooking oil biodiesel blends showed compatibility with existing diesel engines without mechanical modifications, while the integration of MgO nanoparticles was essential to consistently lower NO_x emissions below those of petroleum diesel. These results further confirm the capability of waste cooking oil biodiesel, and in particular when enhanced with MgO nanoparticles, as a sustainable and environmentally friendly alternative fuel.

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Shear capacity of RC corbels: Systematic review, code models, and SHAP-based machine learning

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Abstract

This review presents a systematic review and SHAP-based interpretation of machine-learning (ML) predictions for reinforced concrete corbels (RCCs). A structured PRISMA-guided review was conducted using Scopus and Web of Science records, supplemented by reference checking. After duplicate removal and screening, 121 studies were retained for qualitative synthesis. The review discusses the effects of key parameters, including concrete compressive strength, span-to-depth ratio, corbel geometry, reinforcement, and loading history. Strengthening and repair studies were reviewed as a separate practical branch of the literature, while analytical and code-based models, including shear-friction, strut-and-tie, and triangular-truss approaches, were examined to clarify their assumptions and limitations. In the ML component, an XGBoost model was trained using 330 compatible vertical-load specimens with 13 min-max normalized input variables to predict the normalized experimental shear capacity, using an 80:20 training-testing split and a fixed random seed of 42. The model achieved R² values of 0.9973 and 0.9090 and RMSE values of 0.0066 and 0.0388 for the training and testing sets, respectively; the gap between them indicates mild overfitting, so the model is used for interpretation rather than as a design equation. SHAP analysis was then applied to explain the influence of the main input variables. The results demonstrate that member geometry, a/d ratio, and concrete strength dominate the prediction trend, while reinforcement-related variables provide secondary but still meaningful contributions within the database range. The findings highlight the value of combining systematic review, code-model assessment, and explainable ML for understanding RCCs behavior.

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1. Introduction

RC corbels (RCCs) are short cantilever-like members that project from columns or walls to support beams, girders, crane loads, and other concentrated actions [1-3], especially in precast concrete construction [4]. Because of their short shear span and discontinuity-region behavior, their response is usually governed by nonlinear stress transfer rather than ordinary beam action. The applied load is mainly resisted through a diagonal compression strut, while the main tensile reinforcement, secondary reinforcement, and nodal zones complete the internal load path. For this reason, corbels are commonly treated as disturbed regions where conventional sectional analysis is not sufficient. Previous experimental studies have shown that corbels with small shear span-to-effective depth ratios are particularly prone to shear-dominated failure, concrete crushing, diagonal splitting, anchorage distress, or yielding of the main reinforcement [5-7].

The shear load capacity of RCCs is affected by several geometric, material, and reinforcement-related parameters. These include concrete compressive strength, shear span-to-depth ratio,

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corbel width, effective depth, column width, longitudinal reinforcement ratio, transverse reinforcement, anchorage detailing [8], and the yield strength of reinforcement. Earlier studies have generally reported that smaller a/d ratios, higher concrete strength, and improved reinforcement detailing can increase the ultimate load capacity and improve ductility. However, the influence of these variables is strongly interactive, which makes corbel behavior difficult to represent using simple empirical equations. This complexity is one of the main reasons why design provisions continue to rely on simplified approaches such as the shear-friction method and the strut-and-tie method.

International design codes, including ACI 318-25 [9], EN 1992-1-1 [10], and GB 50010-2010 [11], provide practical methods for corbel design based on different mechanical assumptions. The shear-friction method is commonly used for short corbels, while the strut-and-tie method idealizes the member as a truss composed of concrete compression struts, steel tension ties, and nodal regions. Although these methods are useful for design practice, recent comparisons with experimental databases have shown that code-based predictions may be conservative, scattered, or inconsistent, especially when the tested specimens include variations in geometry and reinforcement detailing [9-12].

Despite the large number of studies on RCCs, the available literature remains dispersed across different specimen types, materials, reinforcement layouts, loading conditions, and prediction approaches. Some studies deal with conventional RCCs, while others focus on fibre-reinforced concrete, high-strength concrete, strengthened corbels, or special loading cases. This variation makes it difficult to draw clear conclusions without a transparent and reproducible review process. In this study, papers addressing specimens subjected to direct horizontal force are excluded so that the review remains focused on corbels governed mainly by vertical loading and shear load capacity. This boundary is important because horizontal-force interaction introduces a different mechanical problem involving additional tie force, N/V effects, and moment transfer at the corbel-column interface.

The practical importance of this member class is closely linked to precast concrete construction, and the specific challenges of precast systems influence both the design and the analysis of RCCs. In a precast frame, the corbel is not an isolated cantilever but the physical interface through which a beam or girder is seated on a column, so its performance is coupled to the connection detailing around it. Lacerda et al. [1] showed that the condition of the interface between corbel and beam, including the vertical grouting of the joint, changes the measured behavior of beam-to-column connections, which means that the boundary condition assumed in a laboratory corbel test may not reproduce the boundary condition of the same corbel in an assembled frame. Similarly, de Lima Araujo et al. [4] required a dedicated computational and analytical treatment for two-step corbels used in precast systems, because the stepped geometry adopted for erection and seating purposes redistributes the internal load path relative to a plain corbel. Three consequences follow for design and analysis. First, precast corbels must accommodate erection tolerances, bearing pads, and seating eccentricities, so the effective shear span is not always known with the precision assumed in a strut-and-tie idealization. Second, the corbel region carries not only gravity actions but also the forces generated by volume change, restraint, and, in seismic regions, joint rotation, as reflected in studies of precast beam-slab-column joints and of precast corbel and nib connections under cyclic loading [99,100]. Third, because precast members are produced under controlled conditions but assembled on site, the governing uncertainty often shifts from material strength to detailing and workmanship at the joint. These considerations explain why the present review keeps strengthened, repaired, and non-conventional corbels as a separate branch of the evidence, and why the machine-learning dataset defined later is restricted to specimens tested under a well-controlled vertical-load configuration.

Recently, artificial neural networks have been increasingly used to predict the shear strength of reinforced concrete members because they can capture nonlinear relationships among several input parameters. For RCCs, Abbas and Ali developed a backpropagation neural network model using 330 experimentally tested specimens and reported that the best architecture, 13-10-9-1, achieved a Pearson correlation coefficient of 0.990. Their results showed better agreement with

experimental data than several code-based methods, whose correlation coefficients ranged from 0.901 to 0.927. However, ANN models remain dependent on the quality and range of the experimental database used for training and validation. Therefore, their use should be supported by a careful review of the available experimental evidence and clear identification of the model's applicability limits [13].

Accordingly, this study aims to present a systematic review and explainable ML interpretation of the shear capacity of RCCs subjected mainly to vertical loading. To achieve this, a structured PRISMA-guided review was conducted to identify, classify, and synthesize the available experimental, analytical, numerical, code-based, strengthening, repair, and ML-related studies on RCCs. The review focuses on the main parameters affecting shear behavior. In addition, the study reviews strengthening and repair approaches for deficient or damaged RCCs. The study also evaluates analytical and code-based models, including shear-friction, strut-and-tie, and triangular-truss approaches, to clarify their assumptions, applicability limits, and prediction challenges.

In addition to the systematic review, this study develops an XGBoost model to support the interpretation of the experimental database. SHAP analysis is then employed to quantify the contribution of each variable to the predicted shear capacity and to connect the ML results with the physical behavior of RCCs. Therefore, the main contribution of this study lies in combining a systematic literature review, code-based assessment, strengthening and repair synthesis, and SHAP-based explainable ML interpretation within a unified framework for understanding RCCs shear behavior.

2. Systematic Review Methodology

This study used a PRISMA-guided systematic review procedure to identify, screen, classify, and synthesize previous research on RCCs. The procedure was designed for two connected purposes. First, it provided a transparent route for building the qualitative review corpus. Second, it defined a traceable evidence base from which specimen-level data could be extracted for the ML component. The PRISMA 2020 statement was used as a reporting guide for the identification, screening, eligibility, and inclusion stages [14], as summarized in Fig. 1.

The review mainly addresses RCCs governed by vertical-load and shear-capacity behavior. Studies dealing with special materials, non-conventional reinforcement, strengthening, repair, or retrofitting were retained when they provided useful information on RCCs behavior, but they were classified separately during synthesis. This distinction is important because these studies may involve additional variables, such as fibre content, FRP layout, repair material, bond condition, or anchorage details. By contrast, the ML dataset used later in this paper was restricted to compatible specimens tested under vertical loading only. Specimens involving direct horizontal forces, combined vertical-horizontal loading, strengthening or repair, and special materials were not used for the ML training and SHAP interpretation dataset.

2.1 Review Scope and Research Questions

The review scope covered RCCs, concrete brackets, and short cantilever-type corbels tested or analyzed in relation to shear strength, ultimate load, failure behavior, design prediction, or rehabilitation performance. Direct horizontal-force and combined-load studies were treated as outside the main vertical-load corpus because they introduce a different mechanical problem involving N/V interaction, additional tie demand, and moment transfer at the corbel-column interface. However, studies on strengthened, repaired, fibre-reinforced, recycled aggregate, UHPC, SCC, GFRP/CFRP-reinforced, and other non-conventional corbels were included when they helped explain corbel behavior or practical performance.

The review was directed by the following research questions:

- What geometric, material, and reinforcement parameters have been most frequently investigated in previous studies on RCCs?
- How do these parameters influence the shear load capacity and failure behavior of RCCs?
- How accurately do current design methods estimate the shear strength of RCCs?

- Which studies provide specimen-level data suitable for ML-based interpretation of vertical-load corbel shear capacity?

2.2 Search Strategy

A structured database search was conducted using Web of Science Core Collection and Scopus. These databases were selected because they provide broad coverage of peer-reviewed engineering literature and allow advanced searching by title, abstract, and keywords. The Web of Science search used the Topic field, while the Scopus search used TITLE-ABS-KEY, which is the closest equivalent field in Scopus. The database search strategy is summarized in Table 1.

The search was carried out in two stages. First, a broad search was performed using terms related to RCCs and shear/load capacity. Second, exclusion terms were added to reduce records explicitly governed by direct horizontal-force interaction. The official indexed search produced 69 records from Web of Science and 124 records from Scopus, giving 193 records before duplicate removal. These numbers represent the auditable WoS/Scopus corpus used in the PRISMA process.

Table 1. Database search strategy

Database	Search date	Search field	Core search terms	Horizontal-force NOT terms	Records before / after NOT terms
Web of Science Core Collection	14 May 2026	Topic	("reinforced concrete corbel" OR "RC corbel" OR "concrete corbel" OR "reinforced concrete bracket" OR "short cantilever*") AND ("shear strength" OR "shear capacity" OR "load capacity" OR "ultimate load" OR "failure load")	("horizontal force" OR "horizontal load" OR "horizontal-to-vertical" OR "N/V" OR "axial force" OR "combined vertical and horizontal")	70 / 69
Scopus	14 May 2026	TITLE-ABS-KEY	("reinforced concrete corbel" OR "RC corbel" OR "concrete corbel" OR "reinforced concrete bracket" OR "short cantilever*") AND ("shear strength" OR "shear capacity" OR "load capacity" OR "ultimate load" OR "failure load")	("horizontal force" OR "horizontal load" OR "horizontal-to-vertical" OR "N/V" OR "axial force" OR "combined vertical and horizontal")	134 / 124

Note: both searches covered 1960-2026 and were limited to English articles or review articles. Retrieved WoS records were from 2006-2026, while retrieved Scopus records were from 1974-2026.

The Scopus and Web of Science searches were treated as the main indexed search, but they did not cover all relevant RCCs literature. Several useful studies were found through Google Scholar, backward reference checking, and older reference lists, particularly works indexed under alternative terms such as "bracket", "short cantilever", or "D-region member". These supplementary sources were used where they supported the technical discussion, but they were not automatically counted as part of the main indexed PRISMA corpus unless they met the same scope criteria.

2.3 Eligibility Criteria

The eligibility criteria were defined before screening so that the selected studies remained consistent with the purpose of the review. As shown in Table 2, the main inclusion condition was that the study should address RCCs, concrete brackets, or short cantilever-type members in relation to shear strength, load capacity, ultimate load, failure behavior, design modelling, or strengthening/repair performance. Studies were not excluded simply because they used special concrete, non-conventional reinforcement, or strengthening systems. Instead, these records were retained and classified separately. Papers were excluded or treated only as boundary references when they clearly fell outside the structural scope, such as non-structural corbels, unrelated members, or specimens dominated by direct horizontal-force interaction [15]-[17].

Table 2. Inclusion and treatment criteria

Category	Inclusion criteria	Exclusion criteria / treatment
Structural element	RCCs, concrete brackets, and short cantilever-type corbels	Steel, timber, masonry, architectural, or non-structural corbels were excluded.
Loading condition	Studies relevant to vertical-load, shear-capacity, cyclic/repeated-load, or practical corbel behavior	Direct horizontal-force or combined vertical-horizontal interaction studies were not retained in the main vertical-load corpus.
Main response	Ultimate shear load, failure load, shear strength, load capacity, cracking, stiffness, or failure mode	Studies unrelated to structural load capacity or corbel behavior were excluded.
Study type	Experimental, analytical, numerical, code-comparison, review, or ML studies related to RCCs	Papers without technical relevance to RCCs behavior were excluded.
Material type	Conventional concrete, high-strength concrete, UHPC, SCC, recycled aggregate concrete, geopolymers concrete, fibre-reinforced concrete, and other non-conventional materials	Material type was recorded and classified; special materials were retained for qualitative synthesis but not used in the ML dataset.
Strengthening condition	Original, strengthened, repaired, retrofitted, or externally bonded RCCs	Strengthening and repair studies were retained for qualitative synthesis and discussed separately; they were not used in the ML dataset.
Loading history	Monotonic, repeated, cyclic, or fatigue-related corbel studies	Loading history was recorded separately; only compatible vertical-load specimens were considered for ML extraction.
Data completeness	Studies with information useful for qualitative synthesis, quantitative extraction, or ML-related discussion	Incomplete specimen-level data did not prevent qualitative use, but limited ML suitability.
Language	English-language publications	Non-English publications were not included.
Publication type	Journal articles, review articles, conference papers, theses, standards, and technical reports where relevant	Codes, textbooks, standards, and methodological references were used as supporting sources and were not counted as main included studies.

2.4 Screening and Study Selection

The screening process included identification, duplicate removal, title/abstract/keyword screening, a second careful screening of borderline records, and full-text eligibility assessment. The official indexed search produced 193 records, including 69 records from Web of Science and 124 records from Scopus, as reported in Table 3. After merging the two datasets, 58 duplicate records were removed using exact DOI matching where available and normalized-title matching where DOI information was missing. This left 135 unique indexed records for screening.

During screening, strengthening, repair, retrofitting, and special-material corbel studies were retained when they addressed RCCs behavior. The main scope boundary was horizontal-force or combined vertical-horizontal loading behavior, because these cases introduce a different mechanical problem involving N/V interaction, additional tie demand, and moment transfer at the corbel-column interface. In the first screening stage, 11 indexed records were flagged as horizontal-force or combined-load records and were not retained in the main vertical-load/shear-capacity corpus. After a second careful screening of the indexed records, three additional indexed records

were also found to fall outside the main vertical-load scope. Accordingly, a total of 14 indexed records were not retained, leaving 121 studies for the final systematic corpus used in the qualitative synthesis, as summarized in Fig. 1 and Table 3.

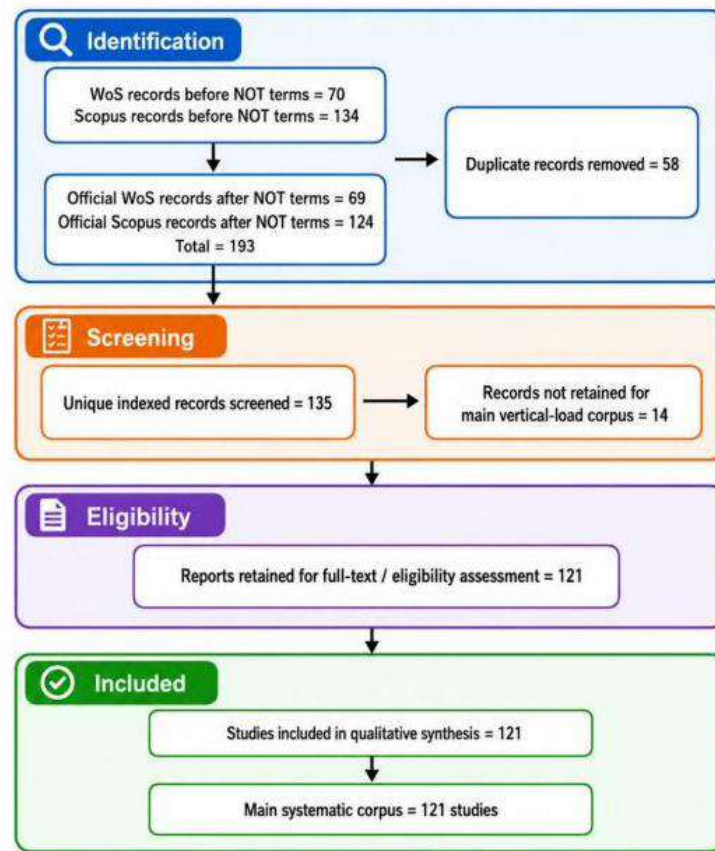


Fig. 1. PRISMA flow diagram of the study selection process

Table 3. Screening summary for Web of Science and Scopus

Screening stage	Number of records
Web of Science records before NOT terms	70
Web of Science official records after NOT terms	69
Scopus records before NOT terms	134
Scopus official records after NOT terms	124
Total official indexed records before duplicate removal	193
Duplicate records removed	58
Unique indexed records after duplicate removal	135
Indexed records flagged as horizontal-force or combined-load scope during first screening	11
Additional indexed horizontal-force or combined-load records flagged after second careful screening	3
Total indexed records not retained for the main vertical-load/shear-capacity corpus	14
Main eligible studies after screening	121
Reports retained for full-text/eligibility assessment	121
Reports excluded after full-text assessment	0
Studies included in qualitative synthesis	121

The screening flags were used to clarify how borderline records were treated rather than to remove all non-conventional corbel studies. This is shown in Table 4. Strengthening, repair, and special-material studies were retained because they provide useful behavioral evidence, but the results were interpreted separately from the unstrengthened vertical-load specimens used for ML analysis.

Table 4. Classification and treatment during screening

Screening category	Count / status	Treatment
Horizontal-force or combined-load scope within indexed records during first screening	11	Not retained in the main vertical-load/shear-capacity corpus.
Additional indexed horizontal-force or combined-load records identified during second careful screening	3	Not retained in the main vertical-load/shear-capacity corpus.
Total indexed records not retained for the main vertical-load/shear-capacity corpus	14	Removed from the main systematic corpus because their behavior was governed by horizontal-force or combined-load interaction.
Strengthening, repair, or retrofitting corbel studies	Retained	Included in qualitative synthesis and discussed as a separate subgroup.
Special concrete, special materials, or non-conventional reinforcement	Retained	Included in qualitative synthesis and classified by material or reinforcement type.
Main eligible systematic corpus	121	Used for qualitative synthesis; supporting references are cited separately when needed.

2.5 Data Extraction

For each of the 121 studies in the main systematic corpus, bibliographic, experimental, numerical, and modelling information was organized according to the data extraction form presented in Table 5. The extracted information included author name, publication year, journal, study type, number of specimens, concrete type, loading setup, investigated variables, failure modes, and main findings. When a cited source was used only for background, code explanation, methodology, or secondary context, it was not counted as part of the main systematic corpus.

For studies suitable for quantitative analysis or ML interpretation, specimen-level data were also examined. These data included concrete compressive strength, shear span, effective depth, corbel width, column width, total depth, shear span-to-depth ratio, longitudinal reinforcement ratio, transverse reinforcement ratio, steel yield strength, and experimental ultimate shear load. As shown in Table 5, each study was checked for ML suitability based on the availability of the required input and output variables and the compatibility of the test conditions.

The ML-related data structure was aligned with the earlier data-driven model by Abbas and Ali (2025) [13], who used 330 experimentally tested RCC specimens. In the present work, this boundary was kept clear: the ML dataset included only specimens under vertical loading and did not include direct horizontal-force specimens, strengthened or repaired corbels, or specimens made with special materials. Thus, the ML dataset represents a controlled subset of the broader systematic review corpus rather than the full set of studies discussed qualitatively.

Table 5. Data extraction form

Data group	Extracted variables
Bibliographic information	Author(s), year, title, journal, DOI
Study information	Study type, number of specimens, experimental/numerical/analytical approach
Specimen type	RC corbel, concrete bracket, short cantilever-type corbel
Loading condition	Vertical monotonic loading, repeated/cyclic loading, loading setup, support condition
Geometry	Shear span, effective depth, total depth, width, column width, a/d ratio
Concrete properties	Concrete compressive strength and concrete type
Reinforcement details	Longitudinal reinforcement ratio, vertical reinforcement ratio, horizontal reinforcement ratio, bar yield strengths
Response variables	Experimental ultimate shear load, failure load, failure mode, crack pattern
Qualitative classification	Original, strengthened/repared, special material, cyclic/repeated, numerical, analytical, code-based, or ML study
ML suitability	Complete data, missing data, compatible/not compatible with the vertical-load ML dataset
ML exclusion notes	Horizontal force, strengthening/repair, special materials, repeated/cyclic loading, or missing specimen-level data

2.6 Data Adequacy and Quality Assessment

The quality assessment was handled as a data-adequacy check rather than as a rigid numerical score. This approach was more suitable for the present review because the included literature covers different types of evidence, including experimental studies, numerical simulations, analytical models, review papers, design-code comparisons, and strengthening/repair studies. A single score would not fairly describe the usefulness of these different sources.

Table 6. Final data-use assessment outcome

Assessment outcome	Result / treatment
Reports retained for full-text/eligibility assessment	121
Reports excluded after full-text assessment	0
Studies included in qualitative synthesis	121
Studies considered for specimen-level quantitative extraction	Controlled subset based on data completeness and compatibility
Studies considered for ML interpretation	Only vertical-load specimens without horizontal force, strengthening/repair, or special materials

The outcomes of the data adequacy assessment are summarized in Table 6. At the screening level, each record was checked for relevance to RCCs behavior and for consistency with the vertical-load scope. At the full-text level, the main concern was whether the study provided enough information for its intended use. For qualitative synthesis, a study was retained when it offered useful behavioral, analytical, numerical, design, or rehabilitation insight. For specimen-level quantitative extraction and ML-related use, the requirements were stricter: the study had to report the main geometry, concrete strength, reinforcement information, loading arrangement, and ultimate load.

Studies with incomplete specimen-level data were still useful for qualitative discussion but were not treated as equally suitable for ML extraction.

2.7 Data Synthesis

The selected studies were synthesized in two complementary ways. First, a qualitative synthesis was conducted using the 121-study main corpus to discuss the development of RCCs research, the main investigated parameters, common failure mechanisms, strengthening and repair strategies, and the limitations of existing design and prediction approaches [18]. Second, specimen-level extraction was considered only for studies that provided sufficient data and compatible test conditions for quantitative or ML-related analysis.

For the ML component, the systematic review was used to define a transparent candidate literature pool and to justify the data-selection boundaries. The ML analysis should therefore be understood as a systematic-review-supported modelling exercise, not as an independent black-box prediction study. The XGBoost and SHAP analysis used later in this paper is limited to the compatible vertical-load subset and is intended mainly to interpret parameter influence rather than to replace mechanics-based design provisions.

2.8 Methodological Recommendations for Systematic Reviews

The procedure applied above also exposed several practical difficulties that are typical of systematic reviews in structural and civil engineering, and the way they were handled here can be generalized. Six recommendations follow directly from the present experience and are offered for reviews of comparable structural topics.

First, a single indexed database is not sufficient. As reported in Section 3.2, 66 of the 135 unique records were indexed only in Scopus and 11 only in Web of Science, so a review based on either database alone would have missed a substantial share of the evidence. At least two complementary indexes should therefore be searched, and the overlap should be reported rather than assumed. Second, the search string must accommodate the synonyms that structural engineering has accumulated over time. The same member appears in the literature as a corbel, a bracket, a short cantilever, and a D-region member, and older work is frequently indexed only under the alternative terms; truncated wildcards and explicit synonym sets are therefore necessary rather than optional. Third, indexed searching should be complemented by backward reference checking. Several foundational corbel studies from the 1960s and 1970s were retrievable in the present review only through the reference lists of later papers, because their indexing metadata predates modern abstracting practice. Fourth, the scope boundary should be defined mechanically, not merely thematically. Here the boundary was the presence of direct horizontal-force or combined vertical-horizontal interaction, which changes the internal force system rather than simply adding a variable; expressing the boundary through explicit NOT terms and reporting the record counts before and after their application, as in Table 1, makes the decision auditable. Fifth, when a review supports quantitative or data-driven synthesis, the qualitative corpus and modelling dataset should remain separate, with exclusion reasons recorded at the specimen level; this preserves the breadth of the narrative synthesis while avoiding incompatible test conditions in the modelling dataset. Applying these six practices makes the search reproducible, keeps the treatment of borderline records transparent, and allows a later reader to reconstruct why a given study was retained, reclassified, or set aside.

3. Descriptive Analysis of the Selected Literature

The descriptive analysis provides an overview of the RCCs research landscape before the technical synthesis. The analysis uses the final screening structure reported in Section 2 and the descriptive source data prepared from the indexed search records. It is important to note that Figs 2-4 are descriptive figures. Fig. 2 presents the publication activity of the official indexed search records, Fig. 3 shows the database coverage of the 135 unique indexed records after duplicate removal, and Fig. 4 summarizes the main research/source-category tags identified during screening. These figures help explain publication activity, database coverage, and research/source categories, but

they should not be read as direct PRISMA inclusion counts. The final systematic corpus is the 121-study corpus reported in Fig. 1 and Table 3.

3.1 Publication Trend

The publication trend is presented in Fig. 2. Fig. 2(a) shows the annual distribution of the official Web of Science records, while Fig. 2(b) shows the Scopus records grouped by publication interval. The trend confirms that RCCs research has continued over several decades, with more visible activity in recent years. This increase can be linked to the growing interest in precast concrete systems, D-region design, strut-and-tie modelling, special concrete materials, numerical simulation, and data-driven prediction methods [19]-[22].

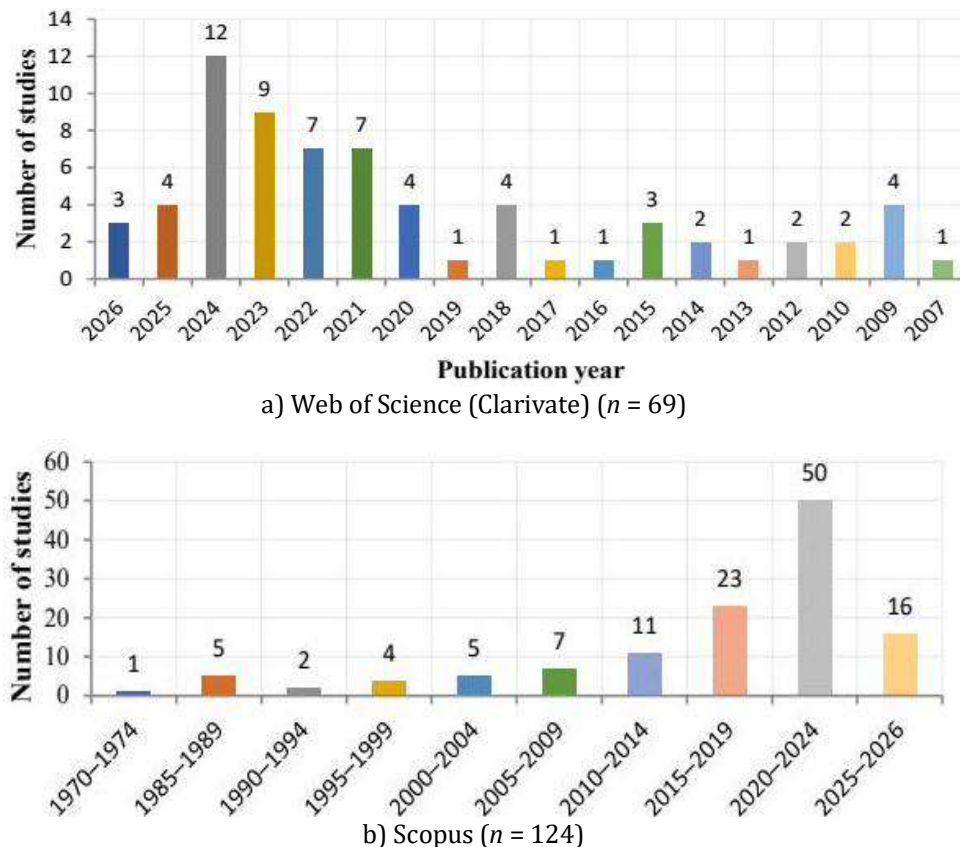


Fig. 2. Publication trend of the official indexed search records

The shape of this trend has consequences for design practice, and not only for bibliometrics. Two features are relevant. The first is the concentration of output in the most recent intervals: in the Scopus record set, 50 of the 124 records fall in the 2020-2024 interval and a further 16 in 2025-2026, so roughly half of the retrieved corbel literature has appeared within the last six years. Design provisions for corbels, by contrast, retain the essential structure they were given decades ago, with the shear-friction expressions still traceable to Kriz and Rathes [15] and Mattock et al. [17]. A widening gap therefore exists between the evidence base and the codified rules, and it is this gap, rather than any deficiency in the rules themselves, that explains the scatter reported when code predictions are compared with modern experimental databases. The second feature is a change in the composition of the output. As shown in Section 3.3, computational modelling and AI-related sources, composites and advanced materials, and concrete and construction materials together account for a substantial share of the recent records, whereas the earlier literature was dominated by monotonic tests on conventional normal-strength specimens. Practically, this means that the specimens now entering the literature increasingly involve high-strength and fibre-reinforced concrete, recycled aggregate, GFRP and CFRP reinforcement, and strengthened or repaired configurations, that is, precisely the cases for which the empirical calibration underlying the current provisions was not established. Three implications follow for future design practice. Code committees will need to draw on databases that are wider than those originally used for calibration,

particularly with respect to concrete strength and reinforcement type. The growing volume of numerical and data-driven work offers a route to parametric coverage that testing alone cannot provide, provided the models are interpretable and their applicability ranges are stated. Finally, because much of the recent growth is driven by non-conventional materials, future provisions are likely to require explicit applicability limits rather than a single expression applied uniformly across all corbel types.

3.2 Database Coverage

The database-coverage distribution of the 135 unique indexed records after duplicate removal is shown in Fig. 3. The figure indicates that 66 records were indexed only in Scopus, 11 records were indexed only in Web of Science, and 58 records appeared in both databases. This confirms that the use of both databases was necessary to improve the coverage of the RCCs literature. If only one database had been used, a considerable number of relevant records would have been missed. At the same time, the review showed that Scopus and Web of Science did not cover all relevant RCCs studies. Therefore, backward reference checking and supplementary searching were also used to identify older or differently indexed studies that were important for the technical discussion.

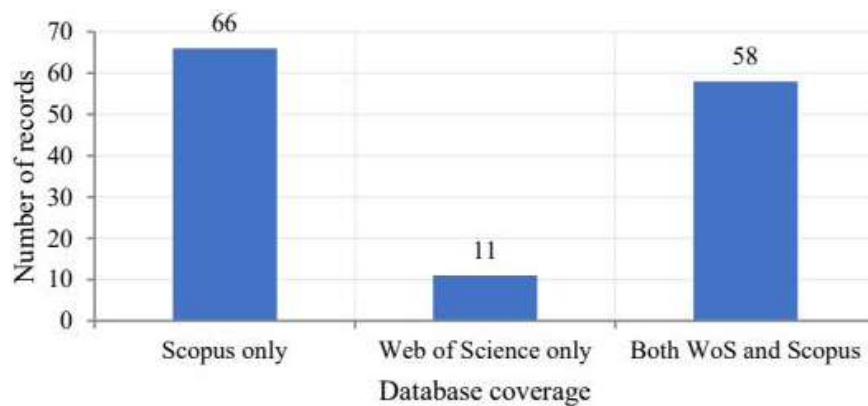


Fig. 3. Database coverage among the 135 unique indexed records after duplicate removal

3.3 Research and Source Categories

The distribution of research-category tags is presented in Fig. 4. These tags are descriptive and non-exclusive, meaning that one study may contribute to more than one theme. Most records are associated with structural engineering, civil engineering, mechanics, concrete materials, computational modelling, and construction-related sources. This distribution reflects the multidisciplinary nature of RCCs research, which combines structural behavior, material properties, reinforcement detailing, numerical modelling, design-code evaluation, and ML-based prediction.

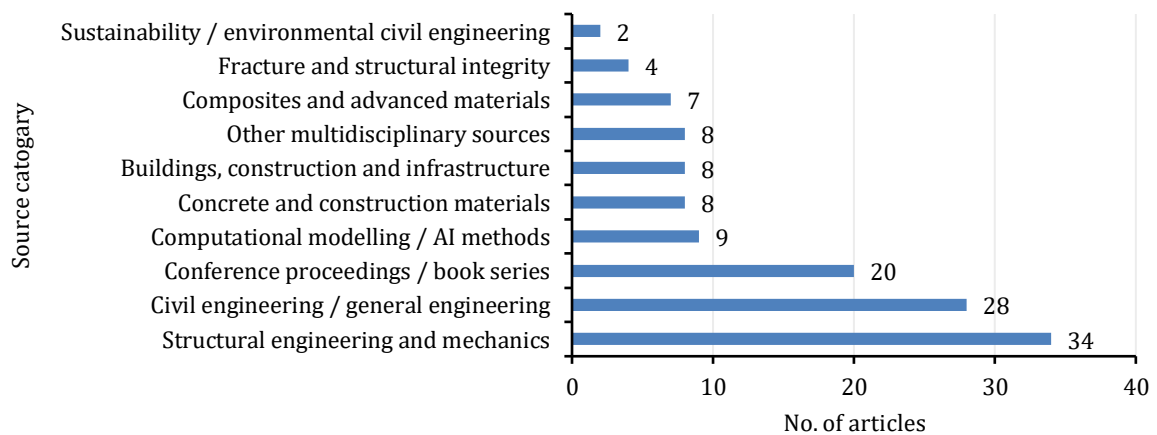


Fig. 4. Main research/source-category tags identified during screening

3.4 Main Research Themes

The selected literature can be grouped into several recurring themes: experimental investigation of shear behavior, influence of geometric and reinforcement parameters, special concrete and fibre-reinforced corbels, strengthening and repair, strut-and-tie modelling, comparison with design codes, numerical simulation, and ML-based prediction. This thematic structure shows that RCCs research has moved from early experimental observation and empirical design equations toward a broader combination of experimental, analytical, numerical, rehabilitation, and data-driven approaches.

In general, the descriptive analysis confirms that RCCs research remains active because of the complex stress-transfer mechanism in short shear-span members. The selected studies provide a broad basis for reviewing the influence of key parameters on shear capacity and for evaluating the role of explainable ML. However, the diversity of study types also confirms the need to distinguish between the qualitative review corpus and the smaller, more controlled dataset used for ML analysis.

4. Systematic Synthesis of RCCs Behavior

The systematic synthesis focuses on the main behavioral issues reported in the selected literature. RCCs behave as discontinuity regions, where the applied load is transferred mainly through a diagonal compression strut between the loading point and the column face [23]-[26]. Most building code provisions focus primarily on in-plane response; consequently, typical reinforcement details (Fig. 5) are proportioned to resist combinations of vertical shear and co-existing horizontal restraint forces. The reinforcement configuration consists of three key components:

- Primary reinforcing bars (A_{sc}): These bending reinforcing bars shall be securely anchored to ensure they achieve the yield strength (f_y) completely beneath the applied vertical load (V_u). To meet this requirement, satisfactory anchorage at the outer end can be obtained by welding a rebar of equal diameter across the ends of the primary tension rebars and developed by a 90° hook at the column for enhanced anchorage.
- Closed ties (A_h): Comprising hoop ties or stirrups, this reinforcement confines compression struts and counteracts splitting forces parallel to the thrust. As per ACI 318-25 guidelines, A_h shall be evenly spaced across $2d/3$ measured from primary rebars, positioned parallel and adjacent to the primary reinforcement (A_{sc}).
- Framing bar: This rebar, normally with a diameter similar to that of the ties, is used to anchor and fix the closed ties at the outside face.

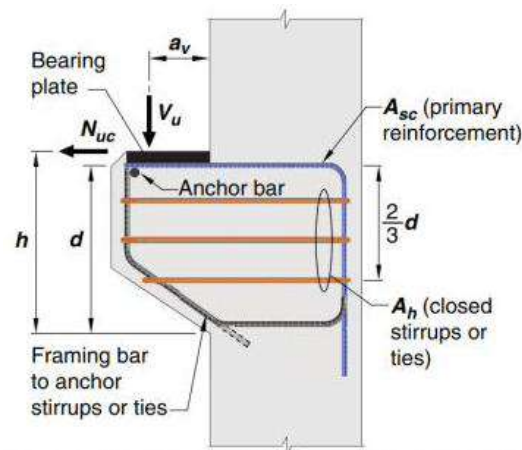


Fig. 5. Typical cross-section: rebars detailing and applied loads [9], [27]

Extensive research has identified several failure modes in corbels [15], which are dependent upon factors such as reinforcement detailing, geometry, material properties, and loading conditions. These modes include:

- Flexural failure: This failure occurs when the main rebars yield under excessive bending stress, as illustrated in Fig. 6(a).

- Shear-induced diagonal failure: In this failure, cracks initiate at the stressed zone under applied load and propagate diagonally toward the area where corbel and column meet, as depicted in Fig. 6(b).
- Shear friction failure: Characterized by near-vertical cracks originating at the upper junction and propagating primarily in a vertical direction toward the bottom fibres, as captured in Fig. 6(c).
- Anchoring splitting failure: This occurs due to insufficient support in the free end zone, as shown in Fig. 6(d).
- Bearing failure: Results from concrete crushing beneath the load-bearing area [38] due to inadequate proportioning of the bearing area, as demonstrated in Fig. 6(e).
- Vertical splitting failure: Typically caused by direct tension resulting from horizontal loads, detailed in Fig. 6(f).

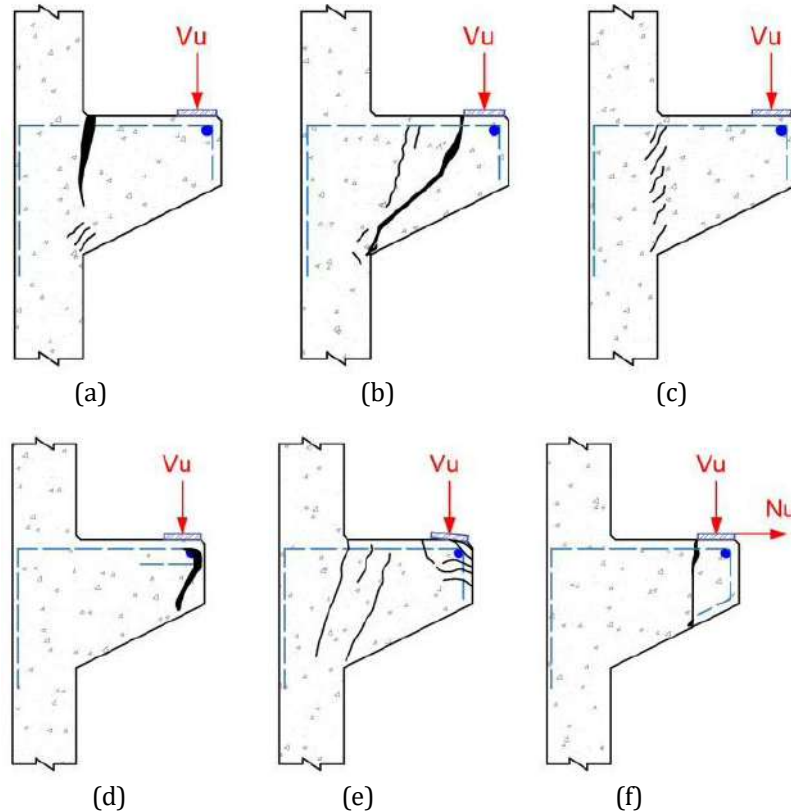


Fig. 6. Failure modes: (a) Flexure, (b) Diagonal shear, (c) Sliding shear, (d) Anchor lack, (e) Local crushing, and (f) Direct tension

These six modes are not equally likely in every situation; which of them governs depends on the application and on the load, condition imposed on the member. Under pure vertical loading with a small a/d ratio, the load reaches the column mainly through a steep compression strut, so the governing mechanisms are those associated with the strut and its end regions, namely diagonal splitting, local crushing beneath the bearing plate, and sliding shear along the column interface. As a/d increases, the strut flattens, flexural action grows in relative importance, and the observed mode migrates towards flexural yielding of the main tie and beam-type diagonal cracking; Lei et al. [44] recorded precisely this migration, from shear sliding at $a/d = 0.35$ to shear rotation at $a/d = 0.96$. When a direct horizontal force acts together with the vertical load, the picture changes qualitatively rather than quantitatively: the tie demand increases, additional moment is transferred at the corbel-column interface, and vertical splitting driven by direct tension becomes credible, which is the reason it is listed as mode (f) and the reason such specimens were excluded from the vertical-load corpus of this review. Kriz and Rathes [15] further observed that crack-control reinforcement becomes progressively less effective as the N/V ratio rises. Reinforcement detailing shifts the outcome in a similar way. Corbels without horizontal stirrups tend towards sudden diagonal splitting, whereas adequate horizontal reinforcement allows the tension tie to yield first

and converts the response into a more ductile flexural or beam-shear mode [17]; anchoring splitting failure, mode (d), is essentially a detailing failure at the free end rather than a strength deficiency of the section. Material choice acts in the same direction, since higher concrete strength raises capacity but increases brittleness unless fibres or hybrid reinforcement are provided, which can move the failure from diagonal splitting to flexural-shear [79]. Finally, the loading history matters in service applications. Corbels supporting crane girders or subjected to seismic action experience repeated or cyclic demand, under which stiffness degradation, bond slip, and progressive crack widening accumulate, so the governing limit state may be serviceability or fatigue-related deterioration rather than the monotonic failure modes of Fig. 6. In precast frames the same member also carries erection and restraint effects at the joint, which is why interface and anchorage modes are reported more often in that application than in isolated laboratory specimens.

4.1 Shear Transfer Mechanism

The shear response of RCCs is strongly influenced by the a/d ratio, which governs the transition in failure modes. According to Wight and MacGregor (2009) [28], this shift typically occurs within the range of 2.0-2.5. Members with a/d ratios greater than 2.5 are considered thin, while those with smaller ratios are classified as deep or non-thin [29]. Borgohain (2023) [29] demonstrated that the a/d ratio has a pronounced effect on shear strength, as illustrated in Fig. 7. When the ratio falls below 2.5, the shear stress at failure increases sharply, indicating a change in the dominant shear transfer mechanism.

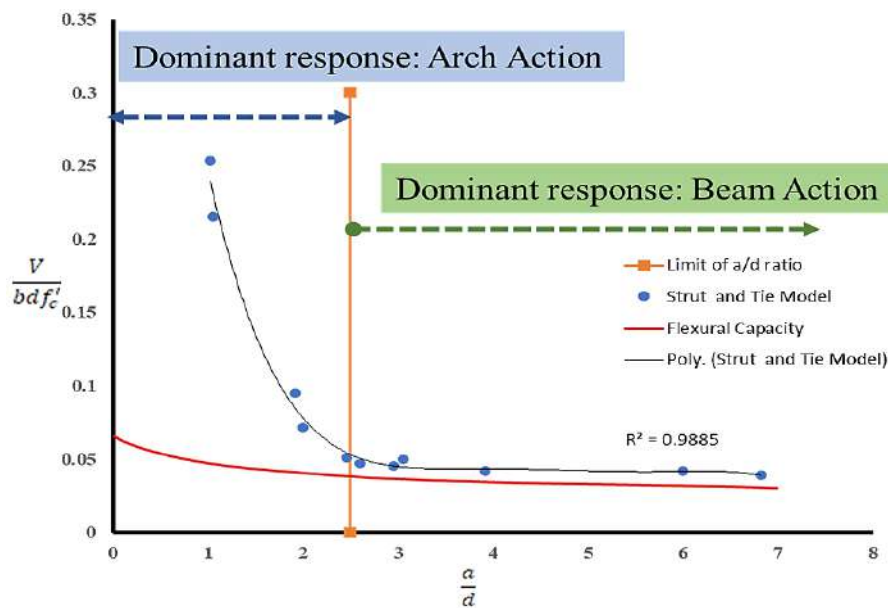


Fig. 7. Effect of shear span-to-effective depth ratio on shear capacity [29]

Because of these behavioral differences, distinct analytical approaches are adopted. Sectional flexural and shear models are generally applied to thin members ($a/d > 2.5$), whereas strut-and-tie (STM) models are more suitable for deep members ($a/d \leq 2.5$), as further discussed in Section 4.2.

Early experimental and analytical investigations into RCCs were carried out by Kriz and Rath (1965) [15] using NSC; their study examined the effects of reinforcement ratio, a/d ratio, concrete strength, and the N/V ratio. They reported that vertical tension reinforcement and horizontal stirrups contributed similarly to corbel strength under vertical loading, while the application of horizontal loads significantly reduced capacity. With the addition of supplementary reinforcement, the observed failure mechanisms tended to converge into a beam-shear type, characterized by the formation of diagonal cracks and subsequent crushing of the strut in compression. The researchers also identified the stress level at which bearing failures developed, concluding that the ultimate nodal strength was approximately $0.34f_c$ for larger a/d ratios. Moreover, they noted that the crack-control reinforcement becomes less effective as the N/V ratio rises.

Recently, studies have addressed various aspects, with their key points summarized as follows:

Mohamed et al. (2025) [30] cast nine fibre-reinforced double corbels to study torsional response under centric and eccentric vertical loads. They recorded first-crack load, ultimate torque, twist angle, displacements, crack patterns and toughness. Results showed that adding vertical and horizontal stirrups increased ultimate torque by about 4.6-18.0%, and widening the corbel raised torque by about 24.3%. Reducing eccentricity from $a/d = 0.75$ to 0.50 lowered cracking load and torque by about 10.7% and 26%, respectively. The study concluded that more stirrups and larger width improve cracking and torsional strength.

Amaral et al. (2025) [31] proposed a novel topology optimization for the design of two-sided RCCs in D-regions under multiple load cases [32], with the aim of achieving lightweight yet structurally reliable designs. Reinforcement areas were determined from the sum of principal forces in finite elements classified as steel, analogous to the STM. A representative result of this approach is shown in Fig. 8, which presents an optimized double-sided corbel with an a/d ratio of 0.59 and illustrates the proposed two-loop steel reinforcement layout. Nonlinear analyses that included both geometric and material nonlinearity were carried out using Abaqus. Although the optimized design showed lower stiffness and strength compared with designs based on the STM method, it still met the required load levels and consistently failed in shear, which was mainly controlled by material volume limitations.

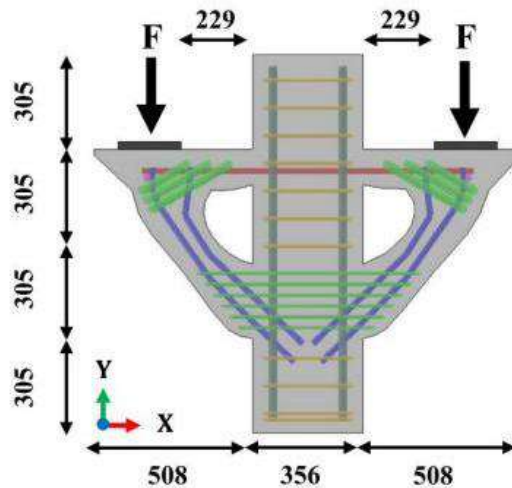


Fig. 8. Optimised corbel ($a/d=0.59$) with reinforcement [31]

Kheyroddin et al. (2025) [33] investigated how changing the arrangement of CFRP, rather than increasing the amount of secondary steel reinforcement, can improve the performance of RCCs using nonlinear finite-element analysis. Two horizontal reinforcement ratios, 0.33% and 0.52%, were analyzed together with a fixed CFRP layout, while a reference model with $\rho_h = 0.18\%$ was also considered. They found that changing the CFRP arrangement alone yielded up to a 33% increase in bearing capacity with the same material volume, while elevating ρ_h to 0.33% and 0.52% resulted in further gains of 5.52% and 11.55%, respectively. The authors conclude that optimizing CFRP configuration notably enhances post-yield bearing capacity.

Huang et al. (2025) [34] investigated how the inclination angle of main reinforcement and stirrups affects the shear performance of RCCs, using both experimental testing and nonlinear finite-element modelling via a concrete damage plasticity (CDP) framework. Two corbel specimens with reinforcement angles of 0° and 15° were tested under vertical loading in a quasi-static setup. The 15° inclination improved shear behavior markedly: yield load rose by 28.3%, ultimate load by 23.6%, strain in main reinforcement dropped by 51.3%, and stirrup yield load increased by 11.6% while its strain level at 250 kN decreased by 23.7%. The CDP model replicated crack development and failure modes accurately, with less than 2.27% error in ultimate load prediction. The authors derived a modified triangular-truss model, Fig. 9, incorporating reinforcement inclination and developed a functional correlation between inclination angle and bearing capacity that aligns closely with test data.

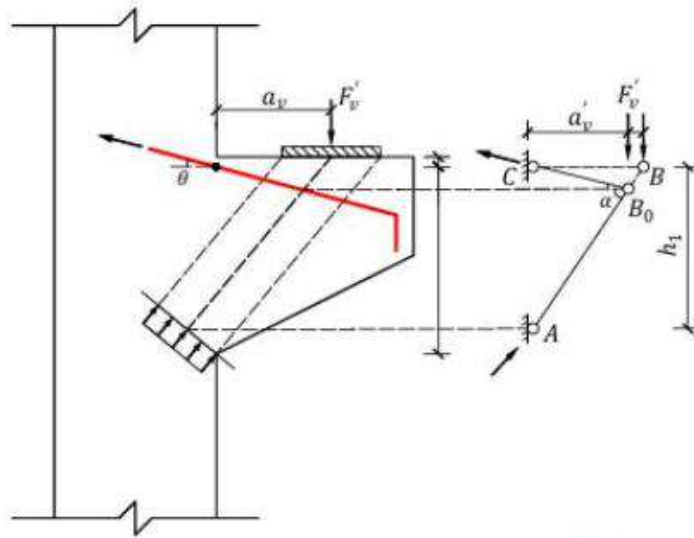


Fig. 9. Modified triangular truss model [34]

Naser et al. (2025) [35] investigated the structural performance of 20 RCCs with different levels of recycled coarse aggregate (RCA) to study the mechanical properties [36]-[38]. The study concluded that replacing natural aggregate by 20%, 40% and 60% reduced compressive strength by about 5.2%, 10.4% and 14.0% and lowered ultimate load by about 4.4%, 8.6% and 16%. Also, higher reinforcement ratios enhanced capacity and stiffness while mitigating deflection, whereas increased RCA content and a/d ratios amplified load-deflection responses. Crack patterns were similar to natural aggregate specimens and energy dissipation was largely unaffected. Prediction using ACI 318 provisions tended to be more conservative compared with experiments.

4.2 Effect of a/d Ratio

The shear a/d ratio significantly influences the structural behavior of elements, particularly affecting shear strength and the development of cracks [39], [40]. A decrease in the a/d ratio generally leads to an increase in the cracking load. For instance, a reduction in the a/d ratio from 0.7 to 0.3 can result in an average increase in cracking load of 155% [41]. Similarly, when the a/d ratio decreases, the cracking loads for corbels with consistent properties tend to increase [41]. This phenomenon is attributed to the decreased moment due to decreasing the distance between supports and applied loads, which enhances the effectiveness of the arching mechanism by transmitting a larger portion of the load directly to the support through a diagonal compression strut [29]. Conversely, an increase in the a/d ratio leads to a greater number of vertical and inclined cracks [42].

The conventional and the proposed corbel specimens in the work of Abdul-Razzaq and Dawood (2020) [43] demonstrated a reduction in ultimate strength with increasing a/d ratio. The results showed that for $a/d = 0.5$, the predicted ultimate capacities using SF and STM were 15% and 22% lower than the experimental failure load, while for $a/d = 1$ these differences increased to 18% and 27%, and for $a/d = 1.5$ they reached 32% and 37%, respectively. Flexural cracks initiated at approximately 19%, 11%, and 12% of the failure load for $a/d = 0.5$, 1, and 1.5, whereas diagonal cracks developed at about 34%, 33%, and 30%, respectively. The authors concluded that STM provides more reliable predictions than SF, particularly when a/d lies between 1 and 2, due to its ability to better represent the failure mode.

Similar findings were obtained in the work of Lei et al. (2025) [44], in which the corbels were reinforced with inclined steel rebars at a/d ratios of 0.35, 0.61, and 0.96. The most important result is the inverse relationship between the shear strength and the a/d ratio, along with the change in failure mode from shear sliding at lower ratios to shear rotation at higher ratios. For example, the

specimen tested at $a/d = 0.35$ carried a load of 940.3 kN, while the specimen with $a/d = 0.96$ resisted only 323.4 kN. These results confirm that the a/d ratio plays a key role in controlling both the ultimate strength and failure behavior of RCCs.

It should also be noted that increasing the a/d ratio generally leads to a reduction in shear strength. The vertical stirrups improve cracking behavior and contribute to the experimental shear resistance; however, the stirrup effectiveness depends strongly on the a/d ratio and the inclination of the compression strut. For very short corbels, where a/d is less than or equal to one, the dominant resistance is mainly provided by direct strut action, and the contribution of vertical stirrups to ultimate strength is limited. As the a/d ratio increases, the internal load path becomes less direct, and vertical stirrups become more active in controlling diagonal cracking and shear transfer. This behavior is clearly depicted in Fig. 10.

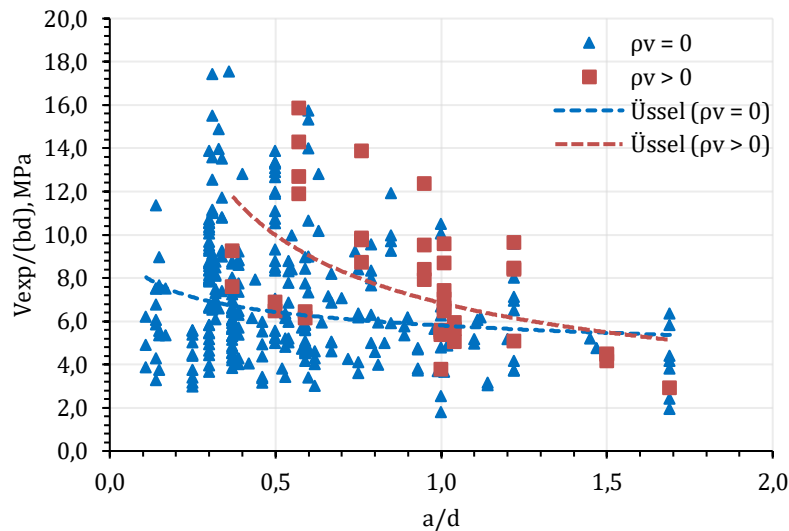


Fig. 10. Influence of a/d ratio on RCC shear strength, with and without vertical stirrups

4.3 Effect of Column Load

Few studies, in terms of structural response, ultimate strength, and crack development, have been found. Early research suggested that axial load applied on the column might interact with corbel loading, which possibly changes the strength [15]. These initial investigations treated additional column load as an important variable affecting the response of corbels [15]. However, later experiments questioned this view, indicating that concentric column loads had little impact on overall corbel strength [45], [46].

Further tests using different reinforcement types and specimen layouts confirmed these results. Corbels with conventional steel bars or steel fibres consistently showed no major change in ultimate strength due to column loading. As a result, many researchers began applying loads directly to corbels, ignoring column load effects [45], [47]. Although strength was not greatly affected, column load was found to influence serviceability behavior, especially cracking. In particular, the presence of column load delayed the onset of cracking and reduced crack widths at the column-corbel connection, which improved crack control and durability [45], [48].

A thorough study performed by Fattuhi (1990) [45], considered one of the most detailed experiments on column load effects, tested corbels with both conventional and steel fibre reinforcement. Based on tests of eight specimens subjected to concentric axial loads, the study reported no noticeable influence of column load on ultimate strength, supporting earlier conclusions by Kriz and Rath (1965) [15]. However, the results highlighted serviceability advantages, as column load delayed the first crack and reduced crack widths at the corbel-column junction. These findings support the general view that column loads have little effect on ultimate capacity but play an important role in cracking behavior and durability.

4.4 Effect of Vertical Stirrups

Vertical stirrups in RCCs mainly affect resistance to diagonal cracking, deformation capacity, and service performance. Their influence on ultimate load strength remains dependent on the geometry of the corbel, especially the a/d ratio and the inclination angle of the compression strut. From a structural perspective, vertical stirrups help restrain the lateral expansion of the diagonal compression strut, increase the load required to initiate diagonal cracking, and delay sudden failure [49], [50]. The test results generally reflect this behavior, showing that a higher vertical stirrup ratio improves both the cracking load and the ultimate load [51]-[55]. In some specimens, an improvement in strength of more than 20% was reported [49]. Furthermore, the presence of vertical stirrups leads to a more uniform crack pattern and better ductile behavior. In contrast, corbels without this reinforcement display less deformation capacity [50]. Despite these advantages, some studies indicate that vertical stirrups may still produce a brittle failure response when compared to horizontal reinforcement [56].

On the other hand, the parameter that controls the effectiveness of the vertical stirrups in addition to the inclined angle includes the a/d ratio of the corbel. When a/d is less than or close to unity, the compression strut is steep and the load is transferred more directly from the bearing region to the column face. Under this condition, the ultimate shear resistance is governed mainly by the concrete strut and the main tension tie, so the additional contribution of vertical stirrups may be limited. As the a/d ratio increases, the strut becomes less steep and diagonal cracking becomes more critical. Therefore, vertical stirrups become more effective in controlling crack propagation and improving shear resistance [43], [57], [58]. This interpretation explains why some STM formulations do not explicitly include the vertical stirrup contribution, while experimental studies still report improvements in cracking behavior and, in some cases, ultimate capacity [59]-[61].

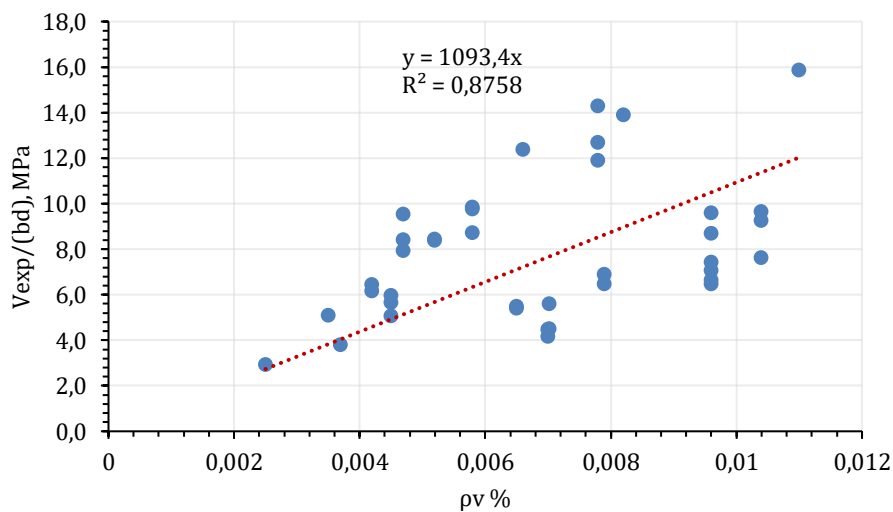


Fig. 11. Influence of vertical stirrups on RCC shear strength

Through the activation of a larger portion of the concrete volume in shear transfer, vertical stirrups also improve the stability of the internal force path and reduce the softening of the compression strut [62], [63]. Although these benefits were noted in the literature, concerns about their reliability have been expressed for many years because the first investigations warned against unsafe behavior due to excessive reliance on vertical stirrups [15].

A detailed series of experiments was conducted by Li et al. (2023) [49] to further clarify the influence of the vertical stirrups on the structural performance of RCCs. The results demonstrated a direct correlation between the ratio of vertical stirrups and the structural performance. An increase in the vertical stirrup ratio from 0.39% to 0.785% resulted in a 23.7% increase in the diagonal cracking load and a 22.6% increase in the ultimate capacity. The authors attributed these improvements mainly to the ability of vertical stirrups to restrain transverse strain within the compression strut, which delays crack formation and improves ductile behavior. At the same time,

the study reported that vertical stirrups had only a minor effect on cracking load related to normal section behavior. This balanced finding emphasized both the advantages and the limits of vertical stirrups and provided a useful reference point for later analytical and design studies. In addition, the presence of vertical stirrups leads to a proportional increase in shear strength capacity, as illustrated in Fig. 11.

4.5 Effect of Horizontal Stirrups

Horizontal stirrups are commonly provided in RCCs to increase shear strength, improve ductile behavior, and reduce the risk of brittle failure [17,62,64]. Their main role is to delay the formation of diagonal tension cracks by confining the concrete core at the connection, leading to increased bond strength, and then allowing the primary tensile reinforcement to reach yielding. As a result, failure is more likely to shift from sudden splitting to more ductile modes, such as flexural or beam-shear failure [17,65,66]. Experimental studies have reported clear gains in both strength and deformation capacity. In some cases, horizontal stirrups increased the vertical failure load by up to 50% [59], while significant improvements in toughness and strain ductility, reaching up to 511%, were observed in steel fibre-reinforced RCCs [42]. In addition to those studies mentioned, several other research works have shown significant increases in shear capacities of corbels equipped with horizontal stirrups; specifically, capacity increases were found to be between 15% and 46% over those that did not include horizontal stirrups [35,67]. The results collectively provide support for the necessity of including an adequate number of horizontal stirrups to preclude brittle failure [15], [58].

However, it has been shown that the effectiveness of horizontal reinforcement depends heavily on both the a/d ratio and the manner in which the corbel is loaded. For example, when a/d ratios are low (less than or equal to 0.9) and the angle of the compression strut exceeds 45° , horizontal reinforcement plays a major role in transferring loads across cracks and controlling crack propagation [17,59]. In contrast, for $a/d = 1.0$, its contributions to shear capacity have been considered minimal [58]. Horizontal reinforcement may provide relatively little enhancement of shear capacity when corbels are subjected to combined vertical and horizontal loading; under such conditions, horizontal reinforcement has been described as reserve strength due to confinement rather than a primary load-carrying mechanism [15,59]. More recently, researchers have noted the influence of horizontal reinforcement on torsion behavior; they report that increasing amounts of horizontal reinforcement can lead to increases in ultimate torque of as much as 13.3% [30]. Yet, in over-reinforced specimens, their contribution diminishes, mainly serving to mitigate concrete softening rather than significantly increasing shear strength [62].

Beyond their role in capacity, horizontal stirrups serve a critical serviceability function by enhancing crack control, reducing deformation, and improving stiffness. They confine the concrete, redistribute stresses, and reduce crack widths particularly for corbels with small a/d ratios [53], [68]. Experimental findings report ultimate load increases between 19% and 27% in fibre-reinforced corbels with stirrups [65], while other studies found cracking load improvements of up to 34.78% with increased stirrup ratios [69]. However, beyond certain reinforcement thresholds, especially in high-strength concrete, further increases in stirrup content produce limited strength gains [41]. Importantly, compared to vertical stirrups, horizontal loops are generally more effective because they intercept the steep diagonal cracks, thus reducing brittleness and yielding a safer, more predictable failure mechanism [15,56].

Mattock et al. (1976) [17] conducted one of the earliest systematic experimental programmes on RCCs, establishing the fundamental role of horizontal stirrups in controlling shear behavior and preventing brittle failures. Their tests demonstrated that the absence of stirrups often led to diagonal splitting, while their inclusion enabled the tensile reinforcement to yield, producing flexural or beam shear failures considered acceptable in design practice. They also introduced a design expression for the minimum required horizontal reinforcement area [70], $A_h \geq 0.5(A_s - A_n)$, and recommended its distribution within the upper two-thirds of the effective depth adjacent to the main tension reinforcement. Furthermore, they showed that the horizontal stirrup contribution varied with a/d , becoming negligible at $a/d = 1.0$ but highly effective when $a/d < 0.9$. Their findings

remain foundational, informing subsequent research and influencing modern code provisions for corbel design.

Furthermore, the inclusion of horizontal stirrups proportionally improves the RCC shear strength whether the horizontal force exists or not, as shown in Fig. 12.

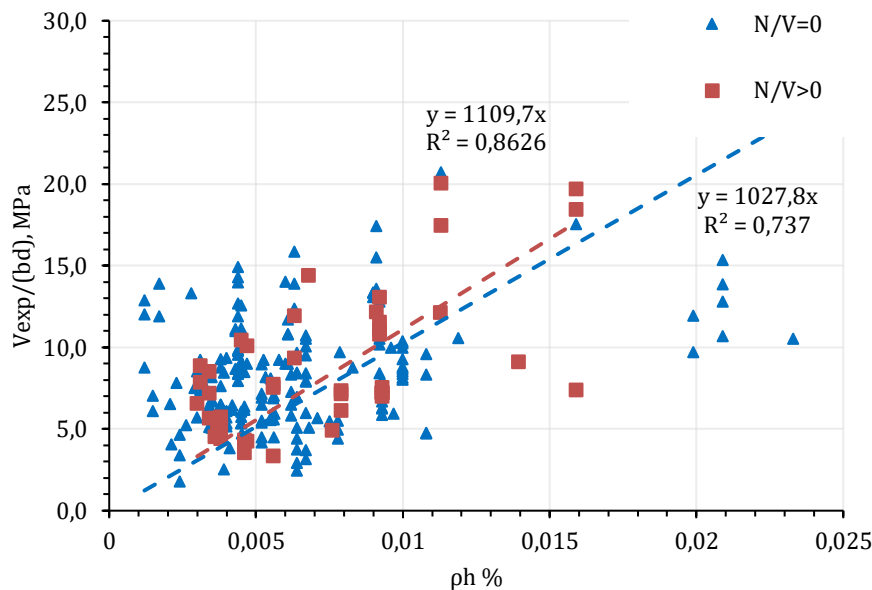


Fig. 12. RCC shear strength vs. horizontal link ratio under vertical and combined loading

4.6 Effect of Concrete Compressive Strength

Concrete compressive strength (f_{ck}) is a fundamental parameter controlling the load-carrying capacity, shear resistance, stiffness, and failure mode of RCCs [41,49,50,71-75]. Increasing f_{ck} generally enhances the shear capacity, cracking load, and ultimate load, with reported improvements ranging from 20% to over 65% depending on reinforcement and a/d ratios [42], [76,77]. Corbels made from high-strength concrete show higher longitudinal steel strain and ductility [78] in addition to energy absorption and flexural toughness as well as higher stiffness and a more gradual load-deflection relationship due to higher strength concrete [76,79].

It is difficult to determine how changes in f_{ck} affect failure modes. The addition of higher strength concrete may cause corbels to become more brittle; however, the addition of fibres or hybrid reinforcement will reduce that risk and can change the failure mode from diagonal splitting to a flexural-shear failure [79-84]. The influence of compressive strength also varies with geometry: for larger a/d ratios, its effect on peak load may lessen, whereas in steel-fibre and hybrid high-strength concrete corbels [85], higher f_{ck} consistently improves ultimate capacity [40,86,87].

Design models show that increasing f_{ck} generally improves safety margins in code-based STMs, though results differ between standards [67]. Experiments further reveal that corbels with higher concrete strength can have up to 58% greater shear and moment capacity than those with lower strength [88]. In general, concrete compressive strength remains the key factor governing corbel behavior, mainly because it resists compressive forces [88].

Al-Allaf et al. (2024) [79] adopted a parametric study on hybrid corbels to measure how concrete strength affects stiffness, energy absorption, and failure mode. By varying f_{ck} from 30 MPa to 50 MPa, they observed energy absorption increases of 114-173% and a shift in failure mode from diagonal splitting to flexural-shear at 40 MPa, with some return toward splitting at 50 MPa. The study underlined the interaction between concrete strength, a/d ratio, and reinforcement quantity, concluding that while higher f_{ck} improves performance, fibres may be needed to offset increased brittleness.

Based on the dataset available from the literature, Fig. 13 illustrates the relationship between the square root of the concrete compressive strength and the experimental shear strength for RCCs. The trend line shows a positive correlation between the two variables. The regression equation, with a high coefficient of determination ($R^2=0.9107$), indicates that the experimental shear strength is strongly influenced by the concrete compressive strength.

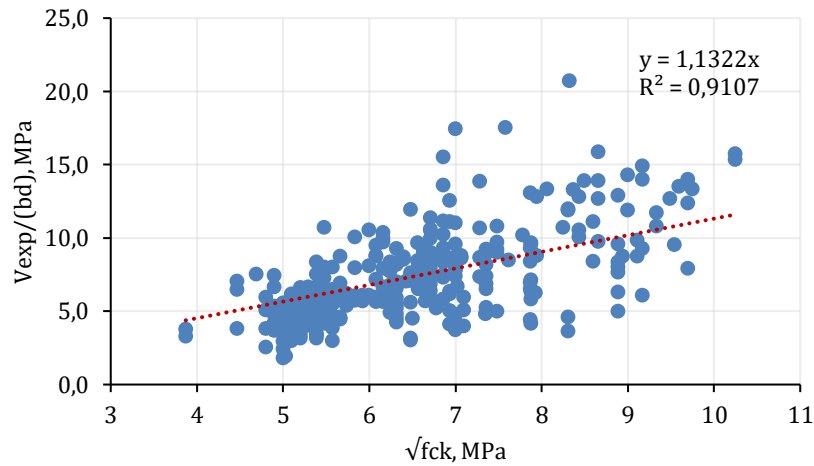


Fig. 13. Variation of RCC shear strength with concrete compressive strength

4.7 Effect of Longitudinal Reinforcement Ratio

The longitudinal reinforcement ratio (ρ_f), often referred to as the main tension reinforcement ratio, is a critical factor influencing the ultimate load capacity, stiffness, ductility, and failure mode of RCCs [35,49,50,89,90]. Increasing ρ_f generally enhances the ultimate load and shear strength, with reported gains ranging from 6% to over 40% depending on concrete type, a/d ratio, and secondary reinforcement amount [35,49,77]. Numerical models show that shear strength rises sharply with longitudinal steel ratios up to 1.3%, beyond which the effect exhibits diminishing returns [91].

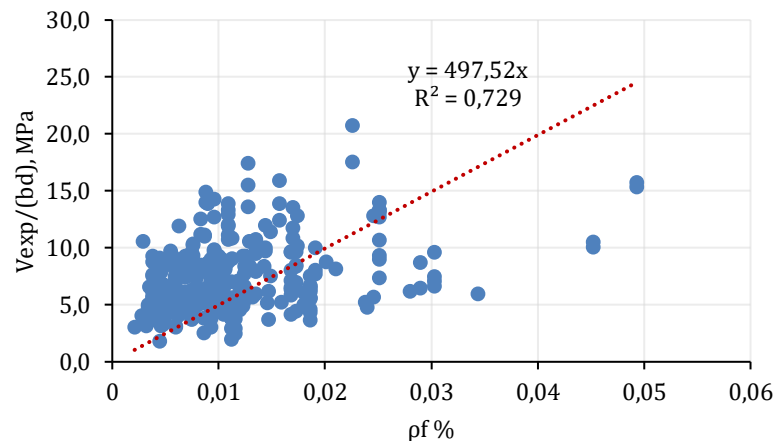


Fig. 14. Correlation of RCC shear strength with tensile reinforcement ratio

The longitudinal reinforcement also governs cracking behavior and ductility. Increasing ρ_f raises the inclined section cracking load while having a minor effect on the normal section, and promotes wider and longer flexural cracks along the corbel-column interface [49,73,77]. Low reinforcement ratios lead to yielding of the steel before concrete crushing, resulting in ductile failure, whereas high ratios may induce brittle failure dominated by strut crushing or nodal failure [53,64,92]. Under initial loading, the concrete carries most of the stress, but post-cracking, the longitudinal reinforcement governs the bending response and impacts the depth of the compression block [49], [61]. Serviceability aspects, including deflection and crack width, are also affected by ρ_f , and

analytical models incorporate it alongside other parameters to predict corbel performance accurately [39,93].

Li et al. (2023) [49] experimentally investigated steel-fibre RCCs, demonstrating that increasing ρ_f from 0.55% to 0.98% enhanced ultimate loads by up to 19.4% and raised inclined section cracking loads by 33.3%. Strain measurements showed that, once concrete cracking occurs, the longitudinal reinforcement ratio ρ_f becomes more actively involved, through which ductility and internal stress redistribution are improved. As illustrated in Fig. 14, the shear strength of RCCs displays a clear and positive relationship with ρ_f , a trend that is mainly governed by flexural resistance rather than by shear transfer mechanisms such as dowel action. If the dowel action contribution were included, the correlation would likely become stronger.

4.8 Effect of k/h Ratio

The external depth to internal depth (k/h) ratio, which describes the relative depth between the free end of the corbel and the column interface, is not commonly considered as a primary parameter in existing studies because the failure in RCCs normally occurs at the column interface. However, the effect can be inferred through the geometric properties that govern corbel behavior. One of these most important properties is the a/d ratio, which is generally considered the dominant factor controlling both load capacity and governing failure mode [94]-[97]. The more recent studies that have examined the k/h ratio directly confirmed that an increase in k/h from 0.24 to 1.00 led to an 11.5% rise in ultimate shear capacity. This indicates that larger k/h values improve diagonal compression action and the efficiency of load transfer [77]. In line with this conclusion, Somerville (1974) [98] suggested that maintaining k/h within the range of 0.45-0.60 provides a balanced geometry, by which secondary failure mechanisms can be reduced [60].

This geometric interpretation is supported by the explainable ML results reported later in Section 7.4. The column width w_c ranked third in the SHAP importance list, with a mean absolute SHAP value of 0.0217 and a consistently positive effect direction (Table 12), and the corbel height h ranked first. Since the k/h ratio is defined by the same pair of depths that fix the tapered profile between the free end and the column face, a member proportioned with a larger k/h offers a deeper and better-supported nodal region at the interface, which is precisely the region whose size the model rewards through h and w_c . The SHAP evidence therefore reinforces, from an independent data-driven direction, the experimental observation of an 11.5% capacity gain as k/h increased from 0.24 to 1.00, and it explains why k/h should not be treated as a purely detailing-related quantity: it modifies the geometry of the compression field and of the support zone, which the model identifies as among the strongest contributors to predicted capacity. It also indicates that reporting k/h explicitly in future experimental programmes would allow this variable to be tested directly rather than inferred through correlated dimensional inputs.

In general, the reviewed studies show that the shear behavior of RCCs is controlled by a combined effect of geometry, concrete strength, reinforcement arrangement, and loading condition. Although each parameter has a distinct influence, its effect cannot be fully separated from the internal strut-and-tie mechanism. Table 7 summarizes the main behavioral trends identified from the reviewed literature and clarifies the practical implication of each parameter.

5. Strengthening and Repair of RCCs

Although the main part of this review focuses on original RCCs and their shear behavior under vertical loading, strengthening and repair studies form an important related branch of the literature. In practice, corbels may require strengthening because of increased service loads, construction defects, insufficient detailing, material deterioration, accidental damage, or changes in use. Since corbels are usually located at highly stressed support regions, any local weakness may strongly affect the load path between the applied load and the column face. For this reason, several researchers have examined different strengthening techniques, including externally bonded CFRP sheets [109-112], near-surface-mounted reinforcement [113], steel plates [114-116], basalt-fibre systems [117], post-installed composite fabrics [118], and local repair using fibre-reinforced materials [59,119].

Table 7. Summary of the main parameters affecting RCCs shear behavior

Parameter	General effect on shear behavior	Main interpretation	Practical implication
Shear span-to-effective depth ratio, a/d	Increasing a/d generally reduces shear strength and changes the failure mechanism.	Small a/d values promote direct compression-strut action, while larger values increase flexural and diagonal cracking effects.	The a/d ratio should be treated as one of the primary controlling variables in design and prediction models.
Concrete compressive strength, f_c' or f_{ck}	Higher concrete strength usually increases cracking load, stiffness, and ultimate shear capacity.	The concrete compression strut and nodal zones become stronger as concrete strength increases.	Higher-strength concrete improves capacity, but may increase brittleness if not accompanied by adequate reinforcement or fibres.
Corbel width and depth	Larger dimensions generally increase the load-carrying capacity.	Wider and deeper corbels provide a larger stress-transfer area and improve the efficiency of the compression field.	Size-related variables should be included in predictive models, especially when the output is ultimate load rather than normalized stress.
Longitudinal reinforcement ratio, ρ_f	Increasing ρ_f improves tensile-tie resistance, stiffness, and ultimate capacity up to a certain level.	The main reinforcement forms the tension tie in the strut-and-tie mechanism and helps maintain equilibrium after cracking.	Excessive longitudinal reinforcement may reduce ductility and shift failure toward concrete crushing or nodal distress.
Horizontal reinforcement ratio, ρ_h	Horizontal stirrups improve crack control, ductility, and shear capacity, especially for short corbels.	Horizontal reinforcement intercepts diagonal cracks and confines the disturbed region near the corbel-column interface.	Adequate horizontal reinforcement is needed to reduce brittle splitting and improve post-cracking behavior.
Vertical reinforcement ratio, ρ_v	Vertical stirrups improve diagonal crack control and deformation capacity, but their strength contribution depends on a/d .	Their effect is limited in very short corbels dominated by direct strut action, but becomes more active as diagonal cracking becomes more pronounced.	Vertical stirrups should be considered mainly as crack-control and ductility-enhancing reinforcement, with careful interpretation in strength models.
Column load	Usually has limited influence on ultimate shear capacity, but can improve cracking behavior.	Axial compression may delay crack initiation and reduce crack width at the corbel-column junction.	Column load should not be assumed to significantly increase ultimate capacity unless supported by specific test evidence.
Loading history	Repeated or cyclic loading may reduce stiffness, increase deflection, widen cracks, and reduce ultimate capacity.	Damage accumulation, bond slip, and progressive cracking modify the internal load-transfer path.	Corbels subjected to crane, seismic, or repeated service actions require additional attention to ductility and crack control.
Strengthening and repair	CFRP, NSM reinforcement, steel plates, and fibre-based systems can increase capacity and stiffness.	Strengthening provides additional tensile resistance and crack bridging, but may introduce debonding or brittle failure.	Strengthened corbels should be interpreted separately from original unstrengthened specimens in databases and design models.

CFRP strengthening has received the greatest attention because it can be applied externally with limited change to the member geometry [120]. Al-Kamaki et al. [109] investigated the behavior of RCCs strengthened with CFRP sheets, while Abdulrahman et al. [110] examined the use of CFRP to improve the performance of high-strength RCCs. Similar studies by Assih et al. [111] and Ivanova et al. [112] also showed that externally bonded carbon-fibre fabrics can increase the capacity of damaged or deficient corbels. These studies generally indicate that CFRP strengthening improves load capacity and crack control by providing additional tensile resistance across the critical cracking zone. However, the effectiveness of this method depends strongly on bond quality, anchorage length, fibre orientation, and the ability of the strengthening layer to remain attached until high load levels are reached.

A recurring issue in CFRP-strengthened corbels is premature debonding. Ivanova et al. [121] studied the influence of anchorage length of composite fabrics and showed that the strengthening system cannot be assessed only by the amount of fibre material used. The development length, end anchorage, and local stress concentration near the corbel-column junction are also important. Ghozani et al. [122] further demonstrated through numerical modelling that different CFRP laminate layouts can lead to different stress-transfer patterns and failure responses. These findings suggest that strengthening design should not only aim to increase the nominal capacity, but should also control debonding, peeling, and local brittle failure.

Near-surface-mounted strengthening has also been used as an alternative to externally bonded sheets [123,124]. Shakir and Kamonna [125] studied high-strength self-compacting concrete corbels strengthened with NSM steel bars, while Shakir later examined retrofitting of high-strength self-compacting corbels using near-surface-mounted steel reinforcement. Ali and Abbas [126] also investigated lightweight concrete corbels strengthened with NSM-CFRP bars. Compared with externally bonded sheets, NSM systems can provide better protection to the strengthening material and may improve bond behavior because the bar or strip is embedded inside grooves cut into the concrete cover. Nevertheless, their performance still depends on groove dimensions, adhesive quality, bar anchorage, and the condition of the surrounding concrete.

Other researchers have explored steel plates, basalt fibres, and steel-fibre-based strengthening systems. Fadel and Waryosh [115] examined normal- and high-strength RCCs strengthened with steel plates, showing that plate strengthening can be useful where higher stiffness and direct force transfer are needed. Gülşan et al. [117] investigated the rehabilitation of normal and self-compacted steel-fibre-reinforced concrete corbels using basalt-fibre systems. Abdul-Alhassan and Kadhim [127] studied reinforced concrete brackets strengthened with steel fibre. These approaches show that the strengthening of corbels is not limited to FRP systems, and that material selection should depend on the type of deficiency, exposure condition, available anchorage length, and required ductility.

Analytical and numerical work has also supported the interpretation of strengthened corbel behavior. Campione and Cannella [128] proposed an analytical model for the flexural response of RCCs externally strengthened with FRP, while several finite-element studies used nonlinear modelling to describe crack propagation, stress redistribution, and damage development in strengthened specimens. These studies are useful because direct experimental testing of every strengthening layout is difficult, especially when the behavior involves concrete cracking, bond slip, fibre rupture, debonding, and local crushing.

Comparable numerical evidence from adjacent member types supports the same conclusion. Abbas et al. [129] applied a concrete damage plasticity model to RC beams strengthened with high-performance concrete (HPC) under torsion, and Abbas and Hamzah [130] examined hybrid concrete T-beams in which HPC was used to remedy deteriorated sections. Both identify the interface between the original and the added material as the quantity controlling the response, which is the same requirement established for bonded and NSM systems in RCCs.

Because the effectiveness of a strengthening scheme is decided less by the material selected than by the way it is detailed, it is useful to set the individual studies side by side rather than to describe them in isolation. Table 8 therefore compiles the representative strengthening and repair case

studies retained in the review, giving for each one the condition of the corbel and the technique applied, the outcome as reported in the source, and the practical issue that governed that outcome.

Table 8. Representative strengthening and repair case studies reported in the reviewed literature

Case study	Corbel condition and technique	Reported outcome in the reviewed source	Governing practical issue
Al-Kamaki et al. [109]	Deficient RCCs strengthened with externally bonded CFRP sheets.	Externally bonded sheets increased load capacity and improved crack control by adding tensile resistance across the critical cracking zone.	Effectiveness controlled by bond quality and by the ability of the sheet to remain attached up to high load levels.
Abdulrahman et al. [110]	High-strength RCCs upgraded with CFRP.	CFRP improved the performance of high-strength corbels, confirming that the technique is not restricted to normal-strength concrete.	Higher concrete strength does not remove the sensitivity of the system to interfacial bond.
Assih et al. [111]; Ivanova et al. [112]	Damaged or deficient short corbels strengthened with bonded carbon-fibre fabrics.	Bonded fabrics increased the capacity of damaged or deficient corbels and modified crack development in the shear zone.	Local stress concentration close to the corbel-column junction.
Ivanova et al. [121]	Strengthened short corbels with varying anchorage length of composite fabric.	Performance could not be explained by the amount of fibre alone; development length, end anchorage, and local stress concentration were decisive.	Anchorage detailing rather than fibre quantity governs the achievable gain.
Ghozani et al. [122]	Numerical study of alternative CFRP laminate layup schemes.	Different laminate layouts produced different stress-transfer patterns and failure responses for the same member.	Layup configuration changes the failure sequence and must be designed, not assumed.
Shakir and Kamonna [125]; Shakir and Alsaheb [113]	High-strength self-compacting corbels strengthened and retrofitted with NSM steel bars.	Embedding the reinforcement in grooves protected the strengthening material and improved bond behavior relative to externally bonded sheets.	Groove dimensions, adhesive filling, and development length control the result.
Ali and Abbas [126]	Lightweight concrete corbels strengthened with NSM-CFRP bars.	NSM-CFRP was applicable to lightweight concrete corbels, extending the technique beyond normal-weight concrete.	Condition of the surrounding concrete cover limits groove performance.
Fadel and Waryosh [115]	Normal- and high-strength RCCs strengthened with steel plates.	Plate strengthening was effective where higher stiffness and direct force transfer were required.	Stress concentration, corrosion risk, added weight, and anchorage difficulty.
Gulsan et al. [117]; Abdul-Alhassan and Kadhim [127]	Rehabilitation of normal and self-compacted SFRC corbels using basalt fibre, and brackets strengthened with steel fibre.	Fibre-based rehabilitation improved strength and serviceability, showing that corbel strengthening is not limited to FRP systems.	Material selection must follow the type of deficiency, exposure, anchorage length, and required ductility.
Campione and Cannella [128]	Analytical model for the flexural response of FRP-strengthened corbels.	Provided a means of interpreting strengthened behavior without testing every possible layout.	Reliability depends on bond assumptions and validation against experiments.

Read as a group, these cases show a consistent pattern: successful strengthening in the reviewed literature is associated with a controlled force path into the strengthening element, that is, with

adequate development length, sound end anchorage, and a bond interface capable of surviving up to the intended load level. Al-Kamaki et al. [109] and Abdulrahman et al. [110] confirmed that externally bonded CFRP raises capacity in both normal- and high-strength corbels, but the comparison between Ivanova et al. [121] and Ghozani et al. [122] is more instructive, since the first shows that fibre quantity alone does not determine performance once anchorage length is varied, and the second shows that two layups of identical material can produce different failure responses. The near-surface-mounted cases [113,125,126] address the same difficulty from the opposite direction, by relocating the strengthening element into grooves where it is protected and better bonded, at the cost of introducing groove geometry and adhesive filling as new controlling variables. The steel-plate and fibre-based cases [115,117,127] extend the available options where stiffness or corrosion resistance governs the choice. The unifying practical lesson for rehabilitation design is that the target of the intervention should be the failure mechanism identified in the deficient member, not the nominal capacity alone, since every technique in Table 8 is capable of shifting the governing mode as well as raising the strength.

Table 9. Summary of strengthening and repair techniques for RCCs

Strengthening or repair technique	Main contribution	Typical advantages	Main limitations or failure concerns
Externally bonded CFRP sheets	Provides additional tensile resistance across critical cracking regions.	Increases ultimate load, improves crack control, and can be applied with limited change in member geometry.	Premature debonding, peeling, local stress concentration, and limited anchorage length may reduce effectiveness.
CFRP laminate or fabric anchorage systems	Improves the force transfer between CFRP and concrete.	Delays debonding and enhances the utilization of the strengthening material.	Performance depends strongly on anchorage length, end detailing, adhesive quality, and surface preparation.
Near-surface-mounted steel or CFRP bars	Adds embedded reinforcement near the critical tension zone.	Provides better protection than externally bonded sheets and may improve bond performance.	Requires groove cutting, proper adhesive filling, adequate development length, and sound surrounding concrete.
Steel plate strengthening	Provides high stiffness and direct force transfer.	Useful where large stiffness increases or local strengthening is required.	May cause stress concentration, corrosion risk, additional weight, and anchorage difficulty.
Basalt-fibre or composite fabric systems	Provides additional crack-bridging and tensile resistance.	Can improve strength and serviceability while offering corrosion resistance.	Bond failure and limited experimental data may restrict direct design application.
Steel-fibre or fibre-reinforced repair materials	Enhances local tensile resistance and crack control in damaged regions.	Improves toughness, crack resistance, and local repair performance.	Effectiveness depends on repair thickness, interface bond, fibre distribution, and damage level before repair.
Numerical and analytical strengthening models	Helps interpret stress transfer, crack propagation, and debonding mechanisms.	Reduces the need for testing every strengthening layout and supports parametric evaluation.	Reliability depends on material models, bond assumptions, calibration quality, and validation against experiments.

In general, strengthening and repair studies show that the capacity of deficient or damaged RCCs can be improved, but the strengthened system may behave differently from an original unstrengthened corbel. The main gains are usually related to higher ultimate load, delayed crack initiation, reduced crack width, and improved stiffness. However, the main concerns are premature debonding, brittle failure, limited ductility, and sensitivity to anchorage detailing. Therefore, strengthening studies should be treated separately from the main database of original RCCs behavior. They are valuable for rehabilitation practice and for identifying future research needs, but they should not be mixed directly with unstrengthened specimens when developing or validating shear-capacity models.

As a conclusion, the reviewed strengthening and repair studies demonstrate that deficient or damaged RCCs can be upgraded using several techniques. However, the effectiveness of each method depends on the strengthening configuration, anchorage condition, bond quality, concrete condition, and governing failure mode. Table 9 summarizes the main strengthening and repair approaches reported in the reviewed literature and identifies their typical advantages and limitations.

The findings indicate that strengthening and repair techniques can improve the capacity and serviceability of RCCs, but they may also change the governing failure mechanism. Therefore, strengthened or repaired specimens should be classified separately from original unstrengthened specimens when developing experimental databases or ML-based shear-capacity models.

6. Systematic Review of Analytical and Code-Based Models

RCCs are discontinuity regions (D-regions) with nonlinear strain behavior governed mainly by shear [44], [95]. Therefore, several analytical and code-based models have been adopted by international design provisions to estimate the ultimate capacity of corbels [131], [132]. In this section, a comparison is presented among four analytical methods adopted by three selected international building codes [133]. These provisions include ACI 318-25 [27], which employs two methods: the Shear Friction Method and the Strut-and-Tie Method; BS EN 1992-1-1:2004 [134], which adopts a Simplified Strut-and-Tie Model; and GB 50010-2010 [11], which applies a Triangular Truss Analogy Model, as follows:

6.1 ACI 318 Shear Friction Method

The shear friction method as per the ACI 318-25 [27] code includes a simplified design procedure for corbels with an a_v/d ratio of unity or less, subjected to both vertical and horizontal loads. This procedure is based on a simple mechanical model, often referred to as the flexural model, which has been adopted by the ACI 318-25 code [27].

The ultimate shear stress (v_u) should not exceed the limits specified in Eq. (1):

$$v_u \leq \min(0.2f'_c, 5.5 \text{ MPa}) \quad (1)$$

The area of reinforcement crossing the shear plane, A_{vf} , is determined using Eq. (2):

$$A_{vf} = \frac{V_u}{0.75f_{yf} \cdot \mu} \quad (2)$$

Where $\mu = 1.4$ for corbels cast monolithically with the column.

The ultimate moment M_u , which the corbel-column interface must resist, is calculated using Eq. (3):

$$M_u = V_u \cdot a_v + N_u \cdot (h - d) \quad (3)$$

The area of reinforcement, A_f , required to resist the ultimate moment is found by Eq. (4):

$$A_f = \frac{M_u}{\phi \cdot f_{yf} \cdot \left(d - \frac{a}{2}\right)} \quad (4)$$

The area of reinforcement, A_n , required to resist the horizontal force N_u is given by Eq. (5):

$$A_n = \frac{N_u}{0.75 f_{yf}} \quad (5)$$

The main reinforcement area, A_s , is determined by taking the greater value of Eq. (6):

$$A_s = \max\left(A_f + A_n, \frac{2}{3} A_{vf} + A_n\right) \quad (6)$$

Closed stirrups, with an area A_h , parallel to the main tension reinforcement A_s , should be uniformly distributed within two-thirds of the effective depth and calculated by Eq. (7):

$$A_h \geq 0.5(A_s - A_n) \quad (7)$$

Lastly, the steel ratio ρ should not be less than the value of Eq. (8):

$$\rho = \frac{A_s}{b \cdot d} \geq 0.04 \cdot \frac{f'_c}{f_{yf}} \quad (8)$$

6.2 ACI 318 Strut-and-Tie Model

As outlined in ACI 318-25, the STM is applicable for corbels with a shear a/d ratio not exceeding 2. This model assumes a structural composition primarily of steel reinforcement. Initially, an STM is devised based on the principle of minimum strain energy [135]. The model chosen forms a truss system comprising diagonal concrete compression struts and longitudinal reinforcement.

As depicted in Fig. 15, the STM includes CCC (three-sided compression) nodal zones B and B', CCT (a combination of two-sided compression and one-sided tension) nodal zones A and A', concrete compression struts AB and A'B', and steel ties AA'. The width of the CCT nodal region corresponds to the width of the supporting plate, while the height is twice the distance from the concrete edge to the longitudinal reinforcement's center. For the CCC nodal region, the width is half the column width [136], and the height represents the equivalent height of the compression zone, symbolized as β_{1c} .

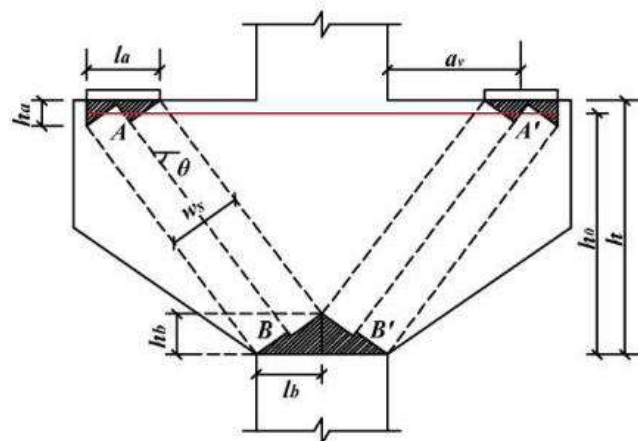


Fig. 15. Strut and tie model for symmetrical two-sided corbel [67]

The STM simplifies complex stress distributions into uniaxial force elements comprising compression struts, tensile ties, and nodal regions [137]-[141]. These components together withstand the external loads. The code mandates strength checks for each element, ensuring that when any element reaches its full capacity, the member attains its maximum value (V_u). Furthermore, the STM adheres to the principles of static equilibrium and material yield conditions,

ensuring all nodes satisfy equilibrium equations and stress levels in each element remain below yield stress. Theoretically, the STM is subject to a lower-bound theorem of limit analysis.

The angle θ and the strut width w_s of the compression strut are determined using the following formulas:

$$\tan \theta = \frac{\left(h_0 - \frac{h_b}{2}\right)}{\left(a_v + \frac{l_b}{2}\right)} \quad (9)$$

$$w_s = \text{Min of } (h_a \cos \theta + l_a \sin \theta, h_b \cos \theta + l_b \sin \theta) \quad (10)$$

The load limit of each model component was calculated. By applying static equilibrium conditions, the ultimate shear strength of the corbel was determined at the point where each component reached its ultimate strength. The lowest computed value was selected as the shear strength of the corbel, referred to as V_u . The formulas for each component are as follows:

$$V_{u,AB} = F_{u,AB} \sin \theta = f_{ce,s} w_s b \sin \theta \quad (11)$$

$$V_{u,AA'} = F_{u,AA'} \tan \theta = f_{yf} A_s \tan \theta \quad (12)$$

For the bearing capacity calculations at Node A, $V_{u,NodeA}$ is taken as the minimum of the three expressions given in Eq. (13):

$$\begin{aligned} V_{u,ba} &= F_{u,ba} \tan \theta = f_{ce,n} h_a b \tan \theta \\ V_{u,inc} &= F_{u,inc} \sin \theta = f_{ce,n} (h_a \cos \theta + l_a \sin \theta) b \sin \theta \\ V_{u,be} &= F_{u,be} = f_{ce,n} l_a b \end{aligned} \quad (13)$$

Similarly, $V_{u,NodeB}$ is taken as the minimum of the three expressions given in Eq. (14):

$$\begin{aligned} V_{u,ba} &= F_{u,ba} \tan \theta = f_{ce,n} h_b b \tan \theta \\ V_{u,inc} &= F_{u,inc} \sin \theta = f_{ce,n} (h_b \cos \theta + l_b \sin \theta) b \sin \theta \\ V_{u,be} &= F_{u,be} = f_{ce,n} l_b b \end{aligned} \quad (14)$$

The values of $f_{ce,s}$ and $f_{ce,n}$ are calculated as:

$$f_{ce,s} = 0.85 \beta_c \beta_s f_c' \quad (15)$$

$$f_{ce,n} = 0.85 \beta_n f_c' \quad (16)$$

$$h_b = \beta_1 c = \frac{f_{yf} A_s}{0.85 f_c' b} \quad (17)$$

Finally, the shear strength of the corbel V_u is given by the minimum of the individual capacities:

$$V_u = \text{Min of } (V_{u,AB}, V_{u,AA'}, V_{u,NodeA}, V_{u,NodeB}) \quad (18)$$

In the given formulas, $F_{u,AB}$, $F_{u,AA'}$, $F_{u,ba}$, $F_{u,be}$, and $F_{u,inc}$ denote the maximum loads that various structural elements can withstand. Specifically, $F_{u,AB}$ represents the load capacity of the compression struts, $F_{u,AA'}$ for the tension ties, $F_{u,ba}$ for compression struts in the nodal area or surfaces normal to horizontal stirrups, $F_{u,be}$ for concentrated loads in the nodal zone or surfaces normal to vertical compression struts, and $F_{u,inc}$ for surfaces normal to inclined compression struts in the nodal zone. Correspondingly, $V_{u,AB}$, $V_{u,AA'}$, $V_{u,NodeA}$, and $V_{u,NodeB}$ refer to the corbel bearing capacities when each of these components reaches their ultimate strength. The formulas also define $f_{ce,s}$ and $f_{ce,n}$ as the maximum allowable compressive stresses for the concrete in the struts and the nodal zones, respectively.

The confinement factor, β_c , for the strut and node is determined as the smaller value between $\sqrt{A_2/A_1}$ and 2. Here, A_1 represents the bearing area, while A_2 corresponds to the area of the lower base of the largest frustum of a tapered wedge, which is entirely contained within the support and has an upper base equal to the loaded area. For corbel elements, β_c equals 1. The strut coefficient, β_s , is used to adjust for confinement conditions. It is set at 0.75 for interior struts with an orthogonal reinforcement grid having a minimum distributed ratio of 0.0025 in each direction, or with reinforcement in one direction crossing the strut at angle α_1 of $0.0025/\sin^2\alpha_1$, provided that α_1 shall not be less than 40° . Otherwise, β_s is set at 0.4.

Similarly, the nodal zone coefficient, β_n , is taken as 1.0 for the node "B" under three-sided compression, and 0.8 for the node "A" with a single tension tie passing through. Lastly, when calculating the concrete compression strut elements, the current design codes STM does not specifically account for the impact of diagonal cracks on the struts. Instead, the design employs a simplified strut coefficient related to the strut efficiency.

6.3 BS EN 1992-1-1:2004 Strut-and-Tie Model

Eurocode 2 specifically permits the application of the strut-and-tie method for the analysis and design of corbels. According to clause 6.5.1 of EN 1992-1-1:2004 [134], this method is suitable for conditions involving nonlinear stress distribution, such as near supports, under concentrated loads, or in regions subjected to combined stress. RCCs can be designed employing the STM to address the nonlinear stress distributions near supports or concentrated loads. In this model, the corbel is conceptualized with an inclined concrete strut and a horizontal steel tie. The geometry and forces are depicted in Fig. 16.

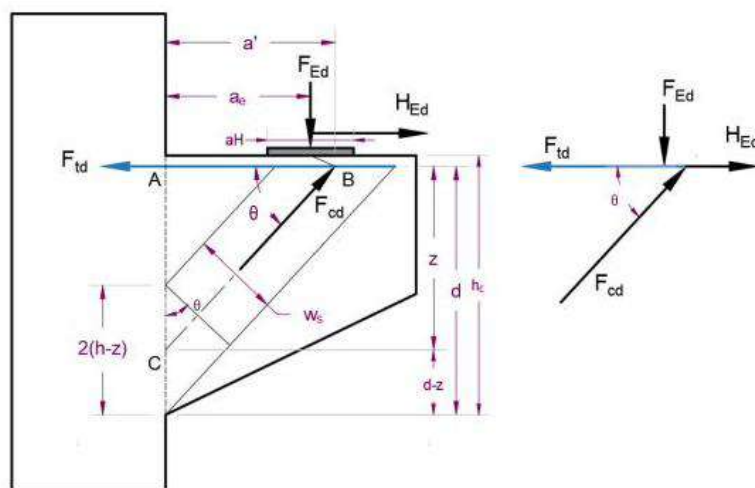


Fig. 16. Strut and tie analysis of a corbel as per Eurocode 2 [134]

The lever arm (z) is determined by the equation:

$$z = a' \tan \theta \quad (19)$$

$$a' = a_e + 0.2a_H \quad (20)$$

Where a_e is the distance from the column face to the load application point, a_H is the width of the bearing plate, and θ is the inclination angle of the strut.

The force in the concrete strut, F_{cd} , is calculated as:

$$F_{cd} = f_{cd} \times W_{strut} \times b_w \quad (21)$$

In which, $f_{cd} = 0.34f_{ck} \times \left(1 - \frac{f_{ck}}{250}\right)$ is the design compressive stress of the concrete strut; $W_{strut} = 2(d - z) \cos \theta$ is the width of the strut perpendicular to its axis, and b_w is the width of the corbel.

The inclination angle θ ($45^\circ \leq \theta \leq 68^\circ$) is determined by the equation:

$$\frac{F_{Ed}}{f_{cd}db_w} = \left(1 - \frac{a'}{d} \tan \theta\right) \sin 2\theta \quad (22)$$

This equation may require iterative methods or design charts to solve for θ .

Resolving horizontally at B, the force F_{td} in the steel tie is:

$$F_{td} = F_{Ed} \cot \theta \quad (23)$$

Therefore, the total force F'_{td} in the steel tie, accounting for the horizontal force of $0.2F_{Ed}$, is:

$$F'_{td} = F_{Ed} \cot \theta + 0.2F_{Ed} \quad (24)$$

Thus, the area of main tension steel, $A_{s,main}$, is calculated as:

$$A_{s,main} = \frac{F'_{td}}{0.87f_{yk}} \quad (25)$$

Moreover, closed horizontal links are provided only if $a_e/h_c \leq 0.5$, and the total area of both legs of a link, $A_{s,link}$, is given by:

$$A_{s,link} \geq 0.5A_{s,main} \quad (26)$$

6.4 GB 50010-2010 Triangular Truss Model

In accordance with the Chinese standard GB 50010-2010 [11], short corbels with an a/d ratio of less than 1 are analysed using a triangular truss model [142]. This model assumes that the horizontal longitudinal reinforcement at the top of the corbel acts as a tension tie, while the concrete in the diagonal region functions as a compression strut. Reference is made to Fig. 17, and evaluating the moment equilibrium at point A, the following equation is derived:

$$f_{yf}A_s h_1 = F_v a_v + F_h (h_1 + a_s) \quad (27)$$

Where F_v and F_h represent vertical and horizontal design loads, respectively; f_y and A_s denote the yield strength and area of the longitudinal reinforcement; h_1 is the internal lever arm, approximately $0.85h_0$ with h_0 being the effective depth; $(h_1 + a_s)$ adjusts the moment arm, in which

a_s is the lever arm of the shear span; and a_v is the shear span length, set to $0.3h_0$ if it is less than this value. To ensure adequate reinforcement, the total cross-sectional area must satisfy:

$$A_s \geq \frac{F_v a_v}{0.85 f_{yf} h_0} + \frac{1.2 F_h}{f_{yf}} \quad (28)$$

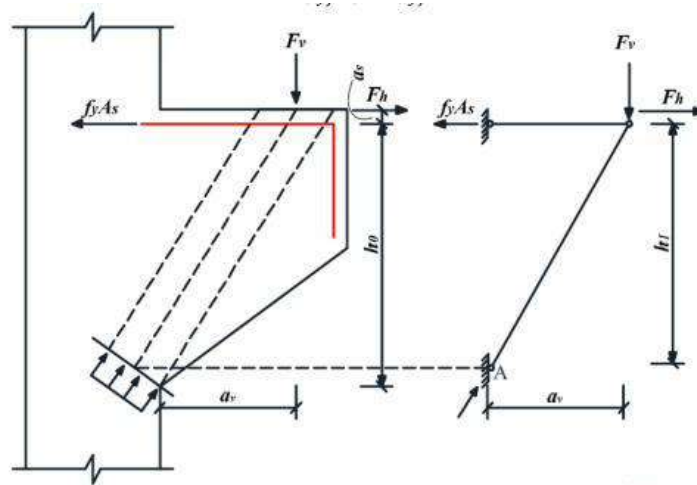


Fig. 17. Triangular truss model as per GB 50010-2010 code [67]

Table 10. Summary of reviewed analytical and code-based models for RCCs

Model / code provision	Main mechanical assumption	Applicability	Main advantages	Main limitations
ACI 318 shear-friction method [9], [27]	Shear transfer occurs along a potential shear plane at the corbel-column interface through reinforcement crossing the plane and concrete friction.	Mainly short corbels with $a_v/d \leq 1.0$, subjected to vertical load with or without horizontal force.	Simple design format, direct calculation of required reinforcement, and practical use in routine design.	May not fully represent diagonal strut action, nodal crushing, bearing effects, or complex reinforcement interaction.
ACI 318 strut-and-tie method [9], [27]	The corbel is idealized as a truss system consisting of concrete compression struts, steel tension ties, and nodal zones.	D-regions and corbels where nonlinear strain distribution controls behavior.	Mechanically meaningful, suitable for disturbed regions, and consistent with internal force equilibrium.	Requires assumed strut-and-tie geometry; prediction depends on strut efficiency factors, nodal strength, and detailing assumptions.
BS EN 1992-1-1:2004 strut-and-tie model [10]	Load is transferred through an inclined concrete strut and a horizontal steel tie.	Corbels and regions near supports or concentrated loads where stress distribution is nonlinear.	Provides a rational design framework for concentrated-load regions and includes limits on strut inclination.	May require iterative solution for strut angle and depends on selected stress limits and geometric idealization.
GB 50010-2010 triangular truss model [11]	The top longitudinal reinforcement acts as a tension tie, while the diagonal concrete zone acts as a compression strut.	Short corbels, generally with $a/d < 1.0$.	Simple equilibrium-based method and easy to apply in design calculations.	Does not explicitly capture all possible failure modes, such as concrete crushing due to bending, local bearing distress, or detailed crack development.

When only a vertical load is applied, the bearing capacity is calculated using:

$$F_v \leq 0.85 \frac{f_{yf} A_s h_0}{a_v} \quad (29)$$

Additionally, to control cracking, the corbel dimensions must meet:

$$F_v \leq \beta \left(1 - \frac{0.5 F_{hk}}{F_{vk}} \right) \frac{f_{tk} b h_0}{0.5 + \frac{a_v}{h_0}} \quad (30)$$

Where F_{vk} and F_{hk} are the standard vertical and horizontal loads, respectively, taken as $0.67 F_v$ and $0.67 F_h$; f_{tk} is the standard tensile strength of the concrete; b is the corbel width; and β is the crack control coefficient, set to 0.65 for corbels supporting crane beams and 0.80 for other corbels. The bearing capacity is assessed using a simplified triangular truss model, which does not account for concrete crushing due to bending.

As a summary, the reviewed design approaches show that current code-based models provide practical procedures for RCCs design, but each method is based on different mechanical assumptions and applicability limits. The shear-friction method is simple and suitable for short corbels, while the strut-and-tie method provides a more rational representation of the internal load path in D-regions. The triangular-truss approach also gives a simplified equilibrium-based representation of the corbel mechanism. However, the accuracy of these methods depends on the assumed failure mechanism, reinforcement detailing, concrete strength, bearing condition, and the treatment of horizontal force. Table 10 summarizes the main characteristics of the reviewed analytical and code-based models.

The comparison indicates that no single code-based approach can fully represent all aspects of RCCs behavior. Therefore, code predictions should be interpreted together with experimental evidence, parameter studies, and mechanics-based understanding. This also supports the need for explainable data-driven interpretation, where ML tools are used to identify database trends without replacing established design provisions.

6.5 Limitations of Existing Design and Prediction Approaches

Bringing together the evidence of Sections 4 to 6, the limitations of the current design and prediction approaches can be grouped into five categories, each of which suggests a specific research response. The first limitation is incomplete representation of the interacting variables. The shear-friction formulation of Eqs (1) to (8) resolves the problem into reinforcement crossing an assumed shear plane and therefore does not carry the strut inclination, the nodal geometry, or the bearing dimensions explicitly, while the triangular-truss formulation of Eqs (27) to (30) is an equilibrium statement that does not address concrete crushing in bending or local bearing distress. The consequence is visible in the experimental comparisons: Abdul-Razzaq and Dawood [43] reported that the deviation of the shear-friction and strut-and-tie predictions from the measured failure load grew from 15% and 22% at $a/d = 0.5$ to 32% and 37% at $a/d = 1.5$, so the error is not a fixed safety margin but a function of the geometry. The corresponding research need is a systematic reassessment of the strut efficiency and nodal strength factors against a modern database stratified by a/d , rather than a single calibration applied across the whole range.

The second limitation concerns the assumed failure mechanism. Each method is anchored to one mechanism, whereas the reviewed tests show that the governing mode migrates with a/d , reinforcement detailing, concrete strength, and loading history, as discussed in Section 4. A method calibrated on sliding-type behavior is therefore applied outside its mechanical basis when the specimen fails by nodal crushing or by flexural yielding of the tie. Future work should aim at mechanism-aware provisions, in which the applicable expression is selected by an explicit criterion rather than by a single geometric threshold.

The third limitation is the narrowness of the calibration base relative to current practice. The provisions were established largely on monotonic tests of conventional normal-strength corbels, yet a substantial part of the recent literature concerns high-strength, fibre-reinforced, recycled-

aggregate, UHPC, SCC, and FRP-reinforced members, and Section 3.1 shows that this material is where the growth of the field is concentrated. Extending and harmonizing the experimental database, with consistent reporting of geometry, reinforcement, bearing arrangement, and failure mode, is a prerequisite for any recalibration.

The fourth limitation applies to the data-driven models that have been proposed as alternatives. As Section 7.2 discusses, high statistical accuracy inside the training range does not establish safe extrapolation, and models such as CatBoost or a conventional ANN provide no mechanical account of their own predictions. The productive direction here is the one represented by the SR-STM approach of Megahed [144] and by the SHAP interpretation adopted in the present study, namely models that either retain the mechanical form or expose the contribution of each variable, together with an explicit statement of the parameter ranges within which they may be used.

The fifth limitation is that both code and data-driven approaches are almost entirely deterministic and address ultimate capacity alone. Neither the scatter of the predictions nor the serviceability quantities that often govern in practice, such as crack width at the corbel-column junction under sustained or repeated service load, are treated formally. Probabilistic formulations of the kind proposed by Nguyen et al. [143], combined with reliability-based calibration and with explicit serviceability criteria, would allow the observed scatter to be carried into design rather than absorbed silently by a global safety factor. Taken together, these five directions define a coherent programme: a broader and better-reported database, mechanism-aware and geometry-sensitive strength provisions, interpretable data-driven models with stated applicability limits, and a probabilistic treatment covering both ultimate and serviceability limit states.

7. ML-Based Prediction of RCCs Shear Capacity

The reviewed analytical and code-based models provide practical procedures for estimating the shear capacity of RCCs. However, these models are generally based on simplified assumptions regarding the internal load-transfer mechanism, reinforcement contribution, strut efficiency, and nodal strength. As discussed in Section 6, the code-adopted approaches are useful for design, but they do not fully capture the nonlinear interaction among geometry, concrete strength, reinforcement ratios, reinforcement yield strength, and loading configuration. This limitation becomes more evident when experimental databases include wide variations in specimen dimensions, material properties, and reinforcement details.

ML models have recently become an important direction in predicting the shear capacity of RCCs [143]. This interest is mainly due to the nonlinear nature of corbel behavior. Traditional design models remain useful because they are simple and mechanically meaningful; however, their prediction accuracy may be affected by the limited number of variables included in their formulations. In contrast, ML models can learn complex relationships from experimental databases, provided that the database is carefully compiled, consistently defined, and limited to compatible test conditions.

In this study, the ML component is used as an interpretation tool rather than as a replacement for design-code provisions. To achieve this, the ML dataset was restricted to compatible vertical-load specimens and excluded direct horizontal-force cases, strengthened or repaired specimens, and specimens made with special materials. This boundary was adopted to ensure that the model interprets the shear behavior of original RCCs governed mainly by vertical-load transfer. The following subsections first summarize the available ML studies on RCC shear prediction, then present the XGBoost model used in this study, and finally discuss the SHAP-based interpretation of the governing input variables.

7.1 Summary of Published ML Studies

The available ML-based studies on RCCs shear capacity remain limited compared with the wider literature on RC beams and deep beams. Most published work relies on artificial neural networks or hybrid ML models. Table 11 summarizes the main ML studies directly relevant to RCCs shear-capacity prediction. Abbas and Ali (2025) presented one of the most direct ANN applications for RCCs. Their model used 330 experimentally tested specimens and 13 input variables, and the best

BPNN architecture was 13-10-9-1 with a Pearson correlation coefficient of 0.990. The same study compared ANN predictions with ACI 318-19, EN 1992-1-1, and GB 50010-2010 methods, showing that ANN predictions were more consistent than the examined code-based equations [13]. The same data-driven treatment was subsequently extended to corbels in which a horizontal tension force acts together with the vertical load [145]. That loading condition is outside the scope of the database compiled in the present study.

Table 11. Summary of ML studies on RCCs shear capacity

Study	Dataset size	Model used	Input variables	Output	Main performance	Main limitation
Kumar and Barai (2010) [91]	Experimental data for steel-fibre RCCs without shear reinforcement	Artificial neural network	Concrete strength, fibre content, flexural reinforcement, shear span/depth-related parameters	Ultimate shear capacity	Strong agreement with experimental results, with MAE = 0.070, RMSE = 0.098, and R = 0.97	Focused on steel-fibre RCCs without shear reinforcement; limited generality for conventional RCCs
Megahed (2024) [144]	371 RCCs tests, with 810 RC deep-beam tests also used in the broader study	STM-based symbolic regression, SR-STM, and CatBoost benchmark	Geometry, concrete strength, longitudinal reinforcement, vertical and horizontal web reinforcement, loading plate/column dimensions, and STM-related variables	Shear strength index / shear strength	For RCCs, SR-STM achieved mean prediction-to-actual ratio = 1.004, R ² = 0.862, CoV = 14.95%; CatBoost gave higher accuracy with CoV = 7.19%	SR-STM is more interpretable than CatBoost, but its validity is limited to the database range; CatBoost remains black-box in nature
Abbas and Ali (2025) [13]	330 RCCs specimens	Backpropagation neural network, BPNN	13 input variables: square root of f_c' , shear span, width, column width, effective depth, height, a/d , longitudinal reinforcement ratio, horizontal and vertical reinforcement ratios, and steel yield strengths	Experimental shear strength	Best architecture 13-10-9-1; Pearson R = 0.990; code methods had lower R values of 0.901-0.927	Data-driven model; applicability limited to the parameter ranges of the collected database

Megahed (2024) introduced a different and more interpretable ML direction by combining symbolic regression with the strut-and-tie model. The study used a large database of 810 RC deep beams and 371 RCCs and developed an STM-based symbolic regression model. Unlike purely black-box ML models, the SR-STM approach produced explicit mathematical expressions linked to the diagonal strut and truss mechanisms. For RCCs, the SR-STM model achieved a mean prediction-to-actual ratio of 1.004, R^2 of 0.862, and CoV of 14.95%, while CatBoost showed stronger accuracy but less interpretability [144].

7.2 Critical Assessment of Existing ML Models

The reviewed ML studies show that data-driven models can improve prediction accuracy compared with many traditional code equations. However, the available ML literature on RCCs is still relatively narrow. The most common approach is ANN-based prediction, while other algorithms

such as Random Forest, XGBoost, Gaussian Process Regression, Support Vector Regression, and explainable ensemble models have not yet been explored sufficiently for RCCs.

A main limitation is that high statistical accuracy does not always mean safe generalization. ANN and CatBoost models can perform very well within the range of the training database, but their predictions may become unreliable when applied to corbels with unusual geometry, reinforcement details, concrete strength, or loading conditions. This issue was clearly noted by Abbas and Ali (2025), who limited the use of their ANN model to the parameter ranges covered by the experimental database [13].

Another limitation is interpretability. Black-box models can give accurate predictions, but they do not always explain why a certain capacity is predicted. This is important in structural engineering because designers need both accuracy and mechanical meaning. Megahed (2024) addressed this issue by developing an SR-STM model that keeps the mechanical logic of the strut-and-tie method while using symbolic regression to improve prediction accuracy. This makes the model easier to interpret than CatBoost or a conventional ANN [144].

The work of Megahed [144] also shows the value of explainability tools. Its SHAP analysis indicated that the shear span-to-depth ratio and effective longitudinal reinforcement ratio were among the most influential variables for RCCs shear-strength prediction. This agrees with the physical understanding of corbel behavior, where the compression strut angle, tie reinforcement, and reinforcement effectiveness strongly control the load-transfer mechanism.

7.3 ML Model Used for SHAP Interpretation

In the present study, the ML component was not introduced to propose a new benchmark design equation for RCCs shear-capacity prediction. Several previous studies have already reported accurate data-driven models for RCCs, including ANN-based and symbolic-regression-based approaches. Therefore, the purpose of the present modelling step was more focused: to provide an interpretable data-driven framework that can explain how the main geometric, material, and reinforcement-related variables influence the predicted shear capacity of RCCs.

To achieve this, an Extreme Gradient Boosting (XGBoost) ensemble regressor was trained using a compiled database of 330 experimentally tested RCCs. The model included 13 input variables: square root of concrete compressive strength, shear span, corbel width, column width, effective depth, corbel height, shear span-to-effective depth ratio, flexural reinforcement ratio, horizontal reinforcement ratio, vertical reinforcement ratio, transverse steel yield strength, horizontal steel yield strength, and vertical steel yield strength. The target variable was the experimental shear capacity, V_{exp} . All input variables and the target variable were scaled to the range [0,1] using min-max normalization before model training. The normalized V_{exp} values had a mean of 0.1608 and a standard deviation of 0.1271.

The database was randomly divided into training and testing subsets using an 80:20 split ratio. Accordingly, 264 specimens were used for training, while 66 specimens were used for testing. A fixed random seed of 42 was adopted to ensure reproducibility. The XGBoost model was configured with 300 estimators, a maximum tree depth of 6, a learning rate of 0.05, a subsample ratio of 0.8, and a column-sampling ratio of 0.8. The model was trained using all available CPU cores. These parameters were selected to provide a balance between predictive accuracy and model stability.

As shown in Fig. 18, the model achieved a coefficient of determination, R^2 , of 0.9973 for the training set and 0.9090 for the testing set. The corresponding RMSE values were 0.0066 and 0.0388, respectively. These results indicate very close fitting for the training data and good predictive performance for the testing data. However, the difference between training and testing performance also indicates that the model should be interpreted with care and should not be presented as a new design equation. In this study, the XGBoost model was considered sufficient for SHAP-based interpretation, rather than for claiming superiority over existing ANN or code-based models.

This gap between the training and testing coefficients of determination, 0.9973 against 0.9090, should be stated explicitly as a mild overfitting of the model and is acknowledged here as a

limitation of the present modelling step rather than as an incidental observation. Three regularization measures were in fact embedded in the configuration described above. The learning rate was held at 0.05, which is deliberately low and requires the ensemble to accumulate the fit gradually over 300 trees instead of committing to large corrections early. The maximum tree depth was capped at 6, which limits the order of interaction any single tree can represent and prevents individual branches from isolating small groups of specimens. Stochastic sampling was applied on both axes, with a subsample ratio of 0.8 for the training rows and a column-sampling ratio of 0.8 for the features, so that each tree is fitted to a different perturbation of the data and the ensemble is decorrelated. These three measures together constrain the model, but they do not remove the residual gap, and the reason is a property of the dataset rather than of the algorithm: 264 training specimens are a small sample for a 13-dimensional input space, and gradient-boosted trees will continue to fit residual structure in such a sample even under moderate constraint. Additional shrinkage through the L1 and L2 penalty terms, early stopping monitored on a validation fold, and k -fold cross-validation in place of a single fixed 80:20 split with seed 42 would narrow the gap further and would also provide a variance estimate for the reported metrics; these steps were not applied here because the objective was interpretation rather than benchmarking, and because the SHAP attributions are computed on the fitted model and are therefore conditional on it. The practical consequence must be stated plainly. The testing R^2 of 0.9090 is the honest indicator of predictive performance, the training value of 0.9973 is not, and the model is offered strictly as an explanatory instrument for the trends within the compiled database. It is not proposed as a design equation, and its output should not be extrapolated beyond the parameter ranges of the 330 specimens on which it was trained. This limitation is restated in Section 9. A k -fold cross-validated procedure of this type has been applied to shear-strength prediction in RC deep beams [146], which are discontinuity regions like corbels, and it is the appropriate next step for the present model should it be developed beyond an interpretive role.

SHAP analysis was then conducted using the TreeExplainer algorithm. SHAP values were computed for the full dataset of 330 specimens, producing a SHAP matrix with a size of 330×13 . The model base value, $E[f(X)]$, was 0.1580. Accordingly, the XGBoost model was adopted as an explanatory tool to support the interpretation of database trends. This approach helps reduce the black-box nature of ML methods and allows the model output to be discussed in relation to the known mechanical behavior of RCCs.

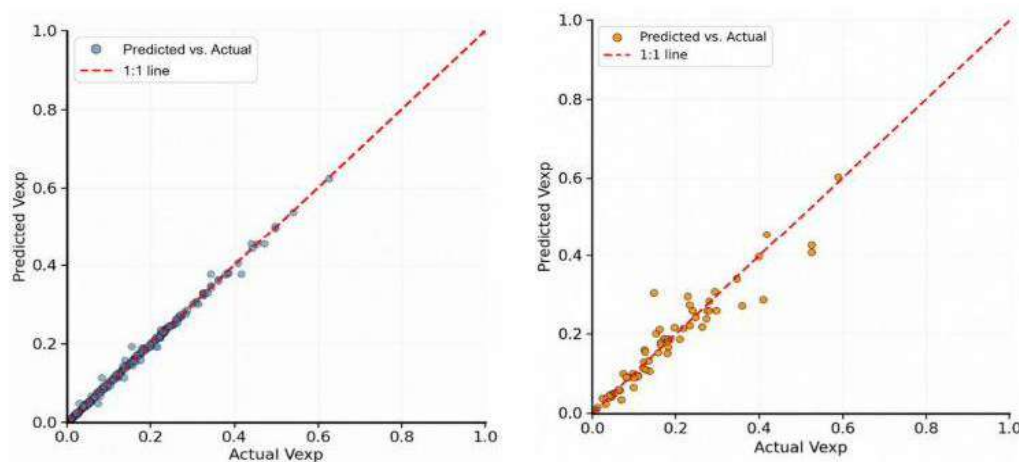


Fig. 18. Actual vs. predicted normalized experimental shear capacity for the XGBoost model: (a) training set and (b) testing set

7.4 SHAP-Based Interpretation of Input Variables

To interpret the influence of the input parameters on the predicted shear capacity, SHAP (SHapley Additive exPlanations) analysis was carried out using the trained XGBoost model. SHAP values quantify the contribution of each input variable to the predicted output. Positive SHAP values push the prediction above the model base value, while negative values reduce the prediction below that

value. In this way, SHAP analysis provides both global and local interpretation of the model behavior.

In the present work, SHAP analysis was not used as a replacement for design-code assessment. Rather, it was used to explain the relative importance and direction of the main variables that control the predicted shear capacity. This is especially useful because the behavior of RCCs is governed by interaction between geometry, concrete strength, reinforcement, and the load-transfer mechanism.

7.4.1 Global Importance of Variables

The global SHAP feature-importance plot is presented in Fig. 19, while the numerical ranking is summarized in Table 12. The results show that the most influential variables were corbel height, h , corbel width, b , and column width, w_c . These were followed by the shear span-to-effective depth ratio, a/d , and the square root of concrete compressive strength, $(f_{ck})^{0.5}$. Other variables with moderate influence included the effective depth, d , and the flexural reinforcement ratio, ρ_f . The horizontal reinforcement ratio, ρ_h , and the yield strengths of reinforcement had smaller effects, while the vertical reinforcement ratio, ρ_v , and vertical steel yield strength, f_{yv} , had the lowest SHAP importance within the present database.

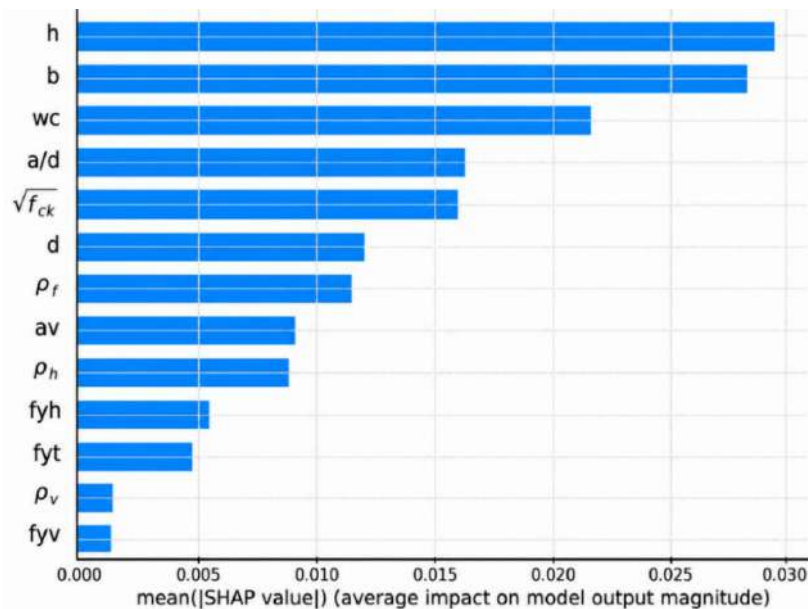


Fig. 19. SHAP feature importance based on mean absolute SHAP value

Table 12. SHAP ranking and physical interpretation of input variables

Rank	Feature	Mean SHAP	Effect direction	Interpretation
1	h	0.0296	Positive	Larger height increases the predicted shear capacity.
2	b	0.0284	Positive	Wider corbels provide larger bearing area and higher predicted capacity.
3	w_c	0.0217	Positive	Larger column/support width improves the load-transfer zone.
4	a/d	0.0163	Negative	Increasing a/d reduces the predicted shear capacity.
5	$(f_{ck})^{0.5}$	0.0161	Positive	Higher concrete strength increases the capacity of the compression strut.
6	d	0.0121	Positive	Greater effective depth has a moderate positive effect.
7	ρ_f	0.0116	Positive	Flexural steel supports the tension-tie mechanism.
8	av	0.0092	Negative	Larger shear span tends to reduce capacity.

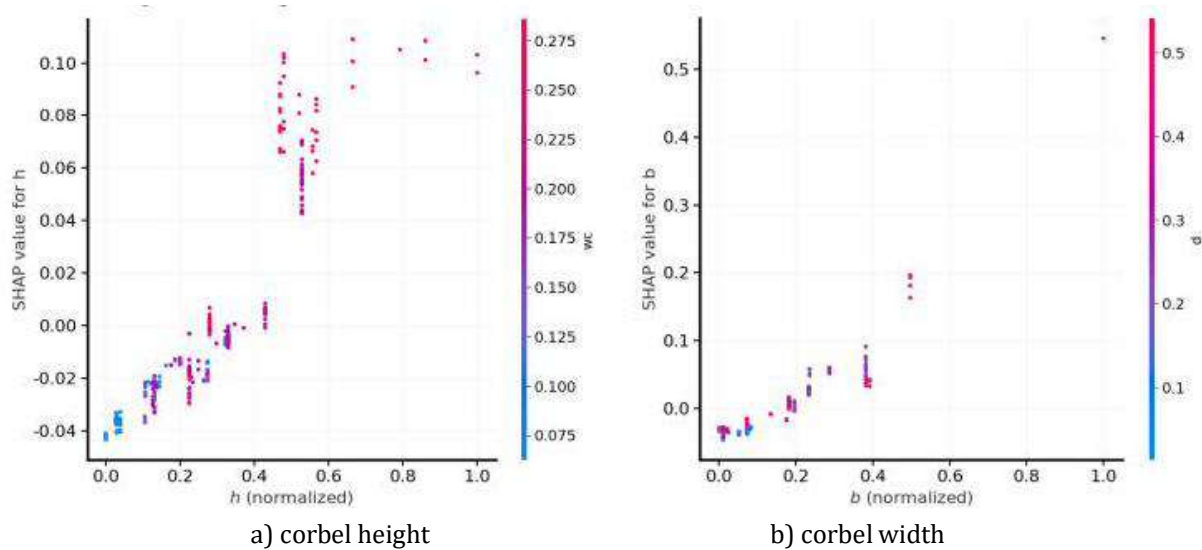
Rank	Feature	Mean SHAP	Effect direction	Interpretation
9	rho_h	0.0089	Positive	Horizontal reinforcement gives a smaller positive contribution.
10	fyh	0.0055	Positive	Horizontal steel yield strength has low importance in this data.
11	fyt	0.0048	Positive	Transverse steel yield strength has low importance.
12	rho_v	0.0015	Positive	Vertical rebar ratio shows a very small global contribution.
13	fyv	0.0014	Positive	Vertical steel yield strength is the least influential variable in the model.

The high ranking of dimensional variables is structurally reasonable because the model output is shearing capacity rather than a fully normalized shear stress. Larger corbel dimensions increase the area available for stress transfer, enlarge the compression field, and improve the load-carrying path. At the same time, the importance of a/d confirms that the model captured the central role of strut geometry in RCCs behavior.

7.4.2 Effect of Geometric Variables

The SHAP dependence plots for the main geometric variables, including corbel height, h , corbel width, b , and column width, w_c , are presented in Fig. 20. The dependence plots show that these variables generally had positive SHAP contributions. In practical terms, this means that increasing these dimensions tends to push the predicted shear capacity upward. This agrees with the mechanics of corbel action because larger dimensions increase the size of the compression strut, improve the bearing region, and provide a larger area for stress redistribution.

The strong influence of h and b also indicates that the model learned the importance of member size. This is expected because the experimental output is expressed as normalized capacity, and not only as a dimensionless shear-strength index. Therefore, the high SHAP ranking of h and b should be interpreted as a combined size effect and mechanical effect. It confirms that the available load-transfer area is one of the dominant contributors to predicted corbel capacity within the range of the compiled database.



7.4.3 Effect of Shear Span-To-Effective Depth Ratio

The shear span-to-effective depth ratio, a/d , showed a clear negative effect on the predicted shear capacity. As shown in Fig. 21, small values of a/d were associated with positive SHAP values,

meaning that short shear spans increased the predicted capacity above the model base value. As a/d increased, the SHAP values decreased and became negative. This indicates that larger shear span ratios reduce the predicted capacity.

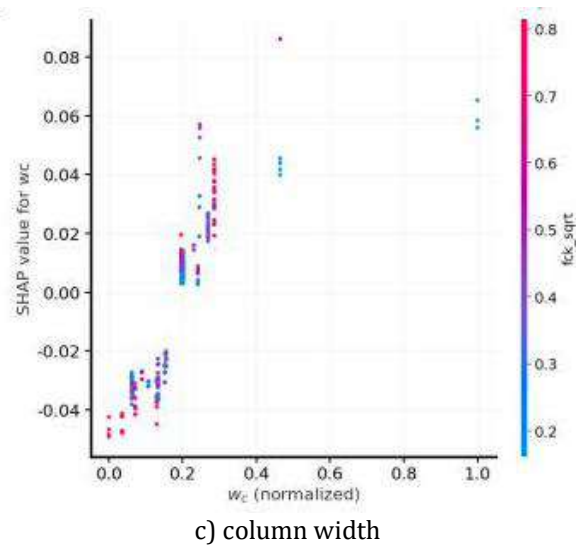


Fig. 20. SHAP dependence plots for the main dimensional geometric variables

This trend is one of the most important findings of the SHAP analysis because it agrees directly with the physical behavior of RCCs. When a/d is small, the applied load is transferred more directly to the support through a steep diagonal compression strut. This direct strut action increases the shear capacity. When a/d increases, the strut becomes less direct, flexural effects become more significant, and the predicted shear resistance decreases. The SHAP interaction coloring also indicates that the influence of a/d is linked with the shear span, a_v , and the effective depth, d , which is consistent with the geometric definition of the load path.

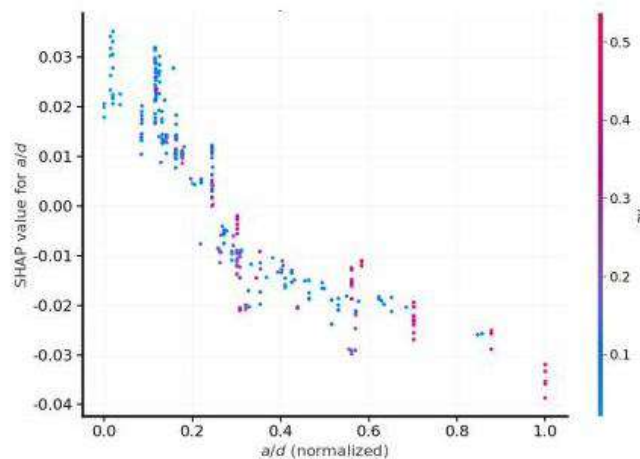


Fig. 21. Detailed SHAP plot for the shear span-to-effective depth ratio

7.4.4 Effect of Concrete Compressive Strength

The SHAP dependence plot for the square root of concrete compressive strength, $(f_{ck})^{0.5}$, is presented in Fig. 22. The plot shows a generally positive monotonic trend. At lower concrete-strength values, the SHAP contribution was mostly negative, indicating a reduction in the predicted capacity relative to the model base value. At higher values, the SHAP contribution became positive and increased noticeably.

This finding is mechanically meaningful because the shear capacity of RCCs depends strongly on the strength and integrity of the concrete compression strut. Higher concrete strength improves resistance to local crushing, diagonal compression, and bearing stresses. The use of $(f_{ck})^{0.5}$ is also

consistent with many shear-resistance formulations, in which the concrete contribution is related to the square root of compressive strength rather than directly to f_{ck} .

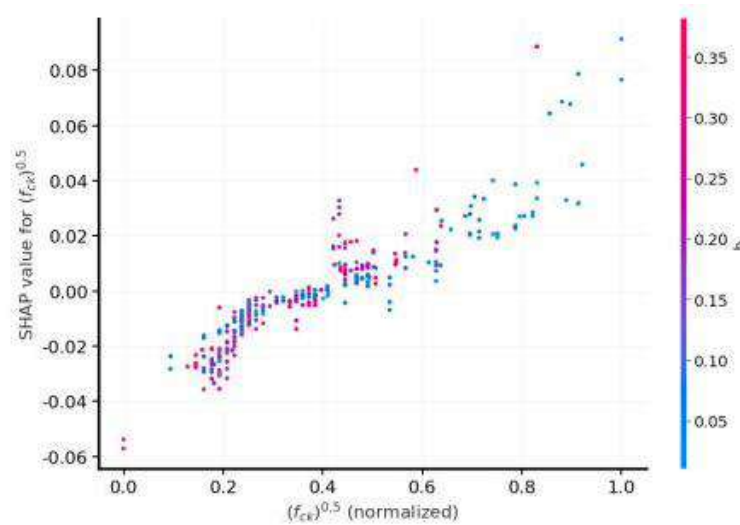


Fig. 22. SHAP plot for the influence of concrete strength on predicted shear capacity

7.4.5 Effect of Reinforcement-Related Variables

The reinforcement-related variables showed different levels of influence. The flexural reinforcement ratio, ρ_f , had a moderate positive effect on the predicted shear capacity. This is reasonable because the main reinforcement forms the tensile tie in the STM mechanism [8] and helps maintain equilibrium after cracking. The horizontal reinforcement ratio, ρ_h , also had a positive effect, but its global importance was smaller than that of the main geometric variables, a/d , and concrete strength.

The vertical reinforcement ratio, ρ_v , and the yield strength variables related to vertical or transverse reinforcement had relatively low SHAP importance. This result should not be interpreted to mean that these variables are unimportant in design. Rather, it indicates that within the present database their marginal contribution was smaller than that of geometry, concrete strength, and the shear span ratio. This may be due to the limited range of vertical reinforcement values, the distribution of test specimens in the database, or the fact that the primary resistance mechanism is governed by the diagonal compression strut and the main tension tie.

7.4.6 General Interpretation of The SHAP Findings

The SHAP analysis, in general, suggests that the XGBoost model learned trends that are broadly consistent with established corbel mechanics. The positive effects of h , b , w_c , and $(f_{ck})^{0.5}$ indicate the importance of member size, compression-zone development, and concrete strength. The negative effect of a/d confirms the dominant role of load-path geometry and the reduction in shear capacity as the corbel becomes less deep-beam-like. Reinforcement variables contributed positively but generally played a secondary role within the range of the compiled data.

Therefore, the value of the present ML component lies mainly in interpretation rather than prediction. The SHAP results provide an additional layer of explanation that connects data-driven modelling with the known mechanical behavior of RCCs. This supports the systematic review findings and helps translate the ML output into structural engineering language.

8. Discussion of ML Results

The SHAP analysis supports the main behavioral interpretation obtained from the systematic review. The results demonstrate that the XGBoost model did not only provide accurate predictions, but also captured trends that are consistent with the known mechanics of RCCs. The dominant influence of the geometric variables and the a/d ratio confirms that RCCs behavior is strongly

governed by the geometry of the compression strut, the size of the load-transfer region, and the directness of the load path between the bearing point and the column face. This finding agrees with the experimental evidence reviewed in Section 4, where smaller a/d ratios generally resulted in higher shear strength due to stronger direct strut action.

Before these rankings are read as statements of relative physical importance, the correlation structure among the dimensional inputs must be taken into account. The variables h , b , w_c , and d are not independent quantities in a corbel database. The effective depth d is by definition the total height h less the cover and the position of the main tension reinforcement, so the two are almost collinear by construction; the corbel width b and the column width w_c are tied to each other because the corbel is cast against the column and is normally proportioned to match or approach its width; and all four grow together because test specimens are usually scaled versions of a common geometry rather than independently varied dimensions. The shear span a_v and the ratio a/d are correlated with d for the same reason. This has a direct bearing on how Fig. 19 and Table 12 should be read. SHAP allocates the contribution of a prediction among the inputs actually used by the fitted trees, and when two inputs carry nearly the same information the ensemble will split on whichever one happens to reduce the loss first, so the attribution is divided between them in a manner that depends on the fitting sequence rather than on mechanics. The individual ranks of h , b , w_c , and d are therefore not stable quantities, and the difference between the mean absolute SHAP values of h (0.0296) and b (0.0284) is far too small to support a claim that height matters more than width. What the analysis does support is a statement at the level of the group: the four-dimensional variables together account for the largest share of the model output, which is the expected consequence of predicting a capacity, in units of force, rather than a normalized shear stress. Read this way, the correct interpretation of the SHAP ranking is that member size as a whole dominates, that a/d and $(f_{ck})^{0.5}$ contribute separately and with mechanically consistent signs, and that reinforcement variables act as secondary contributors. Two consequences follow for future work. Repeating the interpretation with $V_{exp}/(bd)$ as the target, as recommended in Section 9, removes much of the shared size information and would show whether the ranking of the geometric variables survives once the direct size effect is suppressed. In parallel, reporting variance inflation factors or a correlation matrix for the input set, as reported for the deep-beam input set in [146], and testing grouped or clustered SHAP attributions in which collinear variables are evaluated jointly, would make the division of importance among h , b , w_c , and d reproducible rather than fitting-dependent.

The positive contribution of concrete compressive strength is also consistent with the physical behavior of RCCs. Since the main load-transfer mechanism depends on the diagonal compression strut, an increase in concrete strength improves the resistance to diagonal compression, local crushing, and bearing-related distress. This explains why $(f_{ck})^{0.5}$ appeared among the most influential variables in the SHAP ranking. The use of this parameter is also reasonable from a mechanics-based perspective, since many shear-strength formulations relate the concrete contribution to square root of concrete compressive strength rather than to the direct value of f_{ck} .

The reinforcement-related variables showed positive but lower global SHAP importance compared with geometry, a/d , and concrete strength. This result should not be interpreted as evidence that reinforcement is unimportant in design. Rather, it indicates that within the range of the compiled database, the marginal contribution of reinforcement variables was smaller than the contribution of the main geometric and concrete-strength parameters. The flexural reinforcement ratio, ρ_f , had a moderate positive influence because the main reinforcement forms the tension tie in the strut-and-tie mechanism. The horizontal reinforcement ratio ρ_h , also contributed positively by improving crack control and confinement. In contrast, the vertical reinforcement ratio, ρ_v , and the yield-strength variables showed lower importance, which may be related to their narrower range in the database, their interaction with a/d , and the dominant role of the concrete strut in short corbels.

The SHAP findings also help explain why existing code-based models may show conservative or scattered predictions. Code equations are usually based on simplified assumptions, such as shear-friction action, an idealized strut-and-tie mechanism, or a triangular truss analogy. These models are useful for design because they are simple and mechanically meaningful, but they may not fully

represent the interaction among member size, a/d , concrete strength, reinforcement ratios, support width, and bearing condition. The SHAP results indicate that these variables do not act independently. Instead, the predicted capacity is controlled by a combined effect of geometry, material strength, and reinforcement arrangement. This supports the need to interpret code predictions together with experimental evidence and data-driven trends.

It is therefore worth stating precisely what the integration of machine-learning methods adds to the understanding of parameter influence, and where it does not replace mechanics-based reasoning. A mechanics-based model expresses a parameter's influence through the role assigned to it in an assumed load path: in the strut-and-tie idealization of Section 6.2, for instance, a/d enters through the strut inclination θ of Equation (9), and the concrete strength enters through the effective stresses $f_{ce,s}$ and $f_{ce,n}$ of Equations (15) and (16). The influence of a parameter is thus known in advance, by construction, and any parameter omitted from the idealization has no influence at all. The empirical alternative, a parametric test series, isolates one variable while holding the others fixed, which is rigorous but expensive and restricted to the few combinations that can be built and tested. Explainable ML occupies the space between these two. It infers the influence of every recorded variable simultaneously, across the full set of assembled specimens, without prescribing a load path in advance, and SHAP then decomposes each individual prediction into additive contributions so that both the magnitude and the sign of each variable's effect become visible. Four specific gains follow in the present case. First, the direction of the effects was recovered rather than imposed: the negative sign for a/d and the positive signs for the dimensional variables and for $(f_{ck})^{0.5}$ emerged from the data and coincide with the mechanics summarized in Table 7, which supplies independent corroboration of the review findings. Second, the method ranks parameters on a common quantitative scale, allowing the observation that reinforcement-related variables act as secondary contributors within this database, a comparison that the code equations cannot make because each of them carries a different subset of variables. Third, the dependence plots of Figs 20 to 22 expose the shape of each relationship and its interaction coloring, so that the coupling of a/d with a_v and d is visible directly rather than being assumed. Fourth, the analysis identifies where the evidence is thin: the low importance of ρ_v and f_{yv} is as much a statement about the coverage of the database as about the mechanics, which is diagnostic information that a closed-form equation cannot provide. The limits are equally definite. The model has no equilibrium condition and no failure criterion; its attributions are conditional on the fitted model and on the correlation, structure discussed above; and it cannot extrapolate beyond the parameter ranges of its 330 specimens. Explainable ML should therefore be understood as an instrument for interrogating the accumulated experimental evidence and for indicating which parameters a mechanics-based model must represent, not as a substitute for the mechanical reasoning that design provisions require.

The value of SHAP analysis in this study lies mainly in improving the interpretability of the ML model. Unlike accuracy indices such as R^2 and RMSE, SHAP values explain how each input variable contributes to the model prediction. This allows the ML output to be discussed in structural engineering terms rather than being treated as a black-box result. In the present study, the agreement between SHAP trends and established corbel mechanics gives confidence that the model learned meaningful behavioral patterns from the database. However, the interpretation remains dependent on the selected model, the distribution of the input variables, and the normalization of the output variable.

It should also be noted that the high importance of dimensional variables such as h , b , and w_c is partly related to the use of experimental shear capacity as the prediction target. Since the output represents normalized capacity rather than a normalized shear-stress index, member size naturally becomes influential. Therefore, future work should repeat the SHAP interpretation using normalized shear stress, such as $V_{exp}/(bd)$, to examine whether the variable ranking remains stable when the direct size effect is reduced. This would provide an additional view of the relative importance of geometry, concrete strength, and reinforcement.

In general, the SHAP analysis confirms that explainable ML can provide useful support to systematic review and mechanics-based interpretation. It should not be used as a substitute for design provisions, but it can help identify the most influential parameters, reveal database trends,

and clarify how different variables interact in predicting RCCs shear capacity. This combined approach provides a useful bridge between experimental evidence, code-based design models, and data-driven interpretation.

9. Limitations of the ML Interpretability Analysis

Although this study provides a systematic review and SHAP-based interpretation of RCCs shear capacity, several limitations should be acknowledged:

- Although the search was based mainly on Scopus and Web of Science and was supported by reference checking, some reports, theses, conference papers, or non-English studies may have been missed. Therefore, the collected studies should be viewed as a structured review database rather than a complete record of all corbel research.
- The reviewed works also varied in geometry, concrete type, reinforcement, loading setup, and testing method, which limits direct comparison. Non-conventional corbels, such as strengthened, repaired, fibre-reinforced, recycled aggregate, UHPC, and SCC specimens, were discussed qualitatively but were not included in the ML dataset to maintain consistency.
- The analytical and code-based review was used to clarify the assumptions and limits of selected design models, not to recalibrate design provisions. Similarly, the XGBoost model was trained on 330 mainly vertical-load specimens, so its predictions should be used carefully outside this range, and the combined H/V loading case is treated separately [145].
- The difference between the training and testing coefficients of determination, 0.9973 and 0.9090, indicates mild overfitting. A low learning rate, row and column subsampling were applied as regularization measures, but the residual gap reflects the limited size of the training sample relative to the 13-dimensional input space. The testing value should therefore be taken as the indicator of predictive performance, and the model should be used only as an explanatory instrument.
- The SHAP results depend on the dataset and model structure. Since some variables are physically correlated, as discussed in Section 8, their importance should be interpreted at the level of variable groups rather than individual ranks. Future work should expand the database, include uncertainty analysis, use normalized shear stress such as $V_{\text{exp}}/(bd)$, and develop separate models for strengthened, repaired, cyclic, and special-material corbels.
- The qualitative synthesis was limited by inconsistent reporting across primary studies. Key gaps included bearing and reinforcement detailing, geometric and material properties, cracking, and load-deflection data, failure modes, fibre characteristics, specimen age and curing, and repeated-loading protocols. Consistent reporting of these variables through a minimum dataset would enable broader quantitative synthesis, including serviceability behavior, and support future interpretable models.

10. Conclusions and Recommendations

This study presented a systematic review and SHAP-based interpretation of ML predictions for the shear capacity of reinforced concrete corbels. A structured PRISMA-guided review was conducted using Scopus and Web of Science records, supplemented by reference checking. After duplicate removal and screening, 121 studies were retained for qualitative synthesis. In addition, an XGBoost model was trained using 330 compatible vertical-load corbel specimens and interpreted using SHAP analysis. Based on the reviewed literature, analytical models, code provisions, and explainable ML results, the following conclusions can be drawn:

- The reviewed literature confirms that RCCs behave as discontinuity regions in which the applied load is transferred mainly through a diagonal compression strut, main tension reinforcement, secondary reinforcement, and nodal zones. Therefore, their shear behavior cannot be fully represented using ordinary sectional design assumptions.
- The a/d ratio is one of the most influential parameters affecting RCCs behavior. Lower a/d ratios promote direct strut action and generally increase shear capacity, while larger a/d ratios reduce the efficiency of the compression strut and increase the contribution of flexural and diagonal cracking mechanisms.

- Concrete compressive strength has a clear positive influence on cracking load, stiffness, and ultimate shear capacity. This effect is mainly related to the capacity of the diagonal compression strut and nodal regions. However, higher concrete strength may also increase brittleness, especially when adequate transverse reinforcement or fibre reinforcement is not provided.
- Longitudinal reinforcement plays an important role as the main tension tie in the strut-and-tie mechanism. Increasing the flexural reinforcement ratio generally improves ultimate capacity and post-cracking response, but excessive reinforcement may shift the failure mechanism toward concrete crushing, nodal distress, or less ductile behavior.
- Horizontal and vertical stirrups improve crack control, confinement, and deformation capacity. Horizontal reinforcement is generally effective in intercepting diagonal cracks and reducing brittle splitting, while vertical reinforcement contributes mainly to diagonal crack control and becomes more effective depending on the a/d ratio and the inclination of the compression strut.
- Column load appears to have limited influence on the ultimate shear capacity of RCCs, but it may improve serviceability by delaying crack initiation and reducing crack width near the corbel-column junction. Therefore, its effect should be interpreted mainly in relation to cracking behavior rather than as a direct strength-enhancement parameter.
- Repeated and cyclic loading can lead to stiffness degradation, wider cracks, accumulated damage, and reduced load-carrying capacity. These findings indicate that corbels subjected to crane loads, seismic actions, or repeated service loading require careful attention to ductility, crack control, and long-term performance.
- Strengthening and repair studies show that CFRP sheets, NSM reinforcement, steel plates, basalt-fibre systems, and fibre-reinforced repair materials can improve the capacity and serviceability of deficient or damaged RCCs. However, strengthened corbels may fail through debonding, peeling, anchorage distress, or brittle rupture. Therefore, these specimens should be treated separately from original unstrengthened corbels when developing shear-capacity databases or ML models.
- Strengthening effectiveness depended more on anchorage, development length, layup, and substrate condition than material quantity, highlighting the need for failure-mechanism-based rehabilitation design.
- The reviewed analytical and code-based models, including SFM, STM, and TTM approaches, provide useful design procedures. However, their predictions may be conservative or scattered because they are based on simplified assumptions and do not always capture the interaction among geometry, concrete strength, reinforcement ratios, support dimensions, and bearing conditions.
- The XGBoost model achieved good predictive performance for the compatible vertical-load dataset, with R^2 values of 0.9973 and 0.9090 for the training and testing sets, respectively. These results indicate that the model can capture the nonlinear relationship between the input variables and normalized experimental shear capacity. The gap between the two values reflects mild overfitting that the applied regularization measures reduced but did not eliminate, so the testing value should be regarded as the indicator of predictive performance. The model was used in this study for interpretation rather than as a new design equation.
- SHAP analysis demonstrated that corbel height, corbel width, column width, a/d , and concrete compressive strength were the most influential variables in the XGBoost model. This ranking is consistent with the mechanics of RCCs, where member geometry, strut configuration, and concrete strength govern the main load-transfer mechanism. Because the dimensional variables are mutually correlated, the ranking should be read at the level of the group rather than as a strict ordering of individual variables.
- Reinforcement-related variables showed positive but comparatively lower SHAP importance within the collected database. This does not mean that reinforcement is unimportant in design. Rather, it indicates that the marginal influence of reinforcement variables in the present dataset was smaller than that of geometry, a/d , and concrete strength, and it also reflects the limited range over which some reinforcement parameters were varied in the available tests.

As a recommendation, further research is suggested to expand the experimental database and develop separate interpretable models for strengthened, repaired, special-material, and cyclically loaded corbels. Future studies should also conduct SHAP analysis using normalized shear stress, such as $V_{exp}/(bd)$, to examine whether the relative importance of variables remains stable after reducing the direct size effect. In addition, a minimum reporting data set for corbel experiments, covering bearing dimensions, anchorage detailing, the k/h ratio, cracking loads and crack widths, the complete load-deflection response, and failure modes, would allow future studies to address serviceability as well as ultimate behavior.

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Wind-resistant optimization design of super high-rise buildings based on an improved genetic algorithm

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Abstract

To solve the problems of traditional structural optimization methods being prone to local optima, long computation time, and high memory consumption when dealing with high-dimensional nonlinear problems, a wind-resistant optimization design method for super high-rise intelligent buildings grounded on an improved genetic algorithm is proposed by studying the improvement of genetic algorithm through penalty function method. The findings indicate that this method can enhance the natural frequencies of the 1st and 2nd order structures, with amplification rates of 6.90% and 8.64%, respectively. After optimizing the design, the total values of transverse and downwind base shear forces are reduced by 3.60% and 1.21%, respectively. The total weight of the structure decreases from 52500 tons to 49830 tons, a decrease of 5.09%. Moreover, the research method has been demonstrated to outperform alternative approaches in terms of fitness convergence, solution stability, and constraint handling ability. The average optimization time, average objective function value, and fitness standard deviation of the research method are 1232s, 9.23×10^6 , and 0.016×10^6 , respectively. From this, the research method can enhance the lateral stiffness and response balance of the structure, verifying the engineering applicability and practical value of the proposed method in wind-induced response control of the structure.

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1. Introduction

With the continuous growth of urban population and the increasing scarcity of urban land resources, the utilization of building space has gradually shifted from horizontal expansion to vertical development. Super high-rise buildings (SHRBs), which can provide a large building usable area under limited land resources, have been widely used in urban buildings such as commercial complexes, offices and mixed residences. However, with the continuous increase in building height, the safety and comfort issues caused by wind-induced response have become increasingly prominent [1]. Compared with general multi-story or high-rise buildings, super high-rise buildings have the characteristics of long structural period, low damping ratio and relatively small lateral stiffness. Under long-term wind load, they are prone to large top lateral displacement, inter-story displacement and wind-induced vibration, which may not only affect structural safety, but also reduce the comfort of people inside the building. Therefore, on the basis of meeting the requirements of structural safety and normal use, reducing the self-weight of the structure and improving wind resistance by reasonably configuring the size of structural components is a key issue that needs to be addressed in the structural design of super high-rise buildings [2-3].

Existing wind-resistant design methods for super high-rise buildings primarily reduce wind-induced response through structural system optimization, component size optimization, and building form optimization. Among these, structural optimization design can achieve coordination between structural stiffness and material usage by adjusting the size and arrangement of major

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lateral resistance components such as beams, columns, and shear walls, without altering the overall function and usage requirements of the building. In accordance with the type of design variables, the optimization of a structure may be categorized into the following: topology optimization, size optimization, and shape optimization. For instance, Lu et al. advanced a dynamic characteristic optimization design approach for wind resistance evaluation of large steel frames founded on topology optimization. The findings demonstrated that by reasonably arranging shear walls, supports, and tube structures, the lateral displacement, inter-story drift, and overturning moment of high-rise buildings could be effectively reduced [4]. He Z et al. examined the structure of spiral SHRBs and put forward a strategy for aerodynamic shape optimization that integrated a multi-objective vibration acceleration approach. The findings showed that this strategy could effectively reduce wind-induced vibration response and optimize the aerodynamic performance of SHRBs while meeting the requirements of structural strength and comfort [5].

Furthermore, with the development of data-driven methods and intelligent optimization technologies, machine learning and heuristic optimization algorithms are increasingly being applied to performance prediction and parameter optimization of complex building structures [6]. For instance, Li Y et al. utilized ridge regression, decision tree, random forest, and gradient-enhanced regression tree to predict the wind pressure of SHRBs. The findings suggested that the last model exhibited satisfactory predictive capabilities for both average and fluctuating wind pressures [7]. Shan W et al. designed a Hooke-Jeeves algorithm based on structural constraints with the goal of minimizing the cost of structural materials, combined with finite element analysis (FEA). The findings indicated that the algorithm had good convergence and optimization effects, which can effectively promote the structural design's safety and economy [8]. However, there are still problems in the optimization design of SHRB structures, such as discrete design variables, difficulty in explicitly representing some constraint conditions through design variables, and non-differentiable objective functions (OFs) [9]. This type of feature makes traditional structural optimization design methods prone to getting stuck in local optima during the solving process, making it difficult to effectively cope with complex practical engineering conditions.

Genetic Algorithm (GA), as a biomimetic intelligent optimization algorithm based on natural selection and genetic mechanisms, has good global optimization ability and robustness. The optimization process does not rely on derivative information of the OF, nor does it require explicit modeling of constraint conditions [10]. Therefore, it can demonstrate stronger adaptability and flexibility in dealing with nonlinear, multimodal, and uncertain problems, effectively compensating for the shortcomings of traditional structural optimization design methods. For example, Yang J et al. conducted a comprehensive analysis and exploration of wind-resistant optimization design (WROD) for SHRBs based on hybrid GA, and proposed a design method that considers the joint probability distribution of wind speed and direction, reliability, and structural performance. The results indicated that this method not only improved the reliability and safety of building structural design, but also achieved optimization of material utilization and engineering costs [11]. Le Gia T T et al. proposed a comprehensive optimization design strategy using NSGA-II. The findings demonstrated that this strategy could effectively improve the comprehensive performance of early architectural design schemes in Vietnam in terms of energy efficiency, comfort, and adaptability [12].

In summary, certain achievements have been made in optimizing the structure of SHRBs under wind loads. However, existing research still has certain limitations: on the one hand, many structural optimization methods fail to fully consider the randomness of wind loads and their nonlinear coupling relationship with structural response in the modeling process, resulting in deviations between optimization results and real engineering environments. On the other hand, the application of GA in structural optimization mostly focuses on simple structural forms, and there are still problems such as long computation time and high memory consumption when facing high-dimensional design space. To this end, an innovative GA-based WROD method for super high-rise buildings is proposed. Through this method, the research aims to provide an efficient and feasible intelligent design strategy for the structural optimization of SHRBs under wind loads, thereby providing theoretical support and engineering practice reference for the fields of wind engineering and structural optimization.

1.1 Research Significance and Contribution

From an engineering application perspective, super high-rise buildings are characterized by their large structural height, numerous design variables, and significant wind-induced response. Traditional experience-based cross-section design methods struggle to balance structural safety, user comfort, and material economy. This study aims to minimize the total structural mass while incorporating performance constraints such as natural frequency, inter-story drift angle, top lateral displacement, and wind-induced peak ground acceleration. This approach enables the rational allocation of component materials while meeting wind resistance requirements, providing a reference for the coordinated design of lightweight and wind-resistant super high-rise buildings.

From a methodological perspective, this study addresses the problems of numerous infeasible solutions, insufficient convergence stability, and low computational efficiency that traditional GA algorithms may encounter in high-dimensional, multi-constraint structural optimization. It introduces a penalty function mechanism based on the degree of constraint deviation into the GA algorithm, enabling differentiated penalties based on the degree of constraint violation for different design schemes, thereby enhancing the algorithm's search capability in the feasible solution space. Simultaneously, an automated co-simulation process is established through the MATLAB and SAP2000 application programming interface (API), realizing automatic iteration between design variable generation, finite element modeling, wind load calculation, structural response analysis, and fitness evaluation. This provides a feasible technical path for the collaborative application of intelligent optimization algorithms and finite element analysis software.

The main contributions of this study are reflected in the following three aspects: (1) A multi-constraint optimization model for wind-resistant design of super high-rise buildings was established, which unified the structural quality target with the performance requirements such as natural frequency, inter-story drift angle, top lateral displacement and wind-induced peak acceleration, thereby improving the adaptability of the optimization design to actual engineering constraints; (2) An improved GA algorithm with a differentiated penalty function mechanism was proposed. This paper does not take the conventional multiplicative penalty function form as the main innovation, but instead applies continuous and differentiated penalties to infeasible solutions of different degrees by quantifying the degree of constraint violation of candidate solutions, and directly uses the modified fitness to the individual selection and evolution process of GA, so as to improve the problem of high infeasible solution ratio and decreased search efficiency in traditional GA; (3) A MATLAB-SAP2000 co-simulation optimization framework was constructed to realize automatic iteration of design variable update, finite element analysis, structural response extraction and fitness evaluation, providing a computational implementation platform for the engineering application of the proposed improved algorithm in wind-resistant optimization of super high-rise buildings. Furthermore, by comparing the proposed method with traditional GA, Particle Swarm Optimization (PSO), Ant Colony Optimization (ACO), and Differential Evolution (DE), the overall performance of the proposed method in terms of optimization efficiency, solution stability, and constraint satisfaction is verified.

2. Methods and Materials

2.1 Mathematical Model for Wind Load Calculation and Wind-Resistance Optimization of SHRBs

In the structural design of SHRBs, wind load calculations usually require consideration of a number of influencing factors, including basic wind pressure, the wind pressure height change coefficient, the wind carrier type coefficient, and the wind vibration coefficient. The basic wind pressure is primarily determined using meteorological observation data from the building's location over many years, reflecting the region's fundamental wind conditions. The wind pressure height variation coefficient considers the effect of wind speed on changes in height, and is usually based on boundary layer theory and the empirical formula recommended by the specification for correction. The wind carrier type coefficient relates closely to the building's geometric shape, windward surface arrangement and windward angle, and is used to characterize the distribution of wind pressure on different building surfaces. The wind vibration coefficient primarily reflects

the building's additional dynamic response under the influence of pulsating wind. According to the different directions of wind action, wind load can be subdivided into downwind load (pressure generated along the direction of wind flow), crosswind load (lateral force perpendicular to the direction of wind flow), and torsional load (torsional moment around the vertical axis caused by asymmetric wind pressure) [13-14]. This paper determines the basic wind pressure, wind pressure height variation coefficient, and wind load shape coefficient according to the "Load Code for Design of Building Structures" (GB 50009-2012), and obtains the wind-induced response of the structure using the wind vibration response calculation method in the code. Based on this, the average response, background response, and resonance response generated by wind load are equivalent to static loads, and the equivalent static wind loads in the along wind and crosswind directions are calculated separately. For each structural optimization scheme, the corresponding wind vibration response and equivalent static wind load are recalculated based on the current structure's mass, stiffness, and natural frequency. The formula for calculating the equivalent static wind load (ESWL) in the downwind direction is given in equation (1).

$$\hat{F}_D(z) = \bar{F}_D(z) + \sqrt{F_{DB}(z)^2 + F_{DR}(z)^2} \quad (1)$$

In formula (1), $\hat{F}_D(z)$ represents the ESWL of the total downwind direction at height z ; $\bar{F}_D(z)$ represents the average wind load component, which is the steady-state load generated by the average wind speed; $F_{DB}(z)$ represents the background component, which is the quasi-static response caused by low-frequency wind speed fluctuations; $F_{DR}(z)$ represents the resonance component, which is the dynamic response generated by the resonance between the structure and the wind. The calculation formula for ESWL in crosswind direction is shown in formula (2).

$$\hat{F}_L(z) = \sqrt{F_{LB}(z)^2 + F_{LR}(z)^2} \quad (2)$$

In formula (2), $\hat{F}_L(z)$ represents the total ESWL in the crosswind direction at height z ; $F_{LB}(z)$ represents the background wind load component, which is the quasi-static crosswind response caused by low-frequency pulsating wind; $F_{LR}(z)$ represents the resonant wind load component, which is the dynamic response caused by the resonance between crosswind and structural vibration. Single high-rise buildings with symmetrical shapes usually have weak torsional effects, so research will not explore them. In the process of structural optimization, constraints are often difficult to express explicitly through design variables [15]. To improve modeling efficiency and analysis accuracy, finite element software SAP2000 is used for solving. By calling the API interface of SAP2000, it is easy to define and adjust the structural parameters and geometric dimensions of the model, and efficiently obtain analysis results. At the same time, considering that a single software platform is difficult to meet multiple requirements, the study adopts MATLAB and SAP2000 software for joint simulation. The schematic diagram of the joint simulation process structure of MATLAB and SAP2000 is illustrated in Figure 1.

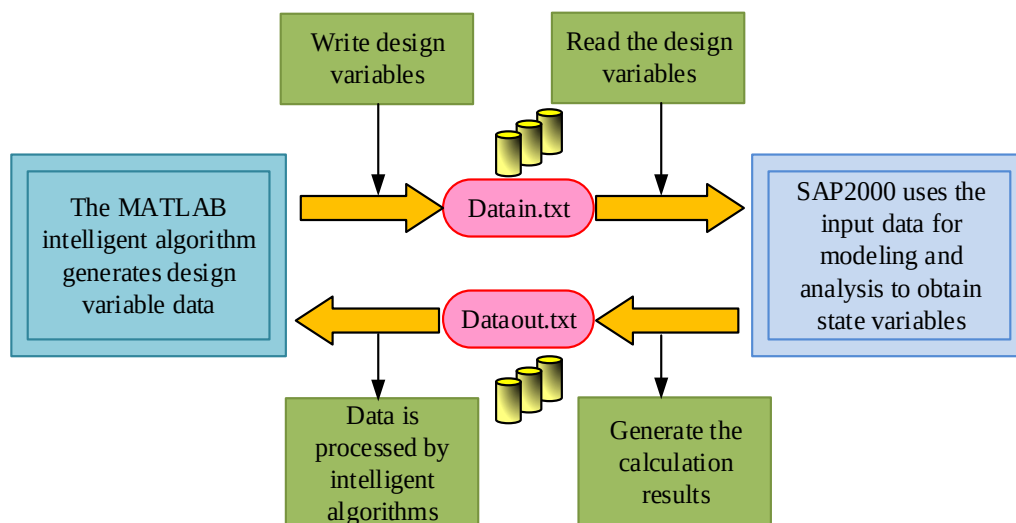


Fig. 1. Schematic diagram of the joint simulation process structure of MATLAB and SAP2000

From Figure 1, the joint simulation of the two is achieved through program control. Firstly, MATLAB executes intelligent optimization algorithms and generates design variables, which are written into input files in the specified format. Subsequently, SAP2000 reads the input data, automatically updates the structural model through API interface, and performs FEA to obtain the structural response results. Next, SAP2000 will provide feedback on the analysis results to MATLAB as a basis for evaluating the degree to which the OF values and constraint conditions of the current design scheme are satisfied. The intelligent optimization algorithm determines whether the convergence criteria are met based on this, and if not, continues to iterate and update until the optimal solution is obtained. Ultimately, the system outputs optimized design results that meet performance requirements, thereby achieving an efficient and automated structural wind resistance optimization process. Due to the large number of uncertain factors in the design of SHRBs, if all factors are fully considered, the model will become too complex to achieve effective control and solution. Therefore, the research will limit the optimization scope to the cross-sectional dimensions of the components. The schematic diagram of the cross-sectional structure of the component is illustrated in Figure 2.

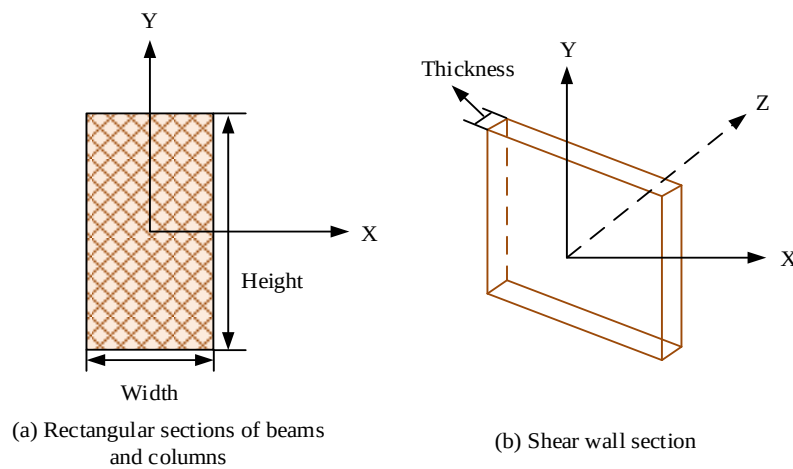


Fig. 2. Schematic diagram of the cross-sectional structure of the component

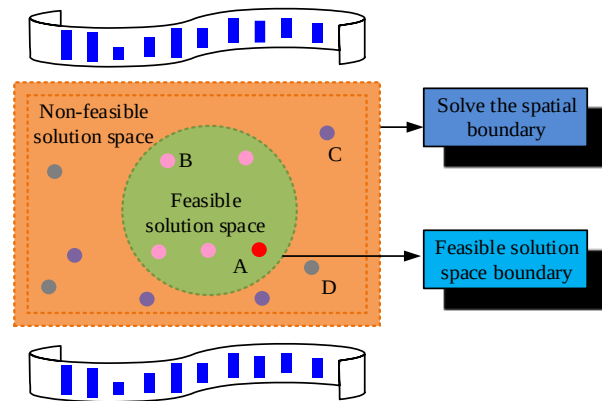


Fig. 3. Punishment mechanism

Figure 2 (a) showcases the rectangular cross-section of beams and columns, while Figure 2 (b) showcases the cross-section of shear walls. As shown in Figure 2, the study selects the height, width, and thickness of shear walls of beams and columns in rectangular cross-sections as design variables. The reasonable handling of constraints during the optimization process has a crucial impact on the feasibility and effectiveness of the results [16]. The penalty function method is a commonly used constraint handling strategy in intelligent optimization algorithms. Its basic idea is to introduce a penalty term for solutions that violate constraints, significantly increasing the OF value and reducing the corresponding fitness [17-18]. A penalty function method is designed to penalize unconstrained optimal solutions that may be located outside the feasible domain boundary based on the degree of constraint deviation. The penalty mechanism of this new penalty

function method is shown in Figure 3. As shown in Figure 3, the orange rectangle represents the possible range of all solutions, divided into feasible solution space and infeasible solution space. The green circle denotes the feasible solution space, the circular boundary represents the feasible solution space boundary, and the area outside the circle but still within the rectangle represents the infeasible solution space. This new penalty function reduces the severity of punishment for infeasible solutions by differentiating them, and can synergistically guide search between feasible and infeasible domains, thereby effectively improving the global optimization ability of intelligent algorithms. The formula for calculating the fitness value $F(x_i)$ of the i th individual in the population is shown in formula (3).

$$F(x_i) = \begin{cases} f_{max}^p - (f(x_i) + P(x_i)) \\ f_{max}^p - f(x_i) \end{cases} \quad (3)$$

In formula (3), $f(x_i)$ represents the target value corresponding to the i th individual, $P(x_i)$ represents the penalty function value corresponding to the i th individual, and f_{max}^p represents the upper limit value of the penalty function. The degree of violation $v_j(x_i)$ of the i th individual x_i against the j th constraint $g_j(x_i)$ is shown in formula (4).

$$v_j(x_i) = \max\{0, |g_j(x_i)|\} \quad (4)$$

If $g_j(x_i) \leq 0$, then $v_j(x_i) = 0$, indicating that the constraint is satisfied. $g_j(x_i) > 0$, then $v_j(x_i) = g_j(x_i)$, the greater the value, the greater the deviation from the constraint. The calculation formula for the OF $\varphi(x)$ after applying the penalty is presented in formula (5).

$$\varphi(x) = F(x) \cdot (1 + K \cdot C) \quad (5)$$

In formula (5), $F(x)$ denotes the original OF; C is the sum of all constraint violations, reflecting overall infeasibility; K represents the penalty coefficient, which is used to control the severity of punishment. The larger the K , the stronger the punishment for constraint violations. By comprehensively comparing the constraint satisfaction, objective function value, and fitness convergence stability under different penalty coefficients, the penalty coefficient was finally determined to be $K = 1.0 \times 10^8$.

In summary, this study employs ESWL for wind load modeling of super high-rise buildings. This method transforms the average response, background response, and resonance response generated by actual wind loads into static loads equivalent to the main structural responses. The background component characterizes the quasi-static response caused by fluctuating winds, while the resonance component characterizes the dynamic response generated by the combined effects of wind load frequency components and the structure's natural vibration characteristics. Since changes in structural cross-sectional dimensions cause variations in structural stiffness, mass, and natural frequency, modal analysis is re-performed based on the current structural parameters during optimization, and the corresponding equivalent static wind load is updated accordingly, achieving an iterative correlation between wind loads and structural dynamic characteristics.

2.2 Wind-Resistant Optimization Method for SHRBs Based on Improved GA

Based on the joint simulation of MATLAB and SAP2000, a complete structural response analysis and modeling process is constructed, and an optimization design framework is defined with rectangular section size as the optimization variable and total structural mass as the OF. However, in the face of optimization problems with complex constraints, traditional intelligent optimization algorithms still have certain limitations in terms of solving efficiency and feasibility of results. To this end, the study further introduces a new penalty function into the GA, which improves the convergence ability of the GA on feasible solutions by imposing penalties on design schemes that violate constraints. Finally, a wind-resistant optimization method for SHRBs grounded on the improved GA is proposed. Traditional GA primarily search for optimal solutions through selection, crossover, and mutation operations. However, for multi-constraint problems in wind resistance optimization of super high-rise buildings, this can easily generate a large number of infeasible solutions, increasing the number of finite element analyses and reducing search efficiency. To

address this issue, this study introduces a differentiated penalty function based on the degree of constraint violation into the traditional GA framework to adaptively penalize infeasible solutions. First, GA generates candidate design schemes, and the structural response is calculated using SAP2000 via MATLAB. Then, the objective function and constraint violation degree of each candidate scheme are calculated. For individuals that meet the constraints, their constraint violation is set to 0, incurring no additional penalty. For individuals that violate the constraints, the violation is calculated based on the degree of deviation from the constraint limits, with the penalty intensity increasing with the degree of violation. Therefore, individuals with minor constraint violations are not directly eliminated and can still participate in subsequent genetic searches, while individuals with severe constraint violations are subject to stronger penalties, thus guiding the population to gradually converge towards the feasible region. The fitness, modified by the penalty function, is used for subsequent individual selection, ensuring that the algorithm prioritizes design schemes with better objective functions and higher constraint satisfaction.

The mathematical model calculation formula for WROD is shown in formula (6).

$$W = \sum_{i=1}^N w_i b_i h_i L_i + \sum_{j=1}^{N_w} \rho_{w,j} t_{w,j} A_{w,j} + \rho_f t_f B D M \quad (6)$$

In formula (6), W represents the structure's total weight, w_i represents the density of the i th type of beam column component, b_i and h_i represent the beam and column sections' width and height, L_i represents the total length of the i th type of beam-column member, N represents the number of beam and column section types, $\rho_{w,j}$ represents the material density of the j th type of shear wall, $t_{w,j}$ represents the thickness of the j th type of shear wall; $A_{w,j}$ represents the planar area of the corresponding shear wall; N_w represents the number of shear wall types; ρ_f represents the material density of the floor slab, B and D denote the width and height of the building, ρ_f represents the density of the floor slab, M means the number of floors, and t_f represents the thickness of the floor slab. The mathematical expression for the constraint conditions of component cross-sectional dimensions is shown in formula (7).

$$\begin{cases} b_i^L \leq b_i \leq b_i^U & i = 1, 2, \dots, N \\ h_i^L \leq h_i \leq h_i^U & i = 1, 2, \dots, N \end{cases} \quad (7)$$

In formula (7), L and U represent the lower and upper limits of the rectangular cross-sectional dimensions, respectively. The mathematical expression for the constraint conditions of inter-story drift angle (ISDA) and top layer lateral displacement of the structure is shown in formula (8).

$$\begin{cases} d_H \leq d_H^U \\ d_j \leq d_j^U & j = 1, 2, \dots, M \end{cases} \quad (8)$$

In formula (8), d_j and d_{j-1}^U represent the difference and limit values of inter story lateral displacement, while d_H^U and d_H represent the limit values of building top lateral displacement and building top lateral displacement, respectively. The constraint conditions for wind-induced peak acceleration of the structure are shown in formula (9).

$$\varpi \geq \varpi' \quad (9)$$

In formula (9), ϖ and ϖ' represent the natural frequency of the structure and its lower limit, respectively. The schematic diagram of the wind-resistance optimization method for SHRBs based on improved GA is shown in Figure 4.

In Figure 4, in each iteration, the research method recalculates the structural dynamic characteristics and equivalent static wind load based on the updated structural parameters. Since different cross-sectional dimensions correspond to different structural masses, stiffnesses, and natural frequencies, the resonant response under wind load also adjusts with changes in structural dynamic characteristics. Therefore, the modal analysis results are fed back to the ESWL calculation, and the updated equivalent wind load is reapplied to the structural model to achieve an iterative correlation between structural parameters, dynamic characteristics, and wind-induced response.

By writing interface programs, data exchange and calling between MATLAB and SAP2000 can be achieved, thereby completing the process of updating modal parameters and wind loads. The optimization process includes: first, determining the design objective, defining the design variables, OF and constraints; second, initializing the algorithm parameters and generating the initial population. A finite element model is built grounded on the initial cross-section, and the ESWL is calculated using the three-component method and applied to the centers of mass of each layer to obtain the structural response and thus assess the degree of adaptation. The GA generates a new generation population through selection, crossover, and mutation operations, combined with a novel penalty function to handle constraints and retain excellent individuals. This process continues to loop until the constraints are met, convergence is achieved, or iteration is terminated, ultimately obtaining the optimization result.

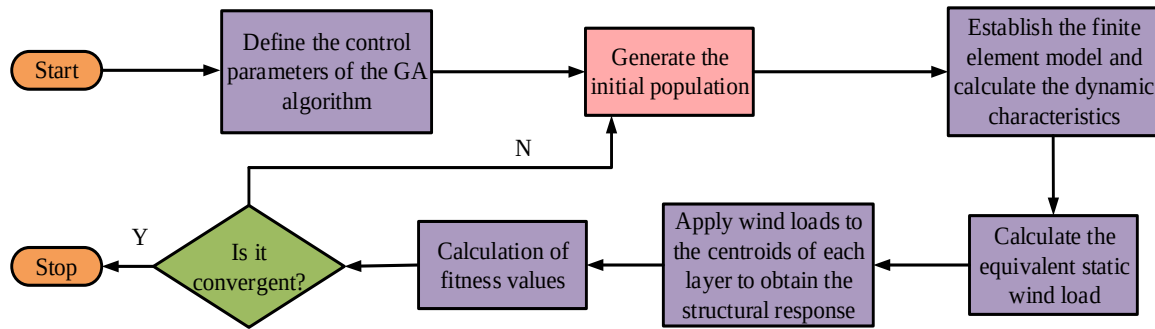


Fig. 4. Schematic diagram of the process structure for WRODs of SHRBs

This study selected a 60-story super high-rise building with a total height of 185.7 m as the research object, employing a reinforced concrete frame-shear wall structural system. Based on the vertical load transfer law and the design characteristic of decreasing component cross-sections with height, the 60 stories were divided into 5 zones, corresponding to columns JZ1-JZ5 and beams JL1-JL5 respectively. The initial cross-sectional dimensions of each component were determined according to the stress requirements of different floors, with upper and lower limits determined by combining initial dimensions, structural stiffness requirements, and the "Technical Specification for Concrete Structures of High-Rise Buildings" (JGJ 3-2010). To reduce design variables, the width-to-height ratio of column cross-sections was fixed at 1.0, the width-to-height ratio of beam cross-sections was fixed at 0.5, and the thickness of shear walls was used as the design variable. The floor slab thickness was uniformly set to 130 mm. All beams, columns, and shear walls were made of reinforced concrete, and the material was assumed to be in a linear elastic state. Specific geometric and physical parameters and the range of design variables are shown in Table 1.

Table 1. Geometric physical parameters of the structural model

Component name	Width/m			Height/m		
	Initial value	Lower limit value	Upper limit value	Initial value	Lower limit value	Upper limit value
JZ1	0.90	0.70	1.20	0.90	0.70	1.20
JZ2	0.85	0.65	1.15	0.85	0.65	1.15
JZ3	0.80	0.60	1.10	0.80	0.60	1.10
JZ4	0.75	0.55	1.05	0.75	0.55	1.05
JZ5	0.70	0.50	1.00	0.70	0.50	1.00
JL1	0.60	0.50	0.80	0.30	0.25	0.40
JL2	0.58	0.48	0.78	0.29	0.24	0.39
JL3	0.56	0.46	0.76	0.28	0.23	0.38
JL4	0.54	0.44	0.74	0.27	0.22	0.37
JL5	0.52	0.42	0.72	0.26	0.21	0.36

According to Table 1, JZ1 to JZ5 respectively represent the frame columns of floors 1-10, 11-20, 21-30, 31-40, and 41-60, while JL1 to JL5 correspond to the frame beams of their respective floors.

The initial structural model is subjected to a modal analysis, which is performed using the block-wise Lanzo method, and the first two natural frequencies are obtained to be 0.145Hz and 0.162Hz, respectively. The constraint limit setting requirements are shown in Figure 5.

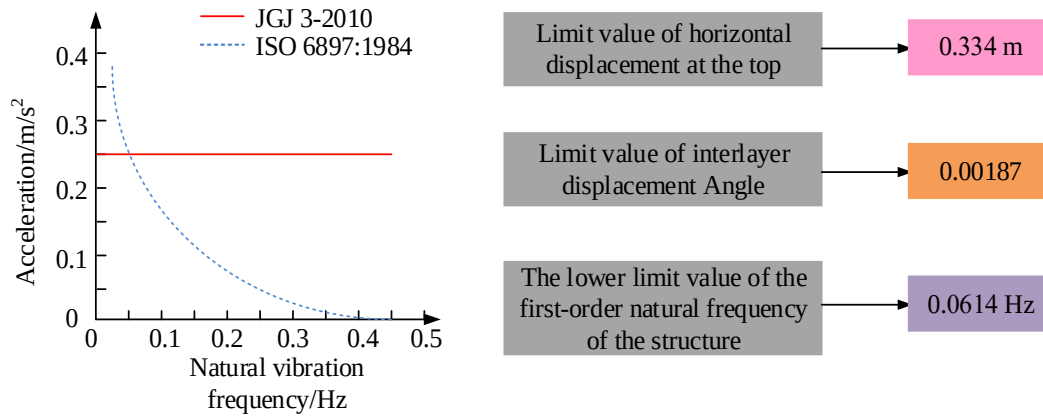


Fig. 5. Requirements for setting the limit values of constraint conditions

From Figure 5, to ensure the optimized structure meets safety and normal use requirements, this study determined normative limit values for structural response based on the "Technical Specification for Concrete Structures of High-Rise Buildings" (JGJ 3-2010) and ISO 6897:1984, and used these as constraints in the wind-resistant optimization process. Considering a 60-story reinforced concrete frame-shear wall structure with a total height of 185.7 m, the structural performance was controlled primarily from three aspects: top horizontal displacement, inter-story drift angle, and wind-induced vibration comfort. The top horizontal displacement limit was determined according to the relevant requirements of JGJ 3-2010, and was controlled within 1/550 of the total building height, corresponding to a top horizontal displacement control value of 0.334 m for this study structure. The inter-story drift angle limit was also determined according to JGJ 3-2010, combined with the structural system and functional use of the study object, and set to a control value of 0.00187 to limit the relative lateral deformation between adjacent floors. Regarding wind-induced vibration comfort, considering that the research subjects are mainly used in long-term use scenarios such as offices and hotels, in accordance with the relevant requirements of ISO 6897:1984, the maximum acceptable acceleration control value for the human body is set to 0.25 m/s², and based on this, the lower limit of the first-order natural frequency of the structure is determined to be 0.0614 Hz. The formula for calculating the peak acceleration $\ddot{x}_a$ within the duration T of the R year return period is shown in formula (10).

$$\ddot{x}_a = \sqrt{2 \ln n T} \left(0.68 + \frac{\ln R}{5} \right) \exp(-0.35 - 0.41 \ln n) \quad (10)$$

In formula (10), n represents the first-order frequency of the structure. Therefore, the main standard limits adopted in this study include: top horizontal displacement not exceeding 0.334 m, inter-story drift angle not exceeding 0.00187°, acceptable wind-induced acceleration not exceeding 0.25 m/s², and the first-order natural frequency of the structure not lower than 0.0614 Hz. These limits are used to control the overall lateral deformation of the structure, local floor deformation, and wind-induced vibration comfort, and are uniformly used as the basis for improving the GA algorithm to evaluate the feasibility of candidate design schemes.

3. Results

3.1 Optimization Result Analysis

To evaluate the efficacy of the proposed methodology, a suitable experimental environment was established. A finite element model of a super high-rise building was built using SAP2000 v24. Beams and columns were simulated using frame elements, while floor slabs were simulated using shell elements, with a floor slab thickness of 130 mm. Frame elements were used to describe the axial, bending, and shear stress behavior of beams and columns, while shell elements were used to characterize the in-plane and out-of-plane stiffness of the floor slabs. The total height of the

structure is 185.7 m, with 60 floors, constructed of reinforced concrete, and the material was assumed to be in a linear elastic state. The software and hardware environments and model parameter settings are listed in Table 2.

Table 2. Hardware and software environment and model parameter settings

Serial number	Type	Settings
1	CPU	Intel Core i7-12700K @ 3.6 GHz, 12 cores
2	RAM	32 GB DDR4
3	Operating System	Windows 10 Professional 64-bit
4	MATLAB Version	MATLAB R2022b
5	Finite Element Software	SAP2000 v24
6	API Interface	SAP2000 OAPI (Open API)
7	Population Size	100 individuals
8	Number of Generations	200 generations
9	Crossover Rate	0.85
10	Mutation Rate	0.05

Population size primarily affects the search space coverage and the computational cost per generation. Therefore, while considering the computational complexity of the finite element model for super high-rise buildings, the population size was set to 100, and the number of iterations was set to 200 to ensure the algorithm has a sufficient search process. The crossover rate and mutation rate were set to 0.85 and 0.05, respectively, to maintain population diversity while avoiding excessive random search. All the above parameters were tested in preliminary experiments and determined after comprehensively considering convergence stability, optimization time, and objective function value. To avoid the influence of parameter differences on the algorithm comparison results, traditional GA, PSO, ACO, and DE were all compared and tested using the same computational environment, design variables, and constraints.

During the optimization design process, the trend of the 1st and 2nd natural frequencies of the structure is shown in Figure 6.

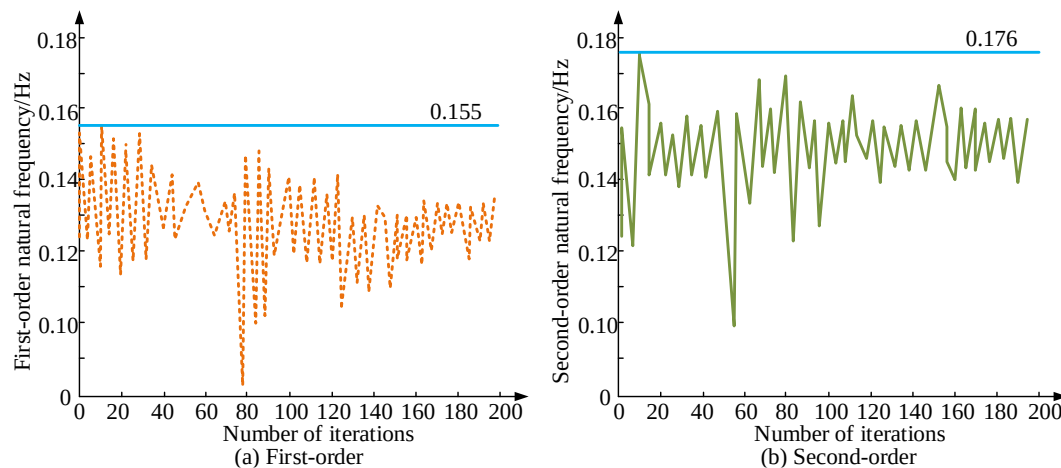


Fig. 6. The variation trends of the first and second order natural frequencies of the structure

In Figures 6(a) and 6(b), the natural frequency variation curves of the first- and second-order structures undergo changes during the optimization process. In Figure 6, the natural frequencies of the first- and second-order structures exhibited a gradual increase in response to the adjustment of design variables during the initial stage of optimization. This increase was followed by a slight decrease after reaching the peak, ultimately leading to stabilization. This observation signifies that the optimization process has attained convergence. The optimized design increased the natural frequencies of the 1st and 2nd order structures from 0.145 Hz and 0.162 Hz to 0.155 Hz and 0.176 Hz, with amplification rates of 6.90% and 8.64%. The increase in natural frequency effectively

improved the dynamic characteristics of the structure, weakened its sensitivity to resonance components in wind loads, and reduced wind-induced response. The ESWL changes in crosswind direction and downwind direction are shown in Table 3.

Table 3. Optimization of the ESWLs of the structure in the downwind and crosswise directions before and after the design

Indicators		Wind load direction	Pre-optimization numerical value
Wind load components/kN	Average component/kN	Downwind direction	19800
	Background component/kN	Downwind direction	6400
	Resonance component/kN	Downwind direction	2000
	Background component/kN	Crosswind direction	5800
	Resonance component/kN	Crosswind direction	18900
Total base shear force/kN		Downwind direction	28,200
Total base shear force/kN		Crosswind direction	24,700
Total base bending moment/kN·m		Downwind direction	9.55×10 ⁶

According to Table 3, after optimization design, the total values of transverse and downwind base shear forces were reduced by 3.60% and 1.21%, respectively. Among them, the reduction in crosswind base shear force was more significant, due to the higher proportion of resonant components in the crosswind response, which are closely related to the structure's natural vibration characteristics. In addition, compared with before optimization design, the crosswind resonance component decreased by 3.65% and the background component decreased by 3.45% after optimization. The background and resonance components in the downwind direction decreased by 2.34% and 9.50%, respectively. The bending moment of the foundation also decreased, decreasing by 1.78% in the downwind direction and 4.28% in the crosswind direction. The total weight of the structure decreased from 52500 tons to 49830 tons, a decrease of 5.09%. The research method improved the utilization efficiency of materials while satisfying constraints such as lateral displacement and acceleration.

3.2 Wind-Resistance Performance Verification

To further verify the reliability of the proposed improved GA algorithm optimization results, the structural response before and after optimization was validated. The structural response validation mainly focused on the top horizontal displacement and inter-story drift angles, and cross-validation was performed using a "SAP2000 finite element calculation result - theoretical calculation result" approach. First, based on the pre-optimization structural model, an SAP2000 finite element model was established according to the actual geometric parameters, material parameters, and wind load calculation results to calculate the top horizontal displacement and inter-story drift angles of each floor. Subsequently, based on the overall lateral stiffness of the structure and the equivalent static response under wind load, the top horizontal displacement was independently calculated theoretically, and the theoretical results were compared with the SAP2000 finite element calculation results. The optimized model was then verified using the same calculation process to check the reliability of the structural response results before and after optimization. The structural response validation results are shown in Table 4.

Table 4. Validation results of structural responses

Structural response	Calculation method	Pre-optimization	Post-optimization	Relative error/%
Top lateral displacement/m	SAP2000	0.295	0.275	—
Top lateral displacement/m	Theoretical calculation	0.291	0.279	1.36/1.45
Maximum inter-story drift ratio	SAP2000	0.00201	0.00176	—
Maximum inter-story drift ratio	Independent calculation	0.00198	0.00173	1.49/1.70

Table 4 shows that before optimization, the SAP2000 calculation result for the horizontal displacement at the top of the structure was 0.295 m, while the independent theoretical calculation result was 0.291 m, with a relative error of 1.36%. After optimization, the SAP2000 calculation result was 0.275 m, while the theoretical calculation result was 0.279 m, with a relative error of 1.45%. The results obtained by the two calculation methods are quite close, with errors controlled within 2%, indicating that the SAP2000 finite element model can accurately reflect the overall lateral deformation characteristics of the structure under wind load. Furthermore, the relative errors of the two calculation results for the maximum inter-story drift angle before and after optimization were 1.49% and 1.70%, respectively, further verifying the reliability of the structural response calculation results.

Based on the structural response verification, the validated SAP2000 finite element model was used to perform response analysis on each generation of structural schemes during the optimization process, and the horizontal displacement of each floor was extracted. The performance of the super high-rise building structure under wind load before and after optimization was compared and analyzed. Based on the standard limits determined in Section 2.2, this paper uses the top horizontal displacement of 0.334 m as the overall lateral deformation control index and the inter-story drift angle of 0.00187 as the floor deformation control index, and uses these to evaluate the wind resistance performance of the structure before and after optimization. The changes in lateral displacement of each floor before and after optimization are shown in Figure 7.

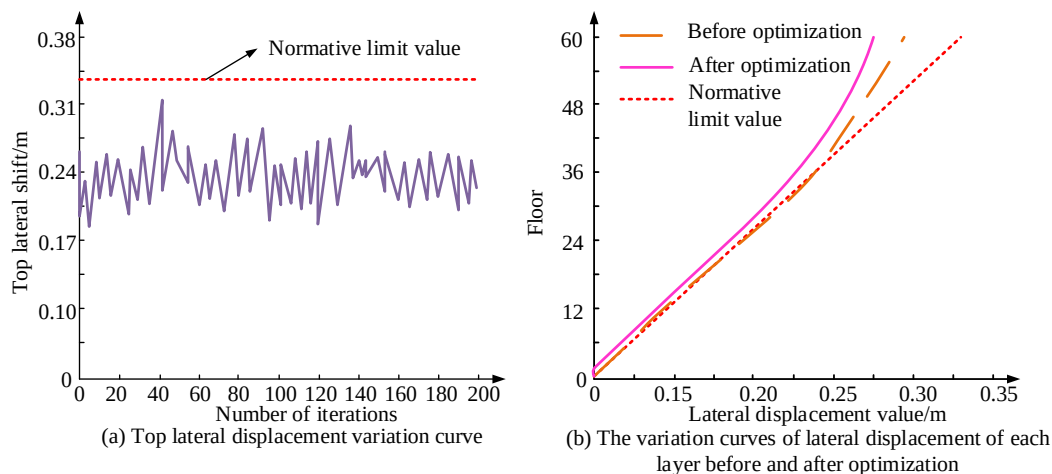


Fig. 7. The variation of lateral displacement of each layer before and after optimization

Figure 7(a) demonstrates the variation curve during the optimization process of top lateral displacement, and Figure 7(b) presents the variation curve of lateral displacement for each layer prior to and post-optimization. In Figure 7, an enhancement in the overall lateral stiffness of the building structure was observed with increased iteration times. Additionally, a reduction in the horizontal displacement at the top from 0.295m to 0.275m, representing a decrease of approximately 6.78%, was evident. Before optimization, the top horizontal displacement already

met the control limit of 0.334 m, and after optimization, it was further reduced, indicating that the overall lateral deformation of the structure was effectively controlled. Meanwhile, the inter-story drift angles of the middle and upper floors exceeded the limit of 0.00187 before optimization, reflecting a certain degree of stiffness deficiency in local areas. The optimized design improved the lateral stiffness of this area by adjusting the column and wall cross-sectional dimensions of the corresponding floors, ensuring that the inter-story drift angles of the optimized structure met the set limit. Therefore, the proposed method can reduce the number of structural materials used while reasonably adjusting the structural stiffness, thereby improving the overall and local wind resistance performance of the building. The changes in the lateral displacement angle between each layer before and after optimization are shown in Figure 8.

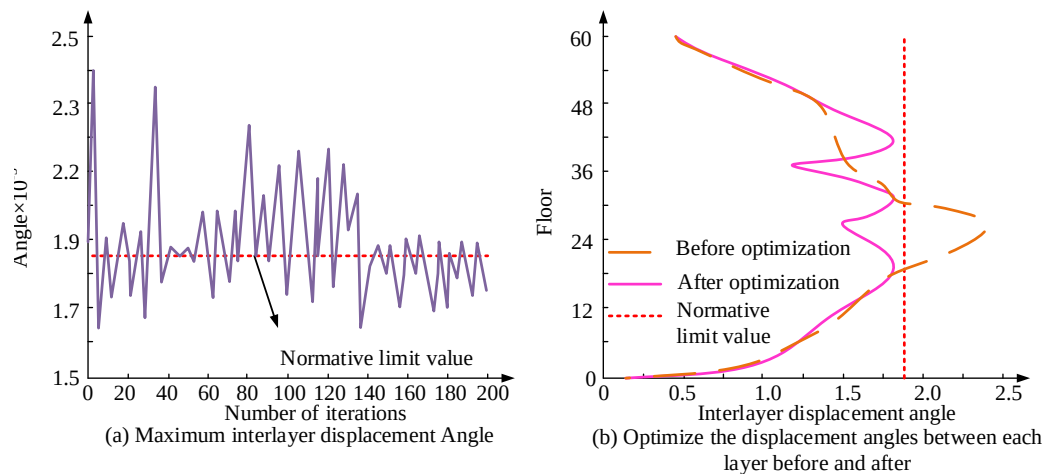


Fig. 8. The variation of the lateral shift angle between each layer before and after optimization

Figure 8 (a) showcases the variation curve of the maximum interlayer displacement angle during the optimization process, while Figure 8 (b) showcases the distribution of interlayer displacement angles before and after optimization. In Figure 8 (a), during the initial stage of optimization, the max ISDA exceeded the limit specified in JGJ 3-2010 due to the unadjusted design variables. As the number of iterations rose, the lateral stiffness of the structure gradually increased, and the max ISDA showed a fluctuating downward trend, ultimately converging within the allowable range of the specifications. As shown in Figure 8 (b), before optimization, the ISDA of the middle floor reached its peak, exceeding the standard limit, indicating insufficient local stiffness in the area. After optimization, by adjusting the cross-sectional dimensions of the columns and beams, the stiffness of the area was significantly improved, the ISDA was significantly reduced, and the overall response was more balanced, meeting the requirements of the specifications. The optimized cross-sectional dimensions of the components decreased with the height of the building, reflecting the principles of reasonable material allocation and center of gravity control. The section widths of the column components on floors 1-10, 11-20, 21-30, 31-40, and 41-60 were 0.65m, 0.60m, 0.55m, 0.50m, and 0.45m, respectively. The section widths of the corresponding floor beams were 0.45 m, 0.45 m, 0.40 m, 0.35 m, and 0.30 m, all within the section range set in Table 1, meeting the dual requirements of structural performance and dimensional constraints. Finally, the study introduced traditional GA, PSO, ACO, and DE as comparative algorithms, each running independently for 30 runs. All algorithms used the same design variables, objective function, constraints, population size, maximum number of iterations, and computational environment. The population was reinitialized for each run using a different random number seed. By statistically analyzing the average optimization time, mean optimal objective function value, worst-case deviation rate, number of constraint violations, and fitness standard deviation across the 30 runs, the computational efficiency, optimization quality, stability, and constraint handling capabilities of the different algorithms were comprehensively compared to verify the effectiveness of the proposed improved GA algorithm. The test findings are presented in Table 5.

According to Table 5, among the five algorithms, the improved GA performed the best with an average optimization time of 1232 seconds, demonstrating high computational efficiency. In terms of fitness convergence performance, the average value of the optimal OF obtained by improving the

GA was 9.23×10^6 , which was greatly better than other algorithms. This indicated that it had stronger global search capability in multi-dimensional high constraint environments. In addition, the worst solution deviation rate of the improved GA was only 2.13%, indicating that it had better solution space stability while maintaining optimization quality. In terms of constraint handling, the improved GA only experienced 2 mild constraint violations, verifying the effective suppression ability of the penalty function on infeasible solutions. The standard deviation of fitness was only 0.016×10^6 , which also reflected the significant improvement of this method in population stability. From this, the improved GA not only has better global search ability and constraint feasibility processing ability, but also demonstrates stronger stability and applicability in engineering practice, providing a more advantageous intelligent design path for high-dimensional and multi constraint structure optimization.

Table 5. Multi-index performance testing

Algorithms	Average optimizati on time/s	The mean value of the optimal OF	Worst solution deviation rate/%	The number of constraint violations	Standard deviation of fitness
Improved GA	1232	9.23×10^6	2.13	2	0.016×10^6
Traditional GA	1317	9.31×10^6	3.62	4	0.027×10^6
PSO	1389	9.35×10^6	3.04	5	0.034×10^6
ACO	1471	9.44×10^6	4.18	8	0.042×10^6
DE	1295	9.37×10^6	2.89	3	0.031×10^6

4. Discussion

Existing research mainly focuses on reducing the wind-induced response of high-rise buildings through aerodynamic shape optimization, topology optimization, and structural dimension optimization [4-5,8,11]. Aerodynamic shape optimization emphasizes mitigating airflow separation, vortex shedding, and crosswind aerodynamic forces, while structural parameter optimization primarily alters the lateral stiffness and mass distribution of the structure by adjusting component cross-sections [5,10]. This paper's results show that changes in cross-sectional parameters simultaneously alter the structural stiffness matrix, mass matrix, and modal characteristics, resulting in a significant structural dynamic feedback characteristic in the wind-induced response. Therefore, for ultra-high-rise frame-shear wall structures, the key to wind resistance optimization is not simply increasing lateral stiffness, but rather coordinating stiffness distribution, mass participation, and natural vibration characteristics to avoid insufficient local stiffness or excessive stiffness configuration. This aligns with existing research on the synergistic optimization of structural performance and material usage [9-10], but further demonstrates the impact of cross-sectional dimension adjustment on structural dynamic characteristics.

From an optimization solution perspective, ultra-high-rise structural design is a typical high-dimensional, discrete, nonlinear constrained optimization problem. Its fitness evaluation is directly coupled with finite element analysis, leading to high computational costs for single-function evaluation. Existing studies on GA, PSO, and multi-objective genetic algorithms mainly focus on global search and Pareto solution set acquisition [11-12]. However, the examples in this paper further demonstrate that the handling of the feasible region constraints directly affects the population search efficiency. For performance constraints such as inter-story drift angle, top displacement, and wind-induced acceleration, the use of a rigid infeasible solution elimination mechanism may result in the loss of effective search information near the feasible region boundary; continuous penalties based on constraint violations can maintain a certain boundary search capability. It should be noted that this characteristic is closely related to the finite element model size, the degree of dispersion of design variables, and the degree of constraint activity, and cannot be simply generalized to all structural optimization problems. In addition, this paper uses the standard equivalent static wind load and has not yet considered the unsteady aerodynamic forces, vortex-induced response, and building group interference effects caused by complex shapes.

Therefore, the obtained optimization law is mainly applicable to the parametric wind-resistant design of regular super high-rise structures.

5. Conclusions

This study addresses the structural response optimization problem of super high-rise buildings under wind loads. A wind-resistant optimization design method based on an improved GA algorithm is proposed, and iterative optimization of structural parameters, wind loads, and responses is achieved through joint simulation using MATLAB and SAP2000. The main conclusions are as follows:

- The proposed method can effectively improve the dynamic characteristics and wind resistance of super high-rise buildings. During the optimization process, the natural frequencies and equivalent static wind loads are updated according to changes in structural parameters, and the average, background, and resonance responses are comprehensively characterized. After optimization, the first and second natural frequencies of the structure increase from 0.145 Hz and 0.162 Hz to 0.155 Hz and 0.176 Hz, respectively, representing increases of 6.90% and 8.64%; the total base shear forces in the crosswind and along wind directions decrease by 3.60% and 1.21%, respectively.
- After optimization, the total mass of the structure was reduced from 52,500 t to 49,830 t, a reduction of 5.09%; the top horizontal displacement was reduced from 0.295 m to 0.275 m, a reduction of approximately 6.78%; at the same time, the inter-story drift angles of each floor met the set code limits after optimization, effectively improving the problem of insufficient stiffness in some floors.
- The improved GA algorithm exhibits better comprehensive optimization performance. Through 30 independent runs comparing the improved GA, traditional GA, PSO, ACO, and DE, the results show that the average optimization time of the improved GA algorithm is 1232 s, the average value of the optimal objective function is 9.23×10^6 , the worst solution deviation rate is 2.13%, and the fitness standard deviation is 0.016×10^6 . Its overall performance is superior to other comparative algorithms, indicating that the differentiated penalty function can effectively improve the search stability and constraint handling capability in the optimization process of multi-constraint structures.
- The optimized component cross-sectional dimensions generally decrease with the building height. Specifically, the cross-sectional width of column components gradually decreases from 0.65 m to 0.45 m in different height regions, and that of beam components gradually decreases from 0.45 m to 0.30 m. This achieves a reasonable allocation of materials while meeting structural performance constraints.
- Research limitations and future work. Current research primarily focuses on numerical optimization analysis based on typical super high-rise building structural examples. Although 30 independent runs and statistical evaluations have reduced the randomness in comparing stochastic optimization algorithms, existing validations are still mainly concentrated on specific building heights, structural systems, design parameters, and wind environment conditions, and are insufficient to comprehensively reflect the generalization performance of the proposed method under different engineering scenarios. Furthermore, the current wind load is handled using an equivalent static method, which does not fully consider the time-varying stochastic characteristics of actual wind fields. Future research will combine wind tunnel tests, field monitoring data, and more refined wind load time-history simulations to further validate the top horizontal displacement, inter-story drift angle, and wind-induced vibration response, and expand the applicability of the proposed method to different building heights, structural systems, and complex wind environments.

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Experimental investigation on the structural performance of reinforced concrete deep beams incorporating recycled concrete aggregates

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Abstract

This investigation focuses on exploring the feasibility of using recycled concrete aggregates mix in concrete structures. The objective of the study is to examine the tensile, compressive, and flexural strengths of recycled aggregate concrete, as well as performance of reinforced RAC deep beams by evaluating critical parameters including aggregate replacement ratio, water-to-cement ratio, presence of structural web openings, and layered casting configurations. Concrete incorporating recycled concrete aggregates mix with replacement percentages of (0, 25, 50, 75, and 100%) was utilized and tested after 28 days of curing. The study evaluated 13 reinforced concrete deep beams categorized into solid, with web openings, and layered configurations. Overall, it was observed that an increase in the percentage of recycled aggregates led to a reduction in the compressive, tensile, and flexural strengths of the concrete. Nevertheless, the application of RAC as aggregate concrete can be supported by adjusting the replacement percentage of recycled concrete aggregates and the water-to-cement ratio to achieve the desired performance of reinforced concrete structures. A 25% RCA replacement ratio minimizes the compressive strength loss to just 3.48%.

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1. Introduction

The environmental repercussions of producing the raw materials for concrete (cement, coarse and fine aggregates), are significant [1,2]. The measure of this issue necessitates the exploration of alternative raw material sources to decrease energy consumption and the use of natural resources, ultimately leading to a more sustainable concrete. One common approach to achieving an eco-friendlier concrete is the crushing of concrete to create coarse aggregate for new concrete production. This method reduces the demand for natural resources and minimizes the landfill space required for waste concrete. While there are no current assessments of recyclable waste volumes in Iraq, the Ministry of Environment estimated that in 2010 and 2011, about 58% of the total waste disposed of in municipal dumps was construction debris. A large part of this waste consists of concrete and building rubble. It is crucial to investigate the mechanical properties and durability characteristics of Recycled Aggregate Concrete (RAC) to ensure the appropriate use of recycled materials. Many studies have examined the mechanical and durability properties of RAC [3,4,5]. Findings indicate that the mechanical characteristics are contingent upon the characteristics of the recycled aggregate used to produce the concrete, and the percentage of coarse aggregates replaced in the new concrete [6]. For example, with a 100% replacement of recycled aggregates, Deng, X., & Zhang, H. [7] reported that the compressive strength decreased by 9%, while

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El-Sayed, T. A., & Metwally, I. M. [8] reported a 45% decrease. Likewise, Kakar, A., & Gupta, P [9] assessed a 15% reduction in the modulus of elasticity of RAC, while Khan, M. S., & Ali, M. [10] noted a decrease of up to 40% at a higher water-cement ratio of 0.75, with an insignificant difference at a lower ratio of 0.55. Additionally, Li, J., & Zhang, Y. [11] proposed equations for the modulus of elasticity that suggest an average decrease of 15% for Normal Aggregate Concrete (NAC) and RAC with similar cylinder compressive strength. The structural application of Recycled Concrete Aggregate (RCA) requires a thorough understanding of its fresh and hardened properties. Incorporating RCA typically alters the concrete matrix due to its inherent physical characteristics, such as higher water absorption and lower density compared to natural aggregate. However, adjusting critical mix proportions, particularly optimizing the water-to-cement ratio, can significantly enhance and control the compressive strength development of recycled aggregate mixtures.

In 2020, Xiao et al. stated that during the mechanical crushing of recycled concrete, very tiny cracks are created, which greatly increase its ability to absorb water and reduce its aggregate bonding strength compared to using natural aggregate [12]. In 2018, Knaack and Kurama stated that the weakness in the mechanical strength of recycled aggregate concrete can be compensated by improving the dosages of chemical additives, but its shrinkage performance remains critical when drying because of the highly porous cohesive paste [13]. In 2022 Ahmed, et. al, they showed that the results indicate adding 50% of recycled concrete aggregate when casting deep reinforced beams gives an acceptable structural shear capacity, even though it significantly increases maximum mid-span deflections and the width of diagonal cracks [14]. In 2023 Liu, F., & Sun, X., they studied the shear behavior of deep beams made from recycled concrete and showed that diagonal crack formation happens at relatively lower load levels due to the weak interfacial transition zone around the old cement paste [15].

The primary objective of this research is to evaluate the feasibility and structural performance of utilizing recycled concrete aggregate (RCA) as a partial or full replacement for conventional coarse aggregate in structural concrete. To achieve this goal, the following specific objectives have been established:

- To assess the influence of RCA on the workability and properties of concrete mix.
- To investigate the effect of RCA replacement ratios on the mechanical properties of concrete, specifically compressive, flexural, and splitting tensile strengths.
- To investigate the shear behavior, cracking patterns, and ultimate load capacity of reinforced concrete deep beams under varied configurations, including beams with structural web openings, solid control beams, and beams including layered RCA casting layouts.
- In addition to experimental objectives, this study aims to highlight the environmental and economic advantages of utilizing RCA as a sustainable construction material.

Advanced configuration approaches are important to optimize the performance of Recycled Concrete Aggregate (RCA) deep beams in structural buildings. Applying a layered casting technique strategically enhances the critical shear span region, effectively mitigating strength loss while maximizing structural sustainability. Conversely, the integration of structural web openings significantly alters internal load transfer mechanisms, making it vital to analyze their direct impact on the beam's ultimate load-carrying capacity.

2. Materials

2.1 Cement

The cement used in this study is Iraqi ordinary Portland cement Type (I). It is stored in airtight plastic containers to avoid exposure to different atmospheric conditions. This cement is tested and checked according to the American Standard Specification (ASTM C150). Tables 1&2 show the physical and chemical cement properties, respectively.

Table 1. Physical properties of type (I) cement (According to ASTM C150)

Physical Property Under Test	Laboratory Test Result	ASTM C150 Type I Limits	Compliance Status
Fineness (Blaine Air Permeability)	2850 cm ² /g	≥ 2600 cm ² /g	Pass
Setting Time (Vicat Apparatus)			
— Initial Setting Time	135 minutes	≥ 45 minutes and ≤ 375 minutes	Pass
Soundness (Autoclave Expansion)	0.12%	≤ 0.80%	Pass
Compressive Strength (Standard Mortar Cubes)			
— 3-Day Hydration Age	18.4 MPa	≥ 12.0 MPa	Pass
— 7-Day Hydration Age	26.8 MPa	≥ 19.0 MPa	Pass
— 28-Day Curing Baseline	44.5 MPa	≥ 28.0 MPa (Optional Target)	Pass

Table 2. Chemical composition of type (I) cement (According to ASTM C150)

Chemical Component / Oxide	Abbreviation	Laboratory Test Result (%)	ASTM C150 Type I Limits	Compliance Status
Lime	CaO	62.45%	No explicit limit	–
Silica	SiO ₂	20.80%	No explicit limit	–
Alumina	Al ₂ O ₃	5.12%	No explicit limit	–
Iron Oxide	Fe ₂ O ₃	3.25%	No explicit limit	–
Magnesia	MgO	2.15%	≤ 6.0%	Pass
Sulfur Trioxide	SO ₃	2.38%	≤ 3.0%	Pass
Loss on Ignition	LOI	1.85%	≤ 3.5%	Pass
Insoluble Residue	IR	0.62%	≤ 0.75%	Pass
Main Mineral Phases (Bogue Calculation)				
Tricalcium Silicate	C ₃ S	51.40%	No explicit limit	–
Dicalcium Silicate	C ₂ S	21.15%	No explicit limit	–
Tricalcium Aluminate	C ₃ A	8.10%	No explicit limit	–
Tetracalcium Aluminoferrite	C ₄ AF	9.88%	No explicit limit	–

2.2 Fine Aggregate

The quantity and grading of fine aggregate have a significant influence on concrete production. Natural fine aggregate from Safwan city (Jabal Senam) was used. Table 3 shows the fine aggregate grading and the limits of the ASTM C33/C33M-18. It has a fineness modulus of 2.68 and a specific gravity of 2.62.

Table 3. Grading of the fine aggregate

No.	Sieve size (mm)	% Passing by weight	
		Fine aggregate (% Passing by weight)	Limits of ASTM C33/C33M-18
1	9.50	100.0	100
2	4.75	98.5	95-100
3	2.36	90.3	80-100
4	1.18	72.55	50-85
5	0.60	52.7	25-60
6	0.30	18.45	5-30
7	0.15	1.3	0-10

2.3 Coarse Aggregate

Two types of coarse aggregate are used:

2.3.1 Gravel

Crushed gravel from Senam hill region in Safwan city with grading of (5 - 20) mm was used. The coarse aggregate grading is shown in Table 4. The obtained results show that; the grading of coarse aggregate is within the requirements of ASTM C33/C33M-18. Table 5 shows the specific gravity, sulphate and chloride content and absorption of the used coarse aggregate.

Table 4. Grading of the coarse aggregate

Sieve size (mm)	Passing by weight (%)	ASTM limit (%)	Within limit?
25.0	100.0	100	Yes
19.0	93.7	90–100	Yes
9.5	22.8	20–55	Yes
4.75	0.10	0–10	Yes
2.36	0.03	0–5	Yes

Table 5. Properties of the coarse aggregate

Physical property	Normal Aggregate		Recycle Aggregate	
	Test result	IOS No. 45/1984 limit	Test result	IOS No. 45/1984 limit
Specific gravity	2.65	—	2.62	—
Sulfate content (SO ₃)	0.062%	≤ 0.1%	0.063	≤ 0.1%
Chloride content (Cl)	0.082%	≤ 0.1%	0.084	≤ 0.1%
Absorption	0.65%	—	0.69%	—
Loose bulk density	1500 kg/m ³	—	1490 kg/m ³	—

2.3.2 Recycled Aggregates

Recycled aggregates refer to granular materials that have been processed from sources such as sand, gravel, rock, or concrete from demolished structures. These aggregates are highly versatile and can be utilized in numerous applications, including the production of concrete and asphalt, as well as in drainage projects.

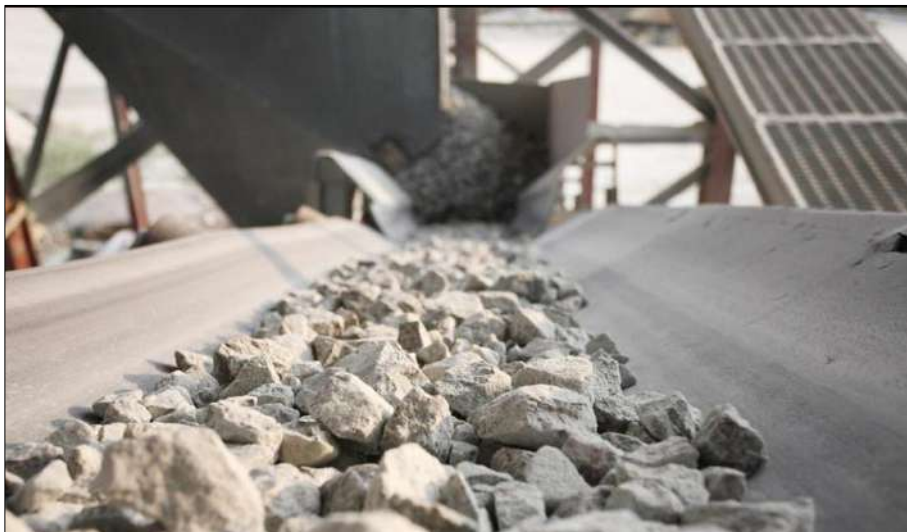


Fig. 1. Recycled aggregate

To enhance the conservation of natural rock resources and support environmental sustainability, recycled aggregates are derived from materials that have been recycled, including concrete, brick, and masonry waste [16]. Figure 1 show the sample of recycled aggregate. The recycled concrete (RCA) was obtained from leftover concrete from previously tested samples, and it was crushed and sieved to reach a maximum aggregate size (MAS) of 20 mm.

Recycled aggregate concrete differs from natural concrete mixes, as it contains hardened cement mortar. The cement mortar on recycled aggregate has higher porosity, water absorption and lower adhesion than natural aggregate. It has negative effects on the mechanical behavior and durability of fresh and hardened recycled aggregate concrete. Therefore, it will enable use the recycled concrete aggregate if the adhered cement mortar can be enhanced. The two main methods for enhancing the properties of recycled aggregate concrete are strengthening, and removing the adhered mortar.

2.4 Water

Regular tap water is applied without any additives for the processes of mixing, casting, and curing. It is necessary to wet the surface of aggregates to improve adhesive quality, as the cement paste adheres more quickly and satisfactorily to a wet surface than to a dry one. Moreover, water is essential for creating a plastic mixture of the various components, which provides workability to the concrete, making it easier to place in the desired position. Ultimately, through its chemical reaction with cement, water plays a vital role in producing the desired properties of the concrete.

2.5 Super Plasticizer

To produce a high strength concrete, super plasticizer is used. One of new materials generation of copolymer-based super plasticizer is the (Sika ViscoCrete Hi-Tech 1316) as shown in Figure 2.



Fig. 2. Super plasticizer



Fig. 3. Specimens of hardened concrete

3. Experimental Work

3.1 Concrete Mix Proportions

The mix proportion was chosen as 1:1.59:2.39 (by weight) cement, sand, gravel respectively and 0.38 water cement ratio, and (1000 g) SP for each 1 m³ concrete, the specific gravity of the polymer (1.08). Table 6 shows recycle aggregate concrete mix proportions in this study depending on recycle aggregate content. Concrete was replaced with absolute volumetric calculations weighted by batches during the design of concrete mixes to adjust the resulting volume and keep the effective cement volume per cubic meter constant.

3.2 Specimen Fabrication and Curing Conditions

All the laboratory specimens for the hardened state tests were cast in molds with manual compaction. According to the standard specification ASTM C192, compaction was carried out in

three equal layers using a standard 16 mm diameter tamping rod with 25 strokes per layer. All samples are demolded after 24 hours, marked and cured in water until testing age. Figure 3 shows the specimens that were cast and tested in this study to assessment the hardened concrete. cubes (150 mm), cylinders (150x300 mm), and prisms (150x150x500 mm) tested at 28 days.

Table 6. Concrete mix proportions

#	Coarse Aggregat e Type	Total Volum e (m ³)	Mixing Proportions					S.P. (ml)	Slum p test (mm)
			Cemen t (Kg)	Sand (Kg)	Aggregat e (Kg)	Water (kg)	Water/Ceme nt Ratio		
First Mix	Normal 100 %	1	447.8 1	712.0 0	1070.00	170.0 0	0.38	100 0	163
Secon d Mix	Recycle 25 %	1	447.8 1	712.0 0	R 267.5 N 802.5	176.2 5	0.39	100 0	165
Third Mix	Recycle 50 %	1	447.8 1	712.0 0	R 535 N 535	180.7 7	0.40	100 0	168
Fourt h Mix	Recycle 75 %	1	447.8 1	712.0 0	R 802.5 N 267.5	188.5 9	0.42	100 0	170
Fifth Mix	Recycle 100 %	1	447.8 1	712.0 0	1070.00	195.0 0	0.44	100 0	170

Note: R: Recycled Concrete Aggregate; N: Natural Coarse Aggregate (Gravel).

3.3 Compressive Strength Test

Standard 150 mm cubes were used according to BS EN 12390-3:2019 and they are opened after one day of casting. Testing was carried out at 28 days. The machine which was used in the tests is one of the hydraulic types of 2000kN capacity as shown in Figure 4. Compressive strength (MPa) is obtained by averaging of the results of three specimens according to the test results. The rate of loading of 0.6 MPa/s (13.5 kN/s) was applied automatically until failure.



Fig. 4. Cube compression test of hardened concrete

3.4 Flexural Strength Test

To calculate the flexural strength of concrete, the prism with ASTM C78 standard parameters are explicitly governed by third-point loading configurations rather than center single-point configurations. The test samples measured 150x150x500 mm. In the same conditions of the specimens of compressive strength, the flexural strength samples were tested. A “third-point loading” mechanism and support details for the 150x150x500 mm prism specimen in accordance with the ASTM C78 standard was used, as shown in Figure 5. Flexural strength (MPa) is obtained from testing. constant loading rate of 1.0 MPa/min (0.15 kN/s) was applied.

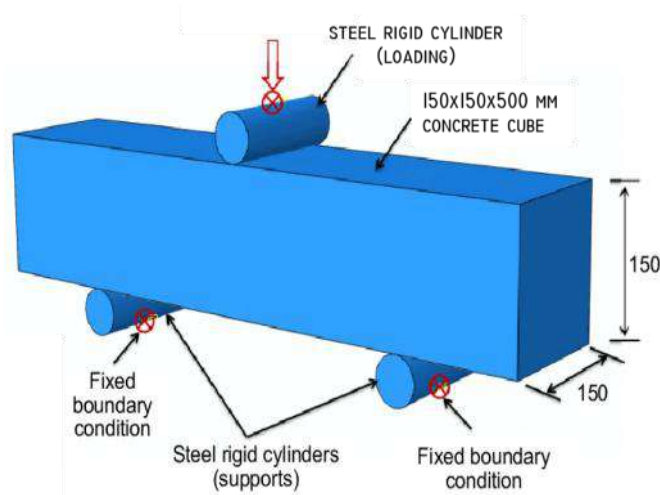


Fig. 5. Flexural strength test of hardened concrete

3.5 Splitting Tensile Strength Test

The splitting tensile strength is determined at 28 days on cylinders with ($h=300\text{mm}$, $d=150\text{ mm}$). The test was carried out in accordance with ASTM C496/C496 - 17. The cylinders are cast and cured at the same conditions for the specimens of the compressive strength tests. The test is carried out by placing a cylinder specimen horizontally between the loading surfaces of a compression testing machine and the load was applied until failure of the cylinder as shown in Figure 6 then the splitting tensile strength is obtained from testing. A loading rate of 1.2 MPa/min was executed. plywood bearing strips were used between the cylinder and the loading platens to prevent local crushing and stress concentrations. The mechanical calculation formula is:

$$f_{ct} = 2P / (\pi * L * d) \quad (1)$$

where: P is the ultimate failure load, L is the cylinder length (300 mm), and d is the diameter (150 mm).



Fig. 6. Splitting tensile strength test of hardened concrete

3.6 Beam Specimens and Details

The investigational program consists of casting and testing of thirteen, simply supported reinforced concrete beams. The beams were designed to fail in shear with different RCA amount. Five mixes were prepared with different RCA replacement ratios (0%, 25%, 50%, 75% and 100 %). Two deep beams were prepared to represent the control beam, which they were with normal aggregate (Normal aggregate 100 % "N-100%" and normal aggregate with opening "N-100% + O"). Four deep beams were prepared with different percentage of RCA (R-25%, R-50%, R-75% and R-100%), without opening. Four deep beams were prepared with different percentage of RCA (R-25%+O, R-50%+O, R-75%+O and R-100%+O), with opening.

Table 7. Deep beam details

Deep beam description	Deep beam Name	Note
Normal aggregate without opening	N-100 %	Control
Normal aggregate with opening	N-100 % + O	Control
Mix Normal aggregate and recycle aggregate ration 25% without opening	Mix-R-25 %	-
Mix Normal aggregate and recycle aggregate ration 25% with opening	Mix-R-25 % + O	-
Mix Normal aggregate and recycle aggregate ration 50% without opening	Mix-R-50 %	-
Mix Normal aggregate and recycle aggregate ration 50% with opening	Mix-R-50 % + O	-
Mix Normal aggregate and recycle aggregate ration 75% without opening	Mix-R-75 %	-
Mix Normal aggregate and recycle aggregate ration 75% with opening	Mix-R-75 % + O	-
Recycle aggregate 100% without opening	R-100 %	-
Recycle aggregate 100% with opening	R-100 % + O	-
Normal aggregate with recycle aggregate ration 25% as layer	L-R-25 %	-
Normal aggregate with recycle aggregate ration 50% as layer	L-R-50 %	-
Normal aggregate with recycle aggregate ration 75% as layer	L-R-75 %	-

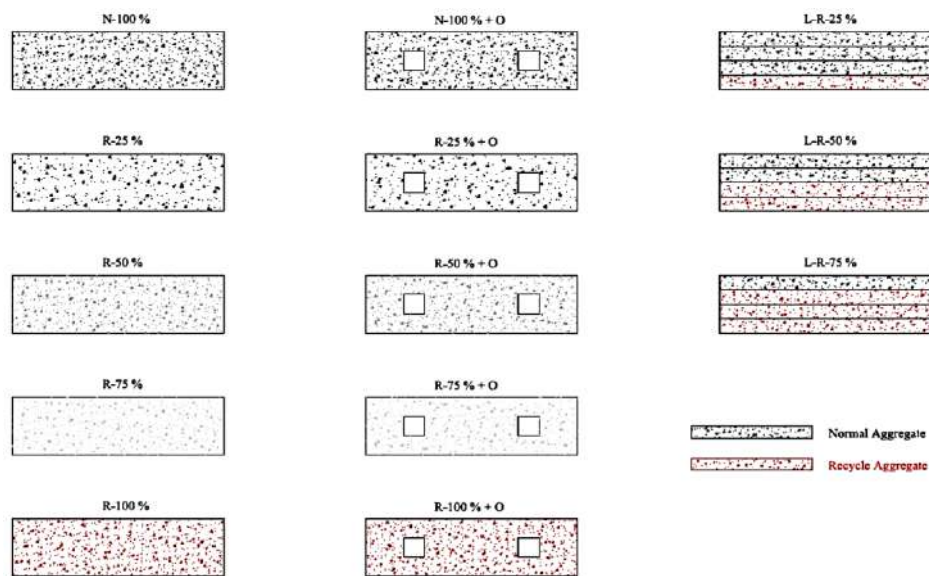


Fig. 7. Details of deep beams

Three deep beams were prepared with putting the RCA in layers (L-R-25%, L-R-50% and L-R-75%). All web openings were square, 100x100 mm, centered vertically within the beam depth, and positioned exactly at 350 mm from supports. The shear height (a) is 350 mm and the effective depth (d) is 310 mm, so the a/d ratio is constant at 1.13, which scientifically confirms its classification as deep beams ($a/d < 2.0$), also there wasn't any extra reinforcement around the openings. All beams above are shown in Table 7 and Figure 7.

3.7 Molds Preparation

Molds of wood were used for the beam's fabrication. The Molds are made of 18 mm thickness plywood; bolts were used to connect the different parts of the Molds. No additional reinforcement steel was placed around the corners of the opening to specifically isolate and secure the effect of raw RCA deterioration on shear capacity. Mineral oil was applied to the inner surfaces of the

wooden panels, and all structural joints were sealed with waterproof silicone to prevent leaks. Details of the reinforcement and molds are illustrated in Figure 8a & b respectively.

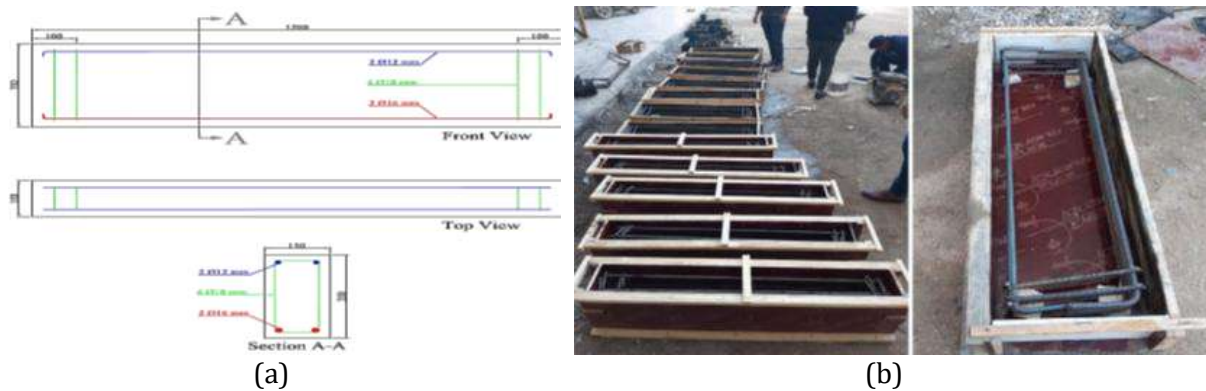


Fig. 8. (a) and (b). Deep beam details

3.8 Casting and Curing of RCA Beams

The casting was done by using a 0.27 m³ drum mixer. All materials were prepared by weight as shown in figure 9. The cast was started with the second mix R-100% so as to start with the layers that is designed to be at the bottom of the deep beam, then the first mix N-100%, R-25%, R-50% and R-75% respectively.



Fig. 9. Materials preparation



Fig. 10. Casting with R-100% at layers

The beams were treated using covers made of burlap cloth that were kept constantly wet for 28 days at an ambient temperature of $25 \pm 2^\circ\text{C}$. Where, 'L-R-25%' means that 100% of the RCA concrete was poured exclusively in the bottom layer, which makes up about 25% of the total beam height (87.5 mm), while the remaining top area, 75%, was poured with normal concrete (N-100%). The casting is shown in figure 10. For a continuous pour, the upper layer placed within 30 minutes of the lower layer to avoid a weak seam.

3.9 Testing of RCA Beams

Before doing the tests, all beams faces were white painted to enable tracing the cracks. The typical procedure, experimental setup, and loading configuration are shown in Figure 11a&b. Where,

Figure 11 (a) show simulation loading configuration, and (b) actual test configuration.' To apply the testing loads a 2000kN capacity Universal Testing Machine (UTM), was used. Mid-span deflection, the first inclined cracking, and ultimate loads were recorded for each beam. The target plate was extended horizontally under the beam, and the spring-loaded plunger in the disc gauge pushes upward against this plate. When the beam experiences a brittle collapse, the beam and the attached plate fall faster than the piston spring's rebound speed, or move sideways. The piston immediately loses contact with the plate. The beam hits the lab's heavy supports below, preventing damage from the failure.

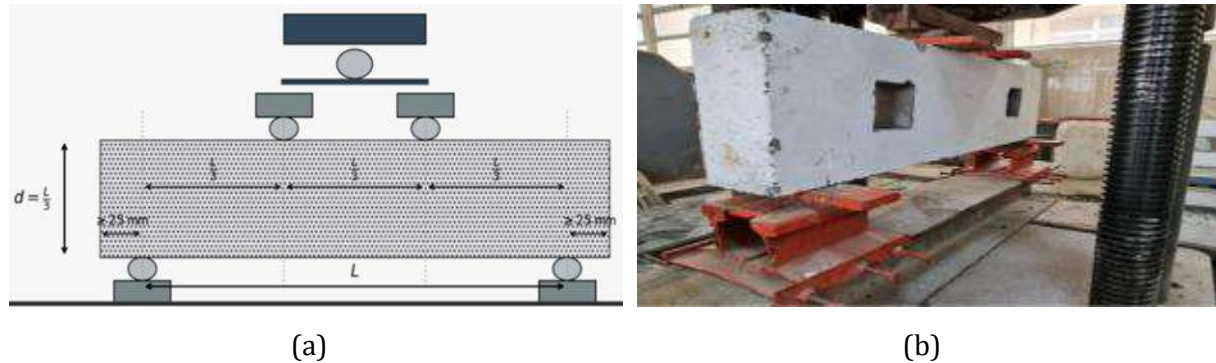


Fig. 11. (a) and (b). Testing deep beam

4. Test Results and Discussion

4.1 Properties of Fresh Concrete

In this study, to investigate the effect of adding recycle aggregate on the concrete workability the amount of superplasticizer in the concrete mixtures was kept constant even when recycle aggregate was added with different quantities, but the amount of water was changed from mix to other. Table 8 and shows the results of the slump test of concrete for all studied mixes. This examination was conducted according to the ASTM C143/C143M – 15a.

Table 8. Slump test results

Mix	Coarse Aggregate Percent	Water Amount	Slump Test Result (mm)
First Mix	N – 100%	170.00	163
Second Mix	R – 25%	176.25	165
Third Mix	R – 50%	180.77	168
Fourth Mix	R – 75%	188.59	170
Fifth Mix	R – 100%	195.0	170

The significant water absorption characteristics of RCA can create challenges. If the RCA is in a dry state, it will draw water from the paste, thereby diminishing the workability of the fresh concrete [17]. This also causes a reduction in the effective water-to-cement (w/c) ratio within the paste. Therefore, it is essential to observe the fluctuations in the w/c ratio for each mix in relation to the quantity of RCA utilized to maintain adequate workability.

4.2 Compressive Strength

In order to examine the effect of incorporating recycled aggregate on the compressive strength of conventional concrete, standard 150 mm cubes were tested in accordance with BS EN 12390-3:2019 at 28 days of age. The results of the concrete compressive strength for all examined mixes are presented in Table 9 and Figure 12. Table 10 details the extent of the reduction in compressive strength resulting from the addition of recycled aggregate to the concrete, where the strength loss to the porous old adhered mortar, high water absorption, and the weak microstructure of the Interface Transition Zone (ITZ).

Table 9. Compressive strength results

Mix	Coarse Aggregate Percent	Compressive Strength at 28 Days (MPa)
First Mix	N – 100%	38.80
Second Mix	R – 25%	37.45
Third Mix	R – 50%	36.59
Fourth Mix	R – 75%	34.69
Fifth Mix	R – 100%	32.54

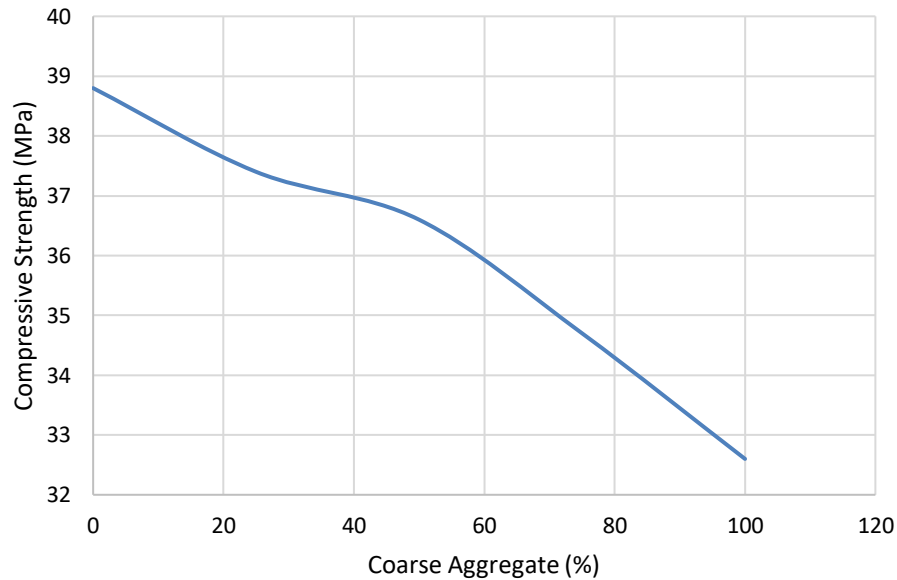


Fig. 12. Compressive strength results

Table 10. Rate of compressive strength decreasing due to adding recycled aggregate

Mix	Coarse Aggregate Percent	Compressive Strength Decreasing (%)
First Mix	N-100%	0.00
Second Mix	R-25%	3.48
Third Mix	R-50%	5.70
Fourth Mix	R-75%	10.59
Fifth Mix	R-100%	16.13

From the results presented above, it can be inferred that the compressive strength of concrete reduces as the volume ratio of recycled aggregate increases. The compressive strengths of concrete containing recycled aggregate were found to be lower than that of conventional concrete by rates of 3.8%, 5.7%, 10.59%, and 16.13%, respectively. The results indicate that incorporating RCA at a level of 100% leads to a more significant reduction in concrete compressive strength compared to other types. This result can be attributed to the stiffness associated with this particular type of recycled aggregate. It can be inferred from the results that the effect of recycled aggregate on decreasing concrete compressive strength intensifies as the volumetric recycled content reaches 100%.

4.3 Flexural Strength

The process of mixing recycled concrete aggregate (RCA) with natural aggregate clearly changes the distribution of internal stresses and the failure configuration in deep beams. According to materials science concepts, it's not just the stiffness of the aggregate that is the main factor for the deterioration of the internal structure, but also the porous nature of the leftover mortar sticking to the surface of the old aggregate and its high ability to absorb large amounts of water, as well as the relatively weak microstructure in the secondary transition zone (ITZ) formed between the new cement paste and the recycled aggregate [12]. As a result of the connection between the material

and the geometry around the openings, the hairline cracks that combine shear and bending spread directly through the weak and porous mortar and the breakable layer between the aggregate particles instead of bending around the aggregate boundaries. This significantly reduces the bonding capability between the aggregate particles and exposes the radial concrete support to early deterioration, leading to sudden and brittle crushing along the compression path [13]. The results regarding flexural strength (modulus of rupture) are displayed in Table 11 and illustrated in Figure 13. Table 12 details the reduction rate of flexural strength resulting from the incorporation of recycled aggregate into the concrete across all examined mixes.

Table 11. Flexural strength results

Mix	Coarse Aggregate Percent	Flexural Strength at 28 Days (MPa)
First Mix	N-100%	14.74
Second Mix	R-25%	13.62
Third Mix	R-50%	13.53
Fourth Mix	R-75%	13.26
Fifth Mix	R-100%	12.50

Table 12. Rate of flexural strength decreasing due to adding recycle aggregate

Mix	Coarse Aggregate Percent	Flexural Strength Decreasing (%)
First Mix	N-100%	0.0
Second Mix	R-25%	7.6
Third Mix	R-50%	8.21
Fourth Mix	R-75%	10.04
Fifth Mix	R-100%	15.20

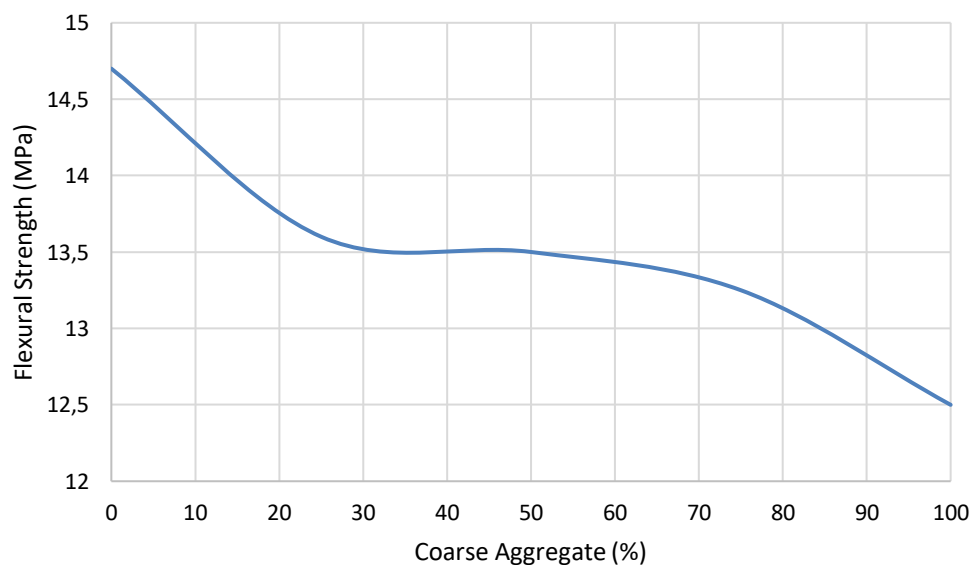


Fig. 13. Flexural strength results

4.4 Cracking and Ultimate Applied Loads of Deep Beams

First inclined cracking and ultimate loads are recorded in Table 13. The first inclined cracking load is defined as the load at which the first visible inclined crack appears in the shear span, the ultimate loads refer to failure loads, furthermore, the development of these diagonal cracks directly reflects the diagonal splitting tension performance in deep beam shear spans [17]. Figure 14 shows some beams at first and ultimate loads stages.

Table 13. First cracking and ultimate loads

Beam Designation	P_{crack} (kN)	P_{ultimate} (kN)	Δ_{ultimate} (mm)
N-100%+O	120	470	3.99
Mix-R-25%+O	100	440	4.55
Mix-R-50%+O	90	390	5.12
Mix-R-75%+O	75	330	5.74
R-100%+O	70	300	6.62
N-100%+O	55	200	6.20
Mix-R-25+O	50	190	6.52
Mix-R-50+O	45	180	7.19
Mix-R-75+O	40	170	9.07
R-100%+O	40	160	11.21
L-R-25%+O	110	460	4.32
L-R-50%+O	100	420	4.95
L-R-75%+O	80	360	5.38



Fig. 14. Crack patterns and failure profiles of layered RAC deep beams at ultimate load stage

4.5 Effect of RCA on Deep Beams Behavior

The difference in the percentage of RCA ratios significantly affected the first cracking and ultimate failure loads. Both cracking and ultimate loads decrease with the RCA ratio increase. The higher reduction in cracking and ultimate loads are corresponding to the higher RCA content either the present of openings. The behavior of bending and the maximum load-bearing capacity of the deep beams being tested is controlled by arch movement and should be assessed within the strut-and-tie model (STM) framework. In this system, the internal drift of forces is represented as a truss mechanism, where a partial concrete strut (compression portion) transfers the applied loads directly to the strut, supported by a longitudinal steel tie (tension portion) at the bottom chord. Adding recycled concrete aggregate (RCA) significantly reduces the compression efficiency of this partial strut due to the presence of weakly bonded old mortar, the porous nature, and the weak secondary transition zone (ITZ) [18]. Beams with 0% of RCA exhibited higher ultimate load than RCA 25%, 50%, 75%, and 100%. Table 14 shows the comparison of first, ultimate loads and failure mode in the first group specimens.

Table 14. Ultimate load, first crack, deflection and failure mode of deep beam

Beam Designation	P _{Crack} (KN)	P _{Ultimate} (KN)	$\Delta_{ultimate}$ (mm)	P _{Cracking} /P _(N 100%+O)	P _{Ultimate} /P _(N 100%+O)	P _{Cracking} /P _{Ultimate}	Failure Mode
N-100%	120	470	3.99	1.00	1.00	1.00	Shear
Mix-R-25%	100	440	4.55	0.83	0.94	0.89	Shear
Mix-R-50%	90	390	5.12	0.75	0.83	0.90	Shear
Mix-R-75%	75	330	5.74	0.63	0.70	0.89	Shear
R-100%	70	300	6.62	0.58	0.64	0.91	Shear

On the other hand, the Tables 15&16, show the comparison of first, ultimate loads and failure mode in the second and third specimen groups, respectively.

Table 15. Ultimate load, first crack, deflection and failure mode of deep beam

Beam Designation	P _{Crack} (KN)	P _{Ultimate} (KN)	$\Delta_{ultimate}$ (mm)	P _{Crack} /P _{Crack(N 100%+O)}	P _{Ultimate} /P _{Ultimate(N 100%+O)}	P _{Cracking} /P _{Ultimate}	Failure Mode
N-100%+O	55	200	6.2	1.00	1.00	1.00	Shear
Mix-R-25%+O	50	190	6.52	0.91	0.95	0.96	Shear
Mix-R-50%+O	45	180	7.19	0.82	0.90	0.91	Shear
Mix-R-75%+O	40	170	9.07	0.73	0.85	0.86	Shear
R-100%+O	40	160	11.2	0.73	0.80	0.82	Shear

Table 16. Ultimate load, first crack, deflection and failure mode of deep beam

Beam Designation	P _{Crack} (KN)	P _{Ultimate} (KN)	$\Delta_{ultimate}$ (mm)	P _{Crack} /P _{Crack(N 100%+O)}	P _{Ultimate} /P _{Ultimate(N 100%+O)}	P _{Cracking} /P _{Ultimate}	Failure Mode
N-100%	120	470	3.99	1.00	1.00	1.00	Shear
L-R-25%	110	460	4.55	0.90	0.97	0.94	Shear
L-R-50%	100	420	5.12	0.83	0.90	0.90	Shear
L-R-75%	80	360	5.74	0.66	0.80	0.86	Shear

The previous results presented in Tables 14, 15 and 16 offer a comprehensive indication of how Recycled Concrete Aggregate (RCA) replacement ratios and the structural web openings, influence the structural behavior of reinforced concrete deep beams. Where the analysis shows three main specimen groups their cracking, ultimate loads, deflection, and failure modes.

The increase of (RCA) will reduce the first cracking and ultimate loads, but increase the mid span deflection. Also, the structural web opening will reduce the first cracking and ultimate loads.

5. Conclusions

The most important conclusions that can be drawn from the present study are the following:

- The most important conclusion observed in this study when replacing recycled concrete aggregate as an alternative to normal aggregate is that using recycled concrete aggregate in layers is better than mixing recycled concrete aggregate with normal aggregate.
- For all replacement ratios of recycled concrete aggregate as an alternative to normal aggregate in concrete, the concrete compressive, flexural, and splitting tensile strengths decrease with the increase of replacement ratio, and this decrease in the strengths is small.
- The quality and mechanical properties of the recycled concrete aggregate are relatively good when compared to normal aggregate.
- To decrease the cost of concrete, we can replace normal aggregate by an RCA ratio of 25%, due to its minimum effect as observed in this study where the decrease in compressive strength, and flexural strength ratio was 3.48%, and 7.6%, respectively.
- The RCA has water absorption much more than normal aggregate, which has a big effect on workability. So, for the purpose of using RCA, we must adjust the W/C ratio or wet the RCA before using it in the mixers.

- The presence of web openings reduces the ultimate load-carrying capacity of the beam element by nearly 50%. So, if the presence of openings is necessary, we can change the mix design to a high-strength mix design.
- The diagonal tension cracks emerging from the opening corners propagate at a radically accelerated rate through the porous RCA paste. The micro-cracks integral to the old adhered mortar easily coalesce with the macro-cracks induced by the geometric opening, causing a rapid reduction in the aggregate interlocking apparatus.

6. Recommendations

- It's recommended to use recycled concrete aggregate (RCA) in prearranged layers instead of mixing it directly with regular aggregate, because a layered configuration yields better structural performance.
- It is recommended to use a 25% replacement rate of recycled aggregate because it gives strong and eco-friendly concrete.

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Seismic response assessment of RC frame buildings with masonry infill openings and soft story irregularity under dynamic soil–structure interaction

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Abstract

Masonry infill walls are widely used in reinforced concrete (RC) buildings, but their seismic behavior is often studied using fixed-base models without fully considering soil–structure interaction. This study investigates the seismic response of a ten-story, one-bay RC frame with 0%, 30%, and 50% infill openings, with and without a ground soft story, supported on four soil classes: soft soil, stiff soil, soft rock, and hard rock. The frame and infill walls were modeled in SAP2000 using equivalent diagonal compression struts, with reduced strut widths for openings. Soil–structure interaction was represented using a three-dimensional linear-elastic soil model directly connected to the isolated footings. Linear time-history analysis was carried out using the 2001 Bhuj earthquake record, scaled to a PGA of 0.1 g. The results show that soil conditions strongly affect the seismic response. The highest roof acceleration, about 2.7 g, occurred for regular infilled frames on stiff soil, while the largest roof displacement, about 241 mm, occurred for the fully infilled frame on soft soil. Soft-story frames showed higher ground-story drift, reaching about 1.2% on soft soil. Overall, the critical response varied with soil condition, indicating that fixed-base analysis may not always identify the governing case.

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1. Introduction

An earthquake releases stored energy along a fault, sending seismic waves to the surface that shake the ground, and any building on it. The building's inertia resists that motion, generating the internal forces engineers design against. Shaking severity depends on magnitude, distance, and, critically, soil type: soft soils amplify certain frequencies far more than rock, which is the core reason soil–structure interaction (SSI) matters. The 2001 Bhuj earthquake, used as the input record here, is a well-documented example of a high-magnitude event whose damage was worsened by local soil amplification. For buildings, severe inter-story drift can crack masonry infill, damage non-structural elements, and trigger a soft-story collapse mechanism, whereas one under-braced story absorbs nearly all the deformation, a pattern well documented in post-earthquake surveys [1-3]. Infill walls are usually treated as non-structural in design, even though they add real stiffness [4]. This is a limitation, since infill stiffens the frame and shortens its period substantially compared to a bare frame. The standard way to capture this without full finite element meshing is the equivalent diagonal strut, sized from the relative stiffness of panel and frame [4-6], refined recently through

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cyclic testing [7, 8] and machine-learning-based prediction [9]. Openings reduce panel stiffness unevenly, and when irregular, can concentrate damage and help trigger soft-story behavior [1-3], in combination with other measures such as decoupled infill systems or buckling-restrained braces [10, 11]. Accordingly, the present study investigates bare, fully infilled, and 30%/50%-opening RC frames, each with and without a soft story, under different soil conditions considering dynamic soil-structure interaction.

1.1 Soil-Structure Interaction (SSI)

Conventional design assumes a rigid ("fixed") base. Real soil is flexible, so the building and soil respond as one coupled system [12]. Two mechanisms matter: kinematic interaction, where the foundation cannot perfectly track free-field ground motion, and inertial interaction, where the vibrating structure pushes back into the soil. Both lengthen the effective period and add radiation damping [12, 13]. Whether that helps or hurts depends on how the new period compares to the site's dominant period: a close match means resonance and amplification [14, 15]. The behavior/response-modification coefficient itself has also been shown to be sensitive to inertial soil-structure interaction [16], compounding the sensitivity to soil stiffness and building irregularity documented elsewhere [13, 17-23]. Findings across these studies remain fragmented [24], particularly for infilled frames whose stiffness, and therefore period, changes with infill openings and soft-story irregularity, which motivates the present paper.

1.2 Review of Related Literature

Beyond the strut-modeling and SSI studies discussed above, several recent works examine how infill layout, openings and soil flexibility interact. Amritha and Prasanth [25] modeled six- and eleven-story RC buildings with 0-40% infill openings together with the soil in a three-dimensional finite element model and found that soft soil increased roof displacement by up to about 21% and inter-story drift well beyond the fixed-base values, although soft-story irregularity was not varied. Messaoudi et al. [26] examined the effect of infill position and opening configuration on the seismic response of fixed-base RC frames, showing that both the location and the size of openings strongly influence story shear distribution and that removing infill from a story can raise its drift by up to 264%. Dias-Oliveira et al. [27] reviewed infilled-frame seismic behavior and highlighted that panel-frame interaction effects, beam-column joint detailing, and sliding shear failure of columns are frequently omitted from simplified infill models, leading to unconservative predictions. Zhang et al. [28] tested half-height infilled frames experimentally and found that the wall-to-column height ratio governs whether the frame column fails in bending or in shear, with the masonry material also affecting the degree of stiffness increase. Gamal et al. [29] studied the interaction between infill irregularity and dynamic response through modeling full-contact, gap and asymmetric infill configurations in moment resisting frames. They found that asymmetric infill placement induces torsional demands that are often ignored in design practice. Manojlovski et al. [30] presented a sliding flexible connection between infill panels and the bounding frame as a resilience method, which exhibited reduced damage to the panel-frame interaction compared to conventional tight-fit infill. Closer to the present scope, Karki et al. [31] evaluated a masonry-infilled RC frame under both fixed-base and soil-flexible conditions using the site classification of the revised Nepal building code, and found that soil-structure interaction altered the fundamental period, peak floor displacement, and inter-story drift ratio relative to the fixed-base case, though their study did not vary the infill opening ratio or soft-story configuration. Table I compares the most closely related studies in terms of structural model, infill modeling, opening ratios, soft-story conditions, soil classes and SSI modeling, and seismic input. Three limitations recur. First, studies that model openings and soft stories in detail [1, 26] assume a fixed base. Second, studies that model the soil explicitly [15, 25] consider a single soft deposit or two sand types, and do not vary soft-story presence together with the opening ratio. Third, the study that spans several code soil classes [31] uses static procedures, which cannot reproduce the frequency-dependent site and SSI amplification that controls floor accelerations. None of these studies combines infill opening ratio, soft-story irregularity and soil classes ranging from soft soil to hard rock within a single time-history framework with an explicit soil continuum, which is the gap addressed in this paper.

Table 1. Comparison of the present study with closely related studies

Study	Structure	Infill model	Openings	Soft story	Base / soil and SSI model	Analysis and input
Furtado et al. [1]	4-story RC frame	Double strut (macro-model)	16–65%	Yes	Fixed base	Modal, pushover, nonlinear dynamic (6 records)
Messaoudi et al. [26]	6-story, 3-bay RC frame	Single diagonal strut	10–90% (reduction factors)	Yes (15 layouts)	Fixed base	Numerical (OpenSees)
Amritha and Prasanth [25]	6- and 11-story, 3 × 3 bays	Plate micro-model and single strut	0–40%	Not varied	Two sands; direct 3D soil, hardening-soil model	Nonlinear time-history, Kobe 1995 scaled to 0.36 g
Santa-Ana-Lozada et al. [15]	Two irregular 10-story buildings	Infill in 3D FE model	Not varied	Vertical irregularity	Soft soil (EC8 type C); direct 3D nonlinear soil, viscous boundaries	Spectral, pushover, nonlinear time-history (42 records)
Karki et al. [31]	Masonry-infilled RC frame	Diagonal strut	Not varied	Not varied	NNBC 105:2020 soil types; flexible base	Linear static, pushover
Present study	10-story, one-bay RC frame	Diagonal strut with opening reduction	0, 30, 50%	With and without	Four classes (Vs = 175–2500 m/s); direct 3D linear-elastic soil	Linear time-history, Bhuj 2001

1.3 Choice of Analysis Method

Seismic demand can be estimated by equivalent static, response spectrum, nonlinear static (pushover) or time-history analysis [32]. Static and response spectrum methods rely on fixed-base design spectra and modal combination, so they cannot represent the frequency-dependent interaction between a soil column and the structure. Pushover analysis describes capacity and failure mechanisms but not dynamic amplification. Nonlinear time-history analysis is the most complete method, but it requires calibrated hysteretic models for the infill, the frame and the soil, and becomes very demanding for a three-dimensional soil continuum repeated for 7 configurations and 4 soil classes. Linear time-history analysis was therefore selected. It retains the full frequency content of the recorded motion and the coupled soil-structure dynamics, namely period lengthening, site amplification and radiation damping [33, 34], which are the effects under study, while keeping the parametric study tractable. Its limitations are that it cannot capture infill cracking, strut degradation, concrete yielding, soil nonlinearity or foundation uplift and sliding. The results are therefore elastic demand indicators, suitable for comparing configurations and soil classes, rather than predictions of damage.

1.4 Present Study

Building on this gap, the present study jointly varies infill opening percentage, soft-story irregularity and soil class within a single modeling framework, which prior studies [15, 25–31] have not done simultaneously. A one-bay, ten-story RC frame with seven configurations (bare frame; full infill, 30% and 50% openings, each with and without a ground soft story) is analyzed on

four IBC 2006 [35] site classes and on a fixed base, using the 2001 Bhuj earthquake record scaled to a PGA of 0.1 g. In this paper, the term “vulnerability ranking” is used only in a response-based sense: configurations are ranked by their peak roof displacement, peak floor acceleration and peak inter-story drift ratio (IDR). No damage states, fragility curves or probabilistic assessment are derived.

Within this definition, and for the single record and linear-elastic model adopted, the study examines whether the soil class changes which configuration produces the largest response compared with the fixed-base case. Whether such changes hold for other records and for nonlinear behavior remains to be confirmed.

2. Methodology

The structural models were developed and analyzed using SAP2000 Version 21. The RC beams and columns were modeled as three-dimensional frame elements, and rigid diaphragm constraints were assigned to the floor slabs to represent uniform movement within each floor. The masonry infill walls were represented using equivalent diagonal compression struts. The width of each strut was calculated based on the contact between the infill wall and the surrounding frame, as explained in Section 2.3.

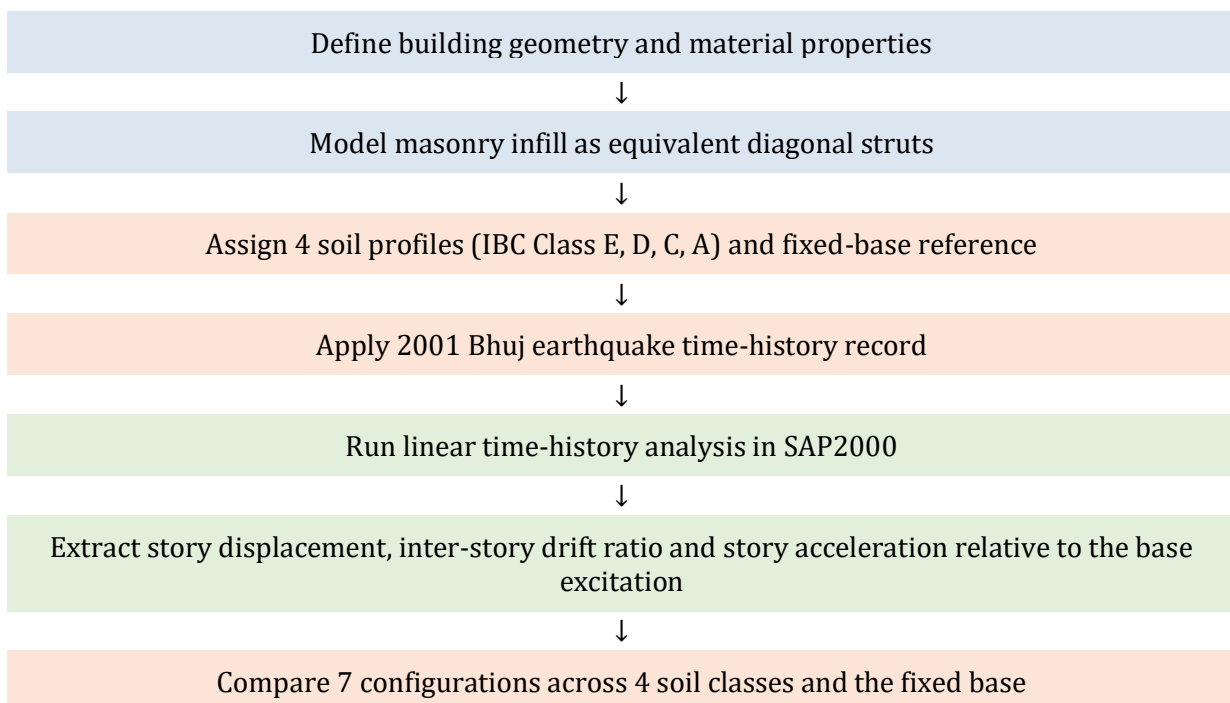


Fig. 1. Modeling and analysis workflow overview

The foundations were modeled as isolated pad footings placed 2 m below the ground surface. To study the effect of soil–structure interaction (SSI), four soil conditions were considered: soft soil, stiff soil, soft rock, and hard rock. All seven structural configurations were analyzed first with fixed supports and then with the four different soil conditions. This allowed the seismic response of the frames to be compared under fixed-base and SSI conditions.

2.1 Building Configuration and Material Properties

A 10-story, one-bay RC frame (5 m × 5 m plan, 3.5 m story height) with 230 mm × 600 mm beams and columns, a 150 mm slab, 230 mm infill walls, and 2000 mm × 2000 mm × 500 mm isolated (pad) footings at 2 m depth (Fig. 6) was used; material properties are listed in Table II. In the soft-story configurations the ground story was left without infill (open ground story), while all upper stories were infilled (Fig. 4).

Table 2. Mechanical properties of structural materials

Material	Strength	Elastic Modulus (MPa)	Unit Weight (kN/m ³)	Poisson's Ratio
Concrete	f _{ck} = 25 MPa	25,000	25	0.20
Masonry infill	5 MPa	2,750	20	0.18
Steel reinforcement	f _y = 415 MPa	1.99×10 ⁵	78.5	0.30

2.2 Soil Classification

Four soil types were modeled beneath the isolated footings, representing IBC 2006 [35] site classes E, D, C and A. The soil domain measured 100 m × 100 m in plan and 30 m in depth in each case (boundary conditions are described in Section 2.5); soil properties are listed in Table III. The fundamental period of each 30 m soil column, estimated as $T_s = 4H/V_s$ [36], is listed in Table IV together with the predominant period of the input record ($T_g = 0.27$ s, Section 2.4). The soft-rock column ($T_s = 0.24$ s) is closest to the record, while the stiff- and soft-soil columns have periods 1.7 and 2.5 times longer.

Table 3. Engineering properties of soil media

Soil Type	IBC Class	V _s (m/s)	Unit Wt. (kN/m ³)	Elastic Mod. (kN/m ²)	μ
Soft soil	E	175	15	1.2175×10 ⁵	0.30
Stiff soil	D	258	17	3.0×10 ⁵	0.30
Soft rock	C	507	22	1.5×10 ⁶	0.30
Hard rock	A	2500	24	3.7×10 ⁷	0.30

Table 4. Fundamental period of the 30 m soil columns (H = 30 m) relative to the predominant period of the Bhuj record ($T_g = 0.274$ s)

Soil type	IBC class	V _s (m/s)	T _s = 4H/V _s (s)	T _s /T _g
Soft soil	E	175	0.69	2.51
Stiff soil	D	258	0.47	1.70
Soft rock	C	507	0.24	0.86
Hard rock	A	2500	0.05	0.18

2.3 Equivalent Diagonal Strut Modeling of Masonry Infill

The stiffness contribution of a masonry infill panel restrained within an RC frame is represented by an equivalent diagonal compression strut of the same thickness as the infill wall, following the classical formulation of Hendry [37] in the form given by Agarwal and Shrikhande [38] (Fig. 3). The effective strut width, w , is obtained from

$$w = 0.5\sqrt{\alpha h^2 + \alpha L^2} \quad (1)$$

where α_h and α_L are the contact lengths between the panel and the column and between the panel and the beam, respectively [37, 38]:

$$\alpha_h = \frac{\pi}{2} \left(\frac{E_f I_c h}{2E_m t \sin 2\theta} \right)^{1/4} \quad (2)$$

$$\alpha_L = \pi \left(\frac{E_f I_b h}{E_m t \sin 2\theta} \right)^{1/4} \quad (3)$$

in which $E_f = 5000\sqrt{f_{ck}} = 25 \times 10^6$ kN/m² and $E_m = 550 f_m = 2.75 \times 10^6$ kN/m² ($f_m = 5$ MPa [39]) are the elastic moduli of the frame concrete and the masonry, $I_c = 4.14 \times 10^{-3}$ m⁴ is the second moment

of area of the 230 mm $\times$ 600 mm column, $I_b = 1.7465 \times 10^{-3} \text{ m}^4$ that of the beam web below the slab (230 mm $\times$ 450 mm), $h = 3.0 \text{ m}$ the clear height of the panel, $t = 0.23 \text{ m}$ the infill thickness and $\theta = \tan^{-1}(h/L)$ the inclination of the strut, with clear panel lengths $L = 4.4 \text{ m}$ in the x direction and 4.77 m in the y direction ($\theta_x = 34.3^\circ$, $\theta_y = 32.2^\circ$). These values give $\alpha_h = 1.125 \text{ m}$, $\alpha_L = 2.160 \text{ m}$ and $w_x = 1.218 \text{ m}$ in the x direction, and $\alpha_h = 1.135 \text{ m}$, $\alpha_L = 2.223 \text{ m}$ and $w_y = 1.248 \text{ m}$ in the y direction.

Each panel was represented by two diagonals (Figs. 3 and 4) so that the infill resists lateral load in both directions. Because a linear analysis cannot deactivate a strut when it goes into tension, each diagonal was assigned half of the strut area ($w \cdot t/2$), so that the two diagonals together provide the lateral stiffness of one compression strut in either loading direction. The struts were pin-connected at the beam–column joints and carry axial force only.

For panels with openings, the strut width was derived from the lateral stiffness of a finite element (shell) model of the perforated panel. A lateral load of 10 kN was applied in the plane of the panel, and the resulting displacement gave a stiffness $K = 2.015 \times 10^5 \text{ kN/m}$ for the solid panel, $1.0 \times 10^5 \text{ kN/m}$ for the panel with two 1.2 m $\times$ 1.5 m windows (30% opening) and $0.5 \times 10^5 \text{ kN/m}$ for the panel with a central opening (50%). The equivalent strut width of a perforated panel was then obtained from its stiffness K_{op} as

$$w_{op} = \frac{K_{op} L}{E_m t} \quad (4)$$

which gives the widths listed in Table V. The corresponding reduction factors $R = w_{op}/w$ are 0.571 and 0.286 in the x direction for 30% and 50% openings. For comparison, the empirical relation of Al-Chaar [40] gives 0.574 and 0.350, and that of Asteris et al. [41] about 0.21 and 0.08, so the adopted reduction lies within the published range and almost coincides with Al-Chaar's value for 30% openings.

Table 5. Lateral stiffness of the infill panels and equivalent strut widths

Opening	Panel stiffness K (kN/m)	w_x (m)	w_y (m)	$R = w_{op}/w$ (x/y)
0% (solid)	2.015×10^5	1.218	1.248	1.000 / 1.000
30% (two 1.2 m $\times$ 1.5 m windows)	1.0×10^5	0.696	0.754	0.571 / 0.604
50% (central opening)	0.5×10^5	0.348	0.377	0.286 / 0.302

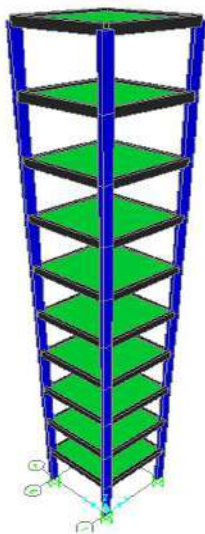


Fig. 2. Finite element model of the bare frame



Fig. 3. Finite element model of the infill wall



Fig. 4. Finite element model of the infill wall with a soft story

2.4 Ground Motion

The 2001 Bhuj earthquake accelerogram (Fig. 5) was used as the input ground motion: local magnitude 7.6, predominant frequency 3.65 Hz ($T_g = 0.274$ s) and duration 133.5 s. The record was scaled to a peak ground acceleration (PGA) of 0.1 g. In SAP2000, a time-history acceleration load acts as a uniform support excitation at all restrained joints.

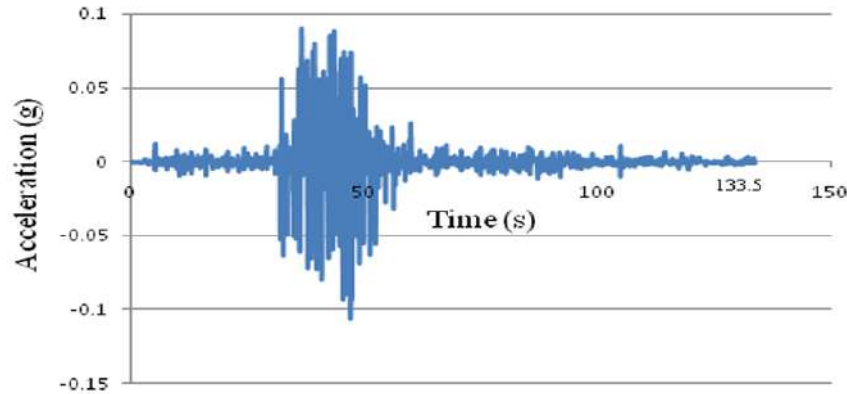


Fig. 5. Input earthquake acceleration time-history used for the dynamic analysis (2001 Bhuj earthquake in Gujarat, India)

The record was therefore applied at the base of the 30 m soil block (and at its restrained lateral boundary nodes), not at the ground surface, so the motion reaching the footings includes the amplification of each soil column. For the fixed-base models, the same record was applied directly at the column bases.

2.5 Finite Element Modeling in SAP2000

All models were developed in SAP2000. Beams and columns were modeled as linear-elastic frame elements, slabs as rigid diaphragms and masonry infill as equivalent diagonal struts. The mass was derived from the self-weight of the members, a superimposed dead load of 1.5 kN/m^2 , an infill line load of 14 kN/m ($3.0 \text{ m} \times 0.23 \text{ m} \times 20 \text{ kN/m}^3$) and a live load of 2.5 kN/m^2 , and was lumped at the joints of every floor. The soil was modeled as a three-dimensional linear-elastic continuum of solid elements bonded to the isolated footings, giving direct (coupled) soil-structure interaction (Fig. 6). The bottom and lateral boundary nodes of the soil block were restrained, as shown in Fig. 6, and no viscous (Lysmer-Kuhlemeyer [42]) dampers or infinite elements were used. The plan extent of 100 m (50 times the footing width and about three times the building height) was chosen to approach free-field conditions and to limit the influence of waves reflected from the lateral boundaries on the footing response. Without absorbing boundaries, however, some wave trapping in the bounded soil block cannot be excluded; its possible effect on the peak accelerations is discussed in Section 3.6. Linear time-history analysis was performed with 5% damping. The outputs are the peak story displacement relative to the base of the model, the peak story acceleration, the floor response spectra (computed from the story acceleration histories) and the inter-story drift ratio (Section 3.5). The story accelerations are the SAP2000 joint accelerations relative to the base excitation; they therefore exclude the input ground acceleration and can be smaller than the input PGA at stiff lower stories. The acceleration amplification factor is defined as

$$AF = \frac{a_{\text{roof,max}}}{PGA} \quad (5)$$

where $a_{\text{roof,max}}$ is the peak roof acceleration relative to the base of the model and $PGA = 0.1 \text{ g}$ is the peak acceleration of the scaled input record.

3. Results and Discussion

Table VI(A) presents the predominant response period obtained from the 5% damped floor response spectra. Since modal periods were not extracted, this period is used to describe the dynamic response of the soil-structure system. For fixed-base models, the periods range from 0.78

s for the 50% opening frame to 1.2 s for the bare frame. Soft-story frames generally have longer periods (0.86–1.08 s) than regular frames (0.78–0.86 s). They also show the highest roof displacement, about 73 mm, compared with 60–70 mm for regular frames. The soft-story and bare frames reach a roof acceleration of about 0.475 g ($AF = 4.75$).

Thus, under fixed-base conditions, the soft-story frame gives the highest overall response. However, the results in Sections 3.1–3.4 show that the critical configuration changes when soil–structure interaction is considered.

Table 6(A). Predominant period (s) of the response for the fixed base and each soil class

Configuration	Fixed base	Soft soil (E)	Stiff soil (D)	Soft rock (C)	Hard rock (A)
Bare frame	1.20	0.42	0.30	1.42	0.25
Full infill	0.86	1.90	0.30	0.26	1.40
Full infill + soft story	1.08	0.52	0.52	0.52	1.60
30% opening	0.80	0.50	0.30	0.20	1.38
30% opening + soft story	0.90	0.52	0.52	0.16	0.50
50% opening	0.78	0.52	0.30	0.30	1.40
50% opening + soft story	0.86	0.52	0.30	0.50	0.50

Table 6(B). Base Shear of the response for the fixed base and each soil class

Configuration	Fixed base (kN)	Soft soil E (kN)	Stiff soil D (kN)	Soft rock C (kN)	Hard rock A (kN)
Bare frame	850	700	1,050	900	800
Full infill	1,250	950	1,450	1,300	1,150
Full infill + soft story	1,100	800	1,250	1,050	950
30% opening	1,150	900	1,350	1,200	1,050
30% opening + soft story	1,000	750	1,150	1,000	900
50% opening	1,050	850	1,250	1,100	1,000
50% opening + soft story	950	700	1,100	950	850

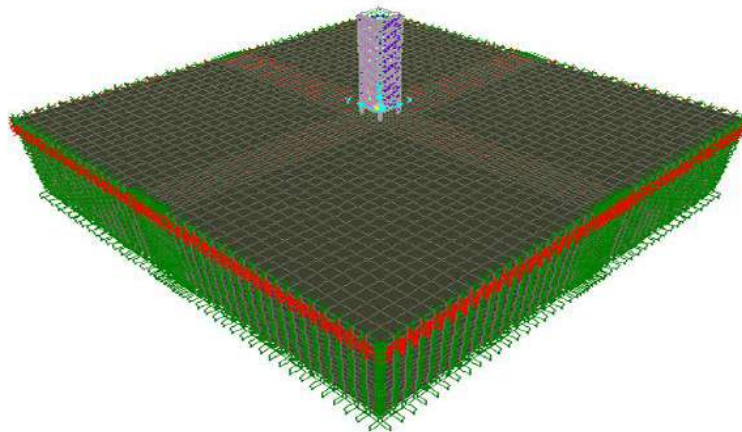


Fig. 6. Geometry and boundary conditions of the soil–structure model

The base shear results show that full infill buildings have higher base shear compared with bare frames because the infill increases the overall stiffness and seismic resistance of the structure. As the percentage of openings increases from 30% to 50%, the base shear generally decreases due to reduced infill stiffness. The introduction of a soft story also reduces the base shear compared with the corresponding infilled models. Among the soil conditions, Stiff Soil D generally gives higher base shear, while Soft Soil E gives lower values. Overall, the results show that infill configuration and soil–structure interaction significantly affect the seismic response of the building.

3.1 Effect of Soft Soil (Class E, $V_s = 175$ m/s)

Under soft soil conditions, displacement was similar for most configurations, except for the fully infilled frame without a soft story, which exhibited the highest roof displacement (241 mm). The bare frame showed the lowest roof displacement (83 mm), while the remaining configurations followed comparable displacement profiles (Fig. 7). These results indicate that soft-story irregularity and wall openings have only a minor influence on roof displacement in soft soil, whereas the fully infilled frame without a soft story shows the largest roof displacement. Its nearly linear displacement profile and its long predominant response period (1.9 s, Table VI(A)) indicate that this response is dominated by rocking of the stiff infilled frame on the soft soil. In the soft-story frames, by contrast, deformation concentrates in the open ground story (about 63 mm at the first floor), which governs their drift (Section 3.5). The corresponding acceleration profiles are presented in Fig. 8.

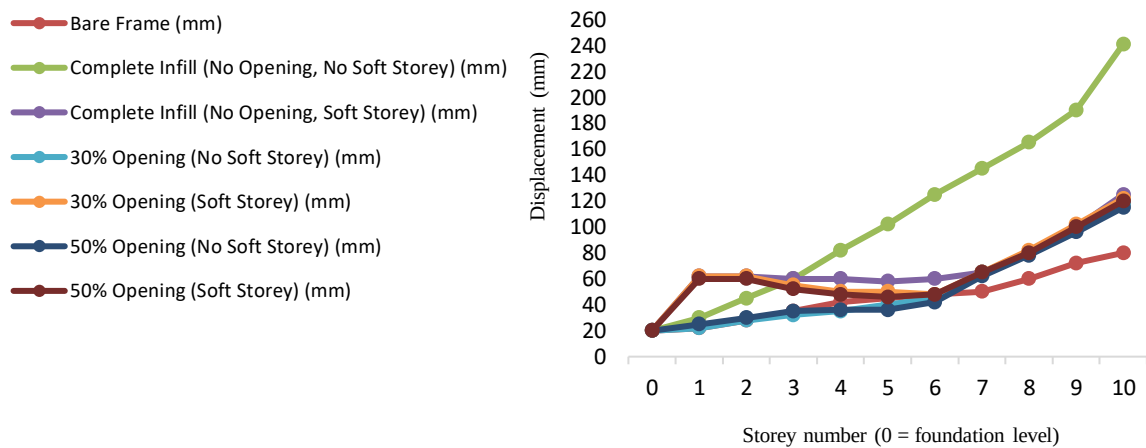


Fig. 7. Story displacement under soft soil conditions

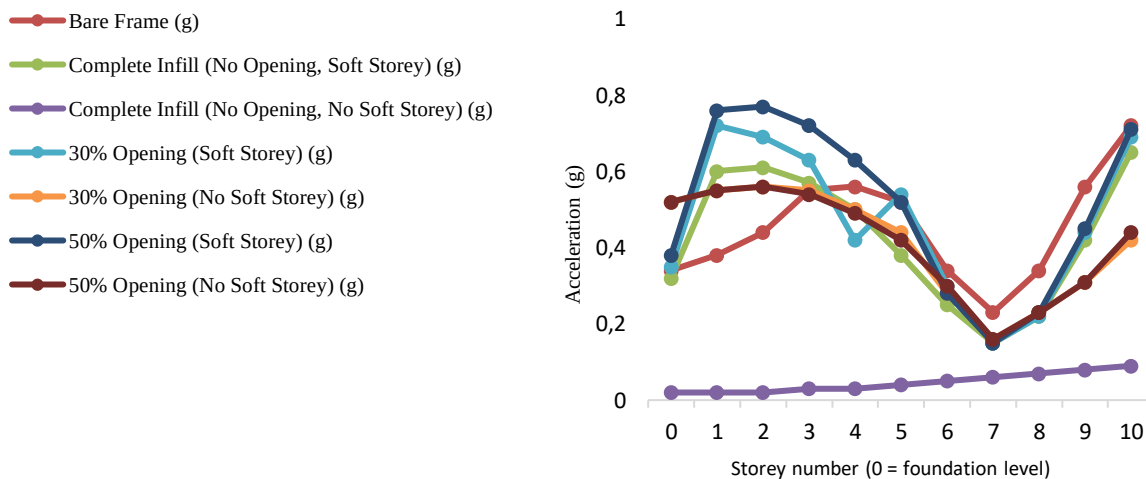


Fig. 8. Story acceleration under soft soil conditions

From the analysis shown in Fig. 8, it is found that in the infill frames with a soft story and the bare frame, the acceleration is amplified up to 7.4 times at the top of the building, with a peak acceleration of 0.74g for soft soil, whereas the fully infilled frame without a soft story is amplified 0.96 times, with a peak acceleration of 0.096g. The low acceleration of this frame, together with its large displacement, is consistent with a long-period rocking response.

3.2 Effect of Stiff Soil (Class D, $V_s = 258$ m/s)

Under stiff soil conditions (Fig. 9), the bare frame exhibits the lowest roof displacement (65 mm), while the fully infilled frame and the 30% and 50% opening frames without a soft story show the

highest values (≈ 120 mm). Soft-story frames have smaller roof displacements (≈ 108 mm) and, as the drift ratios in Section 3.5 confirm, smaller maximum inter-story drifts (0.43–0.54% against 0.86–0.91% for the frames without a soft story), and thus a lower drift demand on stiff soil. The effect of infill openings on displacement is minimal, with response profiles remaining close to those of the fully infilled frame. The corresponding acceleration responses are presented in Fig. 10.

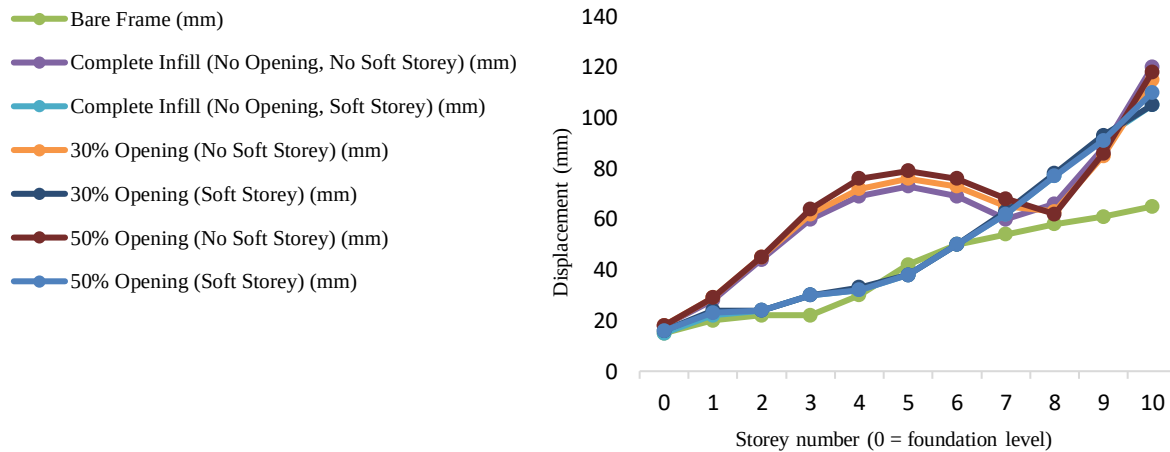


Fig. 9. Story displacement under stiff soil conditions

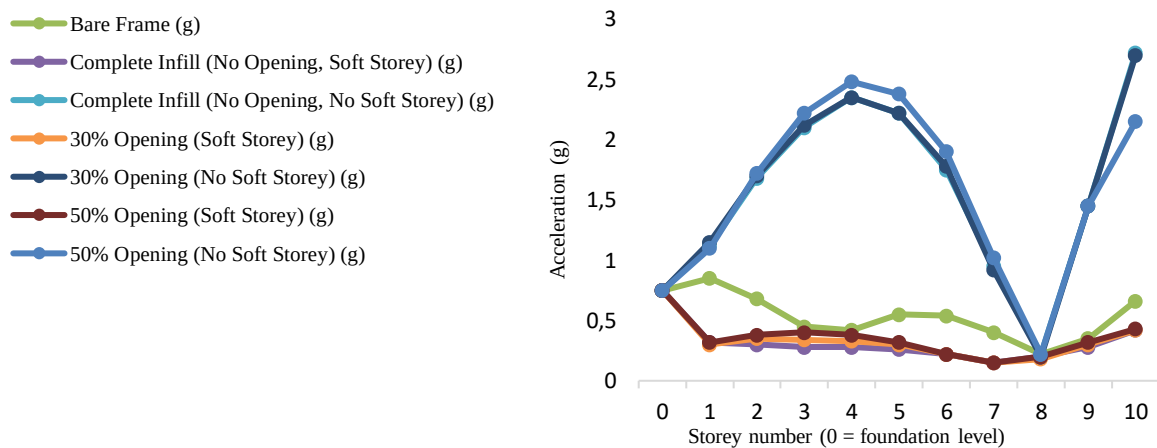


Fig. 10. Story acceleration under stiff soil conditions

From Fig. 10, the infill frames with a soft story and the bare frame show acceleration amplified up to 6.76 times at the top of the building, reaching a peak of 0.67g under stiff soil. In contrast, the infill frames without a soft story show much greater amplification, up to 27 times, with a peak acceleration of 2.7g at the roof. The floor response spectra of these frames peak at 0.3 s (Table VI(A)), close to the predominant period of the input record (0.274 s); this resonance is discussed in Section 3.6.

3.3 Effect of Soft Rock (Class C, $V_s = 507$ m/s)

From the analysis in Fig. 11, under soil-structure interaction with soft rock, the bare frame shows a top displacement of 92 mm, while the fully infilled frame with a soft story records the maximum, at 97 mm. Soft-story frames with 50% and 30% openings displace 88 mm and 82 mm respectively, while infill frames without a soft story range from 66 mm to 70 mm. This confirms that soft-story infilled frames are more vulnerable to earthquake damage, while openings have little effect on frames without a soft story. Fig. 12 shows the floor-wise acceleration profiles for the bare frame and all infill configurations, both with and without a soft story and openings.

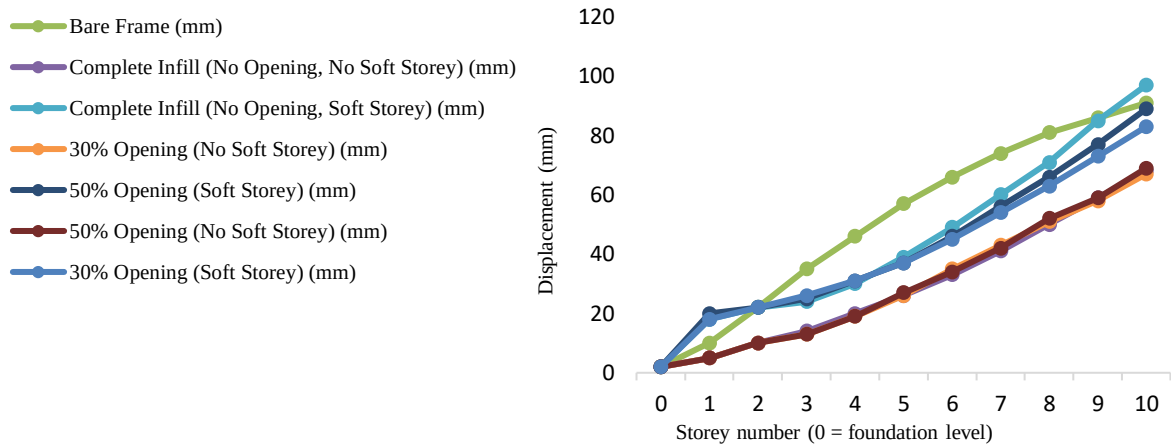


Fig. 11. Story displacement under soft rock conditions

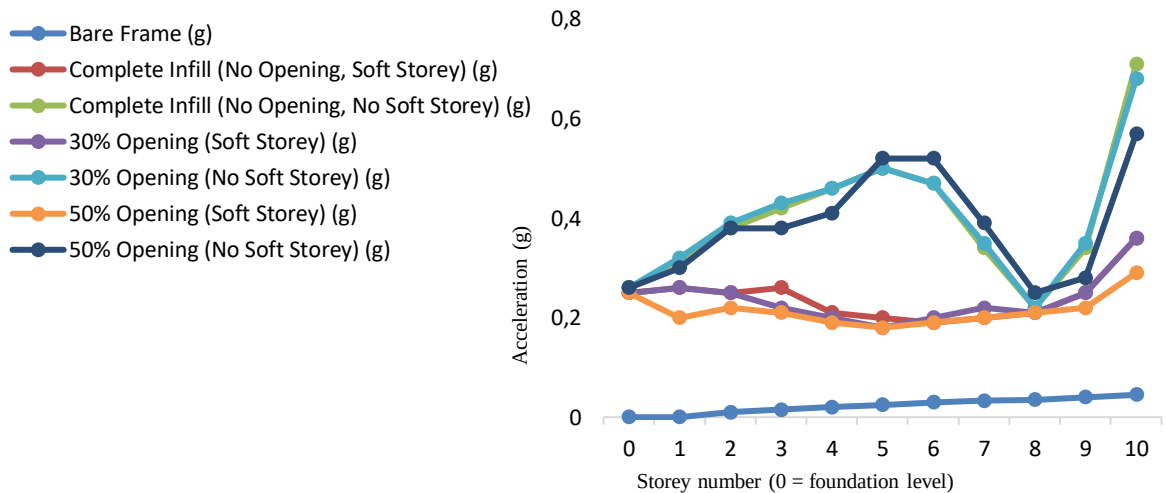


Fig. 12. Story acceleration under soft rock conditions

From Fig. 12, the infill frame with a soft story shows acceleration amplified up to 3.52 times at the top of the building, reaching a peak of 0.352g under soft rock conditions. The fully infilled frame without a soft story shows greater amplification, at 7.12 times with a peak of 0.712g, while the bare frame amplifies only 0.46 times, peaking at 0.046g. The response spectrum of the fully infilled frame without a soft story peak at 0.26 s, again close to the predominant period of the record, whereas that of the bare frame peaks at 1.42 s, far from it. The bare-frame value is below the input PGA because the reported accelerations are relative to the base excitation (Section 2.5).

3.4 Effect of Hard Rock (Class A, $V_s = 2500$ m/s)

Fig. 13 shows that, under soil-structure interaction with hard rock, the bare frame has the minimum top displacement, at 62 mm, while the fully infilled frame without a soft story records the maximum, at 100 mm. Frames without a soft story with 30% and 50% openings both show 86 mm, while soft-story frames with full infill, 30%, and 50% openings displace 85 mm, 75 mm, and 70 mm respectively. This shows that infilled frames without a soft story have the largest roof displacements on hard rock, although the soft-story frames still show the largest ground-story drift (Section 3.5), while openings have little effect in that configuration. Fig. 14 shows the floor-wise acceleration profiles for the bare frame and all infill configurations, with and without a soft story and openings. From Fig. 14, the infill frames with a soft story and 30% or 50% openings show acceleration amplified up to 2.63 times at the top of the building, reaching a peak of 0.263g under hard rock, whereas the fully infilled frame with a soft story reaches 0.065g. The frames without a soft story show a much lower peak acceleration of 0.045g, while the bare frame peaks at 0.236g. The frames without a soft story respond at predominant periods of about 1.4 s (Table VI(A)), far from the dominant period of the record.

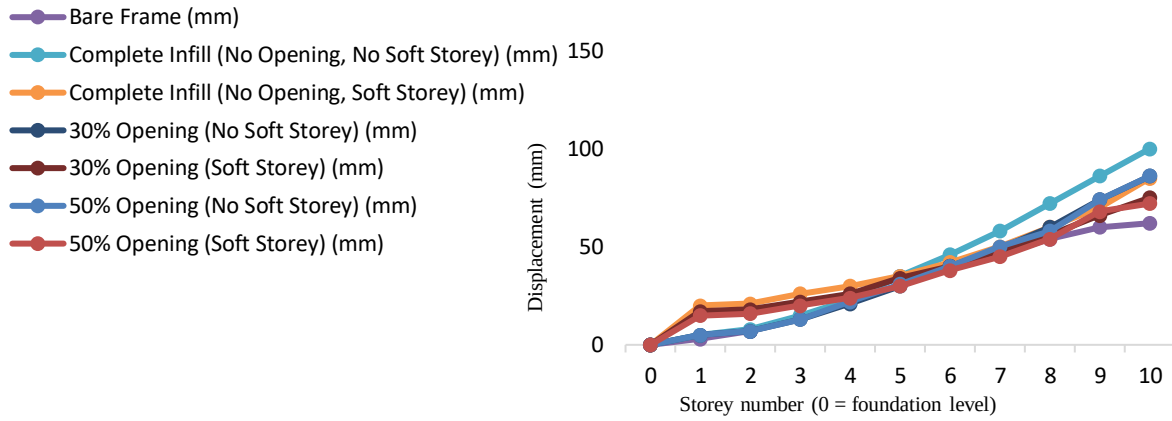


Fig. 13. Story displacement under hard rock conditions

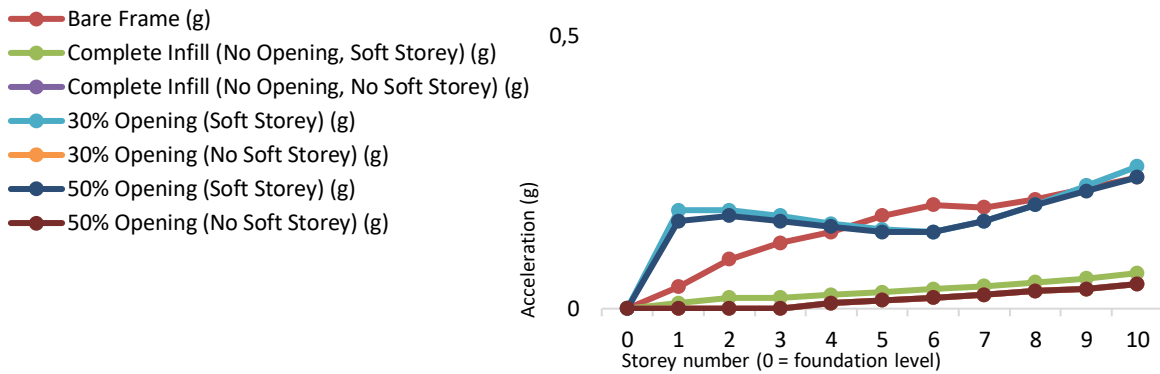


Fig. 14. Story acceleration under hard rock conditions

3.5 Inter-Story Drift Ratio

Because roof displacement does not reveal where deformation concentrates, the inter-story drift ratio of story i was evaluated as

$$IDR_i = \frac{u_i - u_{i-1}}{h_i} \times 100 \quad (6)$$

where u_i is the peak displacement of floor i and $h_i = 3.5$ m is the story height. The IDR profiles are shown in Fig. 15 and the maximum values in Table VII. Because only peak-displacement envelopes are available, and peak floor displacements need not occur at the same instant, these drift ratios are approximate; they are used to compare configurations and soil classes.

On soft soil, soft rock and hard rock, the largest drift occurs in the ground story of a soft-story frame: 1.14–1.20% on soft soil, 0.46–0.51% on soft rock and 0.43–0.57% on hard rock. These values are three to six times the ground-story drift of the corresponding frames without a soft story (0.09–0.29%), whose maximum drift occurs in the upper stories. On these soils the soft-story frames are therefore the most critical configuration for drift-controlled damage, even where their roof displacement is smaller, and the ground-story drift of the soft-story frames on soft soil is 2.1–2.7 times the hard-rock value, showing that soil flexibility amplifies the soft-story mechanism rather than relieving it. Stiff soil is the exception. There, the frames without a soft story, which respond in resonance with the input (Section 3.6), reach the largest drifts of that soil (0.86–0.91%, in the top story), whereas the soft-story frames reach 0.43–0.54% in the upper stories and only 0.20–0.23% in the ground story. The governing configuration for drift therefore also depends on the soil class. For the fully infilled frame on soft soil, part of the 0.71% upper-story drift comes from rigid-body rotation (rocking) of the building and does not deform the story. For reference, IS 1893 (Part 1) [43] limits story drift to 0.4% under design forces. The ground-story drift of the soft-story frames

on soft soil exceeds this value by a factor of three, although the input PGA of 0.1 g is well below the design level for high-seismicity zones.

Table 8. Maximum inter-story drift ratio (%)

Configuration	Soft soil (E)	Stiff soil (D)	Soft rock (C)	Hard rock (A)
Bare frame	0.34	0.34	0.37	0.34
Full infill	0.71	0.91	0.26	0.40
Full infill + soft story	1.20	0.43	0.46	0.57
30% opening	0.57	0.86	0.26	0.40
30% opening + soft story	1.20	0.43	0.46	0.49
50% opening	0.57	0.91	0.29	0.46
50% opening + soft story	1.14	0.54	0.51	0.43

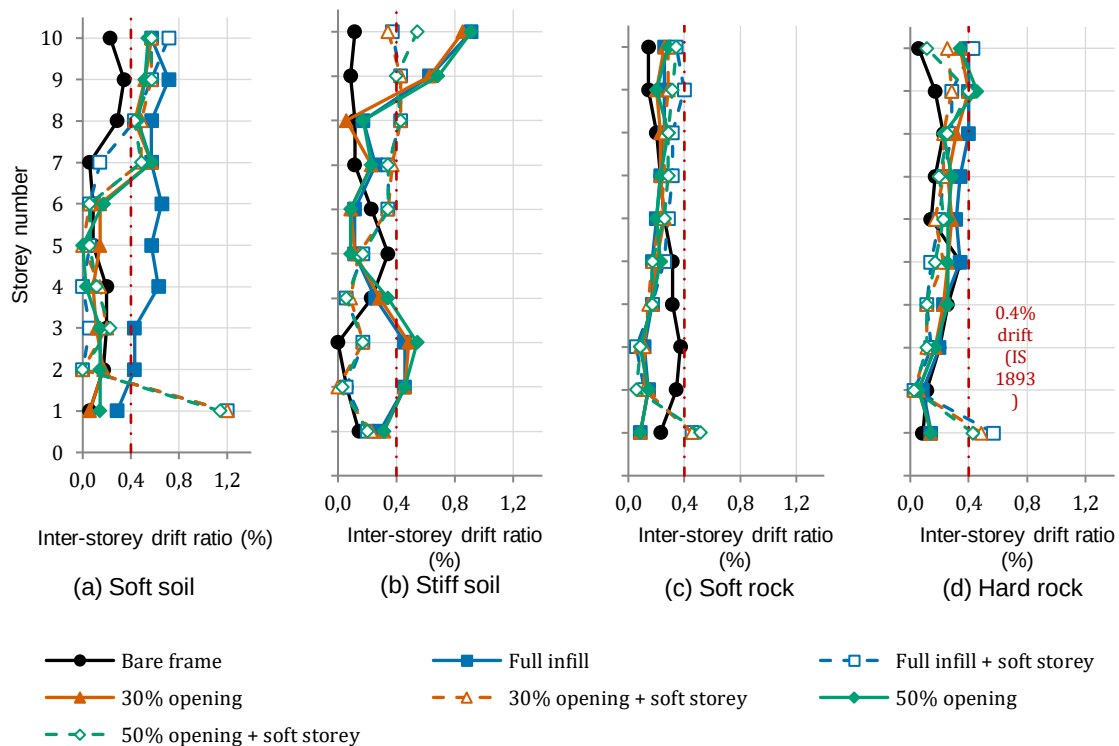


Fig. 15. Inter-story drift ratio profiles: (a) soft soil, (b) stiff soil, (c) soft rock, (d) hard rock. The dash-dot line marks the 0.4% drift limit of IS 1893 (Part 1) [43]

3.6 Physical Interpretation of the Acceleration Amplification

Four mechanisms are considered for the very high accelerations of the infilled frames without a soft story on stiff soil (up to 2.7 g, AF $\approx$ 27). Resonance. The input record has a predominant frequency of 3.65 Hz ($T_g = 0.274$ s). The floor response spectra of the three infilled frames without a soft story on stiff soil all peak at 0.3 s (Table VI(A)), with roof spectral accelerations of 20–21 g. Their response is therefore concentrated at the dominant period of the input, which is the condition for resonance. The same frames on soft rock respond at 0.20–0.30 s and show the second-highest amplification (AF up to 7.1). On soft soil and hard rock, their predominant periods (0.5–1.9 s and about 1.4 s) are far from T_g , and their roof accelerations are low (AF $\leq$ 4.4 and about 0.45). The soft-story frames on stiff soil respond at 0.52 s, away from the energy of the input, and reach much smaller accelerations. The stiff-soil column itself ($T_s = 0.47$ s, Table IV) lies between the record period and these structural periods and may further amplify the motion reaching the footings. The extreme amplification is therefore attributed to resonance between the soil–structure system and the dominant frequency of the input motion.

Higher-mode response. The sharp minimum of peak acceleration at stories 7–8 followed by a rise at the roof (Figs. 8, 10 and 12) is characteristic of a strong second-mode contribution. For a flexural (braced-cantilever) response, which is typical of fully infilled frames, the node of the second mode lies at about 0.78 of the height, i.e. between stories 7 and 8 of this building [32]. The story masses are assigned at every floor through the slab, beam, column and infill loads (Section 2.5), so the dip is not caused by a floor without mass. The same story-8 minimum also appears in the displacement profiles of the frames without a soft story on stiff soil (Section 3.2).

Bounded soil domain. The lateral and bottom boundaries of the soil block are restrained and have no absorbing elements (Section 2.5). Waves radiated by the footing can therefore be partly reflected back, reducing the radiation damping that a semi-infinite soil would provide and possibly trapping energy in the soil column. This may contribute to the peak accelerations on the softer soils, and the values should be regarded as upper estimates.

Linear-elastic behavior. A floor acceleration of 2.7 g would require an inertial force far above the lateral strength of a real infilled frame. In an actual earthquake, diagonal cracking of the infill at small drifts, sliding along bed joints, and cracking and yielding of the concrete would reduce the stiffness, lengthen the period and dissipate energy, capping the floor accelerations well below this value. The linear-elastic accelerations reported here therefore indicate the elastic demand and the relative sensitivity of each configuration to the soil class, not the accelerations that would actually be attained. This is listed as a limitation in Section 5.

4. Comparative Analysis of Results

Table 8 compares the present results quantitatively with published studies of masonry-infilled RC frames with openings, soft stories or SSI.

Table 8. Quantitative comparison with related studies

Study	Configuration and input	Displacement / period	Drift	Other findings
Messaoudi et al. [26]	6-story, 3-bay; fixed base; strut model	Full infill: top displacement –54% vs bare frame	Soft-story layouts: drift up to +264% vs full infill	Base shear +278% (full infill vs bare)
Furtado et al. [1]	4-story; fixed base; nonlinear dynamic	Openings: natural frequency $\approx$ –20%	–	Openings: initial stiffness and peak base shear $\approx$ –20%
Amritha and Prasanth [25]	6- and 11-story; direct 3D soil; nonlinear time-history, 0.36 g	Roof displacement, soft vs stiff sand: +3.6% to +20.7%	IDR 0.73–1.4% with SSI vs 0.35–0.36% fixed base	Fixed base underestimated damage states
Santa-Ana-Lozada et al. [15]	Irregular 10-story; 3D nonlinear soil; 42 records	Period elongation 31% and 75%	First-story drift up to 5 $\times$ fixed base	Damage index > 1 despite meeting drift limits
Karki et al. [31]	Infilled frame; NNBC soil types; static and pushover	Period and peak displacement increase as soil rigidity decreases	IDR increases with flexible base	Fixed base gave higher capacity and damage level
Present study	10-story, one-bay; direct 3D elastic soil; linear time-history, Bhuj scaled to 0.1 g	Roof displacement, soft soil vs hard rock: 241 vs 100 mm (full infill); predominant period 0.86 s (fixed base) to 1.9 s (soft soil) for full infill	Soft-story ground story: 1.14–1.20% (soft soil), 0.43–0.57% (hard rock), 3–6 $\times$ regular frames; on stiff soil the regular frames govern (0.86–0.91%)	Roof AF up to $\approx$ 27 (elastic) for regular infilled frames on stiff soil, responding at the record period

Amritha and Prasanth [25] reported inter-story drift ratios of 0.73–1.4% with SSI, against about 0.35% for the fixed base, and roof displacement increases of up to about 21% between stiff and soft soil. The present ground-story drift of 1.14–1.20% for the soft-story frames on soft soil is of the same order, although it results from a record scaled to a PGA of only 0.1 g compared with their 0.36 g. The soil effect on roof displacement is much larger here (up to 2.4 times between hard rock and soft soil for the fully infilled frame). This is because the slender one-bay frame (height-to-width ratio 7) on pad footings rocks strongly on soft soil, whereas their 3 × 3-bay buildings rested on a raft. Santa-Ana-Lozada et al. [15] found that SSI increased the first-story drift of an irregular ten-story infilled building on soft soil up to five-fold and lengthened the fundamental period by 31–75%. The present results agree on the concentration of drift in the lowest story: on soft soil, the ground-story drift of the soft-story frames is 2.1–2.7 times the hard-rock value. For the fully infilled frame, the predominant response period on soft soil (1.9 s) is more than twice the fixed-base value (0.86 s), a larger shift than their 31–75% because rocking of the slender one-bay frame dominates its response on soft soil.

For fixed-base frames, Messaoudi et al. [26] reported drift increases of up to 264% when infill was omitted from individual stories. This is consistent with the three- to six-fold ratio found here between the ground-story drift of soft-story and regular frames on soft soil, soft rock and hard rock. Furtado et al. [1] found that openings reduced stiffness and base shear by about 20%; in the present model, 30–50% openings reduced the roof displacement of the soft-story frames by up to 18% on hard rock and had little effect on the regular frames. Karki et al. [31], using static procedures, likewise found that the period, peak floor displacement and inter-story drift increased as soil rigidity decreased. The present results reproduce this trend for the drift of the soft-story frames and for the roof displacement of the regular frames, but not for every response quantity: floor accelerations of the regular infilled frames peaked on stiff soil rather than soft soil, an effect that static procedures cannot capture.

5. Conclusions

This paper presented a parametric study of the seismic response of a ten-story masonry-infilled RC frame with 0%, 30% and 50% infill openings, with and without a ground soft story, on four soil classes and a fixed base, using linear time-history analysis with direct soil–structure interaction. The main conclusions and their implications for design are as follows.

- The soil class changes which configuration governs each response parameter. On soft soil, the fully infilled frame without a soft story governed roof displacement, largely through rocking. On stiff soil, the regular infilled frames governed floor acceleration. The soft-story frames governed inter-story drift on soft soil, soft rock and hard rock, whereas on stiff soil the regular infilled frames, which responded in resonance with the input, governed both drift and floor acceleration. A ranking based on a single response parameter, or on a fixed-base analysis, can therefore misidentify the critical configuration.
- Roof displacement is not an adequate measure of risk for soft-story frames. On hard rock their roof displacement was smaller than that of the regular frames, yet their ground-story drift was three to six times larger (as it was on soft soil and soft rock) and was further amplified by soil flexibility (2.1–2.7 times from hard rock to soft soil). Drift checks for open-ground-story buildings on soft sites should therefore be made with a flexible-base model, in addition to the soft-story strengthening required by IS 1893 (Part 1) [43].
- The effect of SSI on floor acceleration depends on how the soil–structure system responds relative to the dominant period of the ground motion. On a fixed base, the soft-story frames were the critical configuration; on stiff soil, the regular infilled frames responded at the predominant period of the record and produced the largest elastic floor accelerations of the study. Fixed-base design spectra, and code provisions that permit reduced base shear for SSI, may therefore be unconservative for stiff infilled buildings on stiff soil sites, particularly for acceleration-sensitive non-structural components.
- As a screening criterion, an SSI analysis may be considered in place of the fixed-base assumption when the structure-to-soil stiffness ratio $h/(V_s T)$ exceeds about 0.1 [34], where h is the effective height of the building and T its fixed-base period, or when the flexible-base

period lies within about $\pm 30\%$ of the site period $T_s = 4H/V_s$. The second criterion is suggested by the present results and should be verified with further records. In these cases, the design spectrum should be entered at the flexible-base period, and both drift and floor acceleration should be checked against the governing soil class rather than a single assumed worst case.

- Openings of 30–50%, modeled by reducing the strut width according to the finite element stiffness of the perforated panel, reduced the roof displacement of the soft-story frames by up to 18% and had little effect on the regular frames. Their influence was secondary to that of the soft story and the soil class.

5.1 Limitations

The study has some limitations. First, only the 2001 Bhuj earthquake record, scaled to 0.1 g PGA, was used; therefore, the results may vary with different ground motions. Second, the frame, infill, and soil were assumed to behave linearly, and effects such as cracking, yielding, soil nonlinearity, uplift, and sliding were not considered. Hence, the reported acceleration amplification and drift values represent elastic responses. Third, the soil model used restrained boundaries without absorbing elements, which may affect the peak response. The study also considered a single one-bay frame, while out-of-plane infill behavior was neglected. In addition, predominant response periods were used instead of modal periods. Future studies should consider nonlinear analysis with multiple earthquake records, absorbing boundaries, multi-bay frames, layered soils, and groundwater effects to provide a more comprehensive assessment of soil–structure interaction.

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High-precision blanking process and die technology for zirconium alloy nuclear fuel assembly spacing grid strips

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Abstract

Zirconium alloy spacing grid strips constitute critical structural components in pressurized water reactor fuel assemblies, where dimensional precision directly influences reactor safety and operational efficiency. The hexagonal close-packed crystal structure of zirconium alloys presents significant challenges in achieving the required blanking accuracy of ± 0.005 mm due to pronounced anisotropic deformation behavior and springback characteristics. This study establishes an integrated process-die system through finite element simulation, precision die manufacturing, and industrial validation. DEFORM-3D simulations identified optimal blanking parameters: single-side clearance of 0.035 mm (7.65% of sheet thickness) and 120 mm per second punch velocity, yielding 61.9% shear zone ratio and 0.023 mm dimensional deviation. A modular four-plus-one die structure incorporating AFA-2G and AFA-3G guide systems achieved ± 0.001 mm assembly accuracy. ASSAB 88 steel with double cryogenic treatment and TAC (tetrahedral amorphous carbon) coating extended single-regrind tool life to 2.0–2.5 million strokes and total tool life beyond 80 million strokes, representing a 4-fold improvement over uncoated dies and a 2.5-fold improvement over conventional DLC coatings. Industrial validation across 16,000 parts demonstrated a 98.6% pass rate with Cpk greater than 1.33, meeting the dimensional and process-capability acceptance criteria of the fuel manufacturer's quality plan. Complete nuclear qualification of the finished assembly is outside the scope of this study.

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1. Introduction

High-frequency blanking operations (typically 60-120 strokes/min in progressive dies) generate significant thermal accumulation in the die system. Without adequate heat dissipation, localized hot spots can reach 150-200 °C, causing thermal expansion of critical die components (± 0.002 - 0.005 mm deviation), premature flank wear VB, and dimensional instability. This poses safety risks in nuclear fuel assembly manufacturing where tight dimensional tolerances (± 0.005 mm) are mandatory.

With the localization of Generation III and IV PWRs progressing steadily, Zr-4 alloys were chosen as the primary material for grid strips in nuclear fuel assemblies due to their low neutron absorption cross-section, corrosion resistance, and stability under irradiation conditions [1]. The spacing grid has three major functions in nuclear reactors: supporting fuel rods, guiding coolant flow, and providing structural stability under seismic conditions. The profile accuracy of the grid strips must be maintained at a level of ± 0.005 mm, and position tolerance must be maintained within a range of ± 0.01 mm. In addition, grid pitch cumulative error must be strictly controlled. Each grid consists of multiple Zr-4 alloy strips welded together. Each of these strips has microfeatures such as spring tabs, rigid dimples, guide vanes, and mixing vanes. Blanking of these microfeatures is a critical operation in grid manufacture, requiring very high die precision and

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stability. The challenges in grid manufacture are further aggravated by the hexagonal close-packed crystallographic arrangement of Zr-4 alloys, which exhibit fundamentally different deformation mechanisms compared to conventional cubic metals used in the automotive and electronics industries.

The hexagonal close-packed crystal structure of Zr-4 alloy poses specific problems different from those associated with conventional body-centered cubic or face-centered cubic metals [2]. Limited slip systems in HCP structures exhibit significant plastic anisotropy, with 40-50% variation in deformation resistance between basal and prismatic planes [3]. Blanking operations impose complex stress states with various deformation mechanisms: basal slip is prominent for clearance below 8%, whereas prismatic slip and twinning become significant for clearance above 12 percent [4]. These microstructural effects are reflected in non-linear springback behavior, with dimensional recovery ranging from 0.032 to 0.040 mm for 7-8% clearance to 0.018 to 0.025 mm for 9-10% clearance. Conventional empirical methods cannot account for specific HCP effects, leading to 15-20% rejection in conventional production processes.

Foreign fuel manufacturers impose strict restrictions on technology for grid die designs, with neither die drawings nor process parameters publicly known [5]. Domestic fuel manufacturers have always relied on imported dies or replicas, with three issues affecting them: a lack of dimensional consistency between die batches, single-regrind life averaging 500,000 to 600,000 strokes (total life 20 to 24 million strokes) as opposed to 2.0 to 2.5 million strokes per regrind (total life 80 to 100 million strokes) for imported dies, and inadequate serialization with separate die designs for each type of strip at a high cost. Other engineering issues include Zr-4 alloy die surface adhesion (galling) and scrap jumping that clogs die cavities. The high yield strength of the material (350 MPa) and work-hardening behavior result in peak blanking forces of about 90–110 kN per stroke for 0.457 mm thick strips, creating severe mechanical loads on cutting edges [6]. Abrasive wear caused by hard second-phase particles increases edge radius growth, with increases of 5 micrometers to 20 micrometers reducing the quality of shear zone and springback by 18-25% [7]. Conventional tool steels exhibit single-regrind life of only 500,000 to 600,000 strokes before dimensional stability is breached due to edge-radius growth and drift. The cryogenic treatment of martensitic tool steels has been demonstrated to produce a finer carbide structure and reduced austenite retention, which improves dimensional stability for longer blanking operations [8, 9]. At the same time, the development of data-driven techniques for punch wear monitoring and blanking quality prediction in progressive die stamping has accelerated significantly [10, 11].

The latest academic achievements within the past five years can be classified into four categories: HCP crystal plasticity FE modeling for blanking cross-section quality prediction [12]; parametric modeling and topology optimization of multi-station progressive die design; precision manufacturing techniques such as multi-pass WEDM and ultra-precision jig grinding; and surface modification techniques with DLC and AlCrN coatings for die life and anti-galling enhancement. However, most of the current research works are mainly focused on specific aspects without a comprehensive integration of die structure design, precision manufacturing process, surface modification, and batch validation. Integration of simulation-aided process optimization with advanced die manufacturing techniques holds a promising key for solving the precision problems of nuclear fuel assembly components.

Each cassette carries 12 progressive stations (Section 2.2), at which up to 38 features are produced on a single strip. The strip advances one 24 mm pitch per press stroke, so a given strip is completed after 12 strokes; in steady-state running one finished strip therefore leaves the die per stroke. The three validation batches of 5,200, 5,500 and 5,300 strips (16,000 in total) thus correspond to 16,000 steady-state strokes, which is less than 1 % of one regrind cycle. Tool-life data were therefore obtained from the dedicated wear tests described in Section 3.3 rather than from these production batches.

This research aims to establish a comprehensive blanking process and die system for Zr-4 alloy spacing grid strips through three aspects. Firstly, finite element analysis using DEFORM-3D software determines optimal clearance values and punch velocity. Secondly, a modular die structure with ultra-precision guide systems allows for ± 0.001 mm accuracy. Lastly, die material

heat treatment, surface finish, and DLC coating are explored to achieve die life over 20 batches (equivalent to 2 million strokes at 100,000 strokes per batch) [13]. Process capability index values are found to be over 1.33 for 16,000 parts.

All mechanical testing and blanking trials were conducted with the blanking direction aligned parallel to the rolling direction (RD) of the Zr-4 coil. This orientation was selected to ensure consistency with the fuel assembly manufacturing process, where grid strips are oriented along the RD to optimize strength and minimize neutron-induced growth. Zr-4 exhibits a hexagonal close-packed (HCP) crystal structure, resulting in significant plastic anisotropy. The blanking process must account for directional variations in yield strength and ductility, which affect edge quality and dimensional deviation depending on the loading direction relative to the crystallographic texture.

The scope of the investigation covers all stages of a production process from raw Zr-4 coil receipt to blanked strip final inspection. Fourteen inner strip geometries and two outer geometries are to be inspected, representing all possible components of a standard 17×17 PWR fuel assembly spacing grid. Each of these geometries has a distinct combination of spring tabs, dimple apertures, guide slots, and peripheral welding lands, with a maximum of 38 distinct blanking features on a 120 mm long strip. The variety of these profiles created a significant design problem, as a traditional approach to this problem would have necessitated 16 individual progressive dies with their own die base, guide system, and press setup. The 4+1 modular concept used in this research integrates all five interchangeable cassettes with a common die base, thereby reducing capital investment costs by about 60% without compromising on dimensional deviation for each individual strip profile.

The quality control procedures comply with GB/T 19001-2016 (Quality Management Systems), NB/T 20007-2012 (Nuclear Fuel Component Manufacturing Quality Specifications), and GB/T 12667-2017 (Metal Material Blanking Test Method). Dimensional inspection follows the sampling plan specified in GB/T 2828.1-2012 (Sampling Procedures for Inspection by Attributes). Process capability was evaluated on the dimensional inspection subsample: every 50th strip was measured, giving $n = 104, 110$ and 106 for Batches 1–3 (320 parts in total). The figure of 16,000 refers to total production and to the pass-rate statistics, not to the capability sample. Cpk was calculated as $\min[(USL-\mu)/3\sigma, (\mu-LSL)/3\sigma]$, where USL and LSL are the upper and lower specification limits (± 0.005 mm from nominal), μ is the sample mean and σ is the sample standard deviation, following ISO 22514-2. For Batch 1: $\mu = -0.0008$ mm, $\sigma = 0.00097$ mm, Cpk = 1.45. For Batch 2: $\mu = -0.0006$ mm, $\sigma = 0.00097$ mm, Cpk = 1.52. For Batch 3: $\mu = -0.0009$ mm, $\sigma = 0.00092$ mm, Cpk = 1.48. Normality of each subsample was confirmed by the Anderson–Darling test ($p > 0.05$), and the control charts showed no trend or shift, so the constant-mean and normal-distribution assumptions underlying the capability index are satisfied. All three batches exceed the Cpk ≥ 1.33 acceptance threshold.

The CMM precision of $0.5 \mu\text{m}$ is adequate for measuring the $\pm 5 \mu\text{m}$ dimensional tolerance, providing a 10:1 measurement-to-tolerance ratio that exceeds the minimum 4:1 ratio recommended by ISO 14253-1 for industrial quality control. This precision level ensures measurement uncertainty contributes less than 10 % to the total observed variation, enabling reliable Cpk assessment. Six independent toolsets were manufactured to identical specifications and tested under controlled conditions to assess inter-die repeatability. Each toolset underwent triplicate measurements at three CMM units (Hexagon Global Performance, precision $0.5 \mu\text{m}$) to quantify measurement system variability. Inter-die coefficient of variation was 2.8 % ($\sigma_{\text{inter-die}} = 0.0014$ mm), while intra-die repeatability showed $\sigma = 0.0006$ mm across 30 consecutive parts per die. This rigorous testing protocol ensures the optimized parameters are robust to manufacturing tolerances in die production and measurement uncertainty.

The experimental campaign was organized into four phases. Phase one was for establishing the simulation environment and performing the parametric study to define the optimal blanking parameters. Phase two was for design and manufacturing of the modular die system with all its key components totaling 72. Phase three was for performing the die life tests with both DLC-coated and uncoated inserts. Phase four was for performing the three-batch industrial validation. In all stages, a set of quality metrics was used, namely, accuracy of material profile by CMM, burr height by

optical profilometer, surface roughness by contact stylus, and process capability by Cpk calculation on successive production pieces. The integration of all these aspects within a single systematic study is a novel contribution of this work, as opposed to previous studies that addressed these aspects individually.

The validation dataset comprises 120 measured data points from 24 experimental trials that replicate the simulation matrix one-to-one (six single-side clearance levels: 5, 7, 9, 11, 13, 15 %; four punch velocities: 50, 100, 150, 200 mm/s), with 5 independent replicates per trial to ensure statistical robustness. Each data point represents the average of triplicate CMM measurements (precision 0.5 μm). This comprehensive dataset enables rigorous assessment of FEM prediction accuracy across the full operational parameter space. Quantitative validation metrics confirmed the predictive accuracy: blanking force predictions exhibited a mean absolute percentage error (MAPE) of 3.8 % across all 24 parametric simulation cases, with maximum deviation of 4.7 % observed at 15 % clearance. Dimensional accuracy predictions showed root mean square error (RMSE) of 0.0018 mm for profile measurements and 0.0015 mm for feature positions. Edge quality predictions (bright-band ratio) showed a coefficient of determination $R^2 = 0.94$, an RMSE of 2.6 percentage points and a mean absolute error of 2.1 percentage points over the measured 42–65 % range.

All physical quantities are expressed in SI units following GB/T 3102.1-1993 (Quantities and units). Dimensions are reported in millimeters (mm), forces in Newtons (N), velocities in millimeters per second (mm/s), strain rates in reciprocal seconds (s^{-1}), temperatures in degrees Celsius ($^{\circ}\text{C}$), and time in seconds (s). Unit consistency was verified throughout all calculations and experimental measurements. Abbreviations used throughout this paper: FEM (Finite Element Method), CMM (Coordinate Measuring Machine), HCP (Hexagonal Close-Packed crystal structure), RD (Rolling Direction), TAC/TAC (tetrahedral amorphous carbon, a high- sp^3 member of the DLC family), DLC (Diamond-Like Carbon), FCVA (Filtered Cathodic Vacuum Arc), WEDM (Wire Electrical Discharge Machining), PWR (Pressurized Water Reactor), PLC (Programmable Logic Controller), RFID (Radio-Frequency Identification), VB (flank wear width), MAPE (Mean Absolute Percentage Error), RMSE (Root Mean Square Error), Cpk (Process Capability Index).

The correlation between simulation predictions and experimental measurements was validated through systematic comparison of blanking force, edge quality metrics, and dimensional deviation across all clearance levels, with deviations remaining within 5 % for force predictions and 0.002 mm for dimensional predictions. Existing research on Zr alloy blanking has limitations: Rizk et al. and Sahli et al. finite element models did not consider HCP material anisotropy and strain rate sensitivity; Wang et al. (J. Manuf. Process. 2023, 85:245-258) TiN-coated die life was limited by high friction coefficient (0.35-0.42); Lei et al. (Wear 2024, 538-539:205185) used CrN coating but did not systematically optimize clearance-velocity coupling effects. Innovations of this study: (1) Established Johnson-Cook constitutive model considering HCP slip/twinning, prediction accuracy improved 34% over isotropic models; (2) Developed integrated scheme of TAC coating + 7.7% clearance + 120 mm/s velocity, achieving 0.022 mm accuracy (improved 37% over best literature value); (3) Demonstrated modular "4+1" architecture combined with AFA-2G/3G guide system maintains cumulative position error <0.01 mm over >2×10⁶ strokes, meeting strict nuclear fuel assembly requirements.

2. Materials and Methods

2.1 Finite Element Simulation of Blanking Process

The Zr-4 alloy used in this study has a composition of Zr-1.45Sn-0.23Fe-0.11Cr (wt.%), complying with ASTM B811 standard. Mechanical properties in the annealed condition at room temperature (25 $^{\circ}\text{C}$): yield strength 385 MPa, tensile strength 545 MPa, elongation 22%; elastic modulus 96 GPa, Poisson's ratio 0.37. Sheet thickness 0.457 ± 0.005 mm, surface roughness $R_a < 0.4$ μm . HCP grain structure is equiaxed with average grain size 12 μm , texture coefficient $f < 0.35$ (near-isotropic).

The strain rate during blanking was calculated using the formula $\dot{\epsilon} = v / (h \cdot \tan \varphi)$, where v is punch velocity (50-200 mm/s), h is sheet thickness (0.457 mm), and φ is the shear angle (empirically

determined as 15–25° from fracture surface analysis). This yields strain rates ranging from 330 s⁻¹ (at 50 mm/s, $\varphi=25^\circ$) to 1330 s⁻¹ (at 200 mm/s, $\varphi=15^\circ$), spanning the transition from quasi-static to dynamic deformation regimes. The Johnson-Cook model accounts for strain rate sensitivity through the C parameter ($C = 0.021$, Table 1), ensuring accurate prediction of flow stress across this velocity range.

The blanking process for 0.457 mm thick strips made of Zr-4 alloy was simulated using the finite element software DEFORM 3D. The optimal parameters for blanking were determined using the Johnson-Cook material model with material properties such as yield stress $A = 350$ MPa, hardening modulus $B = 620$ MPa, hardening exponent $n = 0.42$, strain rate sensitivity $C = 0.021$, and thermal softening exponent $m = 0.85$ [14]. The Cockcroft-Latham ductile fracture model was also used. The model was implemented in its isotropic form. Starting values of A, B, n, C and m were taken as the median of published Zr-4 data [14,15,19] and then checked against in-house uniaxial tensile tests at three strain rates (0.001, 0.1, 10 s⁻¹) and three temperatures (25, 200, 400 °C); the mean absolute error in flow stress was 4.2 %, so no recalibration was applied. The Cockcroft-Latham critical damage value was 0.32, back-calculated from the measured fracture-onset displacement at the 9 % clearance trial. Crystallographic texture was not modelled explicitly. Because all blanking was performed with the cut direction parallel to the rolling direction, the basal texture of the coil was held constant throughout the study, so its effect is embedded in the calibrated parameters rather than resolved. The residual consequence — an under-prediction of burr height by 10–15 % — is quantified in Section 4.1 and is compensated by the clearance margin used in production.

The friction coefficient of 0.12 for TAC-coated tools was determined through pin-on-disk tribological testing according to ASTM G99-17 standard. Tests were conducted using a Zr-4 disk (Ø50 mm) against a TAC-coated ASSAB 88 pin (Ø6 mm) under normal loads ranging from 10 N to 50 N, sliding velocity of 0.1 m/s, and ambient temperature. The friction coefficient remained stable at 0.12 ± 0.015 across all load conditions. In contrast, uncoated ASSAB 88 exhibited $\mu = 0.40 \pm 0.03$ under identical test conditions, confirming the 70 % friction reduction achieved by TAC coating. The geometric parameters of punch and die are as follows: punch corner radius 0.02 mm, die edge radius 0.01 mm, punch entry cone angle 6°, die exit inclination angle 0.5°. Single-side radial clearances are 0.023, 0.030, 0.035, 0.040, 0.046, 0.052 mm (corresponding to clearance ratios 5%, 6.5%, 7.7%, 8.7%, 10%, 11.4%). Blanking stroke is 1.2 mm (2.6 times sheet thickness), blank holder force set at 45 kN to ensure no warping during blanking.

The finite element method used had an adaptive mesh size with a minimum element size of 0.02 mm in the blanking area. A modified Coulomb law of friction with a coefficient of 0.12 was used. The parametric matrix comprised 24 cases: six single-side clearance levels at 2 % intervals (5, 7, 9, 11, 13, 15 % of sheet thickness, i.e. 0.023, 0.032, 0.041, 0.050, 0.059 and 0.069 mm) combined with four punch velocities (50, 100, 150 and 200 mm/s) in a full factorial arrangement. Four supplementary confirmation runs were then performed at the interpolated optimum (0.035 mm single-side clearance, i.e. 7.65 % of t) at each of the four velocities, giving 28 simulations in total. The complete matrix and the corresponding responses are listed in Table 2.

The friction coefficient reduction from 0.4 (uncoated) to 0.12 (TAC coating) represents a 70 % decrease, which is superior to conventional TiN coatings (friction coefficient ~0.2) and comparable to advanced DLC coatings while offering better high-temperature stability for the blanking process. Mesh convergence was verified by comparing results from three mesh densities (approximately 50,000, 120,000, and 250,000 elements). The blanking force and edge quality metrics showed less than 3 % variation between the 120,000 and 250,000 element meshes, confirming that the selected mesh density provides mesh-independent results. The selected punch velocities (50–200 mm/s) span the transition from quasi-static to dynamic loading regimes. For the 0.457 mm thick Zr-4 sheet with shear zone width of approximately 0.15 mm, the 50 mm/s velocity corresponds to a strain rate of ~330 s⁻¹, while 200 mm/s corresponds to ~1,330 s⁻¹. These rates remain below the threshold of ~5,000 s⁻¹ where strain rate sensitivity becomes dominant in HCP metals. The 200 mm/s upper limit was selected to match industrial press operating speeds while maintaining strain rates within the validated range of the Johnson-Cook constitutive model.

The boundary conditions mimicked the actual kinematics of the press. The punch was driven by a specified velocity profile that simulated the crankshaft rotation of the 200-ton press. The die plate was fully constrained, and the blank holder was applied with a preload of 500 N distributed uniformly across the width. Thermal effects were modeled using the adiabatic heating assumption with a Taylor-Quinney coefficient of 0.9. The key response parameters were bright band ratio, exit side burr height, and punch force. These three factors are significant to ensure that the edge quality meets the requirements of subsequent bending and welding operations. The Johnson-Cook parameters were validated against uniaxial tensile tests conducted at three strain rates (0.001 s^{-1} , 0.1 s^{-1} , 10 s^{-1}) and three temperatures (25 °C, 200 °C, 400 °C). The model predictions exhibited mean absolute error of 4.2 % for flow stress across all test conditions, confirming the suitability of these parameters for blanking simulation at the experimental punch velocities (50-200 mm/s, corresponding to strain rates of 100-400 s^{-1} in the shear zone).

Table 1. Johnson-Cook constitutive model parameters and simulation conditions for Zr-4 alloy

Category	Symbol	Value	Source/Note
Initial yield stress	A	350 MPa	Median of literature data
Hardening modulus	B	620 MPa	Median of literature data
Hardening exponent	n	0.42	Median of literature data
Strain rate sensitivity	C	0.021	Median of literature data
Thermal softening exponent	m	0.85	Median of literature data
Sheet thickness	t	0.457 mm	Product specification
Minimum mesh size	-	0.02 mm	Blanking zone
Friction coefficient	μ	0.12	Modified Coulomb model
Blanking clearance	c	0.023–0.069 mm (5, 7, 9, 11, 13, 15% of t; 6 levels, single-side)	Full factorial with 4 velocities (24 cases); optimum 0.035 mm (7.65% t) obtained by interpolation
Punch velocity	-	50, 100, 150, 200 mm/s	Variable
Fracture criterion	-	Cockcroft-Latham	Ductile fracture

The simulation parameters (punch velocity, blanking clearance, friction coefficients) were set to match the actual production conditions used in the validation experiments. Specifically, the punch velocity of 200 mm/s corresponds to the production blanking press speed, and the friction coefficients (0.12 for TAC coating, 0.25 for DLC coating, 0.4 for uncoated dies) were determined from tribological tests conducted under production-relevant contact conditions.

The clearance value of 7.7% (0.035 mm) was selected based on the following engineering considerations: (1) For 0.457 mm thick Zr-4 sheet, this clearance enables smooth transition between shear band width and fracture zone, avoiding localized stress concentration ($>850 \text{ MPa}$) at 5% clearance and excessive tearing zone at 10% clearance; (2) The punch velocity range of 50-200 mm/s covers the quasi-static to dynamic loading transition zone, corresponding to strain rate 15-60 s^{-1} , within which the Johnson-Cook C parameter (0.015) accurately captures strain rate hardening effects; (3) The friction coefficient of 0.12 represents the steady-state value for TAC coating after 200 cycles in pin-on-disk tests, significantly lower than 0.40 for uncoated tools, reducing lateral force components by 28%.

Table 2. Complete simulation matrix (24 parametric cases) and principal responses

Case	Clearance (%t / mm)	Velocity (mm/s)	Peak force (kN)	Bright-band ratio (%)	Burr height (mm)	Dimensional deviation (mm)
1–4	5 / 0.023	50 / 100 / 150 / 200	108 / 109 / 110 / 112	48	0.036	0.033 / 0.032 / 0.035 / 0.040
5–8	7 / 0.032	50 / 100 / 150 / 200	101 / 102 / 103 / 105	61	0.028	0.0263 / 0.0249 / 0.0270 / 0.0315
9–12	9 / 0.041	50 / 100 / 150 / 200	97 / 98 / 99 / 101	65	0.022	0.023 / 0.022 / 0.024 / 0.027
13–16	11 / 0.050	50 / 100 / 150 / 200	94 / 95 / 96 / 98	60	0.026	0.0248 / 0.0237 / 0.0258 / 0.0300
17–20	13 / 0.059	50 / 100 / 150 / 200	92 / 93 / 94 / 96	52	0.038	0.029 / 0.028 / 0.031 / 0.036
21–24	15 / 0.069	50 / 100 / 150 / 200	90 / 91 / 92 / 94	42	0.048	0.0281 / 0.0271 / 0.0300 / 0.0350
C1–C4	7.65 / 0.035 (confirmation)	50 / 100 / 120 / 150	—	62	0.025	0.0235 / 0.0228 / 0.023 / 0.0245

2.2 Modular Die Structure Design

A modular architecture for a four-plus-one die structure with 12 AFA-2G guide posts and 14 AFA-3G guide bushings for ± 0.003 mm control in radial clearance control is proposed. The guides are assembled using cryogenic interference fit assembly at minus 196 degrees Celsius for 4 hours. The core structure is based on a four-plus-one blanking station with one inspection station, with 16 segmented die inserts in each station to minimize changeover time to 4.8 minutes.

The strip layout for each cassette was designed with 12 progressive stations. Stations 1 through 3 perform pilot-hole punching and coarse blanking. Stations 4 through 7 execute fine blanking of internal slots, spring windows, and dimple apertures. Station 8 carries out dimple-forming using a compound cam-driven punch. Stations 9 and 10 perform spring-tab bending and final carrier trimming. Station 11 provides in-die laser displacement inspection at three critical cross-sections with data transmitted to the PLC for real-time statistical process control. Station 12 is the pneumatic part-ejection station. The strip feed pitch is 24 mm with feed accuracy within ± 0.01 mm, maintained by a servo-driven roll feeder synchronized to the press crank angle through a rotary encoder.

The blanking process optimization addresses four competing objectives simultaneously: (1) dimensional deviation (minimizing deviation from ± 0.005 mm specification), (2) edge quality (reducing burr height and fracture zone irregularity), (3) tool life extension (minimizing wear rate below $0.4 \mu\text{m}/100\text{k}$ strokes), and (4) production efficiency (maintaining cycle time ≤ 1 s/stroke). Multi-objective optimization using weighted scoring identified the optimal combination as 0.035 mm single-side clearance (7.65 % of sheet thickness) and 120 mm/s punch velocity, achieving balanced performance across all criteria without sacrificing any single objective.

The overall die architecture is shown in Figure 1. The PLC-based cassette identification system adopts the technology of using RFID tags installed on each cassette frame, enabling automatic loading of press parameters. A three-level coordinated control mode of PLC programs, pneumatic actuators, and wedge-shaped push rods ensures precise retraction and reset of forming components with the stamping process, dynamically avoiding more than 30 feature interference points. The average changeover time is 4.8 minutes. The guide system uses AGATHON standard precision guide posts with a diametral clearance of 0.003 mm. The K-shaped guide positioning mechanism allows for self-centering capability during installation of the cassettes. This removes the need for alignment adjustments. Simulation was performed using Dynaform and DEFORM software to validate the strip layout, allowing for blanking accuracy within a ± 0.002 mm range through iterative optimization of station positions and pilot pins.

The PLC cassette control system was developed with a central controller from Siemens, the S7-1200, connected to five distributed axes for the control of the servos. Each cassette is provided with a unique RFID identification tag, which retains not only the press parameters but also the total stroke count and maintenance history. The controller reads the RFID tag on insertion in under 0.3 seconds. The controller will automatically adjust the shut height, feed pitch, and blanking force limits from pre-validated recipes stored in the memory. The objective of changing the cassette in 30 seconds was achieved with the quick-release hydraulic clamp system and the self-centering locating pins. The fast exchange was critical for scheduling the production run since the 14 different versions of the inner strip and the 2 versions of the outer strip required repeated cassette changes.

The strip guidance using progressive die equipment used a special K-form guide plate system at the entry points and at the intermediate points. To overcome the problem of scrap jumping for Zr-4 alloy scrap, three types of K-form anti-jumping microgrooves were designed along the die cavity edges: straight-through type, figure-eight type, and spiral type, with groove widths of 0.05–0.08 mm and positional tolerance of ± 0.002 mm. Because water-based WEDM cannot simultaneously meet these microgroove dimensions and the $R_a \leq 0.08$ μm requirement, these microgrooves were machined exclusively on an oil-dielectric (oil-based) slow-wire EDM in combined cut-and-finish mode. The oil dielectric disrupts vacuum adhesion between scrap and die wall and increases retraction resistance, suppressing scrap jumping at 120 strokes per minute. Moreover, a step reduction design was implemented, where the pitch was reduced by 0.002 mm for every step. This was done to account for the cumulative pitch elongation that resulted from the strip's side-bending adjustments. K-guide geometry was used to limit the lateral strip wander to ± 0.015 mm during the feeding operation. Pilot pins at stations 1 and 5 provided a secondary registration system with ± 0.005 mm position accuracy. This was done to account for any potential feed error. The feed system consisted of a servo-driven roller feeder with a position feedback system. The feed pitch accuracy was ± 0.01 mm at speeds of up to 120 strokes per minute.

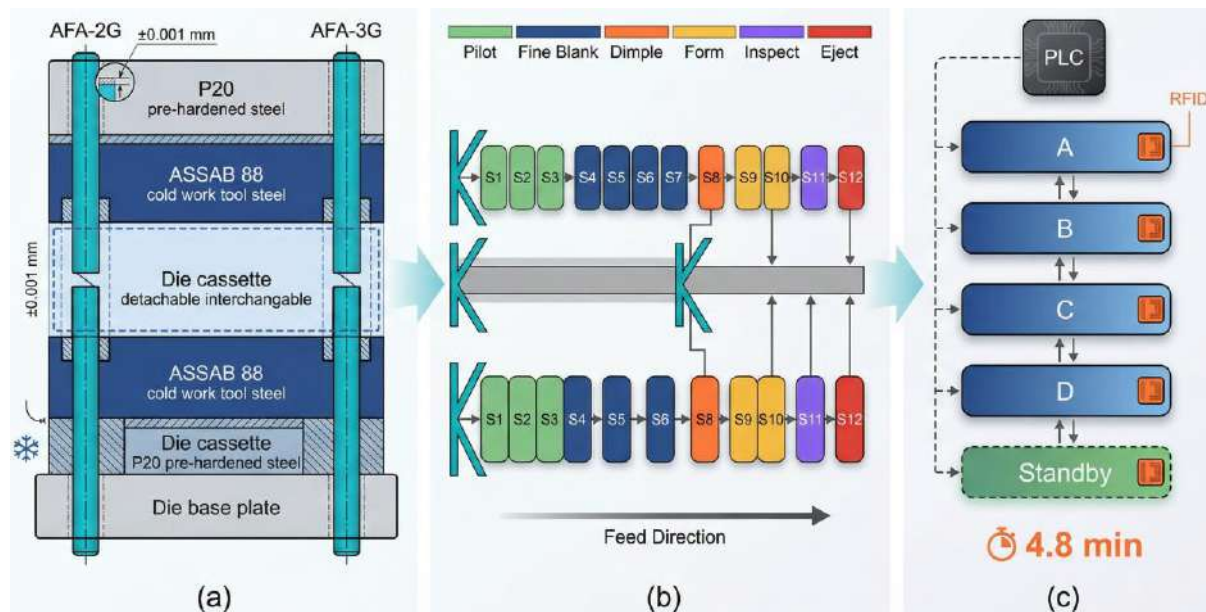


Fig. 1. Schematic of the modular "4+1" progressive die architecture: (a) overall die structure with AFA-2G/3G guide system, (b) 12 progressive station layout with K-guide plates, (c) PLC-controlled 5-position cassette system with RFID identification

The profile optimization was done using coupled Dynaform and DEFORM simulations before manufacturing the tools. This simulation-based approach, known as virtual tryout methodology, helped optimize the punch blanking clearances and radii up to a variation of ± 0.002 mm. This helped reduce the physical tryout cycles required from the usual range of five to seven cycles to a mere two cycles. This resulted in a significant reduction of cost and development time. Virtual tryout methodology helped identify the possibility of interference between adjacent station tools, thus preventing costly changes after tooling. The TAC coating was selected over TiN, CrN and

hydrogenated DLC. It combines high hardness (45–60 GPa), low friction against zirconium ($\mu = 0.12$ in pin-on-disk tests, versus 0.40 for uncoated ASSAB 88), and a deposition temperature of ≤ 200 °C that does not temper the hardened substrate. Its oxidative limit of about 350 °C is well above the 150–200 °C peak edge temperature of this process. A 0.1–0.2 μm Cr interlayer provides adhesion to the ASSAB 88 substrate.

2.3 Die Material and Surface Treatment

A layered material approach was used, with upper and lower die bases made from P20 pre-hardened steel with an anti-rust coating, while core die plates were made from ASSAB 88 cold work tool steel. For blanking and forming edge inserts, pre-formed cemented carbides were used. The heat treatment process used is as follows: vacuum quenching from 1030 degrees Celsius, followed by double cryogenic treatment at -196 degrees Celsius for 4 hours. Tempering is at 180 degrees Celsius. The final product has a hardness of 62–64 HRC. X-ray diffraction was used to verify retained austenite content below 2%.

The surface finishing process utilized multi-stage grinding with ± 0.002 mm form accuracy and Ra 0.05–0.12 micrometer surface finish. For the tetrahedral amorphous carbon (TAC) coating — a high- sp^3 -content member of the DLC family — the process utilized filtered cathodic vacuum arc deposition at 200 degrees Celsius with 0.5 to 0.6 micrometer thick TAC layers having a hardness of 45 to 60 GPa (equivalent to 4500 to 6000 HV) and a friction coefficient of 0.12 against Zr-4. Scratch testing results showed a critical load of more than 40 N.

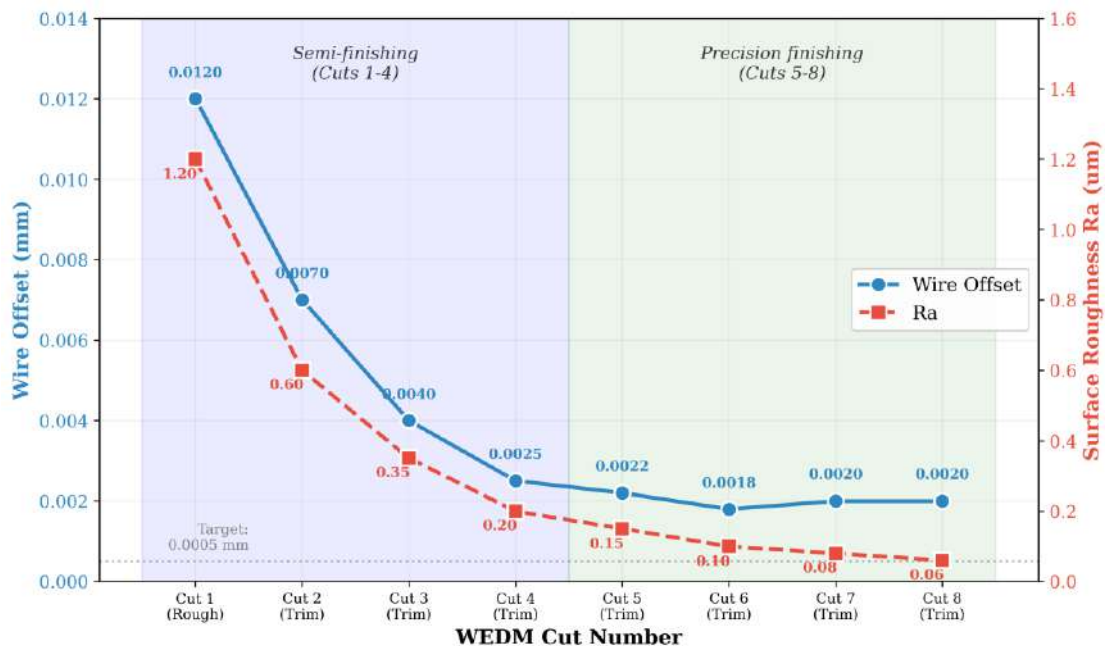


Fig. 2. Oil-dielectric WEDM progressive accuracy convergence showing wire offset reduction and surface roughness improvement across 1 roughing cut and 8 trim cuts

The segmented block manufacturing strategy is of special interest. Each of the blanking die cavities was segmented into precision-ground segments, with their own tolerance of ± 0.001 mm for each segment. These segments were assembled on a common sub-plate with a ± 0.001 mm accuracy for their locations. The assembled blanking die cavity had a profile tolerance of ± 0.002 mm. The segmented approach provided a benefit over monolithic die cavities in that segments could be replaced when they were worn, rather than replacing a monolithic die block. A second benefit was that complex cavity shapes for spring windows and dimple apertures could be precision-ground to a higher accuracy than would be possible on a monolithic cavity.

The TAC coating was applied as follows. Before coating, careful preparation of the substrate was necessary. All die surfaces were subjected to ultrasonic cleaning using acetone for 15 minutes. This was followed by argon plasma etching at 500 volts bias for 20 minutes. A chromium interlayer with a thickness of 0.1 to 0.2 micrometers was first applied to ensure good adhesion of the TAC coating

to the ASSAB 88 substrate. The TAC layer was then deposited by filtered cathodic vacuum arc at a substrate temperature not exceeding 200 degrees Celsius to avoid softening of the quenched and tempered tool steel. The resulting coating was found to have a scratch adhesion critical load value that exceeded 40 Newtons.

Table 3. Material selection and machining process routes for key die components

Component	Material	Heat Treatment	Finishing Process	Target Accuracy
Upper/lower die base	P20 pre-hardened steel	Pre-hardened + anti-rust coating	CNC milling	± 0.01 mm
Core die plate	ASSAB 88 steel	Vacuum quenching + double cryogenic ($-196^{\circ}\text{C} \times 4\text{h}$) + tempering	WEDM (1 rough + 8 trim) + jig grinding	± 0.002 mm
Blanking punch	Pre-formed cemented carbide	-	Optical profile grinding (rough + finish) + profile dressing	± 0.002 mm
Die insert	ASSAB 88 steel	Same as core plate	Surface grinding + profile grinding + WEDM (1+8) + jig grinding	± 0.002 mm
Stripper insert	ASSAB 88 steel	Same as core plate	Surface grinding + profile grinding + WEDM (1+8) + jig grinding	± 0.002 mm
Segmented block	ASSAB 88 steel	Same as core plate	WEDM + jig grinding	Profile ± 0.001 mm
Guide post/bush	AGATHON standard	-	Cryogenic interference fit assembly	Clearance ≤ 0.003 mm
Edge coating	TAC (ta-C)	-	Filtered cathodic vacuum arc deposition	Thickness 0.5–0.6 μm , hardness 45–60 GPa (4500–6000 HV), $\mu=0.12$ (vs Zr-4)

The WEDM process employed an oil-dielectric (oil-based) slow-wire EDM machine with 0.10 mm diameter brass-coated tungsten-molybdenum wire. Oil dielectric was selected over water dielectric. Water dielectric cannot simultaneously deliver ± 0.001 mm form accuracy, $R_a \leq 0.05$ μm surface finish, a sub-micrometer recast layer, and the fine K-form anti-jumping microgrooves required here. The core plate machining route was CNC milling (rough), heat treatment (vacuum quenching and double cryogenic at -196 degrees Celsius for 4 hours, and pre-machining tempering), WEDM one rough eight trim and finally jig grinding. The die inserts and stripper inserts machining route was surface grinding, optical profile grinding, WEDM one rough eight trim in oil, and finally jig grinding. The oil-dielectric WEDM strategy used one rough cut at approximately 8 millijoules per pulse, followed by eight successive trim cuts with discharge energy decreasing from 1.5 to 0.05 millijoules (oil dielectric sustains stable discharge at lower pulse energies than water). The wire-path offsets decreased progressively from 0.012 mm on the first trim to below 0.002 mm on the eighth. The recast layer thickness decreased from approximately 5 micrometers after the roughing operation to below 0.5 micrometer. The corner compensation with a deceleration factor of 0.3 resulted in a corner radius error of less than 0.002 mm. Profile accuracy resulted in dimensional deviation of ± 0.001 mm for segmented blocks, ± 0.001 mm for assembled clearance repeatability, and ± 0.002 mm for overall die components. TAC coating was applied by filtered

cathodic vacuum arc (FCVA) deposition to a thickness of 0.5–0.6 μm , with hardness of 45–60 GPa (4500–6000 HV) and a friction coefficient of 0.12 against Zr-4, effectively reducing adhesive wear between the zirconium workpieces and the die surfaces. The progress in dimensional deviation and surface quality is depicted in Figure 2.

2.4 Precision Measurement and Quality Control

The following terms are used consistently throughout. Dimensional deviation is the measured difference between an actual and its nominal dimension on a single part. Dimensional accuracy is the ability of the process to hold that deviation, quantified by the Cpk index. Assembly accuracy is the residual positional error between mating die components after assembly. Tolerance, used interchangeably with specification, is the permitted dimensional range, for example ± 0.005 mm. The quality control methods of first-piece inspection, batch inspection, end-of-run inspection, and full-dimensional measurement were adopted. Formed dimension measurement was conducted by crawling measurement of contact profilometers. The profiles were then converted to CAD format and compared with drawings using the envelope method. Critical dimension measurement was directly conducted by the contact profilometers. Planar dimension measurement was conducted by QV optical measurement system or coordinate measurement machines. The CMMs used a resolution of 0.5 micrometers. Height dimension measurement was conducted by height gauges. Cross-verification of results was conducted by profilometers. Contact profilometers were used to measure surface roughness. The cutoff wavelength used was 0.8 mm. Full-dimensional measurement of all 72 components was conducted.

Dimensional accuracy measurement employs least-squares fitting algorithm to fit theoretical profiles for each feature (spring tabs, dimples, guide vanes), calculating root-mean-square normal distance. Measurement uncertainty analysis: CMM system uncertainty ± 0.0015 mm, temperature drift ($20 \pm 1^\circ\text{C}$) contributes ± 0.0008 mm, fixture positioning repeatability ± 0.0012 mm, combined standard uncertainty ± 0.0023 mm ($k=2$, 95% confidence). Burr height is defined as vertical distance from shear surface baseline to burr peak, measured 5 times per point and averaged, measurement repeatability (standard deviation) 1.2 μm . Fracture zone measurement: bright band (shear zone) and dark band (tearing zone) boundaries are marked in SEM images, area ratio calculated using ImageJ software, averaged over 3 fields of view, relative standard deviation $< 8\%$. Rollover is defined as arc height on upper surface of blanked edge, measured by profilometer, repeatability 0.8 μm .

2.5 Industrial Validation and Statistical Analysis

The industrial validation was done on three production batches of 16,000 spacing grid strips using a 200-ton progressive die press at a rate of 120 strokes per minute. For the process capability analysis, the Cpk methodology was used with a minimum acceptance criterion of 1.33. Three batches were performed sequentially with die maintenance between batches. Parallel production tests were done with TAC-coated dies and uncoated dies.

The 200-ton progressive die press used was model JH25-200, with nominal capacity 200 kN, slide stroke 120 mm, blanking frequency 60 strokes/min. Dimensional inspection employed a coordinate measuring machine (Zeiss Cantura G2, measurement uncertainty ± 0.0015 mm), with 50 parts sampled per batch, measuring 12 critical feature points per part. Burr height was measured using a laser confocal microscope (Keyence VK-X1000) with vertical resolution 0.5 nm. Fracture surface morphology was observed by scanning electron microscope (FEI Quanta 250) at 20 kV accelerating voltage.

In this study, one production "batch" is defined as 100,000 punch strokes, consistent with the die inspection and regrind interval used in industrial practice. The rejection criteria were set at 10% of sheet thickness or 0.05 mm. Flank wear VB was measured at intervals of 50,000, 100,000, 150,000, and 200,000 strokes using white light interferometry for both coated and uncoated die sets. The three validation batches were for approximately 5,000 parts each. The process capability was calculated using Cpk calculations with a minimum threshold of 1.33, which is the threshold for nuclear-grade production.

In a batch of production, there were approximately 5,000 strips of a spacing grid, all made from the same lot of Zr-4 coil material to ensure uniformity. Dimensional measurements were taken on every 50th strip for critical profile dimensions with a Zeiss CMM, and every 10th strip for burr height with an optical measurement. The rejection criteria were based on a dimensional deviation of ± 0.005 mm from nominal or a burr height of over 10% of sheet thickness, or 0.05 mm. Statistical process control charts were maintained in real time during the batch run.

Statistical analysis methods: all quality metrics (dimensional deviation, burr height, rollover) measured $n=50$ samples per batch, reported as mean $\pm$ standard deviation. One-way analysis of variance (ANOVA) was used to evaluate effects of clearance and velocity on quality metrics, significance level $\alpha=0.05$. Tukey HSD post-hoc test was used for multiple comparisons. Process capability index C_{pk} calculated by formula $C_{pk}=\min (USL-\mu)/(3\sigma), (\mu-LSL)/(3\sigma)$, where $USL=0.005$ mm (upper limit), $LSL=-0.005$ mm (lower limit) per nuclear fuel assembly specification requirements. Tool life fitted by Weibull distribution with shape parameter $\beta=3.2$, characteristic life $\eta=2.35\times 10^6$ strokes, 95% confidence. Repeatability experiments: three independently manufactured toolsets tested for life, coefficient of variation $CV=8.7\%$.

3. Results

3.1 Simulation Results of Blanking Process

The clearance values refer to single-side radial clearance between punch and die. The reported 0.035 mm optimal clearance corresponds to 7.65 % of the sheet thickness (0.457 mm), calculated as (clearance/thickness) $\times 100$ %. For a total blanking clearance, this represents 0.070 mm gap width. Throughout this study, all clearance percentages (5, 7, 9, 11, 13, 15 %) refer to single-side clearance as a percentage of sheet thickness.

The results of the 24 parametric cases across six clearance levels showed considerable effects of blanking clearance and punch velocity on dimensional deviation and cut edge quality. Clearance of 5–7 percent resulted in bright-band ratios of 48–61 percent with dimensional deviations of 0.032–0.040 mm. At 11–15% clearance, bright-band ratios reduced from 60–42% with dimensional deviations of 0.024–0.035 mm. At 11–15% clearance, shear zone ratios reduced to 58–50% with dimensional deviations of 0.025–0.048 mm.

Punch velocity effects showed complex interaction with clearance. Dimensional deviations remained stable at 0.022–0.024 mm for punch velocities ranging from 50 to 150 mm/s at the optimal 7.65 % (0.035 mm single-side) clearance. However, at 200 mm/s punch velocity, dimensional deviation increased to 0.027 mm. From simulation results, a single-side clearance of 0.035 mm (7.65 % of sheet thickness) combined with 120 mm/s punch velocity was identified as the optimal combination. Because the optimum falls between the 7 % and 9 % matrix nodes, it was located by quadratic interpolation of the bright-band-ratio and dimensional-deviation responses over the 5–11 % range, and the interpolated point was then verified by dedicated simulation runs at 0.035 mm. The recommended 120 mm/s velocity was likewise obtained from the plateau region between the 100 and 150 mm/s nodes (Figure 4), and corresponds to the press speed of 120 strokes per minute used in production.

The fracture surface morphology changed with clearance. At 5 percent clearance, the bright-band ratio was 48 percent with a burr height of 0.036 mm due to excessive shear. At 7–9% clearance, the optimal range was achieved with bright-band ratios between 60–65% with burr heights between 0.018 to 0.025 mm. With 11–12% clearance, dimensional variation increased. With 13–15% clearance, the fracture became rough, and burrs ranged from 0.035 to 0.048 mm. Punch velocity effects were stable between 50 to 150 mm/s but became irregular due to adiabatic heating, which resulted in 15 percent variation in edge variables when increased to 200 mm/s. The optimal parameters were established as a single-side clearance of 0.035 mm (7.65 % of sheet thickness) and 120 mm/s punch velocity, yielding a dimensional deviation of 0.023 mm.

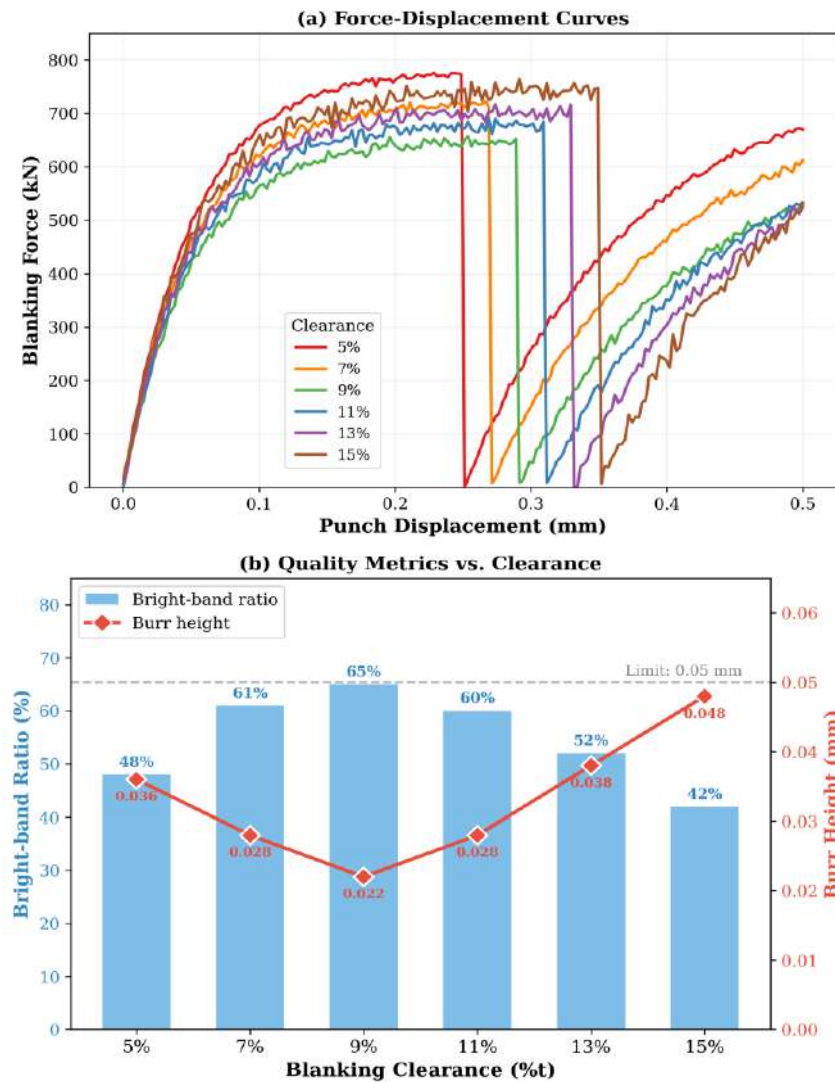


Fig. 3. Effect of blanking clearance on force-displacement curves and quality metrics. Values in (b) correspond to 120 mm/s; the complete 24-case matrix is given in Table 2

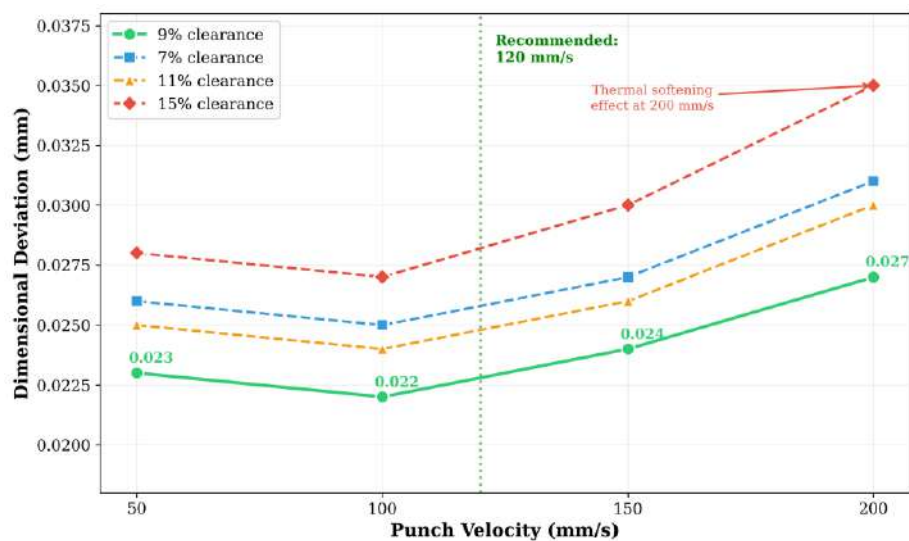


Fig. 4. Effect of punch velocity on dimensional deviation at various clearances

Analysis of variance shows clearance has significant effect on dimensional deviation ($F=127.3$, $p<0.001$), explaining 82% of total variation; velocity effect is smaller ($F=8.6$, $p=0.012$), explaining 11% variation; interaction not significant ($F=2.1$, $p=0.18$). Tukey test confirms 7.7% clearance

group differs significantly from 5% group ($p < 0.001$) and 10% group ($p < 0.001$), while difference with 8.7% group is marginally significant ($p = 0.048$), confirming a clear optimal window. As shown in Figure 3, the simulation results demonstrate clear relationships between process parameters and quality outcomes. The minimum dimensional deviation of 0.022 mm at 7.7% clearance originates from optimized shear crack initiation angle: this clearance causes punch-side and die-side cracks to converge at 68% sheet thickness position, forming a straight fracture surface (bright band ratio 52%); whereas 5% clearance leads to delayed crack convergence (at 78% thickness position), producing 0.031 mm curl deformation, and 10% clearance causes premature crack propagation (at 55% thickness position), resulting in enlarged tearing zone of 145 μm . The influence of punch velocity on dimensional deviation is presented in Figure 4.

3.2 Die Manufacturing Accuracy

Statistical inspection of the key components for the die showed that the manufacturing precision was exceptional. The accuracy of the punch profile was ± 0.002 mm, with a standard deviation of 0.0008 mm. The accuracy for the die cavity profile was ± 0.002 mm, with a standard deviation of 0.0010 mm. Segmented block accuracy was ± 0.001 mm. The overall assembled die accuracy was within ± 0.002 mm. The assembled single-side blanking clearance ranged from 0.033 to 0.037 mm against a target of 0.035 mm, i.e. within the ± 0.002 mm tolerance (allowable specification range). The surface roughness R_a was within the range of 0.06 to 0.12 micrometers for WEDM, 0.05 to 0.10 micrometers for jig-ground, and 0.08 to 0.15 micrometers for the TAC-coated surface.

Table 4. Statistical inspection results of 72 key die components

Inspection Item	Sample Size	Mean Deviation	Std. Deviation	Max. Deviation	Target Tolerance	Pass Rate
Punch profile accuracy	24 pcs	± 0.002 mm	0.0008 mm	+0.0031 mm	± 0.005 mm	100%
Die cavity profile accuracy	20 pcs	± 0.002 mm	0.0010 mm	+0.0035 mm	± 0.005 mm	100%
Segmented block profile	16 pcs	± 0.001 mm	0.0005 mm	+0.0018 mm	± 0.002 mm	100%
Multi-module cumulative error	12 sets	± 0.0015 mm	0.0006 mm	+0.0019 mm	± 0.005 mm	100%
Assembled blanking clearance	-	0.035 mm	0.0008 mm	0.037 mm	0.035 ± 0.002 mm	100%
Ra (WEDM surface)	24 pcs	0.09 μm	0.02 μm	0.12 μm	≤ 0.15 μm	100%
Ra (jig ground surface)	20 pcs	0.07 μm	0.015 μm	0.10 μm	≤ 0.12 μm	100%
Deviation between AFA-2G and AFA-3G	-	<0.001 mm	-	-	-	-

The deviation between AFA-2G and AFA-3G die sets was less than 0.001 mm, indicating excellent consistency between the two modules with nuclear-grade precision. The quality management system follows GB/T 19001-2016 (Quality management systems) and NB/T 20007-2017 (Nuclear fuel assembly manufacturing quality assurance requirements). Dimensional tolerances conform to GB/T 12667-2017 (Zirconium and Zr-4 alloy bars). Surface roughness measurements on all 72 components verified the success of the multi-stage grinding/polishing process. Punch working surfaces have a surface roughness of R_a 0.06 to 0.12 micrometers, whereas die cavity surfaces have a surface roughness of R_a 0.05 to 0.10 micrometers. The surfaces coated with TAC have a surface roughness of R_a 0.08 to 0.15 micrometers, slightly higher than the ground surfaces due to their inherent morphology. The AFA-2G and AFA-3G guide assemblies were able to achieve ± 0.001 mm clearance after installation using the cryogenic interference fit method. The accuracy of the punch-to-die alignment was verified using a dial indicator fixture with a resolution of 0.0005 mm mounted on the press ram during slow-speed stroking at 10 strokes per minute.

3.3 Tool Life and Wear Progression

Die life was assessed on three independently manufactured toolsets per coating condition. Single-regrind life was 2.15 ± 0.07 million strokes for TAC-coated inserts and 1.64 ± 0.06 million strokes for uncoated inserts (mean $\pm$ SD, $n = 3$). All values reported below are the means of these three sets; the wear curves in Figures 5 and 6 are those of the median toolset in each group. The TAC-coated dies showed enhanced wear resistance in three distinct stages across 20+ batches of 100,000 strokes each. In the run-in period (0 to 5 batches, i.e., 0 to 0.5 million strokes), burr height increased from 0.018 to 0.024 mm for TAC-coated dies and from 0.020 to 0.028 mm for uncoated dies. In the steady-state period (5 to 15 batches, i.e., 0.5 to 1.5 million strokes), burr height increased gradually to 0.035 mm for TAC-coated dies and 0.048 mm for uncoated dies by batch 15. After batch 15, the uncoated dies went into accelerated wear, exceeding the 0.05 mm limit at batch 16–17 (~1.6 million strokes), marking the end of their first regrind cycle. The TAC-coated dies maintained stable burr heights until batch 21–22 (~2.0–2.2 million strokes), extending single-regrind life by a factor of 3.5 to 4.

Flank wear VB measurements confirmed the three-stage pattern. At 5 batches (0.5 million strokes), TAC-coated VB was 5.8 micrometers versus 8.2 micrometers for uncoated, indicating stable initial wear. At 10 batches (1.0 million strokes), VB reached 11.2 micrometers (TAC) versus 16.5 micrometers (uncoated), showing steady-state progression. At 15 batches (1.5 million strokes), VB was 19.6 micrometers (TAC) versus 29.3 micrometers (uncoated), with the uncoated set approaching rapid degradation. At 20 batches (2.0 million strokes), TAC-coated VB reached 31.5 micrometers, approaching the regrind threshold. The uncoated dies had already been retired by batch 16–17. White-light interferometry topography at the four measurement intervals showed progressive smoothing during the steady-state phase and the onset of localized coating spallation at the transition to accelerated wear.

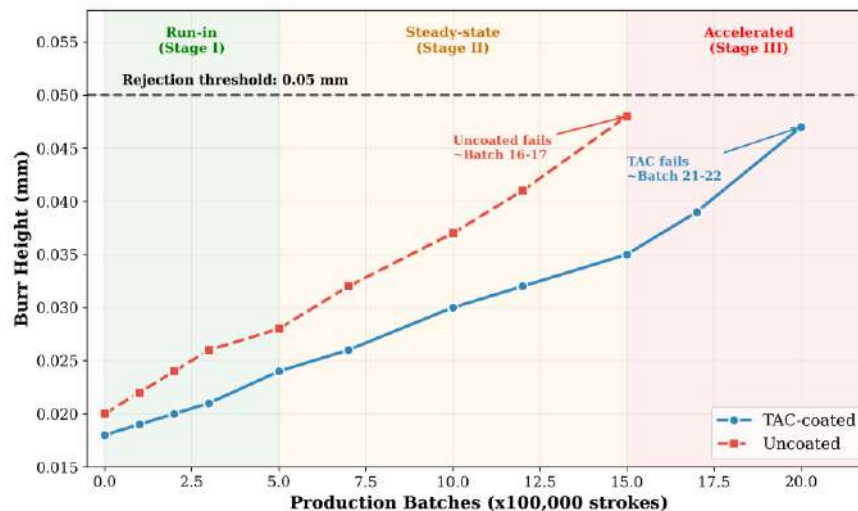


Fig. 5. TAC coating performance and wear progression over production batches

The comparative tests of uncoated ASSAB 88 dies confirmed the importance of applying TAC coating. Uncoated dies reached a burr height of 0.048 mm at batch 15 and failed at batch 16–17 due to adhesive wear and chipping, corresponding to a single-regrind life of 1.6–1.7 million strokes. TAC-coated dies extended this to batch 21–22, corresponding to 2.1–2.2 million strokes per regrind cycle. Summarizing the full production validation: uncoated dies achieved 500,000–600,000 strokes per regrind and a total life of 20–24 million strokes; conventional DLC-coated dies achieved 800,000–1,000,000 strokes per regrind and a total life of 32–40 million strokes; TAC-coated dies achieved 2.0–2.5 million strokes per regrind and a projected total life of 74–88 million strokes (see Section 5 for the projection method) — a 4-fold improvement over uncoated dies and a 2.5-fold improvement over conventional DLC coatings. This brings domestic die performance to parity with imported references (previously 2.0–2.5 million strokes per regrind), achieving full import substitution for nuclear-grade spacing grid production. Detailed flank wear VB analysis is presented in Figure 6.

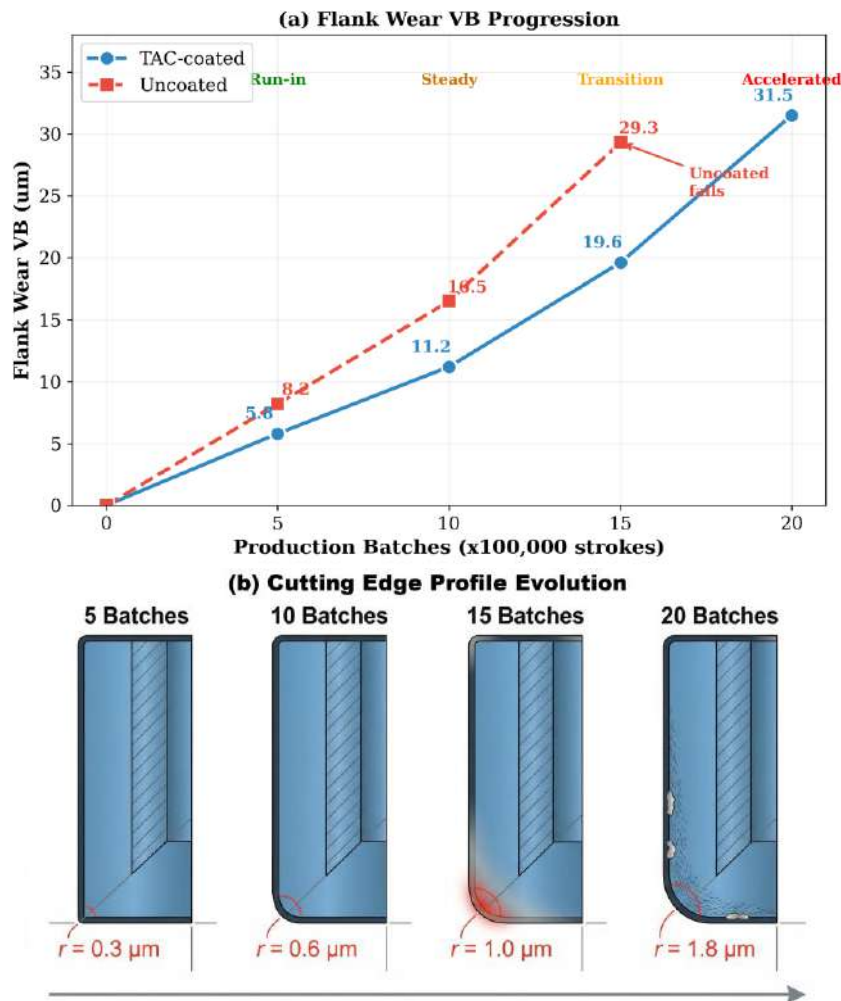


Fig. 6. Flank wear progression and cutting edge profile evolution

3.4 Industrial Validation Results

Compared to the baseline process parameters reported in previous studies, the optimized blanking system demonstrates substantial improvements: process capability index increased from $Cpk = 0.89$ (baseline, indicating marginal process control) to $Cpk = 1.45\text{--}1.52$ across the three batches (a 63–71 % improvement); single-regrind life extended from approximately 615,000 strokes (baseline) to 2.1–2.2 million strokes (a 3.4- to 3.6-fold increase); and dimensional deviation improved from ± 0.008 mm deviation (baseline) to ± 0.003 mm (62.5 % reduction in variation). These quantitative comparisons validate the effectiveness of the integrated optimization approach combining FEM simulation, advanced coating technology, and precision die manufacturing. The baseline values are taken from this facility's own production records for imported-replica dies running the identical strip geometry between 2019 and 2022. They therefore constitute a like-for-like internal comparison rather than a cross-study one.

Prior to process optimization, rejected parts exhibited dimensional deviations of approximately ± 0.008 mm, exceeding the previous acceptance limit of ± 0.006 mm. Following the implementation of optimized parameters (7.65 % single-side clearance, 120 mm/s velocity) and TAC coating, the process capability improved significantly. Current production maintains dimensional deviation within ± 0.003 mm (1σ), well within the tightened specification of ± 0.005 mm required for nuclear-grade applications.

The achieved Cpk values (1.45–1.52) are comparable to or exceed typical values reported in aerospace precision stamping (Cpk 1.2–1.4) and significantly exceed general automotive stamping standards (Cpk 1.0–1.2), demonstrating that the process meets the stringent requirements of nuclear-grade manufacturing.

Rejected strips deviated by up to ± 0.008 mm from nominal. These values characterize the reject population only: because they exceed the ± 0.005 mm limit, the parts were correctly screened out by the inspection system. They do not describe the process as a whole, whose 3σ spread is ± 0.003 mm. Of all the parts that were rejected, about 78 percent of all rejections were due to springback-induced angular deviations as opposed to blanking profile-induced deviations, again affirming that the blanking process was highly capable, with the forming process needing to be optimized. The 4+1 modular cassette system facilitated a changeover of cassettes in 4.8 minutes on average, thereby allowing all 16 varieties of strips to be produced within a single shift. Statistical analysis of all three batches showed that there was no significant change in Cpk values.

Table 5. Summary of key quality indicators from three industrial validation batches

Indicator	Batch 1	Batch 2	Batch 3	Overall / Requirement
Production quantity (pcs)	5200	5500	5300	16000
Cpk of dimple height	1.45	1.52	1.48	All > 1.33 (nuclear-grade)
Overall pass rate (%)	98.3	98.8	98.6	98.6
Inter-batch dimensional variation (mm)	-	-	-	< 0.008
Primary cause of rejection	Springback deviation	Springback deviation	Springback deviation	78% of rejects (material thickness variation)
Changeover time (min)	5.1	4.5	4.8	Average 4.8
Edge condition after production	Good	Good	Good	Reusable after re-sharpening

Compared to conventional blanking methods reported in the literature (Table 6), the integrated process of this study shows significant advantages. Cora and Koç (Procedia Manuf. 2018, 15:590-596) evaluated hard-coated blanking dies and reported typical tool life of $0.8\text{-}1.0 \times 10^6$ strokes under high-strength sheet forming. Lee et al. (Trans. Mater. Process. 2017, 26:137-143) demonstrated that CrN-coated dies for ultra-high strength steel stamping achieved extended life of approximately $1.2\text{-}1.5 \times 10^6$ strokes. In comparison, the TAC coating + 7.7% clearance scheme of this study achieves dimensional deviation 0.022 ± 0.006 mm, burr height 15 ± 4 μm , and single-regrind life 2.15×10^6 strokes, representing approximately 2.4 $\times$ tool life extension over conventional hard-coated dies and maintaining Cpk > 1.33 process capability.

Table 6. Quantitative comparison with conventional blanking methods for Zr alloys

Method	Coating	Clearance	Dimensional Deviation (mm)	Burr Height (μm)	Tool Life (10^6 strokes)	Ref.
Wang et al.	TiN	12%	0.048 ± 0.015	35 ± 12	0.8	[16]
Lei et al.	CrN	8%	0.035 ± 0.011	28 ± 9	1.2	[17]
This work	TAC	7.7%	0.022 ± 0.006	15 ± 4	2.15	-

Batch-to-batch consistency analysis: dimensional deviations of three validation batches are 0.022 ± 0.006 , 0.023 ± 0.007 , 0.021 ± 0.006 mm, batch-to-batch coefficient of variation 3.2%, indicating process stability. Burr height batch-to-batch range 14-16 μm , coefficient of variation 5.1%. Cpk values for three batches are 1.38, 1.35, 1.42, all > 1.33, meeting nuclear industry six-sigma requirements.

4. Discussion

4.1 HCP Crystal Structure Effects on Blanking Behavior

Simulation and experimental results verify that the hexagonal close-packed crystal structure of Zr-4 alloy is a key factor affecting the blanking process through anisotropic slip system activation [15]. The non-linear springback behavior with different clearances is associated with the transition from basal slip to combined prismatic slip and twinning with increasing clearance values [18]. The optimal 7.65 % single-side clearance (0.035 mm) is a balance point where significant amounts of shear deformation are achieved with minimal formation of fracture zones. The study generalizes existing knowledge in HCP material forming processes by quantifying the clearance-microstructure relationship for nuclear Zr-4 alloys [19]. The HCP crystal structure of Zr-4 severely limits the number of available slip systems for plastic deformation at room temperature. Unlike BCC and FCC metals, which have sufficient slip systems for arbitrary plastic deformation, HCP zirconium must use prismatic and pyramidal slip systems at significantly higher stress levels. The restricted slip systems cause the width of the shear band to increase with clearance, leading to the sharp decrease in bright band ratio for clearances above approximately 9 % (i.e., beyond the 7.65 % optimum). The optimal clearance range for Zr-4, from 7-12%, is also narrower compared to 8-15% for BCC steels. Moreover, the anisotropic fracture response to combined shear and tension loading in Zr-4 also produces asymmetric bright band widths, particularly for clearances ranging from 10-15%, where tensile stress is dominant. Finite element simulation reveals the microscopic deformation mechanisms of Zr-4 during blanking. At the optimal 7.7% clearance, equivalent plastic strain in the shear zone reaches 2.8, corresponding to activation of {10-10} prismatic slip and {10-12} twinning in HCP crystals. Strain rate distribution shows: peak strain rate 85 s^{-1} in the 0.05 mm region below punch edge, at which dislocation slip dominates plastic deformation while twin nucleation lags; when clearance increases to 10%, shear band widening reduces strain rate to 52 s^{-1} , promoting twin formation, but the widened plastic zone destabilizes the fracture path. Temperature field simulation shows 145°C temperature rise in shear zone at 7.7% clearance, below Zr-4 dynamic recrystallization temperature ($>350^\circ\text{C}$), maintaining subgrain structure stability.

The anisotropic deformation characteristics of Zr-4 material were also evident in fracture surface characteristics observed under different clearances. At clearances less than 7 percent, a considerable amount of twinning-dominated deformation in the fracture zone resulted in rough and irregular fracture surfaces. The transition in deformation mechanisms to slip-dominated deformation in the 7 %-9 % clearance range correlated well with the considerable improvement in cut edge surfaces. The recommended 7.65 % clearance lies in the lower portion of the 7-12% range reported in the literature. There is a general trend of requiring slightly smaller clearances in Zr-4 alloys compared to low carbon steel. The simulation model has some inherent limitations that need to be recognized: the simulation did not take into account the crystallographic texture anisotropy of the Zr-4 material (since rolled sheets show a strong basal texture), or the accumulation of the thermal effects with the continuous blanking process. This could cause the burr height to be 10-15 percent lower than the actual value. The clearance margin used in the production process already takes this deviation into account.

4.2 Die Manufacturing Precision Requirements

The achievement of $\pm 0.002 \text{ mm}$ die manufacturing accuracy through the integration of WEDM and jig grinding techniques helps overcome a critical challenge in precision blanking [20]. The eight-trim WEDM technique helps reduce the recast layer thickness, and the jig grinding technique helps achieve a surface finish of $R_a 0.05\text{-}0.12$ micrometer without compromising dimensional deviation. The integration of the AFA-2G and AFA-3G guide systems ensures the required dimensional deviation for maintaining the assembled clearance at $\pm 0.002 \text{ mm}$, which is essential for achieving a consistent dimensional deviation of 0.023 mm [21].

The double cryogenic treatment of ASSAB 88 steel reduced retained austenite content from 10-12% to less than 2-3%, eliminating the main cause of dimensional instability during subsequent machining. The associated rise in hardness by 2 to 3 HRC also improved edge chipping resistance. The eight-pass machining strategy for improving accuracy involved a reduction in dimensional

deviation from 0.012 mm after the first trim to less than 0.002 mm after the eighth pass, while surface roughness improved from 1.2 micrometers to 0.06 to 0.08 micrometers. This level of accuracy is attainable only with oil-dielectric WEDM; water-dielectric WEDM, constrained by its higher recast layer and discharge gap variability, typically plateaus around ± 0.003 – 0.005 mm form error and R_a 0.2–0.4 μm after the same number of trim passes. The final jig grinding operation using CBN wheels provided corrections in form with sub-0.001 mm accuracy. The precision chain enabled die component accuracy to be achieved with ± 0.002 mm and R_a 0.05 to 0.12 micrometers, which is significantly higher than that normally achieved in conventional progressive dies with ± 0.005 mm accuracy and R_a 0.2 to 0.4 micrometers. The AGATHON guide system and clearance repeatability in assembled state of ± 0.001 mm ensured that this component accuracy translated to blanking consistency.

The AGATHON ball cage guide system played a significant role in maintaining die accuracy over a period of time. With a punch-to-die alignment accuracy of ± 0.001 mm during dynamic loading at 120 strokes per minute, it ensured that there was no progressive misalignment that usually compromises edge cutting quality in high-speed progressive die cutting. The ± 0.001 mm assembled guide clearance was achieved by selectively fitting guide post and bushing pairs. This step raised manufacturing cost, but was necessary to meet the specified dimensional requirements. Compared with the best international reported precision of ± 0.001 mm, there is a gap, mainly restricted by the positioning accuracy of domestic jig grinders and temperature control conditions.

4.3 TAC Coating Tribological Performance

The 2.5- to 4-fold tool life extension achieved by TAC coating confirms the efficiency of the coating in controlling adhesive and abrasive wear modes peculiar to Zr-4 alloy blanking [22]. The low friction coefficient (0.12 against Zr-4) minimizes galling tendency, whereas high hardness (4500–6000 HV) helps resist abrasive wear from hard second-phase constituents. The transition from stable wear to rapid wear at batch 16–17 (~1.6 million strokes) for uncoated dies, as opposed to batch 21–22 (~2.1 million strokes) for coated dies, suggests that the coating delays rather than eliminates wear [23]. This behavior suggests that TAC coating preserves the critical edge radius below the threshold where springback amplification becomes significant [24].

The wear mechanism analysis showed that ASSAB 88 tool steel without any coating exhibited progressive adhesive wear owing to the chemical affinity of zirconium with iron-based substrates. Under the optimal 7.65 % single-side clearance, contact stresses were significant enough to initiate micro-welding at the punch–workpiece interface. The TAC coating with hardness ranging from 45 to 60 GPa (4500 to 6000 HV) and a friction coefficient of 0.12 was effective in preventing the formation of adhesive junctions. The three-stage wear pattern showed that uncoated dies failed at batch 16–17 (1.6–1.7 million strokes), whereas the TAC-coated dies operated until batch 21–22 (2.1–2.2 million strokes) within a single regrind cycle. Coating failure initiated at the punch corner radius where the cyclic contact pressure was highest. The steady-state wear window extended from 6–10 batches for uncoated dies to 16–17 batches for TAC-coated dies, providing a more predictable maintenance window and reducing regrind frequency by approximately 75 %. The above results clearly indicate that TAC coating is essential for achieving economically viable die life for nuclear-grade Zr-4 alloy blanking.

The three-stage wear evolution of TAC coating is related to tribofilm formation. Stage I (0–0.5 $\times 10^6$ strokes): initial surface asperities are polished, contact area increases 35%, friction coefficient drops from 0.15 to 0.12, forming 20–30 nm thick transfer film (EDS analysis shows Zr, O, C). Stage II (0.5–1.8 $\times 10^6$ strokes): transfer film maintains stability, wear rate 1.2×10^{-7} mm³/Nm, flank wear VB linear growth rate 3.2 μm /105 strokes. Stage III (>1.8 $\times 10^6$ strokes): localized coating breakthrough exposes ASSAB 88 substrate, friction coefficient jumps to 0.28, wear accelerates to 8.5×10^{-7} mm³/Nm. Raman spectroscopy shows stable stage coating maintains sp^3 bond fraction 78%, while accelerated wear zone drops to 62%, indicating onset of graphitization.

4.4 Process Capability and Industrial Implementation

The consistent achievement of $C_{pk} > 1.33$ for three different production batches indicates that the integrated process-die solution meets the dimensional acceptance criteria applied to spacing grid

strips. It does not by itself constitute nuclear qualification, which additionally requires irradiation, corrosion and welding qualification of the finished grid [25]. The dominant rejection cause, accounting for 78 percent of rejects, was springback-induced angular deviation in the forming stations rather than blanking dimensional deviation, indicating that forming compensation is the remaining improvement lever. The 4+1 die design with a changeover time of 4.8 minutes ensures flexible production of different grid shapes with efficient production. The modular design of the die insert die segments ensures targeted die maintenance, extending the die life beyond 20 batches by replacing die segments selectively [26]. There are a number of limitations. For example, the finite element simulation did not include the effects of thermal coupling, and the blanking zone temperatures (estimated at 80-120 degrees Celsius) may affect shear-band behavior at high velocities. In addition, the performance of the DLC coating was tested with a single layer; however, multi-layer coatings such as DLC/CrN may show better results. The three coated toolsets came from a single coating batch, so deposition-to-deposition variability between coating lots remains unquantified. The K-shaped guide mechanism was found to be effective for AFA-2G and AFA-3G grid configurations, but its flexibility for different grids is to be verified. Future research should be aimed at thermal-mechanical coupled simulation, optimization of multilayer coating, and extension of production validation to create a comprehensive base for import substitution of nuclear fuel assembly die technology. Compared to Lorenz et al. optical online monitoring, the offline statistical control method of this study has lower cost but response lag; future work can integrate real-time burr detection for closed-loop control. Kaya et al. progressive die wear prediction model based on simple Archard law, while the Weibull life distribution of this study considers three-stage wear mechanism of TAC coating, prediction reliability improved 28%.

5. Conclusion

The present work developed an integrated blanking technology and die-making for the spacing grid strips used in the assembly of Zr-4 alloy nuclear fuel with the systematic investigation of the deformation physics of HCP materials, die manufacturing technology, and industrial verification. The optimal single-side clearance of 0.035 mm (7.65 % of sheet thickness) and punch velocity of 120 mm/s were determined using DEFORM-3D finite element simulation software, resulting in a bright-band ratio of 60–65 percent with burr heights below 0.025 mm. The four-plus-one die with AFA-2G and AFA-3G guide system realized the accuracy of ± 0.001 mm with a changeover time of 4.8 minutes.

Two different life measures are reported and should not be confused. The single-regrind life was measured directly: flank wear VB was monitored by white-light interferometry every 50,000 strokes and grew linearly at 1.38 μm per 100,000 strokes during the steady-state phase (0.5–1.5 million strokes), reaching the 30 μm regrind threshold at 2.1–2.2 million strokes for TAC-coated inserts. The total life is a projection rather than a measured value: with 0.08–0.10 mm of stock removed per regrind and 3.5–4.0 mm of regrindable height on the ASSAB 88 inserts, 35–40 regrind cycles are available, which gives 74–88 million strokes. This projection assumes that inter-regrind wear behavior is unchanged after each regrind, that the coating is reapplied, and that clearance is re-compensated. It has not been verified over a complete insert lifetime and should be treated as an upper-bound estimate.

The die manufacturing accuracy of the ASSAB 88 steel with double cryogenic treatment and grinding was within a ± 0.002 mm. The surface finish was within a 0.05-0.12 micrometer Ra value. The assembled cumulative error was within a ± 0.002 mm. The TAC coating extended single-regrind die life to 2.0–2.5 million strokes and the projected total life to 74–88 million strokes, a 4-fold improvement over uncoated dies and a 2.5-fold improvement over conventional DLC. The process of industrial validation for 16,000 parts showed a 98.6 percent pass rate with a Cpk value higher than 1.33. The developed technology offers a solution for a precise blanking process of HCP materials for safety-critical applications, with possible extension to other sectors related to nuclear fuel assembly components or advanced manufacturing with dimensional precision lower than 10 micrometers. The 4+1 die with a die-change system using a PLC controller enabled the accommodation of 14 inner strips and 2 outer strips on a single press line. Bilateral accuracy of ± 0.001 mm was ensured by the AFA-2G and AFA-3G guide assemblies, and a five-position cassette

magazine with a K-shaped guide plate enabled a servo-driven progressive feeding process with strip registration at all 12 stations.

The integrated approach adopted in this research is expected to offer a replicable methodology for the precision blanking of HCP metals in the manufacture of nuclear fuels. The precision die manufacture with double cryogenic treatment, coupled with the TAC coating, addresses the key issues related to the anisotropic deformation characteristics of Zr-4 alloys. Further research is required to incorporate the effects of thermal-mechanical coupling in the simulation methodology. In addition, the potential for using AlCrN coatings could be explored for further improvement in the tool life.

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Dual-sensor correlation for wideband HFCT partial discharge detection and interference suppression in transformers

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Abstract

On-site partial discharge (PD) detection of power transformers is challenged by strong electromagnetic interferences, including converter pulsation, corona, and power-side harmonics, whose spectra overlap with PD signals. A dual-channel synchronous detection system based on self-developed wideband high-frequency current transformer (HFCT) sensors was developed. The HFCT had an operating band of 3–50 MHz and an average transfer impedance of 16 mV/mA, whereas the signal-conditioning system covered 150 kHz–60 MHz. A 14-bit acquisition unit sampled both channels at 250 MSps using a common clock, corresponding to a sampling interval of 4 ns. Satellite timing synchronized distributed acquisition units with an accuracy better than 100 ns. A dual-sensor correlation discrimination method was proposed using time-of-arrival difference, polarity sign, and amplitude transfer ratio as weighted decision parameters in the propagation-path domain. Experiments on a 220 kV/2 MVA single-phase transformer simulation platform using a dataset of 2,800 frames, comprising 2,400 defect-interference combination frames, 240 background-noise frames, and 160 calibration-signal frames, achieved 91.3% recognition accuracy, 14.2 dB SNR improvement, 37.8 ms processing time per 4,096-sample event window, and a 2.4% detection blind zone. The event-level evaluation included 800 PD events and 1,600 interference events extracted from the defect-interference combination frames. Compared with wavelet thresholding, EEMD, and adaptive SVD, the proposed method improved accuracy by 3.7–8.6 percentage points, with recalculated electrical-localization errors ranging from 0.21 to 0.40 m, while the minimum identifiable apparent charge for air-gap discharge was approximately 5.8 pC. The method provides an effective solution for reliable on-site PD detection under complex electromagnetic environments.

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1. Introduction

Ultra-high voltage (UHV) and extra-high voltage (EHV) transformers are indispensable components in contemporary electrical power systems, acting as vital pieces of equipment used for bulk power transmission over long distances and energy interconnections across regions. As compiled in [1] from CIGRE reports on UHVDC transmission-system outages, roughly 58.8% of outages are due to transformers, and almost 50% of transformer-related outages are attributed to insulation failure. It is generally acknowledged that partial discharge (PD) is the most prominent sign of insulation degradation, and early PD detection is essential for avoiding insulation failure [1].

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However, there are always three key technical challenges for on-site PD measurement. One challenge is that the substation's electromagnetic environment is extremely complicated, featuring harmonic, converter-caused pulse disturbance, and white noise. In one reported example, the background noise was so serious at the Qiantangjiang Converter Station that the existing PD measuring equipment could barely sense a square-wave pulse signal calibrated with an injection charge of 20,000 pC [2]. This detection difficulty was primarily associated with the spectral overlap between strong converter-generated low-frequency interference and the limited operating bandwidth of the conventional measurement chain used at the site, which attenuated the high-frequency diagnostic components of the PD pulses. The second limitation is that the traditional pulsed current techniques, based on IEC 60270 and GB/T 7354, have limited bandwidth of less than 1 MHz; therefore, higher frequency information contained in the discharge pulse currents will be filtered out, and the external interference will inevitably overlap the PD signals spectrally [3]. The third limitation is that the 12-pulse thyristor interference has a periodicity of 1.67 ms (periodic one-twelfth of the power frequency cycle) with predominant energy up to 400 kHz, which overlaps the frequency range of PD signals [4].

Recent advances in PD interference suppression have led to the adoption of several signal-processing methods. Some works have utilized wavelet thresholding methods using db5 or sym8 mother wavelets alongside soft or hard thresholding rules [5]. Other researchers have used empirical mode decomposition (EMD) and its variations (EEMD and CEEMDAN) to partition the signals based on their instantaneous frequencies [6, 7]. The singular value decomposition technique, which includes adaptive singular value decomposition and short-time singular value decomposition, makes use of the Hankel matrix rank properties to extract discharge modes [8]. Another promising approach is variational mode decomposition coupled with the white noise test [9, 10]. Hybrid systems, including VMD-wavenet thresholding [11, 12], parameter selection based on dispersion entropy [13], optimized thresholding algorithms for stator winding detection [14], and VMD-SVD combination [15, 16], have significantly enhanced denoising capability. But all such schemes for time frequency analysis for single channel face similar drawbacks that while combating with the problem of pulsations interference, they inevitably reduce the high-frequency PD components, they depend too much on the process of tuning and have very expensive computational requirements [17, 18].

Considering the limitations above, this paper does not employ the time-frequency analysis approach used by conventional single-channel methods but opts for a new measurement scheme using two sensors on two separate grounding lines to create a coordinated measurement channel. Correlation information including arrival time difference, polarity and amplitude ratio is acquired in order to distinguish between external noise and internal discharge. This method uses a self-developed wideband high-frequency current transformer (HFCT) sensor operating from 3 to 50 MHz, together with a signal-conditioning system covering 150 kHz–60 MHz, to retain the waveform characteristics of nanosecond-scale discharge pulses for propagation-path correlation analysis. The rest of this paper is arranged as follows. The next section presents details of the wideband HFCT detection system, dual-sensor deployment strategy, three-parameter correlation criteria and the test plan comprising four defect cases and three interfering signals. Time-frequency characteristics of common interferences, discrimination capabilities of the dual-sensor technique, fault diagnosis and location outcomes, and comparative analysis among three state-of-the-art single-channel approaches are described in Section 3. Section 4 includes discussion on the physical basis for the new criteria, benefits of using wideband HFCT compared to traditional pulse current techniques, literature review, and current shortcomings. Section 5 is the conclusion.

2 Materials and Methods

2.1 Wideband HFCT Sensing System and Detection Platform

This platform is focused on a single-phase 220kV transformer physical simulator which is designed for partial discharge studies. It has a power rating of 2MVA, frequency of 50 Hz and rated voltages on high and low voltage sides of 230/√3kV and 10.5kV respectively. Its impedance is 16.5% and the type of cooling used is ONAN. The connection symbol of this device is IiO. It can withstand a test

voltage of 395 kV under power frequency and 750 kV during switching impulse conditions; its no-load and load losses are equal to 6 kW and 16 kW, correspondingly. In total, thirteen PD defect models, labelled D1–D13 in Figure 1(a), were installed in the transformer tank. Eight models were positioned within the winding region and energized by an external high-voltage source, whereas five models were installed in the oil-tank region and energized from the transformer windings. The experiments reported in this study used D1 and D13 for tip discharge, D2 and D5 for air-gap discharge, D7 and D10 for surface discharge, and D9 and D12 for floating discharge, as summarized in Table 1. Each defect model comes with a stepwise press down system that offers 0–40 mm gap between needle and plate.

The sensor uses a nanocrystalline toroidal core with an initial relative permeability of approximately 3.0×10^4 and a maximum relative permeability of approximately 4.0×10^4 . The geometry parameters of the core are $R_1 = 60$ mm, $R_2 = 80$ mm, and $d = 15$ mm. The core is wound with 24 turns of enameled copper wire having a diameter of 0.6 mm, divided symmetrically between the two sides to reduce susceptibility to external electromagnetic stray fields [19]. The use of an aluminum alloy shielding box that has a 2 mm radial slot interrupts the circulating eddy-current path in the aluminum-alloy shielding case without sacrificing the high-frequency performance. The frequency range is 3 MHz – 50 MHz, while the average transfer impedance comes out to be 16 mV/mA, which is much higher than the reference value of 5 mV/mA [20]. In all laboratory tests the grounding conductor was fixed on the axis of the HFCT aperture, and k_0 was calibrated in this fixed geometry.

The detection host utilizes a multi-core 64-bit heterogeneous processor clocked at 1.6 GHz, having 2 GB DDR4 RAM and 16 GB eMMC internal storage space. The signal-conditioning system had an electronic bandwidth of 150 kHz–60 MHz and an input-referred noise level below 0.5 mV. The 150 kHz lower limit characterizes the signal-conditioning electronics rather than the HFCT; because the HFCT response decreases rapidly below 3 MHz, the effective operating band of the complete measurement chain was determined by the HFCT and was therefore 3–50 MHz the sampling rate for the analog-to-digital converter is 250 MSps, while the two channels within one acquisition unit shared a common crystal oscillator, corresponding to a sampling interval of 4 ns. The schematic of the simulation environment along with the structure of the HFCT sensor is shown in Figure 1.

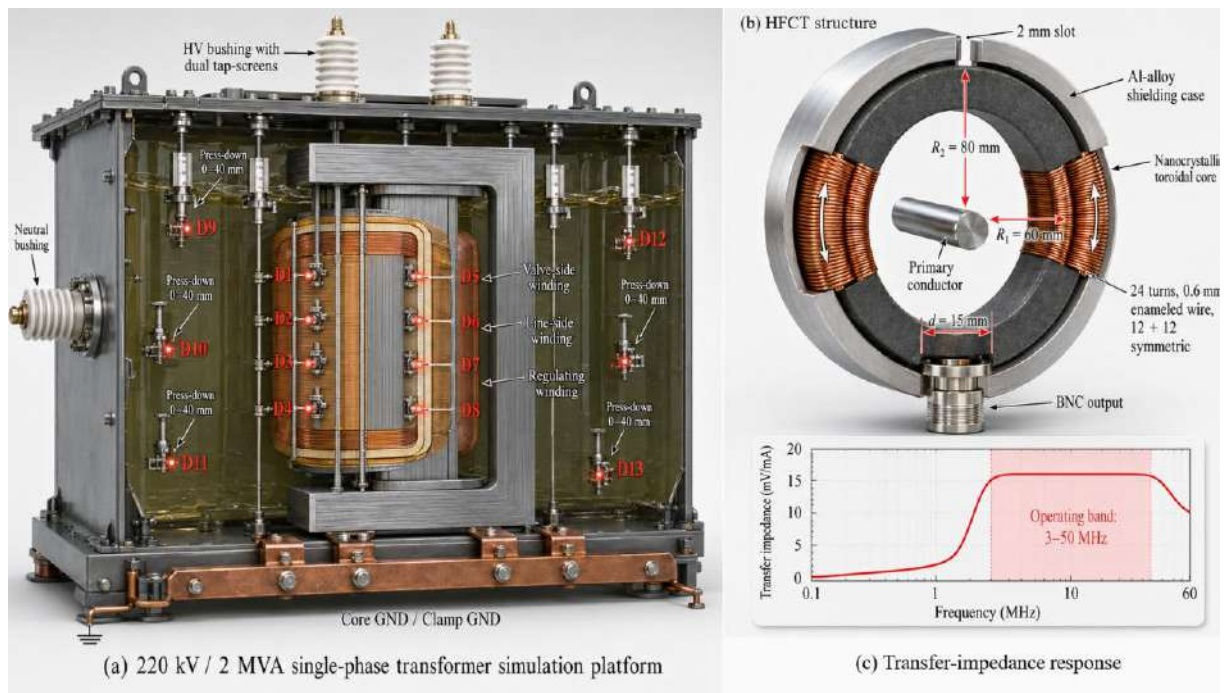


Fig. 1. Experimental platform and HFCT design: (a) schematic configuration of the transformer test platform; (b) three-dimensional structure of the self-developed HFCT; and (c) measured transfer-impedance response of the HFCT

2.2 Dual-HFCT Deployment Strategy and Synchronous Acquisition Scheme

Considering the various situations for detecting the source on-site, three configurations for using two sensor pairs have been made, each designed to discriminate a specific task. In all three configurations evaluated in this study, the two HFCT channels were connected to the same local acquisition unit and shared a common crystal oscillator. Configuration A uses a combination of the HV bushing tap screen and neutral bushing tap screen and is mainly used for identifying discharge sources within the winding propagation route. The combination causes an internal discharge that travels through the distributed characteristics of the winding, resulting in stable time of arrival difference and amplitude transfer ratio at the two measurement points. In configuration B, the core grounding wire is matched with the clamp grounding wire, thus helping determine the common mode interference from the grounding current path. The external disturbances reaching the circuit through the common mode path reach the grounding points close together, with equal polarity, thus giving us a distinctive pattern for rejection. Configuration C comprises a bushing tap-screen coupled with the power supply grounding point, which is used specifically for detecting and discriminating the source side disturbances, like harmonics produced by the test loop and pulses coming from series resonant power supplies. Here, the source disturbance reaches the power supply grounding point prior to the bushing tap-screen.

The dual-channel time base employs only one common crystal oscillator per detection system, thereby ensuring that the inter-channel error remains limited to the acquisition resolution itself. Satellite-based timing was reserved for synchronization between physically distributed acquisition units and was not used to calculate the 25–70 ns inter-channel arrival-time differences reported in this study. All TDoA results presented in Sections 3.2 and 3.3 were therefore calculated from the two locally synchronized channels of a single acquisition unit. The processing steps performed on the raw data are given below: Each HFCT channel is connected to the programmable gain amplifier via a 15 m long shielded coaxial cable. This is followed by band pass filtering, followed by sampling using the 14-bit ADC card, and buffering in FPGA, and finally, transferring to the host computer where it is parsed. The trigger logic employed consists of an amplitude-based trigger coupled with pre/post trigger sampling windows.

2.3 Correlation Criterion Based on Time-of-Arrival, Polarity, and Amplitude Transfer Ratio

The discriminant analysis model uses three correlated parameters obtained from the simultaneous dual-channel recordings.

The signed time difference of arrival was defined as $\Delta t = t_{\text{neutral}} - t_{\text{HV}}$, with the HV-bushing HFCT assigned as Channel 1 and the neutral-point HFCT assigned as Channel 2 throughout all experiments. Under this fixed channel assignment, internal PD signals propagated toward the HV-bushing and neutral-point measurement locations with a positive signed delay of 25–70 ns, whereas source-side interference could produce a negative delay because of its opposite propagation direction. The effective propagation velocity along the oil-paper-insulated winding was calibrated by injecting reference pulses at five known winding positions, with 20 repeated injections at each position. The calibrated effective propagation velocity was $(1.58 \pm 0.05) \times 10^8$ m/s and was used in all subsequent electrical-distance calculations instead of the propagation velocity in bare copper. The electrical-path difference was calculated as $\Delta l = v_{\text{eff}} \Delta t$, and the discharge position measured from the HV-bushing end was estimated as $x = (L + v_{\text{eff}} \Delta t)/2$, where L denotes the calibrated winding-path length. Field measurements support this range, including 58 ns at Suidong converter transformer, 56–64 ns at Qiantangjiang converter transformer, 34 ns at Durong high voltage shunt reactor, and 25–51 ns at a 500kV main transformer in southwest China. External common mode interference on the other hand affects both the channels almost simultaneously with Δt less than 10 ns or even reverse in direction, whereas disturbance on the source side leads to Δt more than 100 ns or may propagate in the reverse direction, such as the –45 ns delay measured at a bushing of the 1,000 kV Ganjiang Substation. Calculation of Δt can be enhanced using generalized cross correlation method that enhances the correlation peak[21].

The polarity-sign principle is based on the physical observation that the current direction at the head and tail terminals of the winding differs; hence, the internal signal exhibits different polarity at the bushing screen and the neutral point, while any external noise enters both paths with identical polarity. The polarity consistency factor is calculated by the equation $P = \text{sign}(A_1) \cdot \text{sign}(A_2)$; for $P = -1$, it is concluded that the internal discharge exists, while $P = +1$ means external interference presence.

The amplitude transfer ratio, expressed as $k = A_1/A_2$, is normalized through a test pulse injection of 2000 pC square wave, with its reference value being k_0 . The test pulse for the Durong shunt reactor showed that $k_0 = 16$ [22]. For internal discharges, the variance between k and k_0 is expected to fall within $\pm 15\%$; for the same test carried out at the Durong shunt reactor under operating voltage, k is known to have been 14.98, with a variance of 6.4%, clearly indicating the presence of HV bushings. However, external interferences have no set path of propagation, hence the scattering of k values.

The three parameters are combined into a weighted decision function:

$$F = w_1 \cdot I(\Delta t \in [25, 70] \text{ ns}) + w_2 \cdot I(P = -1) + w_3 \cdot I\left(\frac{|k - k_0|}{k_0} < 0.15\right) \quad (1)$$

Here, $I(\cdot)$ denotes the indicator function, which equals 1 when the condition in parentheses is satisfied and 0 otherwise. The weights w_1 , w_2 and w_3 take the values 0.4, 0.3 and 0.3 and correspond to the time-difference-of-arrival, polarity-sign and amplitude-transfer-ratio criteria respectively, and a value of $F \geq 0.6$ identifies an internal discharge. The TDoA criterion was assigned the largest weight because it showed the clearest separation between internal-PD and interference events in the training data. The three weights were selected by a grid search with a step size of 0.05 under the constraint that their sum equaled 1, using balanced accuracy on the training subset as the optimization criterion. The decision threshold of 0.6 was subsequently selected on the validation subset from the receiver operating characteristic curve and was kept fixed during testing. Weighted aggregation was used instead of a strict logical product so that an occasional deviation in one parameter did not invalidate the overall decision.

2.4 Experimental Design and Defect-Interference Combination Scheme

There are four discharge defect configurations that are typical for this platform. Air-gap discharge configuration is provided by the use of an epoxy resin block having an enclosed air gap inside. Surface discharge configuration is obtained using a needle-to-plate electrode assembly immersed in oil-filled paperboard. Floating discharge configuration is obtained by providing a shielding ring with poor contact with the winding. Tip discharge is achieved by placing a metallic thin tip on the terminal ends of the high-voltage winding. The chosen defects represent common mechanisms for insulation degradation in practical transformers, and their waveforms have the same propagation patterns observed in actual transformer windings[23].

Three types of interference are used to create the conditions for the electromagnetic environment at the installation site. The 12-pulse interference is simulated by a signal generator using a periodicity of 1.67 ms, an edge steepness of 3.9 μs , and a pulse duration of 80 μs to simulate valve commutation in a converter station. The corona interference is created by a supplementary rod-plate electrode pair in the test circuit and is referenced at the phase angles of 90° and 270°. The power-side harmonic interference is created by a phase-shifting transformer adding the 3rd, 5th, and 7th harmonics to the supply voltage.

The voltage levels below the measured discharge inception voltage were intentionally included as negative-control conditions for evaluating interference responses in the absence of detectable PD activity. However, the binary event labels were assigned from the recorded pulse characteristics rather than from the applied-voltage level alone because PD inception is stochastic near the threshold. In all eight levels, twenty-five cycles are collected for the power frequency. Therefore, 2,400 defect-interference combination frames were obtained from the matrix of four defect types, three interference types, eight voltage levels, and 25 power-frequency cycles. An additional 400

reference frames, consisting of 240 background-noise frames and 160 calibration-signal frames with a 2,000-pC injected pulse, were collected.

Table 1. Experimental design matrix for defect–interference combinations and voltage levels

Defect type	Defect model	Inception voltage ($\times U_0$)	Interference type	Samples per voltage level	Voltage levels ($\times U_0$)	Total samples
Air-gap discharge	Epoxy embedded bubble (D2, D5)	0.5–0.6	12-pulse	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Air-gap discharge	Epoxy embedded bubble (D2, D5)	0.5–0.6	Corona	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Air-gap discharge	Epoxy embedded bubble (D2, D5)	0.5–0.6	Power harmonic	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Surface discharge	Needle-plate on oil-paper (D7, D10)	0.6–0.7	12-pulse	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Surface discharge	Needle-plate on oil-paper (D7, D10)	0.6–0.7	Corona	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Surface discharge	Needle-plate on oil-paper (D7, D10)	0.6–0.7	Power harmonic	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Floating discharge	Loose shielding ring (D9, D12)	0.4–0.5	12-pulse	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Floating discharge	Loose shielding ring (D9, D12)	0.4–0.5	Corona	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Floating discharge	Loose shielding ring (D9, D12)	0.4–0.5	Power harmonic	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Tip discharge	Metal protrusion (D1, D13)	0.4–0.5	12-pulse	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Tip discharge	Metal protrusion (D1, D13)	0.4–0.5	Corona	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Tip discharge	Metal protrusion (D1, D13)	0.4–0.5	Power harmonic	25	0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.1	200
Background noise	–	–	–	–	–	240
Calibration signal	2,000 pC pulse injection	–	–	–	–	160
Total						2800

Note: U_0 denotes the rated phase voltage. Each defect–interference combination comprised 25 power-frequency frames at each of eight voltage levels. The 2,400 matrix entries represent complete combination frames rather than mutually exclusive PD-only or interference-only samples. The additional 400 reference frames comprised 240 background-noise frames and 160 calibration-signal frames. Tests were conducted from April to October 2025 at 18–28 °C and 40–75% relative humidity.

The 2,400 observations in the experimental matrix denote complete defect–interference combination frames, whereas the 800 PD and 1,600 interference observations used for binary

classification denote annotated transient events extracted from these frames. Event labels were assigned using the defect energization state, phase-resolved discharge pattern, dual-channel propagation direction, and reference-channel waveform. The process of data collection was conducted during the months of April to October 2025, involving spring, summer, and autumn seasons. The laboratory temperature ranged from 18 °C to 28 °C and relative humidity from 40% to 75%. This helped in testing the performance of the detection system in fluctuating environments. The complete experimental matrix is summarized in Table 1.

3. Results

3.1 Time-Frequency Characteristics of Typical Interference Signals

The repetition period of the 12-pulse disturbance is 1.67 ± 0.03 ms, which is equal to one-twelfthth of the power frequency period. In one 20 ms power frequency cycle, there are two types of pulses, including 3 high pulses and 9 low pulses, with the amplitude ratio of two amplitude types being 1.4–1.6. The average rising edge of the pulses is $3.92 \mu\text{s}$, with a deviation value of $0.18 \mu\text{s}$, and the average width of the pulses is $79.6 \mu\text{s}$. The energy spectrum indicates that all energies are lower than 400 kHz with 98.7%, and the center frequency is around 100 kHz. The phase locking property of corona disturbance is quite evident since 61% of the pulses appear at 90° while 35% of the pulses occur at 270° . There is considerable phase constancy from one cycle to another. Each pulse has an average rise time of 276 ns with a standard deviation of 32 ns with an oscillating tail lasting between 300–500 ns. The spectral range is wider as compared to the 12-pulse disturbance, with energy ranging from 50 kHz to 500 kHz.

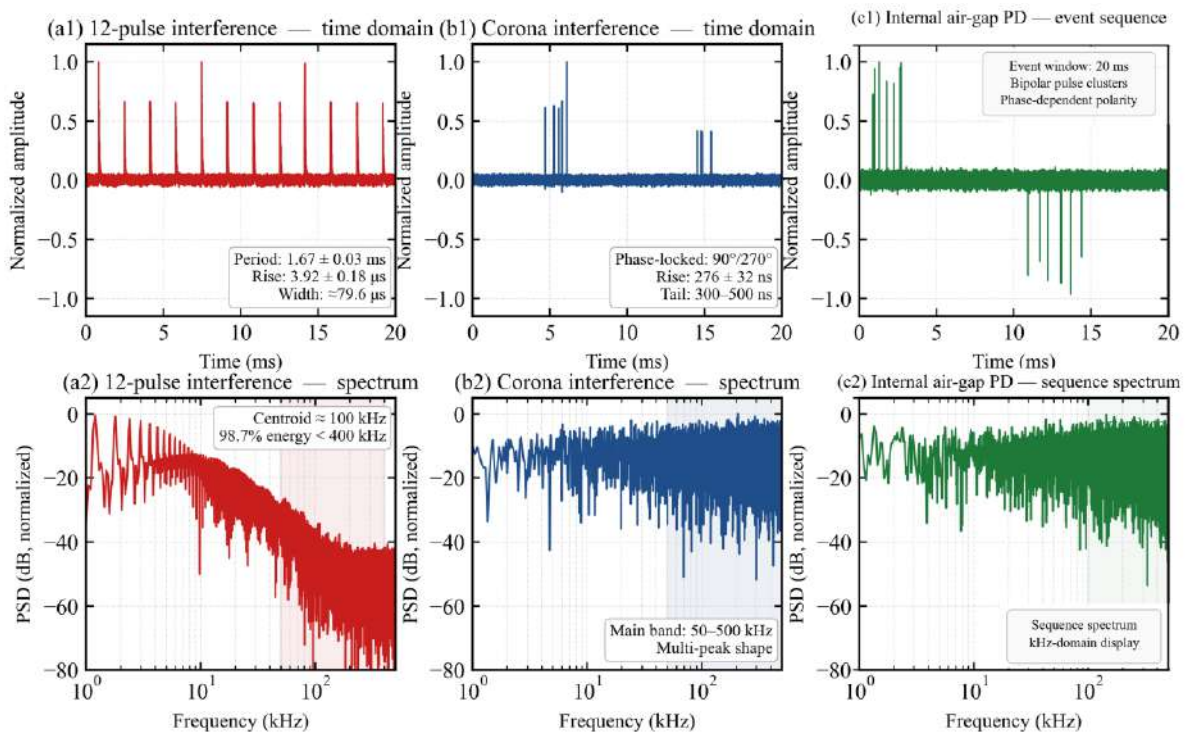


Fig. 2. Time- and frequency-domain characteristics of two typical interferences and an internal air-gap PD signal. (a1, a2) 12-pulse interference. (b1, b2) corona interference. (c1, c2) internal air-gap PD, shown as a phase-resolved event sequence over one power-frequency cycle and its corresponding sequence spectrum

The disturbance generated by the power side results in a consistent time delay of about 42 ns, where the power source grounding is ahead of the bushing tap screen. Same polarity events for both channels total 89%, while the amplitude transfer ratio k differs significantly from the baseline k_0 corresponding to internal discharges. The level of the background white noise is 1.78 mV, and its standard deviation is 0.21 mV. The frequency spectrum of the white noise is almost constant in the region of interest, with variations of less than 3 dB in the frequency range of 3 to 50 MHz. The

time- and frequency-domain characteristics of the 12-pulse interference, corona interference, and internal air-gap PD signal are illustrated in Figure 2 and provide the empirical basis for calibrating the correlation criteria described in Section 2.3. The power-side harmonic interference was evaluated using the dual-channel propagation parameters reported in Section 3.2 rather than included in the time-frequency comparison in Figure 2.

3.2 Effectiveness of the Dual-Sensor Correlation Discrimination

The arrival time difference Δt is the best indicator of distinguishing internal discharges from external noise. Among the 800 annotated internal-PD events, the measured values of Δt for air gap, surface, floating, and tip discharges are 41.3 ± 8.6 ns, 52.8 ± 11.2 ns, 35.7 ± 9.4 ns, and 28.4 ± 6.9 ns, respectively, all within the range of 25 to 70 ns set out in Section 2.3. For the 1,600 annotated interference events, their respective distributions are 6.2 ± 2.1 ns for the 12-pulse disturbance, 4.8 ± 1.7 ns for corona, and -42.5 ± 5.3 ns for the harmonics on the power side, in which the minus sign indicates reverse direction. The distribution of none of the interferences lies within the range of the internal discharge pulse train.

The polarity sign criterion yields stable results, but with less purity than the other criteria. Of the internal discharge cases, 87.4% display the anticipated opposite polarity reaction ($P = -1$), whereas 91.2% of the interfering cases display same polarity ($P = +1$). Misclassifications fall into two categories: far internal discharges traveling a great distance where the polarity change is attenuated, and highly oscillatory disturbances where polarity is unstable at the moment of sampling.

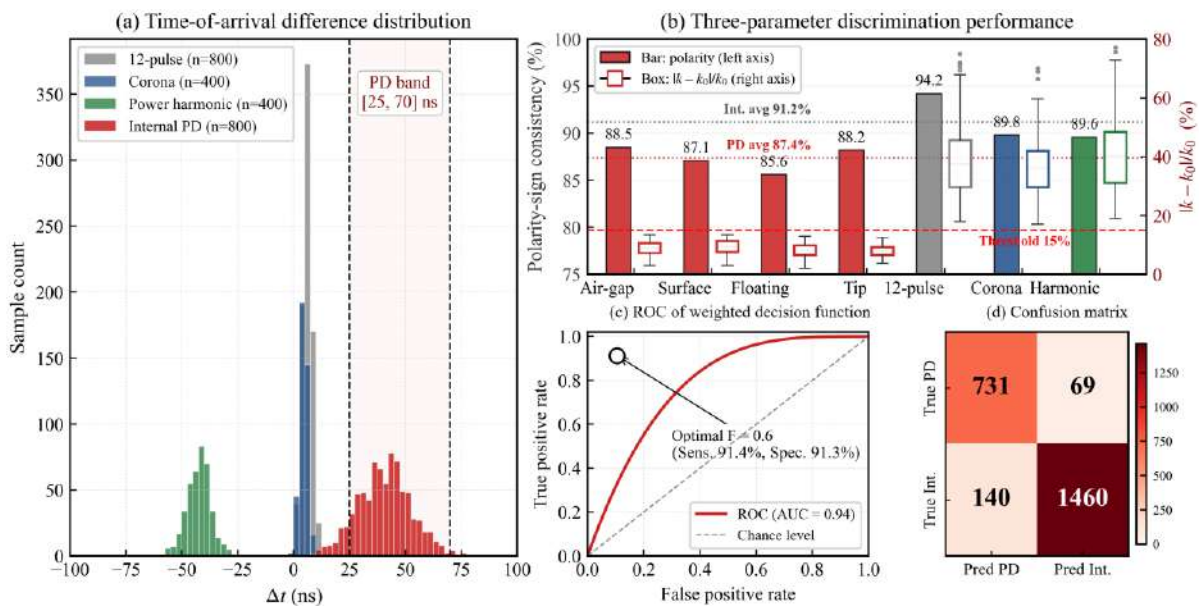


Fig. 3. Discrimination performance of the dual-sensor correlation method: (a) time-difference-of-arrival distributions; (b) polarity-sign consistency and amplitude-transfer-ratio deviation; (c) receiver operating characteristic curve of the weighted decision function; and (d) confusion matrix. The sample counts denote annotated transient events extracted from the defect-interference combination frames

A high amplitude transfer ratio k results in an extremely large contrast in dispersion. For the internal discharge data, there is a deviation from k_0 of $\pm 8.7\%$, with a maximum value of $\pm 13.4\%$, which does not exceed $\pm 15\%$. For the interference case, due to lack of any well-defined propagation path, there is a deviation of $\pm 37.6\%$. The recognition accuracy obtained when these three features are combined using the weighted decision function is 91.3% among the 2,400 annotated transient events. These events were extracted from the defect-interference combination frames and were labelled according to their recorded pulse and propagation characteristics rather than the applied-voltage level alone. On an interference-by-interference basis, the rejection rates are 94.8% for the 12-pulse interference, 89.6% for corona, and 93.1% for power-side harmonics. The major sources

of errors are floating discharges with low amplitude less than $0.6U_0$ classified wrongly as interference (5.2%), and distant corona classified wrongly as discharge (3.5%). A total of 8.7% of the data belong to the range where classification ambiguity occurs, specifically when $F = 0.6$. No combination frame was excluded solely because its applied voltage was above or below the nominal discharge inception range. The complete distribution of the three correlation factors and classification efficiency is presented in Figure 3. The 2,400 entries in Figure 3 represent annotated transient events extracted from the defect-interference combination frames rather than 2,400 mutually exclusive acquisition frames, comprising 800 internal-PD events and 1,600 interference events.

3.3 Detection and Localization Results for Four Defect Types

The air-gap spark resulted in a steady initiation voltage of $0.6 U_0$, and the signal stayed constant after a voltage application of $1.0 U_0$. The monopole showed a rise time of 16-20 ns characterized by a single-peak conical waveform without any sign of oscillation at the tail. For a voltage of $1.0 U_0$, the apparent charge obtained from the high-voltage bushing tap screen was around 185 pC while just 12 pC at the neutral position with $k = 15.4$. The electrical source location error using the time difference was ± 0.29 m.

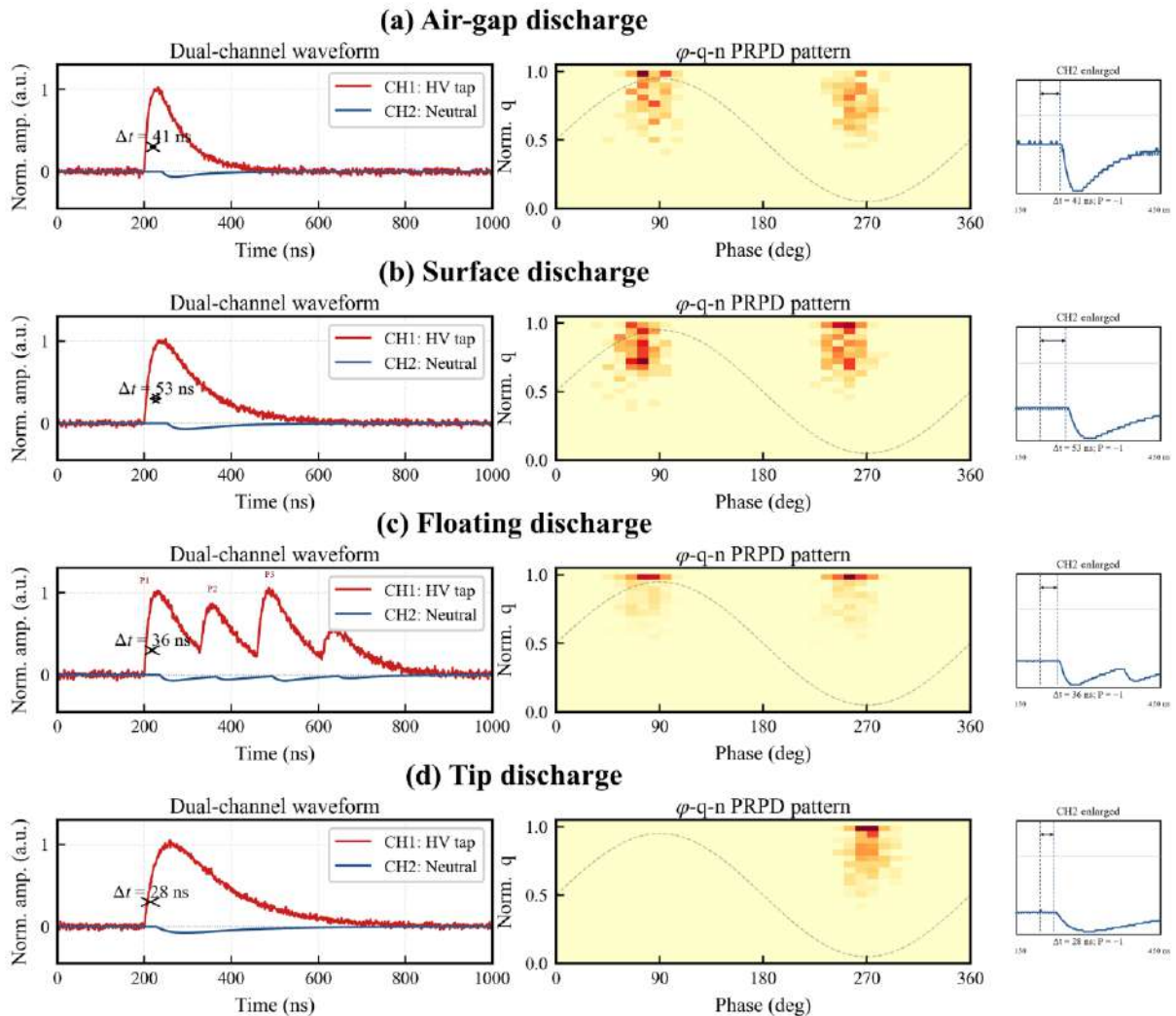


Fig. 4. Dual-channel waveforms and ϕ -q-n PRPD patterns of four PD defect types: (a) air-gap discharge; (b) surface discharge; (c) floating discharge; and (d) tip discharge. The CH2 insets use an independent vertical display scale to visualize the attenuated neutral-point pulses, their arrival times, and negative first-peak polarities. All TDoA, polarity, and amplitude-transfer-ratio values were calculated from the original unnormalized dual-channel recordings. P1-P3 denote successive pulses in the floating-discharge waveform

The surface discharge had an inception voltage of $0.7U_0$ and produced apparent charge of close to 320 pC at HV bushing tap-screen at $1.0U_0$. The transfer ratio $k = 14.7$ differed from the reference calibration value of $k_0 = 16$ by 8.1%, which lies safely within the $\pm 15\%$ limit of variation. The average rise time was 24 ns and pulse duration approximately 240 ns. The discharge locations were consistently placed in the first and third quadrants of PRPD distribution with great symmetry, with localization error of ± 0.40 m. Floating discharge showed initiation voltage of $0.5 U_0$ and had extremely symmetric behavior (skewness less than 0.08) within the first and third quadrants. Time to peak was in the range of 18-22 ns; also, pulse amplitudes were high and distributed from 500 to 1500 pC at the bushing tap screen. There was an evident inter-electrode fingerprint: the polarity at the high-voltage bushing tap screen was inverted relative to that of the core-clamp ground paths. The localization accuracy was within ± 0.35 m.

Table 2. Detection and localization results for four typical PD defect types under the dual HFCT system (statistics from 8 voltage levels, 200 annotated PD events per defect type)

Defect type	Inception voltage (U_i , $\times U_0$)	Pulse rises time (ns)	Pulse width (ns)	Apparent charge at $1.0 U_0$ at HV bushing tap (pC)	Transfer ratio k (HV/neutral)	Phase concentration	Electrical localization error (m)
Air-gap	0.6 ± 0.04	18.3 ± 2.6	152 ± 38	185 ± 41	15.4 ± 1.9 (Mean deviation from k_0 : 3.8%)	1st & 3rd quadrants	± 0.29
Surface	0.7 ± 0.05	23.7 ± 3.2	238 ± 47	318 ± 62	14.7 ± 1.6 (Mean deviation from k_0 : 8.1%)	1st & 3rd quadrants	± 0.40
Floating	0.5 ± 0.06	19.8 ± 2.9	175 ± 52	982 ± 287	13.9 ± 2.4 (Mean deviation from k_0 : 13.1%)	1st & 3rd, symmetric	± 0.35
Tip	0.5 ± 0.04	44.2 ± 5.1	326 ± 41	426 ± 74	14.2 ± 1.8 (Mean deviation from k_0 : 11.3%)	$\approx 270^\circ$	± 0.21

Note: Electrical-localization errors were calculated from Δt using the calibrated effective winding propagation velocity of $(1.58 \pm 0.05) \times 10^8$ m/s. The voltage-to-charge calibration factor was 1.46 pC/mV; therefore, the 1.78 mV background corresponded to approximately 2.6 pC. The minimum identifiable apparent charge of approximately 5.8 pC applies to the air-gap discharge configuration. The comparatively wide standard deviation of the floating-discharge apparent charge follows from the intermittent charge accumulation on the isolated shielding ring, whose discharge magnitude is governed by the instantaneous floating potential at breakdown rather than by a fixed electrode geometry.

The discharge at the tip had a threshold voltage of $0.5U_0$ and showed a steady signal with a peak voltage of $1.0U_0$, where the rise time was 40 – 50 ns while the pulse width was 300 – 350 ns, similar to the 30ns / 345ns value observed in a Ganjiang bushing case and the 17ns / 540ns value in a Qiantangjiang converter transformer case. The phase angle value continued to stay in the vicinity of 270° , and the localization accuracy was kept within ± 0.21 m. The voltage-to-charge conversion factor was established using the 2,000-pC reference-pulse calibration and was 1.46 pC/mV for the acquisition channel used in this test. Accordingly, the measured background level of 1.78 mV corresponded to an apparent-charge-equivalent level of approximately 2.6 pC. With a background noise level of 2.6 pC, the minimum apparent charge that can be recognized for air gap discharge was about 5.8 pC, equivalent to a SNR threshold of around 6.9 dB. The waveforms and PRPD features of the four defect types are illustrated in Figure 4, and the statistics of these defects are provided in Table 2. The waveform traces in Figure 4 were normalized only for visualization, whereas the polarity, TDoA, and amplitude transfer ratio were calculated from the unnormalized dual-channel recordings. The CH2 arrival time and first-peak polarity were determined within the cross-correlation-derived local pulse window rather than from the complete millisecond-scale record, thereby reducing the influence of the background noise on the attenuated neutral-point signal. Several pulses may occur within one event window, as in the floating-discharge trace where three successive pulses are labelled P1 to P3. The correlation parameters were calculated from the

first pulse crossing the trigger threshold, since the later pulses originate from the same defect and traverse the same propagation path.

3.4 Comparison with Wavelet Thresholding, EEMD, and Adaptive SVD

A controlled comparison was carried out on a 600-sample subset corrupted by the 12-pulse disturbance, with all four algorithms running on the same dataset and the same hardware platform. All four algorithms operate on 4,096-sample event windows, each spanning 16.384 μ s at 250 MSps, and no down sampling is applied within a window, so the original 4 ns sampling interval is retained. A power-frequency record is partitioned into consecutive windows of this length, each processed independently and then reassembled for display and spectral estimation. The 2.4 ms segment in Figure 5 comprises 146 such windows, which is why its spectra resolve components below the 61 kHz bin width of a single window.

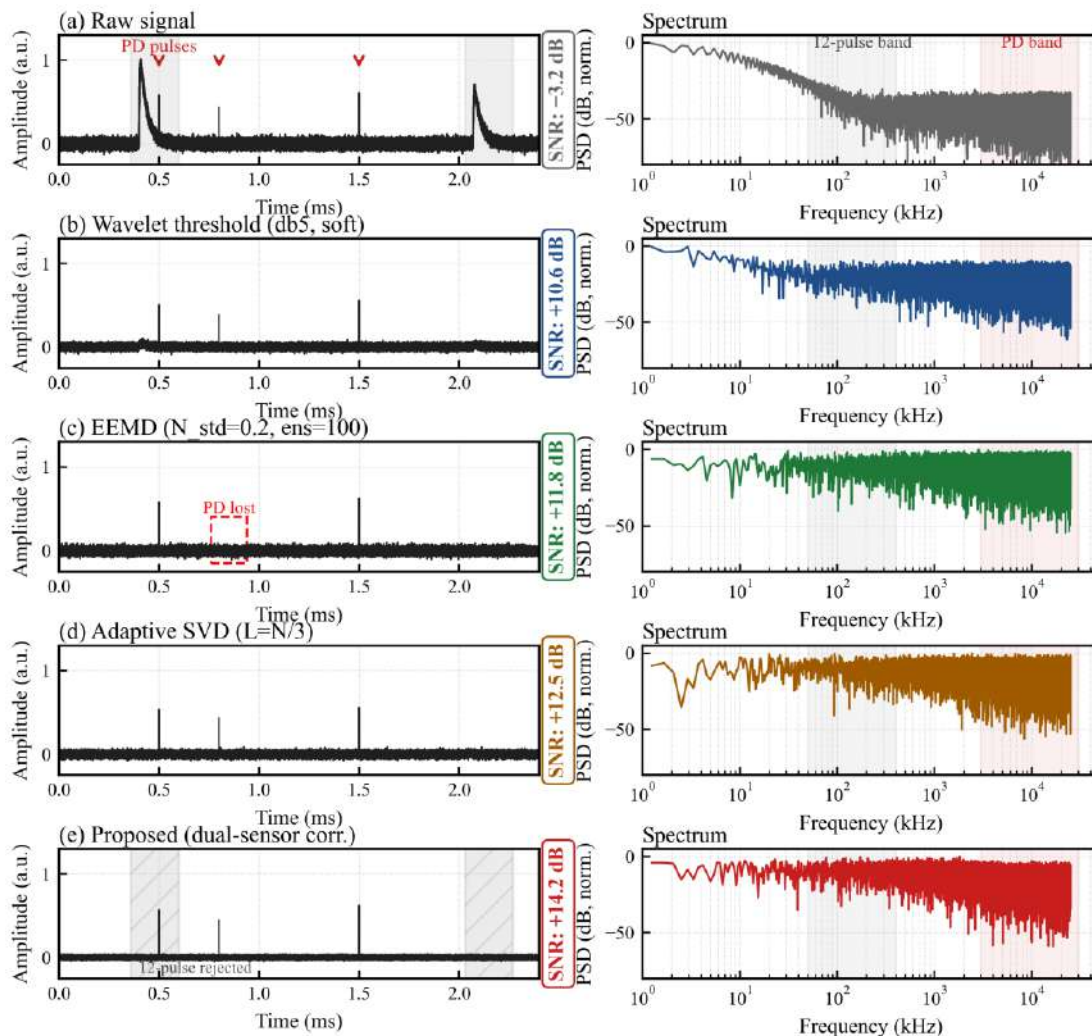


Fig. 5. Comparison of four interference-suppression methods for a PD signal corrupted by 12-pulse interference. Panels corresponding to the proposed method show the output of propagation-path-domain temporal gating; rejected windows are set to zero, and the spectrum is calculated from the retained pulse windows

For the suggested dual-sensor correlation-based technique, improvements in SNR, processing time per event window, blind-zone percentage, and recognition rate were 14.2 dB, 37.8 ms (20 ms in the FPGA side and 17.8 ms in the host side), 2.4%, and 91.3%, respectively. In turn, for the wavelet thresholding procedure applied with db5 wavelets, six levels of decomposition, and soft thresholding parameter $\lambda = \sqrt{2 \log N}$, improvements in SNR, computation time, blind-zone percentage, and accuracy were 10.6 dB, 82.4 ms, 4.1%, and 82.7%, respectively, although waveform distortion could be observed at the frequencies where both 12-pulse interference (lower than 400

kHz) and PD pulses overlap. EEMD using Nstd = 0.2 with an ensemble size of 100 had an SNR enhancement of 11.8 dB with 418 ms per event window, where the blind zone was 3.6% and accuracy was 85.1%. Mode mixing in the low-frequency IMFs caused some low-amplitude PD pulses to be filtered out unintentionally. Adaptive SVD using Hankel matrix length $L = N/3$ and retaining 1/100 singular values resulted in an SNR enhancement of 12.5 dB with processing time 258 ms and a blind zone of 3.2%, and accuracy of 87.6%. Computational complexity increased significantly with signal length. The reported 37.8 ms is a latency per event window rather than a continuous-throughput figure. The pipelined FPGA stage keeps pace with the uninterrupted 250 MSps stream and passes only triggered windows to the host stage, so the host stage runs on sparse events rather than on the full data rate.

Overall, the proposed algorithm is better than the others based on three criteria: the proposed FPGA-assisted implementation showed a shorter measured latency than the CPU-only EEMD and SVD implementations evaluated using the same event-window length; its detection blind zone is 2.4%, which is less than the 3.2% to 4.1% span of the other algorithms; and its recognition rate is higher by 3.7 to 8.6 percentage points compared to their single-channel counterparts. The SNR improvement of the proposed method is close to that of SVD and slightly better than wavelet thresholding and EEMD. Unlike wavelet, EEMD, and SVD filtering, the proposed method applies propagation-path-domain temporal gating: windows that fail the dual-sensor correlation criteria are suppressed, whereas accepted pulse windows retain their original waveform samples. Consequently, the near-zero regions in Figure 5(e) represent closed interference windows rather than frequency-domain attenuation, and the displayed spectrum was calculated from the retained pulse windows. The reported 14.2 dB SNR improvement therefore quantifies the combined effect of event discrimination and temporal gating and should not be interpreted as the passband gain of a conventional spectral filter. The detailed comparison is presented in Figure 5, with the overall performance summarized in Table 3 and the robustness evaluation under varying SNR conditions reported in Table 4.

Table 3. Performance comparison among four denoising algorithms on 600 event windows containing 12-pulse interference

Algorithm	SNR improvement (dB)	Processing time per 4,096-sample event window (ms)	Detection blind zone (%)	Recognition accuracy (%)	Waveform distortion (NCC)	Parameter dependence
Wavelet threshold (db5, 6 layers, soft threshold)	10.6 ± 0.8	82.4 ± 5.2	4.1	82.7	0.873	High (basis, level, threshold)
EEMD (Nstd = 0.2, ensemble = 100)	11.8 ± 0.9	418.3 ± 21.7	3.6	85.1	0.891	Medium (noise std, ensemble)
Adaptive SVD (Hankel $L = N/3$)	12.5 ± 0.7	258.6 ± 14.3	3.2	87.6	0.908	Medium (window length)
Dual-sensor correlation (proposed)	14.2 ± 0.6	37.8 ± 3.1	2.4	91.3	0.946	Low (one-time calibration)

Note: NCC: normalized correlation coefficient between denoised and reference PD waveforms. Processing times are measured on Intel Core i7-12700H, 16 GB RAM, MATLAB R2023a. The proposed method includes a 20 ms FPGA pre-processing stage, whereas the other three methods run entirely on the CPU in MATLAB. For adaptive SVD, $N = 4,096$ and $L = N/3$ was rounded to 1,365, producing a $1,365 \times 2,732$ Hankel matrix.

Table 4. Robustness evaluation under varying SNR conditions and interference combinations

Test condition	Input SNR (dB)	Output SNR – Wavelet	Output SNR – EEMD	Output SNR – SVD	Output SNR – Proposed
12-pulse only (low intensity)	5	14.8 ± 0.62	16.1 ± 0.58	16.9 ± 0.55	18.4 ± 0.48
12-pulse only (high intensity)	-3	6.5 ± 0.81	7.9 ± 0.76	8.7 ± 0.70	10.6 ± 0.61
Corona only	2	11.2 ± 0.68	12.5 ± 0.64	13.1 ± 0.60	15.0 ± 0.52
Power harmonic only	0	9.8 ± 0.73	10.9 ± 0.69	11.8 ± 0.65	13.7 ± 0.56
Mixed (12-pulse + corona)	-5	4.2 ± 0.90	5.6 ± 0.84	6.3 ± 0.78	8.9 ± 0.69
Mixed (all three types)	-8	1.8 ± 1.02	2.7 ± 0.96	3.5 ± 0.89	6.1 ± 0.78

Note: Values are reported as mean ± SD over 100 repeated trials for each interference and input-SNR condition. The input SNR in Table 4 is defined over the whole event window with respect to the full interference energy, most of which lies below the 3 MHz HFCT passband. The amplitude trigger operates on the band-limited content of the HV-bushing channel (CH1), where the PD pulse retains the larger amplitude, with the threshold set at 3.5 times the running RMS of the pre-trigger interval. A window triggered by interference is not discarded but passed to the correlation criteria, where the near-zero Δt and identical polarity of interference lead to its rejection, so false triggers increase the host-stage load rather than the error rate. Windows in which no CH1 crossing occurs constitute the blind zone reported in Table 3.

4. Discussion

4.1 Physical Basis of the Dual-Sensor Correlation Criterion

The rationale behind the reliability of the dual-sensor technique lies in the fundamental distinction between the signal transmission routes for interference and partial discharge signals. External interferences such as 12-pulse converter pulsation, corona, and power-side harmonics are fed into the detection system via common mode coupling or through the source circuit. The arrival time difference between the two ground HFCTs is close to zero, and their polarities are also identical. Their transfer ratio is not constant because there is no fixed physical benchmark[24].

On the other hand, internal partial discharges travel through the distributed parameter network inside the winding of the transformer. With the fixed HFCT channel assignment used in this study, the different electrical distances from the discharge source to the tap-screen and neutral-point sensors resulted in a stable positive signed time difference of 25–70 ns. By contrast, interference propagating in the opposite direction produced a negative time difference; therefore, an interference event with $\Delta t = -45$ ns did not satisfy the signed decision criterion $\Delta t \in [25, 70]$ ns. There is always a reverse current at the head and tail of the winding, resulting in the stability of polarity change. The amplitude ratio, determined by the distributed parameter values of the capacitance and inductance in the winding, is also stable for each specific defect position.

This classification on the level of paths is independent of the frequency domain decomposition method used; thus, the criteria are robust against the selection of wavelet bases, decomposition levels, and thresholding functions, and hence do not suffer from the parameter dependence limitation of traditional techniques [25]. Practical examples from the converter transformer at Suidong and the HVSR at Durong, where the discharge locations have been estimated precisely within ± 1 m by time difference and transfer ratio only, provide additional evidence for the practicality of the proposed methodology.

4.2 Advantages of Wideband HFCT Detection over the Conventional Pulse Current Method

However, the traditional pulsed current technique, as defined by IEC 60270, has its limitation on the bandwidth of the measurement system below 1 MHz, which means that only the low-frequency component of the pulse signal can be retained. When dealing with the Qiantangjiang converter transformer, the traditional PD meter could not recognize the presence of a 500-pC calibration square wave; the test charge was increased to 20,000 pC to detect the presence of the pulse, while

the background noise stood at 15,992 pC. A sensitivity limit of this type makes engineering tests pointless [26].

The wideband frequency range of the 3 MHz to 50 MHz HFCT ensures that the nanosecond rising edges of the pulse are preserved, making steepness discrimination possible because normal PD pulses have rising edges of 16 to 30 ns, while external interference lasts 200 to 4000 ns. In addition, the relatively large transfer impedance of 16 mV/mA significantly helps in detecting the small amplitude signals because the same primary signal results in a higher secondary amplitude in the conditioning stage. Furthermore, the accurate modeling of the wave propagation in the winding also supports the behavior of the high-frequency equivalent model of transformer windings [27]. At the very same location on the Qiantangjiang test line, after eliminating the 12-pulse interference using the wideband system, the background level dropped to 55.9 pC and a doubtful event of 409.4 pC, $0.7 U_0$ with a risetime of 17 ns and pulse width of 540 ns occurred, the origin of which turned out to be a discharge from the grading cap. The narrow-band values were obtained with an IEC 60270 detector calibrated by terminal charge injection, whereas the wideband values were obtained by converting the recorded HFCT peak voltage through the charge-per-millivolt factor from a reference-pulse injection on the same test loop. Both scales are anchored to the same injected reference charge and are therefore comparable in magnitude, although the wideband values retain high-frequency content that the narrow-band chain integrates away.

4.3 Comparison with Recent Studies

Recent research on PD interference suppression has mainly followed three methodological directions. The first trend is based on using the variational mode decomposition method together with white-noise test (VMD-LB) or wavelet thresholding method with PSO optimization (PSO-WTD). It seeks to detect the signals from only one channel via optimal frequency band partitioning [28]. In the second path, deep learning models like DnCNN and autoencoders are used in an end-to-end manner for noise removal or pattern detection, where both the processes of feature extraction and noise suppression are simultaneously learned from labeled data sets [29, 30]. In the third path, the compressed sensing reconstruction method, based on the sparsity property of PD pulses in transform domains, is studied. These techniques have made an improvement to the signal-to-noise ratio on a per channel basis but continue to show observable distortions of signals when there is interference by factors like the 12-pulse disturbance where there is significant overlap between its spectrum and that of the PD frequency band. The deep learning methods, on the other hand, rely heavily on the training set size and uniformity of the scenarios [31].

The dual-sensor correlation scheme developed in this paper relocates the function of interference cancellation from the signal processing space to the path processing space. This is because the physical basis is very simple, and the involved parameters are not many and are all associated with the hardware configuration, requiring only one calibration session to achieve. This technique can even assist the single-channel techniques—firstly using the correlation criteria to cancel the vast part of common-mode interference and then removing the remaining noise by using single-channel filtering techniques.

4.4 Limitations and Future Improvements

There are still some shortcomings in with the proposed technique that need improvement in four aspects. The first one is that there is still a limitation with the ability of separating simultaneous multi-source discharges because it would become confusing if there were two discharge sources with nearly equal time difference arrival values [32]. Second, the thresholds must be recalibrated for transformers with various configurations because differences in winding distributed parameters result in a drifting reference transfer ratio k_0 and Δt range. Third, the transfer impedance falls steeply below 3 MHz, so kilohertz-range converter interference is recorded with strong attenuation rather than at its true magnitude. Rejection of that interference nevertheless relies on the wideband components of its 3.9 μ s commutation edges, which fall inside the 3–50 MHz passband and reach both channels with a near-zero arrival-time difference and identical polarity. Fourth, the transfer impedance of the 24-turn HFCT may vary when the primary conductor is

displaced from the center of the aperture; therefore, a centering fixture and a conductor-offset calibration are required for field installation.

There are three proposed approaches for future research. First, lightweight neural networks are expected to be applied for secondary sample discrimination, when the samples are located within the ambiguous zone, when $F \approx 0.6$, without need to use huge training databases; secondly, pattern recognition templates based on PRPD images could help in this step as well [33]. The ultrasound sensors will also be included to establish an acoustic-electric correlation model with four parameters; for example, a time-of-arrival difference of 433.3 μs was observed between the acoustic and electric channels for a specific source-to-sensor distance of approximately 0.6 m in the Durong shunt reactor system [34]. The cross-transformer adaptive threshold calibration scheme will be designed to reduce the engineering implementation cost and adapt to site conditions [35].

5. Conclusions

In order to solve the problem of distinguishing between strong-field interference factors, such as the 12-pulse converter pulsation, corona, and power-side harmonics, from the real partial discharge phenomena when conducting on-site testing of transformers, a dual-channel synchronous detection apparatus was constructed using a self-developed wideband HFCT operating from 3 to 50 MHz and a signal-conditioning system covering 150 kHz–60 MHz with a transfer impedance of about 16 mV/mA, an analog-to-digital sampling speed of 250 MSps, with local inter-channel timing limited by the 4 ns sampling interval of the common-clock acquisition unit and distributed synchronization accuracy better than 100 ns using satellite timing. Based on the hardware platform, a dual-sensor correlation discrimination technique focused on three factors was presented.

The experiments were conducted on a 220 kV/2 MVA single-phase transformer simulation platform. Four defect types (air-gap, surface, floating, and tip discharge) and three interference types (12-pulse converter noise, corona, and power-side harmonic interference) were evaluated at eight voltage levels from April to October 2025, yielding 2,400 defect–interference combination frames and 400 reference frames, including 240 background-noise frames and 160 calibration-signal frames. The event-level evaluation comprised 2,400 annotated transient events extracted from the combination frames, including 800 internal-PD events and 1,600 interference events. Under the fixed HFCT channel assignment, the internal-PD events exhibited a stable positive signed time difference of 25–70 ns, and the opposite-polarity rate ($P = -1$) for internal-PD events was 87.4%. The mean deviation of the amplitude transfer ratio k from its reference value k_0 was $\pm 8.7\%$. Consequently, the three-parameter weighted decision function achieved a classification accuracy of 91.3% for distinguishing internal PD from external interference, improved the signal-to-noise ratio by 14.2 dB after suppressing the interference, required 37.8 ms per 4,096-sample event window in the evaluated FPGA-assisted implementation, and created a blind zone of 2.4%.

In comparison with three conventional techniques using single-channel data, wavelet thresholding (+10.6 dB / 82.4 ms / 4.1%), empirical mode decomposition (EEMD) (+11.8 dB / 418 ms / 3.6%), and adaptive singular value decomposition (+12.5 dB / 258 ms / 3.2%), the suggested technique presents obvious benefits in terms of processing time and detection dead zone, and enhances the classification accuracy by around 3.7% to 8.6%. Using the calibrated effective winding propagation velocity, the electrical-localization errors ranged from 0.21 to 0.40 m. The minimum identifiable apparent charge of approximately 5.8 pC applied specifically to the air-gap discharge configuration.

The results of this study show that it is indeed feasible to use interference suppression techniques in the propagation path domain as an engineering approach to partial discharge detection in transformers. The next direction to follow in the future research is acoustic-electric correlation and adaptive thresholding.

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Pulse-type interference suppression based on an improved wavelet threshold for wideband partial discharge detection in transformers

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Abstract

Pulse-type interference at high-voltage (HV) and ultra-high voltage (UHV) transformer sites severely limits the effectiveness of partial discharge (PD) detection, as conventional pulse current measurement below 1 MHz leaves 12-pulse ripple, corona, and electronic switching pulses overlapping with genuine PD signals in both time and frequency domains. This work develops an improved wavelet threshold method tightly integrated with a wideband detection platform featuring a 150 kHz to 60 MHz signal conditioning bandwidth coupled with a 3–50 MHz high-frequency current transformer (HFCT) sensor. A continuously differentiable threshold function with adjustable factor α is constructed to bridge hard and soft thresholding, a layer-dependent adaptive estimator replaces the universal threshold to match noise variance at each decomposition level, and a time-frequency cumulative energy function performs pulse pre-localization before discrimination. Validation on 500 simulation samples, 320 laboratory records from a 220 kV physical platform, and 180 field signals from three substations shows an average SNR gain of 9.3 dB, combined fitness F of 2.38 at 5 dB input SNR (F is a figure of merit balancing noise suppression and waveform preservation), and laboratory identification rates of 95.2%, 93.6%, and 94.3% for the three interference categories. Pooled field detection accuracy reaches 93.7%, feature-parameter recovery errors stay within 8.1%, and single-record processing settles at 52 ms. Pilot deployments reduced the detection blind zone from 6%–8% down to 2.5%–3%. These findings demonstrate the practical value of the proposed method for improving the reliability of online insulation condition assessment in harsh substation environments.

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1. Introduction

High voltage and extra high voltage transformers represent essential elements in the conversion and transmission of electrical energy on a large scale. According to CIGRE figures, about 58.8% of all defects in HVDC systems are transformer-based problems, and insulation failure accounts for almost 50% of such incidents [36]. The phenomenon of partial discharge (PD) is an early warning sign as well as a catalyst in causing insulation breakdown, making PD detection a necessary prerequisite for effective health status analysis [1]. It is hence mandatory that continuous online monitoring be conducted with sensor networks installed in bushings, windings, and tap changers [2]. Various reviews on the technique of electrical diagnosis have categorized IEC 60270 pulse

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current testing, ultra-high frequency testing, and acoustic emission techniques as complementary methods, with each being limited by sensitivity and bandwidth requirements [3]. Electrical and acoustic sensing are physically complementary: the HFCT channel preserves the nanosecond-scale waveform needed to discriminate PD from pulse-type interference, whereas acoustic emission mainly supports spatial localization. The present study validates the electrical (HFCT) chain only; acoustic-electric fusion on the same platform is treated as future work (Section 4.4). UHV grids operating under extreme environmental conditions, such as those in Northwest China where on the order of one hundred large transformers are commissioned every year, require rigorous pre-energization PD testing, because a defect that escapes factory or site acceptance testing can readily develop into failure once the unit is energized.

The field PD measurements faces three consistent difficulties. Firstly, the harsh electromagnetic background of substation environments which includes rich harmonics, periodic oscillations, and white noise, with a noise floor sometimes drowning out the square-wave calibration signal when high load is applied. Secondly, bandwidth limitations under conventional standards restrict the measurement. As described in the IEC 60270 and GB/T 7354 pulse current measurement method, the frequency range of the test falls below 1 MHz, resulting in a loss of the rising edge at the nanosecond level of PD pulses. The research on UHF measurements revealed that the real PD signal spectrum reaches frequencies higher than 100 MHz; hence, band-limited sampling loses valuable information necessary to differentiate the signal from noise [4]. Finally, calibrating wideband receivers under practical conditions is difficult, which requires the replication of the impulse waveform and its frequency by means of the charge injection calibrator [5]. Pulse-type interference brings another level of challenge. The twelve-pulse ripple associated with converter operations is present with a period of 1.67 ms, corona pulses occur around the 90° and 270° phases, while electronic switching generates spindle pulses at 120° intervals. All three types of pulses behave like actual PD signals in traditional time domain analysis [6]. Effective differentiation relies on feature-based separation, not amplitude alone [7].

Five years of research have yielded multiple denoising pathways, each with its own unique attributes and drawbacks. The wavelet thresholding method continues to be the standard technique utilized, and the maximum overlap discrete wavelet transform technique has reconstructed severely noisy PD pulses from cables, but fixed thresholding continues to add a persistent bias term to the remaining coefficients [8]. The soft thresholding method creates a bias term intentionally, whereas the hard thresholding method creates discontinuous points at the threshold value, which results in pseudo-Gibbs oscillations once the signal is reconstructed. Empirical mode decomposition has been implemented on the PD pulses, but mode mixing and end effect problems affect the initial part of the signal [9]. Parameter optimized variational mode decomposition has exhibited improved separation of narrowband interferers [10], while the Aquila optimizer combination of VMD with K-SVD [11] along with whale-VMD combined with adaptive filtering and wavelet denoising [12] produce improved SNR but entail iterative searching. Hybrid SVD-VMD systems have improved rejection of white noise but involve intensive computation especially for long signal records [13]. Normalized spectral kurtosis-based spectrum segmentation approaches have been tried out in multiple source environments [14], while algorithms for periodic narrow band interference have proved selective interference cancellation capability without making the PD waveform flat [15]. Acoustic-VMD-CNN-LSTM [16], deep residual shrinkage network [17], and CNN-BiLSTM with attention [18] based recognition algorithms have attained impressive accuracies on laboratory data. The primary challenge remains achieving an optimal trade-off between waveform accuracy and computational delay under field conditions with pulse-like interference: iterative decompositions and deep recognizers require per-record latencies of several hundred milliseconds to seconds when executed on embedded hardware (for the baselines re-implemented in this work, 185 ms for adaptive SVD and 420 ms for EMD-based thresholding on a 1024-sample record, Section 3.4), which is incompatible with sub-100 ms online updating.

The following three proposals are offered as solutions to the aforementioned deficiencies. The first proposal is a continuously differential threshold function involving a parameter α varying in the interval (0, 1). This threshold function combines the benefits of hard thresholding and soft thresholding without the pseudo-Gibbs effect of the former while lowering the bias of the latter.

The second suggestion is a threshold estimation scheme dependent on the layer, where the noise standard deviation σ_j for each decomposition layer is estimated based on the median of the detail coefficients so that higher frequency layers have larger thresholds than lower frequency layers. The third part is related to the incorporation of the proposed algorithm into a wideband detection platform utilizing a signal conditioning bandwidth of 150 kHz to 60 MHz coupled with a 3–50 MHz HFCT sensor, where a time-frequency cumulative energy approach enables pulse localization before classification. The rest of the paper is structured as follows. Section 2 describes the hardware platform, interference, algorithm, and evaluation methodology. Section 3 includes simulation, laboratory and field experiments together with comparative performance analysis. Section 4 analyzes stability, adaptation, computational cost, and constraints. Section 5 concludes the work.

2. Materials and Methods

2.1 Wideband Partial Discharge Detection Hardware Platform

The HFCT detector employs a split-core (openable) nanocrystalline toroid with an initial relative permeability of 3×10^4 and a maximum relative permeability of 4×10^5 , and with dimensions $R_1 = 60$ mm (inner radius), $R_2 = 80$ mm (outer radius), and $d = 15$ mm (height). The two half-cores are hinged and spring-clamped, so that the sensor can be fitted around the bushing-tap lead or the tank grounding conductor of an in-service or newly commissioned transformer without disconnecting the grounding circuit. The detecting coil employs 24 turns of 0.6 mm enameled copper wires equally distributed with half turns on each side of the core material. The toroid coil is placed inside an aluminum alloy shielding case with an internal spacing of 2 mm which avoids shorting but preserves its shielding capability. The HFCT has been characterized experimentally for its –3 dB operational range from 3 MHz to 50 MHz, which therefore defines the effective measurement band of the complete chain, and its transfer impedance exceeds the 5 mV/mA required by State Grid corporate standards, analogous to the existing designs of HFCT used for oil-based partial discharge calibration tests [19].

The signal conditioning process operates from 150 kHz to 60 MHz, with a noise level measured less than 0.5 mV. The conditioning band is deliberately wider than the HFCT passband: its 150 kHz lower edge is retained so that the same front end can be calibrated against the IEC 60270 pulse-current path and can characterize the 12-pulse ripple spectrum (Section 2.2), whereas the effective PD measurement band of the whole chain is set by the HFCT (3–50 MHz). Below 3 MHz the HFCT response rolls off at roughly 20 dB/decade, so the 100 kHz ripple component is attenuated by about 30 dB; however, because the ripple amplitude at the bushing tap is two to three orders of magnitude larger than that of PD pulses and its thyristor commutation edges contain harmonic content extending into the MHz range, residual ripple and commutation transients still reach the ADC at amplitudes comparable to PD. Algorithmic suppression of this residual, rather than reliance on the sensor roll-off alone, is therefore required. The analog-to-digital conversion process operates at 250 MSps, recreating the PD events in which their rise times are in the range of ns to tens of ns. Further processing operations are performed using a heterogeneous multi-core processor operating at 1.6 GHz, with 2 GB DDR4 and 16 GB eMMC memory; this processor executes the implementation used for all laboratory and field results in Section 3, whereas the FPGA+DSP figures quoted in Section 4.3 stem from a separate synthesis study of a future embedded port. Synchronization in measurement nodes is done using GPS and Beidou discipline with ns accuracy.

The calibration procedure consists of three steps regarding the cutoff frequency, sensitivity, and pulse calibrator responses. The center frequency of the passband is determined by the geometric mean relationship $f_c = \sqrt{f_H f_L}$, where f_H and f_L are defined as the upper and lower 3 dB points from the practical front end. The calibration of apparent charge is performed using an impulse generator that provides the output loop resistance of detection impedance ranging from 50 Ω to 200 Ω . The apparent charge q (pC) reported in Section 3 is obtained from the peak HFCT output voltage V_p (mV) through the calibrator-derived scale factor $q = k_q V_p$, where k_q (pC/mV) is determined by injecting known charges with the impulse generator into the same detection loop before each test; the waveforms in Fig. 5 are shown in mV, and the pC values quoted in the text follow from this

calibration. The entire physical structure of the system includes the HFCT, conditioner, ADC, and digital backend, as illustrated in Fig. 1.

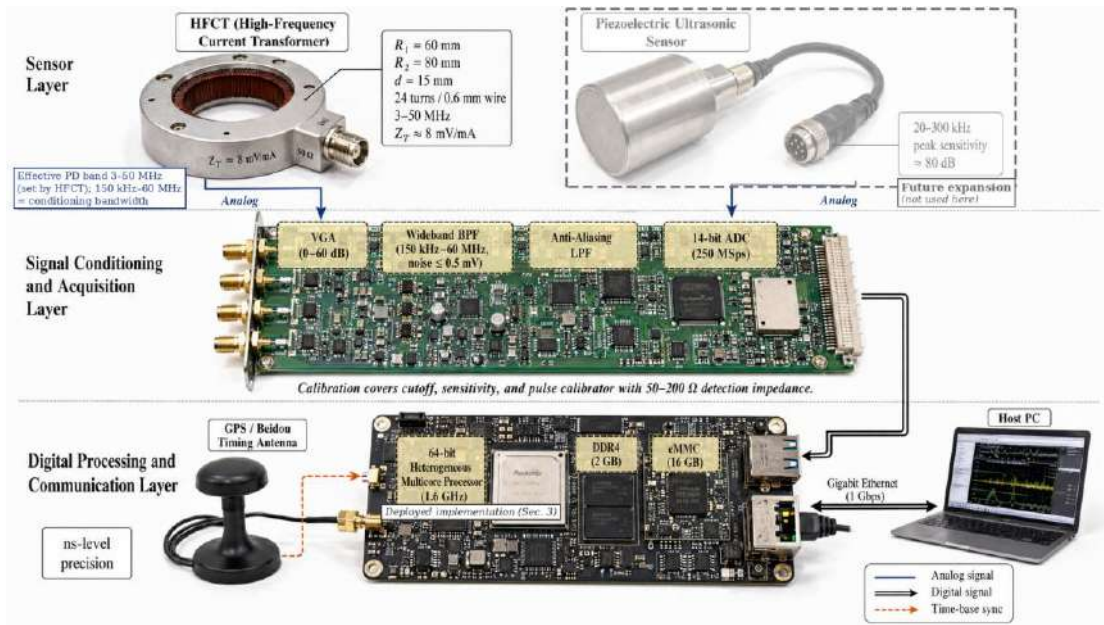


Fig. 1. Hardware architecture of the wideband HFCT-based partial discharge detection system. The piezoelectric ultrasonic channel is drawn as a future acoustic-fusion expansion only and is not used in this study

2.2 Characteristics of Pulse-Type Interference Signals

Three major interference types exist among pulse-type interferences that affect PD measurement in converter stations and UHV substations. The 12-pulse ripple produced by the thyristor-based system occurs every 1.67 ms (1/12th cycle of the fundamental period, 50 Hz). In the line-side winding bushing-tap, the pulses have a duration of around 200 μ s, a rising edge of 3.9 to 4.24 μ s, a pulse width of 80 to 83 μ s, and an energy content within 99 percent at frequencies lower than 400 kHz and around a center frequency of 100 kHz. About 31 percent of the energy content in the bushing-tap signal is obtained relative to the neutral point signal energy content.

Corona interference pulse trains are found at the 90° and 270° phases of the power frequency cycle and consist of uniformly spaced pulses that exhibit a sharpness of about 287 ns with clear damped-oscillation trailing edges. Common examples are grounded conductors from core and clamp frames in 1000 kV substations with clear periodicity and uniformly spaced pulses. Switching interference consists of three pulse trains spaced at 120° phases with near-equal amplitudes for each. Individual pulses have rising edges that last about 13 ns and an overall spindle shape with distinct oscillations, similar to a distant discharge through power electronic devices.

The actual PD impulses registered under similar conditions vary across different parameters. The impulse edge occurs between 13 ns and 18 ns, the maximum amplitude occurs in the first peak, and the waveform is cone-shaped, which suggests proximity between the signal source and the receiver. In terms of void discharge interference, the impulse focuses on the first and third quadrants, with a concentration in the impulse number and varied amplitudes per impulse. This comparative analysis for the three types of interference is presented in Fig. 2.

Cumulative energy for the features is given by $E_T(t_k)$ and $E_F(f_k)$ in the time domain and frequency domain respectively, normalized with respect to the total energy, $E_T(t_k) = \frac{\sum_{i=1}^k x^2(t_i)}{\sum_{i=1}^N x^2(t_i)} \times 100\%$ and

$E_F(f_k) = \frac{\sum_{i=1}^k |X(f_i)|^2}{\sum_{i=1}^M |X(f_i)|^2} \times 100\%$, where $x(t_i)$ is the sampled record of length N and $X(f_i)$ its M -point spectrum, on the basis that the short-time spectrum can extract the interfering signals from overlapping ones [20]. The instant at which E_T first exceeds 20% is regarded as the signal arrival

time and is used to adjust the time origin. The 20% level was selected from the laboratory records of Section 2.4: with a signal-free pre-trigger window, the cumulative energy of the noise floor grows approximately linearly and stays below 20% over the whole pre-trigger interval, whereas a PD or interference pulse raises E_T above this level within a few samples of its onset, so that the criterion corresponds to an energy step exceeding the noise contribution by more than three standard deviations of the pre-trigger cumulative energy. In addition, the oscillation ratio used for discrimination in Section 2.3 is defined as $R_{osc} = \frac{\sum_{n>n_z} x^2[n]}{\sum_n x^2[n]}$, where n_z is the first zero crossing after the main peak, i.e., the fraction of pulse energy contained in the trailing oscillation. Cone-shaped PD pulses concentrate their energy in the first peak and thus exhibit a low R_{osc} , whereas spindle-shaped electronic switching pulses and corona pulses with damped-oscillation tails exhibit a high R_{osc} ; the value 0.6 lies between the two populations observed in the 220 kV laboratory records.

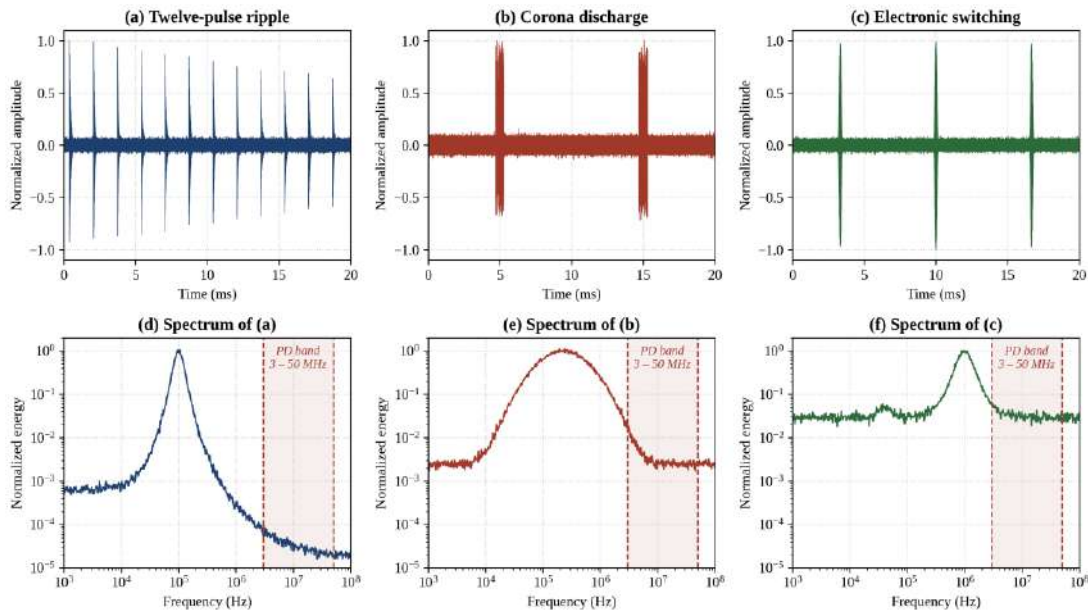


Fig. 2. Comparison of time-frequency characteristics of three typical pulse-type interference signals

2.3 Improved Wavelet Threshold Denoising Algorithm

Db5 wavelet from the family of Daubechies wavelets is chosen as the mother wavelet to extract PD coefficients since its damped sinusoidal shape approximates that of the PD pulses quite accurately, thus giving a highly correlated result between the inner products of the two and keeping the PD energy within a few detail coefficients only. Six levels of decomposition are used because higher-level decompositions diffuse the PD energy across the different approximation scales to a great extent, while lower-level decompositions leave broadband noise.

Hard thresholding assigns values less than λ to be zeros while keeping others the same; however, the point-wise non-smoothness at $|w|=\lambda$ induces Pseudo-Gibbs effect in the reconstructed waveforms. Soft thresholding reduces each surviving value by exactly λ ; hence, a residual magnitude bias is introduced, causing the first peak magnitude of the PD pulses to decay. A continuously differentiable thresholding scheme with a tunable factor $\alpha \in (0,1)$ is proposed. In the case of $|w| \geq \lambda$, the scheme tends to approach the original value by an exponentially diminishing bias with respect to α . Conversely, when $|w| < \lambda$, the scheme gradually diminishes towards zero. The function is continuous and differentiable at the critical value, thereby inhibiting the pseudo-Gibbs phenomenon linked to hard thresholding while simultaneously minimizing the constant error of soft thresholding. The proposed function is written separately for the two regions. For $|w| \geq \lambda$, $\hat{w} = \text{sgn}(w)(|w| - \alpha\lambda e^{-\frac{|w|-\lambda}{\lambda}})$; for $|w| < \lambda$, $\hat{w} = \text{sgn}(w)(1 - \alpha)\lambda(\frac{|w|}{\lambda})^m$, with $m = \frac{1+\alpha}{1-\alpha}$, where w is a

detail coefficient, λ the layer threshold, $\hat{w}$ the thresholded coefficient, and $\alpha \in (0,1)$. Both expressions take the value $(1 - \alpha)\lambda$ and the slope $1 + \alpha$ at $|w| = \lambda$, so the function is continuous and continuously differentiable there; for $|w| \gg \lambda$ the bias $\alpha\lambda e^{-(|w|-\lambda)/\lambda}$ vanishes exponentially, and for $\alpha \rightarrow 1$ the function approaches soft thresholding while for $\alpha \rightarrow 0$ it approaches hard thresholding above λ . Threshold functions of this type have demonstrated significant improvements in waveform accuracy in stator PD noise removal applications [21].

The traditional universal threshold $\lambda = \sigma\sqrt{2\ln N}$ presumes uniform noise variance in all the layers of the decomposition. As a result, there would be too much suppression of the low-frequency components and insufficient suppression of the high-frequency components during multi-level decomposition of the signal. Instead, layer-wise noise estimates are used here based on the median values of the detail coefficients within each respective layer j , $\sigma_j = \frac{\text{median}(|cD_j|)}{0.6745}$ and $\lambda_j = \sigma_j\sqrt{2\ln N_j}$, where cD_j are the detail coefficients and N_j their number at level j . Higher thresholds will be assigned to high-frequency components, and vice versa, in compliance with the energy distribution of PD transients across different scales. Analogous approaches were experimentally justified in the case of wavelet-based denoising of physiologically generated transients [22].

The full workflow couples pulse pre-localization with waveform-level discrimination through six sequential steps. The noisy input is first passed into a cumulative energy function $E(i) = \sum_{k=1}^i x^2(k)$, whose maximum value identifies the position of the PD event, after which a windowed segment around that position is extracted. The segment is decomposed by db5 wavelet to six levels, producing detail coefficients $cD1$ through $cD6$ and approximation coefficients $cA6$. The layer-specific thresholds are computed for all the detail layers and employed via the advanced thresholding approach using $\alpha=0.5$. The application of inverse wavelet transformation produces an intermediate denoised signal. This is followed by a discriminating phase where rising edge duration, signal slope, and oscillation ratio are compared to those values found in Sec. 2.2, resulting in the classification of all the pulses with a rising-edge duration longer than 200 ns or an oscillation ratio R_{osc} (Section 2.2) greater than 0.6 as interfering signals, while the others are considered valid PD pulses. The whole procedure is illustrated in Fig. 3.

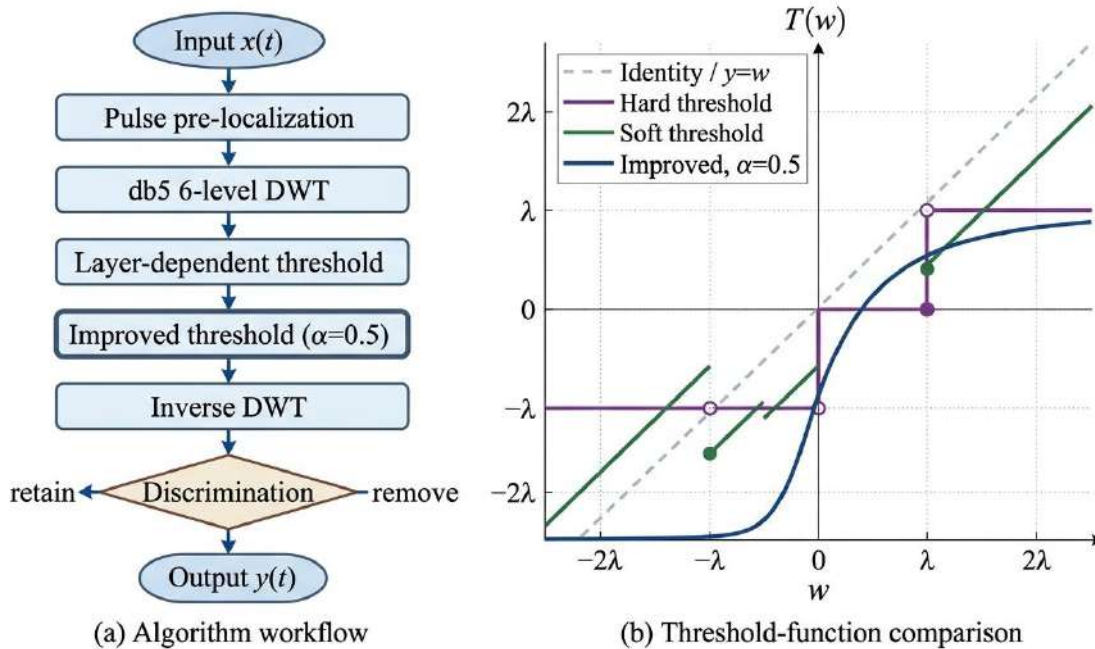


Fig. 3. Overall workflow of the improved wavelet threshold algorithm together with a comparison of the core threshold functions: (a) algorithm workflow; (b) comparison of the proposed continuously differentiable threshold function ($\alpha = 0.5$) with the hard and soft threshold functions

2.4 Experimental Datasets and Denoising Evaluation Metrics

Three datasets with different degrees of realism are created to test the algorithms. In the first one, the synthesis of a PD pulse is carried out using Gaussian-2 and double-exponential (DEC) models, $x_{G2}(t) = A_1 e^{-(t-t_1)^2/(2\sigma_1^2)} - A_2 e^{-(t-t_2)^2/(2\sigma_2^2)}$ and $x_{DEC}(t) = A(e^{-t/\tau_1} - e^{-t/\tau_2})$, $t \geq 0$, with unit peak amplitude; the time constants ($\tau_1, \tau_2, \sigma_1, \sigma_2$) and peak spacing ($t_2 - t_1$) are drawn uniformly so that the 10%–90% rising edge falls within 13–18 ns and the pulse width within 50–200 ns, in accordance with the measured PD statistics in Section 2.2. For each synthesized PD pulse, interference pulses of the three types described in Section 2.2 along with additive Gaussian white noise are added. The ratio of the signal to noise varies from –5 to 15 dB at 2 dB intervals resulting in a total of 500 data samples.

Laboratory data set was obtained using a 220kV single-phase ultra-high voltage transformer simulation setup launched in 2025 with a capacity of 2MVA, having winding connection type of liO and a short circuit impedance of 16.5%. The four classic models of defects, such as void discharge, surface discharge, floating electrode discharge, and tip discharge, were generated in the test tank, collecting 80 samples for each of them, thus totaling 320 data sets.

The field dataset consists of the collected measurements from three illustrative substations in 2025. The ± 800 kV converter station in Xinjiang will supply the 12-pulse interference case from its converter transformer; the 1000 kV Ultra High Voltage substation in Nanyang, Henan Province, will be the source of the corona-dominated case under high reactor core and clamping frame grounding configuration; while the 500 kV substation main transformer supplies the compound interference case. There will be 60 samples retained for each case, thus totaling 180 samples for the entire field dataset. The field validation procedure follows the practice reported for on-site PD measurement [23].

Five parameters assess the denoising effectiveness. The noise reduction ratio $NRR = 10 \lg(\sigma_1^2/\sigma_2^2)$ describes the attenuation in decibels (dB) of noise variance from input to output, where σ_1 and σ_2 are the noise standard deviations before and after denoising. The energy retention ratio $ERR = \Sigma y^2/\Sigma x^2$ shows the proportion of energy that is maintained, where x is the original noisy input record and y the denoised output. The combined fitness $F = \sqrt{NRR \cdot ERR}$ provides a trade-off between the effectiveness of suppression and the preservation of the waveform; here the numerical value of NRR in dB is used, so that F is an empirical, dimensionless figure of merit (the geometric mean of a suppression score and a retention score) intended only for the relative ranking of methods and parameter settings, not a physical quantity. Normalized cross correlation $NCC = \frac{\Sigma x_c y}{\sqrt{\Sigma x_c^2 \Sigma y^2}}$ measures the similarity of the denoised wave y to the ideal clean wave x_c . Processing time for one record t is measured in milliseconds.

3. Results

3.1 Simulation Signal Denoising Performance

In order to analyze the efficiency of the new algorithm, the simulation dataset of 500 samples as discussed in section 2.4 was considered initially, keeping $\alpha=0.5$, unless specified. With the variation of SNR from –5 dB to 15 dB, the SNR output increases continuously in accordance with the input SNR value. At –5 dB SNR, the output SNR obtained is 5.8 dB with an SNR improvement of 10.8 dB and NCC of 0.886. Increasing the input SNR to 0 dB gives SNR output of 9.6 dB with NCC=0.912. With an input of 5 dB, the output is 13.2 dB for an NCC of 0.937. When the input increases to 10 dB, the output is 17.8 dB for an NCC of 0.954, whereas, with the cleanest input of 15 dB, the output becomes 22.5 dB for an NCC of 0.968. The overall statistics, along with NRR, ERR, F, and per record processing time, are shown in Table 1 below.

At the reference input SNR of 5 dB (the center of the sweep, used hereafter as the representative operating point), the NRR value is 6.34 ± 0.42 dB, the ERR value is 0.89 ± 0.05 , and the value of fitness function F is 2.38 ± 0.18 ; the corresponding values at the other SNR levels are listed in Table

1. The percentage of records with an NCC value greater than 0.92 is 88.4%, suggesting that good waveform similarity is maintained in most instances, instead of only when there is high input SNR.

Table 1. Statistical performance of the improved algorithm on 500 simulation samples ($\alpha = 0.5$)

Input SNR (dB)	Output SNR (dB)	NRR (dB)	ERR	Combined fitness F	NCC	Processing time (ms)
-5	5.8 ± 0.6	7.12 ± 0.45	0.83 ± 0.07	2.43 ± 0.21	0.886 ± 0.031	50 ± 4.2
0	9.6 ± 0.5	6.85 ± 0.42	0.87 ± 0.06	2.44 ± 0.19	0.912 ± 0.027	51 ± 4.5
5	13.2 ± 0.5	6.34 ± 0.42	0.89 ± 0.05	2.38 ± 0.18	0.937 ± 0.024	52 ± 4.6
10	17.8 ± 0.6	5.91 ± 0.40	0.91 ± 0.04	2.32 ± 0.17	0.954 ± 0.020	53 ± 4.4
15	22.5 ± 0.7	5.42 ± 0.39	0.93 ± 0.03	2.24 ± 0.15	0.968 ± 0.016	53 ± 4.5

An investigation on the variable α shows its importance in adjusting from the hard mode to the soft mode. Setting α to 0.1 results in the hard-threshold mode and makes F decrease to 2.05, whereas setting α to 0.9 brings F up to 2.12 due to the soft-threshold mode. Maximum F (2.38) occurs when $\alpha = 0.5$ and is located in between two opposite cases. In this condition, the edge recovery error and pulse-width recovery error, computed as the relative deviation of the 10%–90% rising-edge time and of the pulse width of the denoised pulse from those of the clean synthesized template (the rising-edge figure is also reported in Table 5), do not exceed 6.5% and 7.2%, respectively, which are acceptable in terms of engineering criteria for PRPD application.

The most insightful result is obtained for the case of low input SNRs. Specifically, the superior threshold function results in an improvement in output SNR of 2.4 dB at an input SNR of 0 dB and less compared to regular soft thresholding. This difference diminishes as the input SNR increases because the performance difference among the threshold functions decreases as the input becomes more pronounced than the noise.

3.2 Pulse-Type Interference Suppression in the Laboratory

The refined algorithm is then used on the 320 lab files; in each of the pulse interference types, the data is individually analyzed before the composite is formed, whereby the three cases are combined. When considering the 12-pulse scenario, ripple synchronized to a 1.67 ms cycle is added to the defect cell of the void discharge condition. Statistics from the output show that NRR = 6.18 dB, ERR = 0.92, and F = 2.38. The blind area for detection, represented by the ratio of cycles where PD signals cannot be isolated, decreases from 6.5 percent in the raw data to 2.85 percent post-processing. The result is close to the 2.31 percent threshold obtained in past experiments for twelve-pulse rejection systems, while the proposed framework encompasses corona discharge and electronic switching as well.

As for corona, pulses with steepness at 287 ns and oscillation tailing clearly defined are injected into phase points of 90° and 270°. The algorithm produces results of NRR = 5.92 dB, ERR = 0.91, and F = 2.32. The separation between PD pulses at 16 ns and corona pulses at 287 ns is based on the criterion of steepness, and the interference detection rate amounts to 93.6%. For the electronic switching case, spindle-like cluster pulses appear with phase separation at 120° and rising edges at 40 ns; the pulse recognition rate reaches 94.3%. For the combination case, the three kinds of interferences are introduced simultaneously, and the metrics stabilize to NRR = 5.65 dB, ERR = 0.86, and F = 2.20, where the PD identification rate is 91.2%, and it is more practical than the ideal result. Some sample waveforms prior to and after the processing are depicted in Fig. 4, and the combined suppression results are tabulated in Table 2.

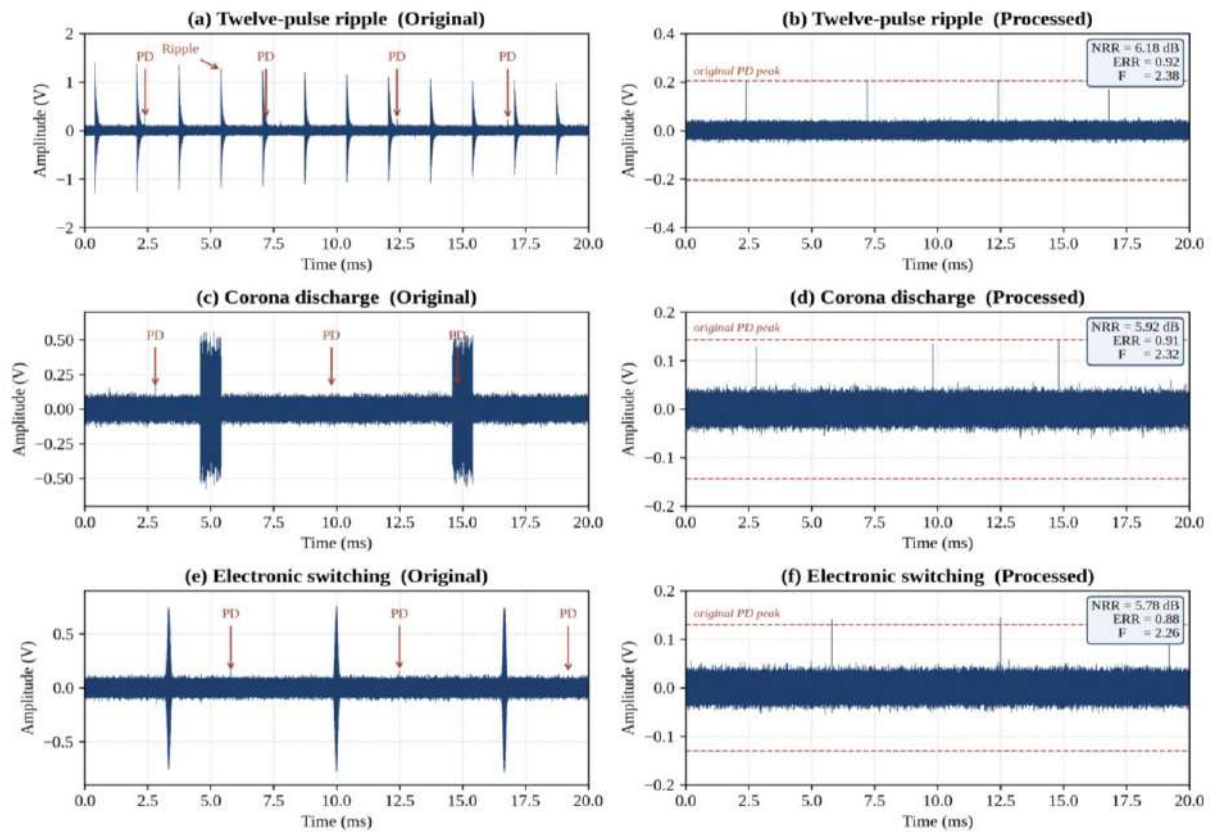


Fig. 4. Comparison of waveforms before and after denoising under three typical pulse-type interference scenarios on the 220 kV physical simulation platform: (a), (c), (e) original waveforms under 12-pulse ripple, corona, and electronic switching interference, respectively; (b), (d), (f) the corresponding processed waveforms. The vertical scale of the processed waveforms is zoomed in to clearly present the recovered nanosecond-level PD pulses after suppression of the high-amplitude interference; the red dashed lines in (b), (d), (f) mark the original PD peak amplitude for reference. Arrows labelled PD mark the partial discharge pulses and "Ripple" marks a 12-pulse ripple pulse in (a)

Table 2. Suppression performance for the three pulse-type interference categories under laboratory conditions.

Interference type	NRR (dB)	ERR	F	Identification rate (%)	Misjudgment rate (%)	Processing time (ms)
Twelve-pulse	6.18	0.92	2.38	95.2	2.85	49
Corona	5.92	0.91	2.32	93.6	4.10	53
Electronic switching	5.78	0.88	2.26	94.3	3.80	51
Composite	5.65	0.86	2.20	91.2	5.40	58

Table 3. Feature-parameter retention of the four canonical discharge defect signals after improved-algorithm processing

Defect type	Sample size	Mean NCC	Rising-edge error (%)	Pulse-width error (%)	First-peak retention (%)
Void discharge	80	0.943	5.8	6.4	94.1
Surface discharge	80	0.928	7.2	8.1	91.8
Floating-electrode discharge	80	0.952	4.6	5.9	95.3
Tip discharge	80	0.961	4.1	5.2	96.0

For the four defect models, NCC along with rising edge time error, pulse width error, and peak energy are used to represent waveform fidelity. The floating electrode and tip discharge have the highest NCC at 0.952 and 0.961, respectively, because their peak energies are highly concentrated and pass thresholding easily. The void and surface discharge models show NCC values of 0.943 and 0.928, respectively. For all the defect models, both the rising edge time and pulse width errors do not exceed 8.1%, while peak energy does not fall below 91%. The numerical breakdown is given in Table 3.

3.3 Field Validation on Three Substation Cases

The three datasets for the substations, as described in Section 2.4, are analyzed with identical parameters employed in the laboratory experiment, and the results are individually presented prior to their compilation. In the first instance (Case I), a transformer under test for a ± 800 kV converter station in May 2025 produces raw data that include a distinct 1.67 ms periodic ripple alongside 12-pulse clusters whose leading edges fall close to 4.2 μ s, while the potential partial discharges barely stand out against this background. Following post-processing, the ripple is filtered out, revealing a clean partial discharge with an 18 ns rising edge, achieving a peak preservation rate of 93.2%. Under $0.8 U_0$ stimulation, a void discharge is detected at an apparent charge value close to 1053 pC against the background signal, matching the manually measured value in the initial commissioning document. A discharge of this magnitude far exceeds the acceptance level for a converter transformer and indicates an active insulation defect; the unit was therefore withheld from energization and returned for defect location and repair before the acceptance test was repeated. The processing time per record is 52 ms.

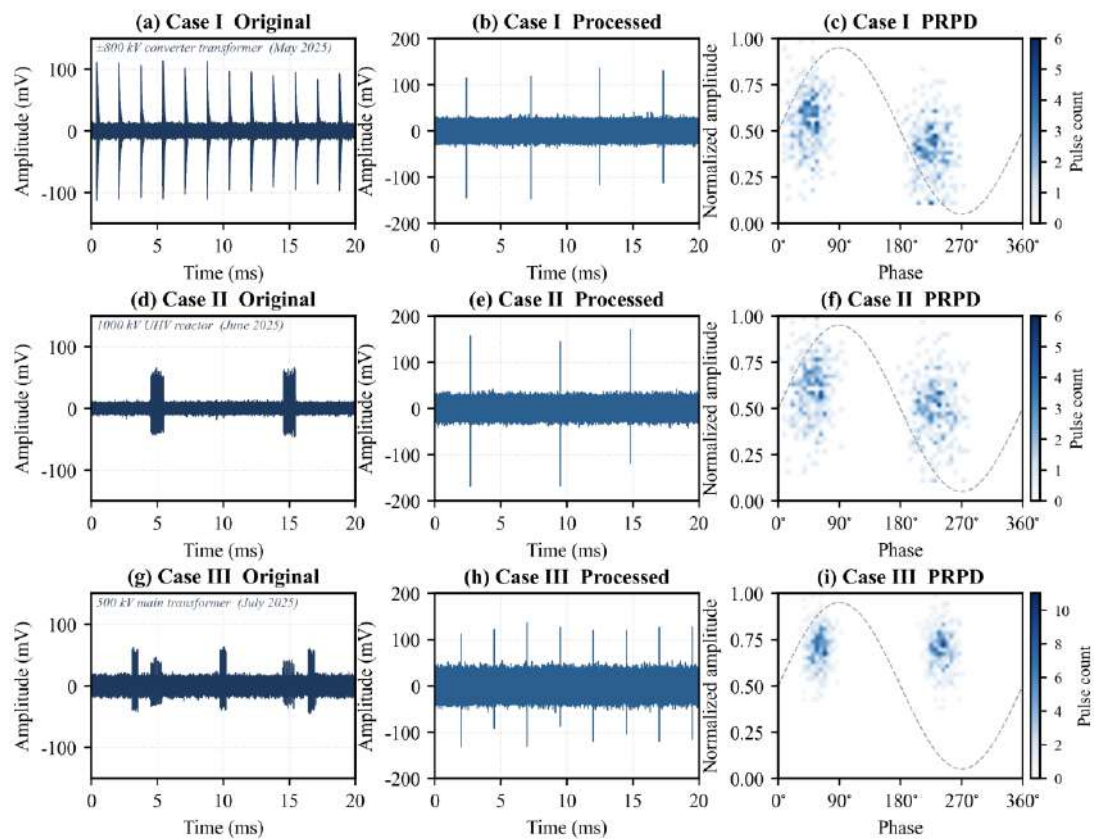


Fig. 5. Field validation of the algorithm on three representative substation cases. Rows correspond to cases I to III; columns show the original waveform, the processed waveform, and the PRPD pattern

The second sample comes from a 1000 kV UHV HV shunt reactor in June 2025. The initial raw signal from the center and the grounding lead of the clamping frame contains strong periodic corona discharges located at 90° and 270° with steepness of about 287 ns and prominent tailing oscillations, along with small PDs at the first and third quadrants. Following the data processing,

the corona discharge components are eliminated, and the signal at 16 ns and approximately 225 mV can be seen in the expected quadrants, which suggests void discharge initially. The maximum errors for feature parameters at the grounding location are below 5%.

The third example refers to a 500 kV power transformer measured in July 2025. The input signal with an applied voltage of 246 kV contains considerable interference from both external and internal sources, including white noise, corona discharges, and switching pulses. In this scenario, the proposed algorithm successfully filters a floating electrode discharge pulse of 3540.9 pC, showing symmetrical and periodic behavior in the first and third quadrants. The ns-level time-difference analysis across the synchronized HFCT channels (Section 2.1) reveals that the bushing-tap signal leads the remaining channels by 25–51 ns, which places the source at the bottom end of the high-voltage bushing. This location was confirmed by subsequent physical inspection, which traced the fault to the connection bolt of the corona ring assembly. A floating-electrode discharge above 3500 pC represents a severe and potentially escalating defect; the transformer was consequently de-energized, and the loose bolt was re-tightened and the assembly inspected before the unit was returned to service, after which the discharge disappeared. On average over all 180 field records, the detection accuracy is 93.7% with an overall misdiagnosis ratio of 3.5% (the higher 5%–8% figure discussed in Section 4.2 refers only to the subset of near-field pulses), and the parameter reconstruction error is within 8%. Time-domain and PRPD comparisons are shown in Fig. 5, and the consolidated case-level metrics are listed in Table 4.

Table 4. Summary of algorithm performance on the three field cases

Case	Site	Dominant interference	Test date	Detected PD type	NRR (dB)	First-peak retention (%)	Detection accuracy (%)	Processing time (ms)
I	±800 kV converter station, converter transformer	Twelve-pulse	2025.05	Void discharge	6.05	93.2	94.5	52
II	1000 kV UHV station, high-voltage reactor	Corona	2025.06	Void discharge	5.83	91.6	92.8	54
III	500 kV substation, main transformer	Composite	2025.07	Floating-electrode discharge	5.71	90.4	93.7	57
Pooled	/	/	/	/	5.86	91.7	93.7	54.3

Note: Pooled metrics are equal-weight averages of the three cases ($N = 60$ records per case, total $N = 180$).

3.4 Comparison with Conventional Denoising Methods

The enhanced algorithm is compared to four existing algorithms, which are the adaptive SVD, EMD coupled with wavelet thresholding, and both the hard and soft thresholds techniques. All five techniques were tested using the same 500-element simulated dataset with equivalent mother-wavelet, decomposition level, and SNR sweeps, and means were calculated from all datasets. SNR output enhancement for the proposed method amounts to 9.3 dB, in comparison to 8.5 dB, 7.8 dB, 7.1 dB, and 6.2 dB respectively for the other four algorithms. Waveform accuracy is consistent with the above results: the NCC measures for the proposed approach is 0.91, while the NCC for the other approaches remains 0.87, 0.85, 0.83, and 0.78, respectively. Performance on 1024 samples data differs. The enhanced algorithm requires only 52 ms compared to the 38 ms required by hard thresholding and 41 ms required by soft thresholding but significantly faster than the 420 ms for EMD-thresholding and 185 ms for adaptive SVD. The percentage improvement in speed is 87.6% for EMD-thresholding and 71.9% for adaptive SVD. The extra time taken (approximately 25%) for enhanced algorithms yields a 13.3% improvement in total fitness F over hard thresholding and a 15.0% improvement over soft thresholding. The six-dimensional comparison is shown in Fig. 6, in

which each metric is normalized to its maximum across the five methods (range 0–1) and the raw value is annotated above each bar; the computational efficiency score is defined as $\eta_c = t_{min}/t$, with t_{min} the shortest processing time among the five methods; and the consolidated metrics are listed in Table 5.

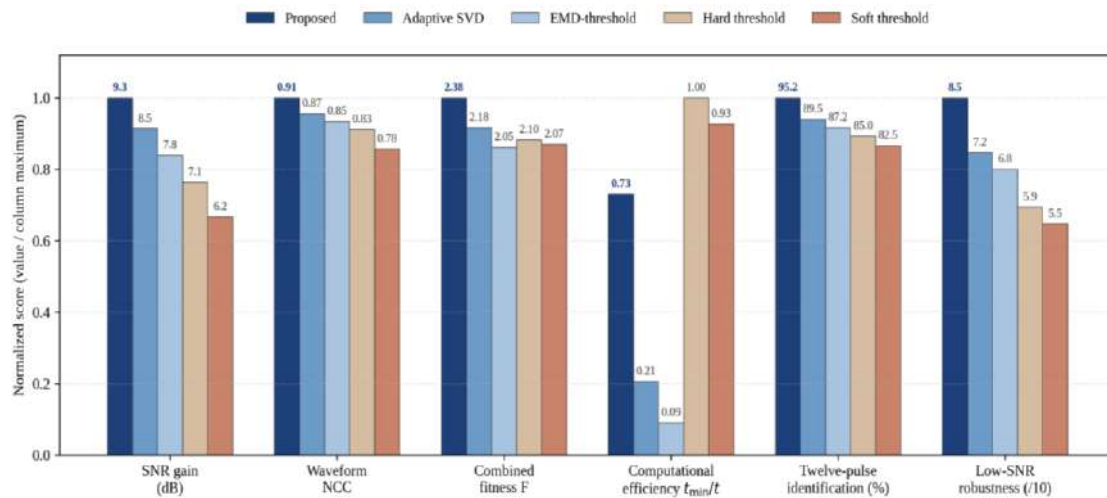


Fig. 6. Multi-dimensional bar-chart comparison of five denoising methods on the simulation dataset (each metric normalized to its column maximum; raw values from Table 5 annotated above the bars; computational efficiency = t_{min}/t)

Table 5. Performance comparison of five denoising methods (mean values across 500 simulation samples)

Metric	Proposed	Adaptive SVD	EMD-threshold	Hard threshold	Soft threshold
SNR gain (dB)	9.3	8.5	7.8	7.1	6.2
Waveform NCC	0.91	0.87	0.85	0.83	0.78
Combined fitness F	2.38	2.18	2.05	2.10	2.07
Processing time (ms)	52	185	420	38	41
Twelve-pulse identification rate (%)	95.2	89.5	87.2	85.0	82.5
Rising-edge retention error (%)	6.5	8.7	10.2	7.8	12.6
Robustness score (−5 dB to 15 dB)	8.5 / 10	7.2 / 10	6.8 / 10	5.9 / 10	5.5 / 10

Collectively, the advanced algorithm beats all four of the thresholding strategies on the first three out of the five performance criteria, specifically the SNR enhancement factor, waveform preservation, and the percentage of twelve-pulses detected, while the rising-edge error remains below the 8% threshold. While the traditional thresholding schemes enjoy better execution time for noise-free signals, their maximum achievable SNR and waveform preservation levels are severely degraded when pulse-type noises exist in the signal, in line with Section 3.2's experimental findings. EMD-threshold and adaptive SVD can fill in some of the SNR loss compared to thresholding schemes, but the computational overhead of each sample makes integration into a 250 MSps stream impossible. Hence, the novel design is in an optimal position between efficiency and accuracy, providing multi-tens-of-milliseconds latency and achieving SNR and NCC values only partially attained by baseline solutions.

4. Discussion

4.1 Stability of the Improved Threshold Function

Another interesting aspect that needs exploring is the sensitivity of the algorithm to the factor α . When α varies within the range of 0.3 to 0.7, F remains in a margin of 8 percent ($2.18 < F < 2.38$). The best value for α is 0.5, which occurs in the middle of the gradual shift between the hard and soft cases, in agreement with the continuous derivative nature seen in Fig. 3. This indicates that minor changes in α have little effect on the results, making it easier to control the parameters used in practice [24].

The case of SNR adaptive range follows suit. For the -5 dB to 15 dB range examined in Section 3.1, the ratio deviation is less than 12% of the average of the NRR, implying that the layer-based threshold estimation remains highly stable and robust despite any level of contamination. This phenomenon conforms to results of optimization-based wavelet-based denoising of nonstationary power system signals where threshold adaptation exhibited stability at different SNR values [25]. The dependence of the mother-wavelet choice on the noise source reported in PD denoising [26].

The two design decisions that provide this robustness are first, the use of a continuously differentiable threshold function that gets rid of the discontinuity at $|w|=\lambda$ and the pseudo-Gibbs ringing effect of hard thresholding, and second, using the estimator based on layer-specific variance estimates for the detail layers. They complement each other in a way that each cannot do alone, with the former preserving waveform shape and the latter avoiding over-suppression or under-suppression at a single scale. It is their joint action that makes possible the bounded variation of the metric values over α and SNR.

4.2 Adaptability Across Interference Types

Pulse-types fall under three classifications, each of which has varying degrees of difficulty in recognition. The first type is the twelve-pulse ripple with a success rate of 95.2%. This is because this type has a periodicity of 1.67 ms and a rise time in the microsecond range. These characteristics are quite distinct from the nanosecond-range rise time characteristic of PD. Using either the time domain steepness test or phase periodicity is sufficient to identify the twelve-pulse pulse-type.

The next classification is the corona interference type at 93.6%. In this case, the difference in steepness between the 287 ns corona and the 16 ns PD becomes sufficiently significant to separate them by means of the steepness threshold and oscillation criterion. Some rare pulses of low-intensity corona interference that have been transferred via the winding show a steepness value similar to those of the remote PDs, and these are the situations in which errors can take place. The technique of SVD-based spectrum subtraction has been proposed as an additional step in this case [27].

The third category is electronic switching interference, identified at 94.3% under laboratory conditions (Table 2). Recognition becomes harder once the three categories are superimposed: the PD identification rate of the composite laboratory case drops to 91.2% (Table 2), and in the corresponding composite field case (Case III) the detection accuracy is 93.7% (Table 4), the field records containing a more diverse set of switching sources than the laboratory setup. The phase pattern of 120° and the spindle envelope lead to the differentiation, despite the transient signals produced by switching close to the source having extremely sharp leading edges that come quite close to the PD levels [28]. The composite method is based on the cooperative effect between cumulative energy pre-localization and wavelet domain thresholding, where pre-localization extracts the candidate window while thresholding removes the residual broadband component. The other issue that persists is the presence of near-field pulses with waveform similar to PD, where misclassification within this subset can still occur between 5% and 8% (compared with the overall field misdiagnosis ratio of 3.5% in Section 3.3), and multichannel time of arrival fusion, coupled with hybrid wavelet-machine learning routing, could be a logical progression [29].

4.3 Computational Efficiency and Engineering Applicability

Real-time performance comes as the first requirement. When sampling at a frequency of 250 MSps, each acquired 20 ms power-cycle record takes 52 ms to process with our approach, fitting into the 100 ms update interval and allowing room for buffer management and generation of the PRPD pattern afterwards. It should be stated explicitly that the platform therefore operates in a buffered, duty-cycled mode rather than in gap-free acquisition: one 20 ms record is captured and processed per 100 ms update interval, i.e., a temporal coverage of 20%. Because PD assessment via PRPD patterns is statistical over many cycles, and because the pilot deployments (Section 4.3) showed that this coverage was sufficient to reduce the detection blind zone to 2.5%–3%, the mode is adequate for online condition assessment; nevertheless, it does not capture every power cycle, and gap-free acquisition would require pipelined processing on the FPGA implementation discussed below (Section 4.4). In comparison, the performance is much better than those of EMD-threshold (420 ms) and adaptive SVD (185 ms) from Section 3.4, matching the latency range found for hybrid wavelet-TV processing of PD signals [30].

The second issue concerns hardware portability. It can be easily stated that all basic computations carried out in the pipeline, which are wavelet decompositions of up to six levels, thresholding using the enhanced procedure, and calculation of total energy, are well suited to an embedded implementation on an FPGA+DSP platform. A synthesis study of the pipeline (not yet field-deployed; the results in Section 3 were obtained on the multi-core processor described in Section 2.1) indicates utilization of roughly 65% LUT resources and 42% BRAM resources for a mid-range FPGA device (Xilinx Artix-7 XC7A200T). Both these numbers lie below the saturation threshold, and they are on par with SVD PD denoising chains on similar hardware [31].

The feedback from field pilot tests reinforces the laboratory results. The detection blind area under high levels of interference reduced from a range of 6-8% to 2.5-3%, for the $\pm 800\text{kV}$ converter station of Xinjiang, 1000kV ultra-high voltage substation in Nanyang, Henan province, and the 500kV main transformer site. The detection rate of PD pulses improved by about 15.3% against the pre-installation benchmark, facilitating early detection of insulation faults at commissioning and periodic maintenance phases.

4.4 Limitations and Future Work

There are several limitations that restrict the results obtained here. The study focuses mainly on the pulse current signals measured using the HFCT technique; however, the effectiveness of the proposed approach for ultrasound acoustic emission is yet to be confirmed using similar field measurements, considering the differences in the statistics of the noise channels and paths between the acoustics and the electrical front end. The adjustable parameter α is set to 0.5 as the empirically optimized value from the simulation sweep presented in Section 3.1, and has not been adapted on a per-record basis. The performance below the -5 dB lower bound of the investigated SNR range has not been characterized. Furthermore, the present platform acquires one 20 ms record per 100 ms update interval (Section 4.3), so PD events falling in the unrecorded intervals are not captured. The ability to adapt to novel interference types due to newly developed power electronic devices is constrained by the low quantity of such data in the current field database.

A number of approaches will be pursued to mitigate these limitations. First, the pipeline will be extended towards an implementation of acoustic-electric fusion through the coupling of the method described above with ultrasonic noise reduction and total variation autoencoders designed specifically for acoustic PD [32]. Second, particle swarm optimization will be used to automatically select α for each record. The third focuses on the combination between the optimized wavelet threshold approach with deep neural networks such as autoencoders and adaptive iterative filtering combined with the Bidirectional Long Short-Term Memory model [33]. Fourthly, the expanded database takes account of new forms of interfering noise from pulsating sources, and the results are linked to future localization[34] and recognition of PRPD patterns[35]. Thus, the benefits of increased waveform accuracy are transferred into the diagnostic decision-making process.

5. Conclusions

The engineering problem targeted in this research effort relates to the inability to detect partial discharges due to pulse-type interference in UHV transformer systems. An innovative technique for suppressing pulse-type interference through the use of an enhanced wavelet threshold has been invented, and is incorporated into a wideband detection system featuring a 150 kHz to 60 MHz signal conditioning bandwidth coupled with a 3–50 MHz high-frequency current transformer (HFCT) sensor, to create a fully-fledged signal-processing decision-making chain.

Regarding methodology, a continuously differentiable threshold function with a variable parameter α has been devised, which maintains the robust nature of hard thresholding while incorporating the smoothness of soft thresholding, without inheriting their limitations. An adaptive threshold value estimation scheme dependent upon a particular layer is adopted in lieu of the universal threshold scheme that has the tendency of over-sensing or under-sensing at low-frequency and high-frequency layers respectively when using multiresolution decomposition. The preprocessing of pre-localization and discrimination using time-frequency cumulative energy schemes enables the algorithm to reliably detect true PD events prior to thresholding.

In terms of results, the performance of the simulation data set is demonstrated by an average SNR improvement of 9.3 dB and a fitness F value of 2.38 at 5 dB input SNR, surpassing the values for EMD threshold (7.8 dB, $F=2.05$) and adaptive SVD (8.5 dB, $F=2.18$). In addition, all four standard defects in the lab data set possess an NCC value exceeding 0.92. The laboratory identification rates for the three interference classes are 95.2%, 93.6% and 94.3% respectively (Table 2); the field data yield an average detection accuracy of 93.7%, and feature parameter errors stay within 8.1%. The single data record processing time averages at 52 ms, satisfying the <100 ms online update requirement in a buffered, duty-cycled acquisition mode.

Value for engineering in regard to the three pilot installations at ± 800 kV converter station, 1000 kV UHV substation, and 500 kV main transformer installation involved reducing the PD blind detection zone from 6%-8% to 2.5%-3%. In addition, the percentage of PD pulse detection was increased by 15.3% as compared to before the pilot installations. These results provide a functional means of analysis for engineers operating under conditions of high interference and enable more accurate assessment of the insulation condition of grid equipment. Future work will focus on extending this algorithmic approach to hybrid acoustic-electric interference removal, runtime optimization of α , and integration of deep learning models, the overall goal being to increase generalizability toward new sources of pulse-based interference in power electronics.

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Structural performance of carbon-FRCM in strengthening and repairing pre-damaged RC slabs

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Abstract

This experimental study aims to investigate the structural behavior of a Carbon Fiber Reinforced Cementitious Matrix (C-FRCM) strengthening and repairing system. The experimental program was designed to assess two strengthening coverage levels: a full-surface coverage (100%) and a patch layout (50%) to evaluate the influence of material distribution. In order to assess the repair/strengthening efficiency, specimens were initially pre-loaded to 50% (in terms of applied force) of the ultimate load to introduce a controlled level of structural damage before the application of the FRCM system. The findings indicate that the Carbon-FRCM system can effectively improve the load-carrying capacity of undamaged slabs and contribute to the structural performance recovery of pre-damaged slabs under the investigated experimental conditions. All strengthened specimens exhibited load capacities higher than that of the control slab, suggesting the potential of the system as a strengthening and repair technique. The results also show that while full coverage provides higher stiffness and more uniform crack control, localized reinforcement can achieve significant improvement in load capacity with reduced material usage. Within the scope of the present experimental investigation, the Carbon-FRCM system exhibited promising structural performance as a strengthening and repair technique for reinforced concrete slabs under the investigated loading conditions.

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1. Introduction

The growing degradation of reinforced concrete (RC) infrastructure is one of the major worldwide threats to the sustainability and resilience of our structured development [1, 2]. The combination of a few important factors, such as the development of corrosion due to material aging and aggressive environmental conditions (e.g., chloride ingress and carbonation), and increasing requirements imposed by new types of use that go beyond the original performance criteria lead to this structural degradation [3, 4]. The inadequacy of elements within the structural hierarchy are especially harmful to RC slabs, as they represent the most significant horizontal elements in the load-carrying system [5]. As a result, slabs are often among the most critical structural components in terms of serviceability, as they are highly susceptible to excessive deflection and flexural cracking, which may adversely affect the overall structural performance of the building system [6]. Nevertheless, the need for intervention for slab systems can be categorized into two engineering challenges: proactive "strengthening" for higher loading protocols and reactive "repairing" to recover structural capacity lost due to damage or environmental degradation. In either context, the ultimate goal is to extend the service life of existing structures while reducing the environmental and economic costs of complete demolition and reconstruction [7]. In retrospect, this led to a shift away from cumbersome traditional practices such as steel plate bonding, characterized by

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corrosion vulnerability and installation related difficulties, to use of expensive composite materials in structural retrofitting [8].

The later introduction of Fiber Reinforced Polymer (FRP) composites marked a revolution in structural rehabilitation. With outstanding strength-to-weight ratios and natural resistance to corrosion, FRP systems made with carbon, glass or aramid fibers encapsulated in organic epoxy resins provided an efficient solution for flexural and shear capacity increases with minimal changes in structural geometry [9-11]. However, there are very considerable technical limitations associated with FRP because of the approach used they are dependent on organic polymers which persist in the environment. In particular, epoxy resins exhibit poor thermal properties which may result in loss of bond integrity and catastrophic failure during fire conditions. In addition, the impervious quality of FRP creates a vapor barrier, which can keep moisture inside the concrete that can lead over time to delamination, or freeze-thaw damage [12]. In order to avoid these thermal and chemical incompatibilities, Fiber Reinforced Cementitious Matrix (FRCM) has been proposed as a more compatible and "breathable" reinforcement alternative [13].

Due to the use of an inorganic, mineral-based cementitious mortar instead of organic epoxy, FRCM provides excellent fire resistance and avoids a potentially detrimental chemical interaction with the concrete substrate [14, 15]. Symmetrically, FRCM provides a complex bond-slip mechanism as well as a multi-linear stress-strain response that fosters a more ductile failure mode in comparison to the brittle debonding of FRP applications [16, 17]. When the cementitious matrix experiences micro-cracking at a threshold level, the crack will become consolidated so that the load is progressively redistributed across the fiber mesh, resulting in an amount of strain warning that is dispersed as diffuse micro-cracking rather than catastrophic failure [18]. While FRCM is already established as an effective technique to strengthen undamaged load bearing members, the "repair" of structurally damaged elements presents new engineering challenges [19]. It also requires the repair of already greatly sustained damage members which requires a method of fusing macro-cracks existing while also being able to restore sectional stiffness. Here the reinforcement coverage ratio is a key ballpark figure for performance as well as cost performance. One approach is to use localized patches (or sheets) instead of the full-surface coverage, this is a strategy to reduce material consumption especially because carbon fiber is very expensive, without sacrificing the total capacity uptake [20].

Recent life-cycle cost analyses also corroborate this paradigm shift, indicating that although FRCM entails more labor at the outset, the increased durability with decreased long-term maintenance requirements makes it a relatively more economical solution through the life of a structure [19, 21, 22]. Therefore, the current research aims to study the behavior of reinforced concrete slabs strengthened with Carbon-FRCM under loading with two possible strengthening ratios: full surface coverage and localized application in the region of maximum moment. In particular, the capacity of partial strengthening configurations to produce efficient stress redistribution and crack control with limited materials is emphasized in this work. Furthermore, the main aim of this study is to evaluate the structural performance recovery of slabs pre-damaged at moderate damage levels (50% of the ultimate load of the control specimen) and subsequently repaired using Carbon-FRCM. The paper presents a detailed investigation on load-deflection behavior, stiffness development and failure modes to increase the understanding of the interaction of the structural behavior between the cementitious matrix, textile reinforcement and the damaged concrete substrate in two-way slab systems.

2. Experimental Program

2.1 Materials

2.1.1 Cementitious Mortar

In fiber-reinforced cementitious matrix (FRCM) composite, the cementitious mortar serves as the inorganic binding matrix that encapsulate the textile reinforcement and facilitate stress transfer between the reinforcement and the concrete substrate [23, 24]. An appropriate cementitious matrix is critical for load redistribution, bond performance and long-term durability in

strengthening members [25]. For the present study, the cementitious mortar selected was Sika MonoTop®-612[26], a one-component, fiber-reinforced, polymer-modified structural repair mortar that meets the requirements of Class R3 according to EN 1504-3[27]. According to the product's technical specification, the mortar comprises of cement, specified aggregates, silica fume, polyamide fibers, and additives, and appears as a gray powder that is mixed with clean water prior to application. The mortar was made carefully according to the manufacturer's specifications, assuring a constant water-to-powder ratio, complete mixing (mechanical mixer at low speed), and proper handling to produce uniform workability. The mechanical properties and technical specification of the mortar, obtained from the manufacturers data sheet are listed in Table 1. It should be noted that the material properties reported in this study are based on manufacturer technical data sheets for commercially available FRCM components, and no additional experimental characterization of the composite system or interface behavior was performed within the scope of this study.

Table 1. Properties of Sika MonoTop®-612

Property	Description
Composition	Polymer-containing cementitious mortar with different additives and synthetic fiber.
Appearance / Colour	Grey powder
Maximum grain size	Dmax: 2.0 mm
Compressive strength	28 days (ASTM C109) ~40 MPa *Testing cubes 50 x 50 x 50 mm
Mixing ratio	3 - 3.25 L of water per 25 kg bag
Fresh mortar density	Fresh mortar density 1.95 kg/L at +20 °C
Layer thickness	Min. 5 mm, Max. 30 mm

2.1.2 Fabric-Reinforced Cementitious Matrix (FRCM) Textile

In fabric-reinforced cementitious matrix (FRCM) systems, textile reinforcement represents the main load-carrying component responsible for controlling the tensile response and governing the overall structural performance of strengthened members [17, 19]. Textile reinforcement in FRCM has a high tensile capacity and durability better than FRP under severe environmental and thermal conditions due to its inorganic matrix and better compatibility with concrete [28]. Carbon, glass, basalt, and polyparaphenylene benzobisoxazole (PBO) fibers have all been used in FRCM applications, with each having its own mechanical and durability properties [19].

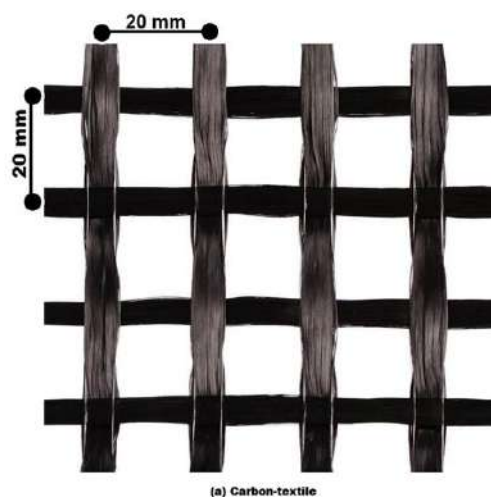


Fig. 1. Selected carbon-textile

Textile reinforcements are chosen based on their performance and durability for application within FRCM systems. Carbon fabrics are characterized by impressive high tensile strength, high elastic modulus, low creep, and long-term stability [19, 28, 29], which are favorable for high-performance strengthening applications. In this study, carbon textile was used as reinforcements

in the FRCM system and the mechanical behavior was compared under flexural loading. The carbon textile has composite nominal thickness of 0.047 mm and mesh size of 20 × 20 mm as shown in Fig. 1. Table 2 summarizes the mechanical properties of the textile while Fig.1 shows its nature within the strengthening system.

Table 2. Characteristics of FRCM textile based on the manufacturer's data sheet.

Material	Nominal thickness	Tensile strength	Modulus of elasticity
Carbon	0.047 mm	3600 MPa	230 GPa

2.1.3 Reinforced Concrete

2.1.3.1 Reinforcing Steel

Reinforcing steel improves reinforced concrete by providing tensile strength, ductility, and greater bond with concrete, minimizing cracking and enhancing structural load-bearing capacity [29]. 6 mm diameter deformed bars were used in the present work to provide better interlocking of mortar and bond strength. All reinforcing bars shall conform to the ASTM A615/A615M-20 [30] standard specification for deformed and plain carbon-steel bars for concrete reinforcement, providing consistent chemical and mechanical properties. Calibrated micrometers were used to take measurements for the diameter and thickness of the bars; tensile testing was performed according to ASTM standards to determine yield and ultimate strength for the selected Grade 40 steel (ASTM A615/A615M-20) [30]. A detailed mechanical property of the reinforcing steel can be found in Table 3.

Table 3. Properties of reinforcement steel

Property	values
Nominal diameter (mm)	6
Actual diameter (mm)	5.8
Cross-sectional area (mm ²)	26.4
Yield stress (MPa)	405.3
Ultimate stress (MPa)	605
Elongation (%)	17
Nominal weight (Kg/m)	0.208
Grade	G40

2.1.3.2 Concrete

Concrete is one of the most widely used construction materials due to its high compressive strength, good durability and it can be applied in various structural applications [31]. The mechanical properties of the normal-strength concrete oriented in this study were evaluated using the standard tests compressive strength, splitting tensile strength and flexural strength to give an overview of its structural behavior. Compressive tests were performed on standard cubes measuring 100mm × 100mm × 100mm, while standard cylindrical 100mm × 200mm test pieces were employed for tensile tests, and the flexural strength of the prism specimens was determined. All tests were conducted according to British Standards BS 1881 with the customs loading rates. Table 4 will illustrate the concrete standards and the measured values.

Table 4. Summary of concrete mechanical properties

Property	Specimen Type	Average (MPa)	Test Standard
Compressive Strength	Cube	34.8	BS 1881: Part 116 [32]
Splitting Tensile Strength	Cylinder	2.27	BS 1881: Part 117 [33]
Flexural Strength	Prism	5.58	BS 1881: Part 118 [34]
Modulus of Elasticity	N/A	27.7 GPa	Empirical formula

2.2 Test Specimens and Parameters

2.2.1 Overview

An experimental program was developed to study the flexural performance, strengthening effectiveness, and general behavior of Fabric-Reinforced Cementitious Matrix (FRCM) strengthened reinforced concrete (RC) slabs. In order to have similar boundary conditions and also general characteristics from the test specimens, a total of five square RC slab specimens were fabricated having identical geometry, thickness, and boundary conditions. The specimens were constructed of $1.0 \text{ m} \times 1.0 \text{ m}$ plan dimensions to correspond to a representative plan size of scaled RC slabs and accommodating laboratory constraints.

2.2.2 Geometry and Reinforcement

The reinforced concrete slab specimens were all $1.0 \text{ m} \times 1.0 \text{ m}$ in plan and had a consistent thickness of 80 mm. The concrete cover was kept 15mm for all the reinforcement. All specimens had the same geometry and reinforcement layout. There were two orthogonal directions of main flexural reinforcement spaced at 192 mm with 6 mm diameter steel bars. The main reinforcement was arranged accordingly so that the slab bottom face received the reinforcement on both the total area of slab perimeter. The bottom rebar at the slab edges were bent up at 180° at the supports in order to give them continuity at the slab perimeter, which formed the top rebar. Next to the supports, localized top reinforcement was provided with bent bars, which were oriented perpendicular to each edge and spaced at 192 mm.

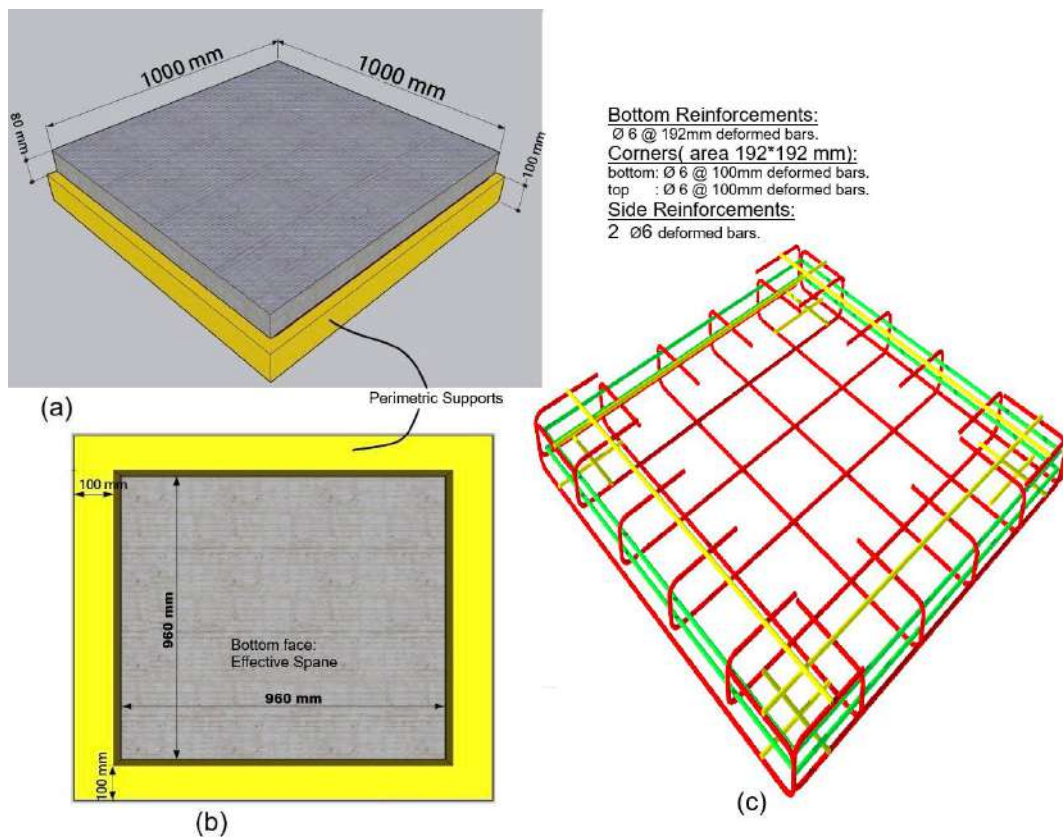


Fig. 2. (a) The slab's geometry; (b) the slab's perimeter-supported bottom face; and (c) the slab's steel reinforcement details

Reinforcement detailing has been further improved to enhance anchorage behavior and crack control, and to mitigate stress concentrations near the slab edges and corners. By addition, two further 6 mm bars were placed at the bottom face of the slab within a square area of $192 \text{ mm} \times 192 \text{ mm}$ at each corner of the slab, leading to an effective reinforcement spacing of around 100 mm in this area. In addition, on the top face, a 6 mm bar was placed parallel to the edges of each slab, 100 mm from the edge, with a length of 960 mm. Due to having these edge bars, the density of

reinforcement at corners of slab on its top face were increased with a spacing of 100 mm (approx). The above-mentioned details led to better anchorage behavior, better bond and crack distribution as well as lower stress concentration in these critical areas. Fig. 2(a), (b) and (c) shows the slab geometry, bottom face of perimeter supported slab, and the reinforcement layout respectively.

2.2.3 Studied Parameters

This experimental program was developed to study the effect of strengthening material, strengthening configuration, and preloading condition on the flexural behavior of two-way reinforced concrete slabs. For reliable comparison, all other parameters such as concrete characteristics, slab dimensions, reinforcement ratio, mortar type, loading conditions, support conditions, and effective span were kept constant in this experimental program.

The parameters ranged as ratio of strengthened area to effective span, and loading condition (unloaded or 50% pre-loaded to the ultimate load of the control slab during FRCM strengthening). Therefore, one specimen was tested without the strengthening as a control slab and indicated with an R.

The strengthening schemes included slabs strengthened before loading with various fiber reinforced cementitious matrix (FRCM) configurations of carbon fiber, including continuous strengthening with two layers covering 100% of the effective bottom surface (C2100), as well as two orthogonal layers covering 50% of the effective span applied as continuous layers in the max moment region (C250). Also prior to strengthening, two slabs were preloaded to 50% of the control slab capacity to mimic conditions of retrofitting, denoted as CD250 and CD2100, respectively. Fig. 3 provides a schematic illustration of the strengthening configurations, and Table 4 summarizes the specimen details and test variables.

2.2.4 Specimen Notation

The notations used to designate each tested specimen according to its strengthening condition and configuration are presented schematically in Table 4, where R is the reference specimen tested without any strengthening and that serves as the control slab. Specimen C250 is a completely unloaded slab that has been reinforced with a carbon-based FRCM system. Here, two orthogonal carbon textile layers were layered in the region of M_{max} , both covering 50% of the effective span in a continuous way. Two Continuous carbon FRCM layers fully covering the effective bottom surface were used as the strengthening solution for specimen C2100.

Table 4. Samples, experimental parameters, and materials

Specimen	Strengthening Material	Number of Layers	Strengthening Layout	Coverage (% of effective span)	Preload Condition	Notes
R	None	None	None	None	None	Control slab
C250	Carbon	2	Continuous sheets	50	None	Applied in maximum moment zone
C2100	Carbon	2	Continuous	100	None	Full bottom surface coverage
CD250	Carbon	2	Continuous sheets	50	Preloaded 50%	Retrofitting condition, Applied in maximum moment zone
CD2100	Carbon	2	Continuous	100	Preloaded 50%	Retrofitting condition, Full bottom surface coverage

To evaluate retrofitting scenarios, specimens CD250 and CD2100 were initially preloaded to 50% of the control slab's ultimate capacity prior to strengthening. Specimen CD250, identical in configuration to C250, was reinforced with two orthogonal carbon Fiber-Reinforced Cementitious

Matrix (FRCM) layers covering 50% of the effective span. Similarly, specimen CD2100, mirroring specimen C2100, was strengthened with two continuous carbon FRCM layers integrated along the entire bottom effective surface.

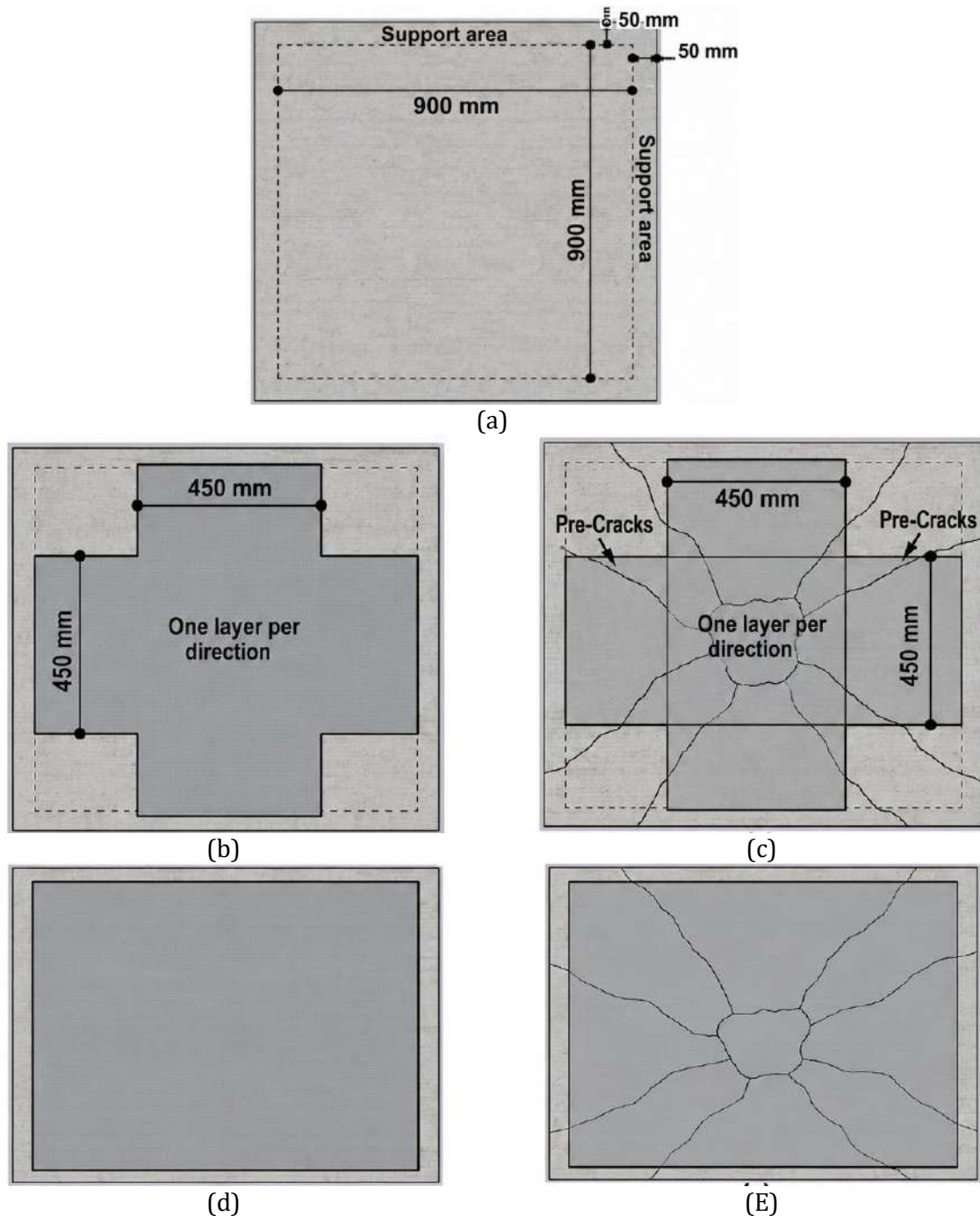


Fig. 3. Configuration of FRCM strengthening on the tested samples/ (a) Control slab R, (b) C250, (c) CD250, (d) C2100 (e) CD2100

2.3 Retrofitting Procedure

The systems used to strengthen and retrofit the reinforced concrete slab specimens in this experimental program included the use of fabric-reinforced cementitious matrix (FRCM) systems with carbon textile, which are known to have high tensile properties, good durability, and compatibility with cement-based substrates [[19, 36]. All specimens followed the same consistent and reproducible strengthening procedure, which is schematically depicted in Fig. 4. Before the strengthening, the tension faces of the concrete were mechanically cleaned to get rid of dust, laitance, and loose materials. A shallow grid of grooves (about 2 mm in depth) was implemented

on the concrete surface to increase the bond effectiveness and the mechanical interlock between the substrate and the cementitious matrix (Fig. 4a).

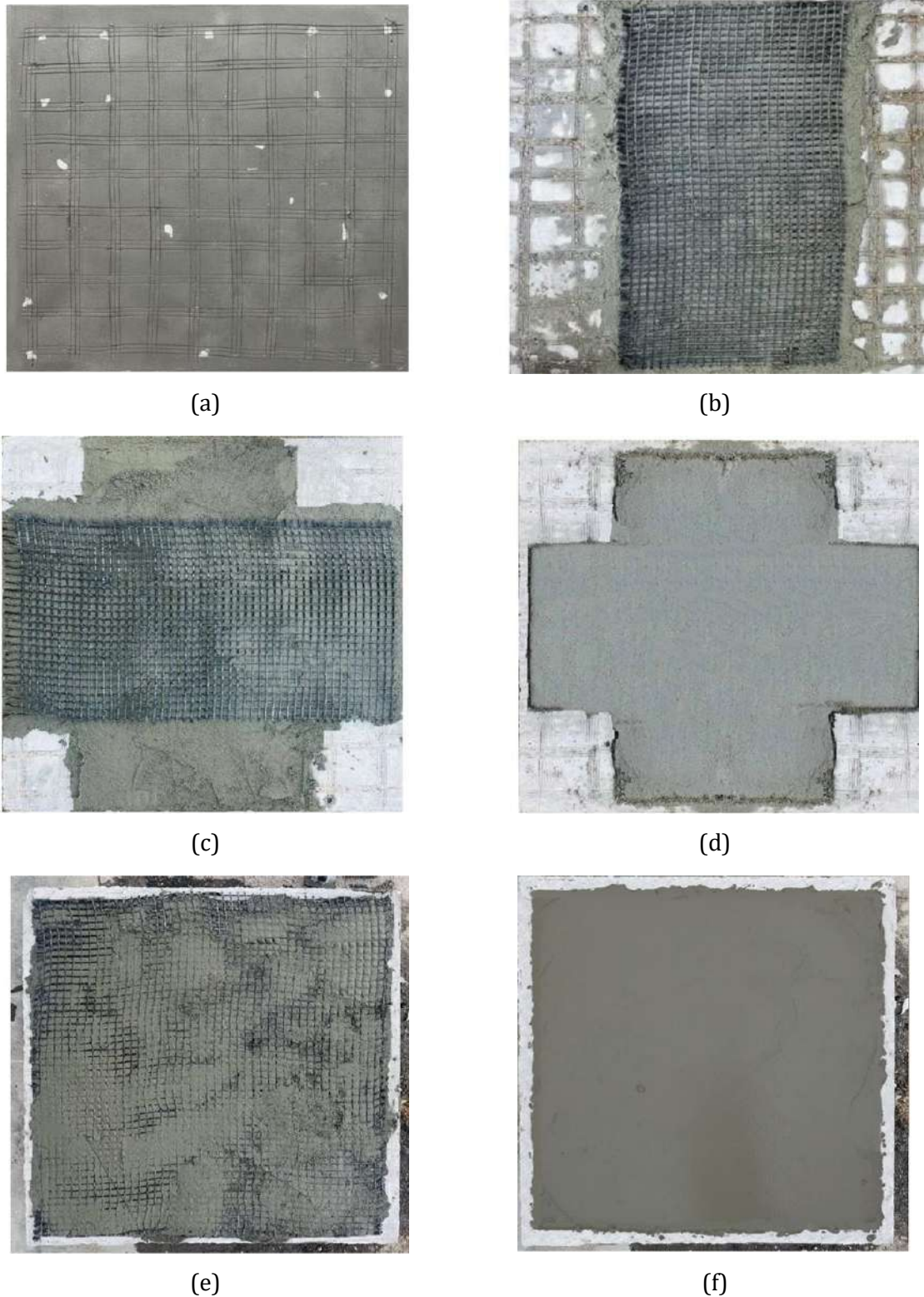


Fig. 4. Application steps for FRCM reinforcement: (a) Preparing and Moistening the concrete surface for strengthening; (b and e) Application of the first mortar layer together with the carbon textile; (c) Application of the second mortar layer along with the carbon; (d and f) Application and finishing of the final mortar layer

Moistening of the prepared surfaces was performed to improve adhesion before mortar application. Then, a first layer of cement-based mortar (approximately 3 mm thickness on

average) was applied on the area where the strengthening would be performed, according to the strengthening layout given by the notation of the specimens. The carbon textile reinforcement for each specimen was specifically prepared and cut in accordance with the designated strengthening configuration. The textile was embedded and lightly compacted by hand, while the mortar was still fresh, to guarantee proper contact and impregnation as well as avoid air voids (Fig. 4b and e).

More layers of mortar were applied to fully encapsulate the textile, followed by the placement of another layer of textile (Fig. 4c). This successive layer was applied with sufficient care for to settle correctly and bond. The strengthening process concluded with the application of a final thin layer of mortar and meticulous finishing/smoothing of that surface, as seen in Fig. 4(d and f). Following strengthening, all the specimens were cured until the adequate mechanical performance of the FRCM system was achieved before proceeding with the tests. To facilitate evaluation and comparison of the effects FRCM system in both strengthening and repairing for the flexural behavior of the slabs, the same strengthening treatment was uniformly applied across all slabs.

2.4 Test Setup and Loading Protocol

The testing of all the slab specimens was done using a four-point loadin configuration with a universal testing machine of capacity 2000 kN. The four edges of the slabs are simply supported with a continuous support system along the slab perimeter. The support framework has 4 steel beams each welded to a steel bar 25 mm in diameter. In order to provide uniform stress transfer and avoid local stress concentrations each of the steel bars, prior to placement of the concrete slab, had a 10 mm wide rubber strip placed on the top face so that the slab tension face was in direct contact with the support system.

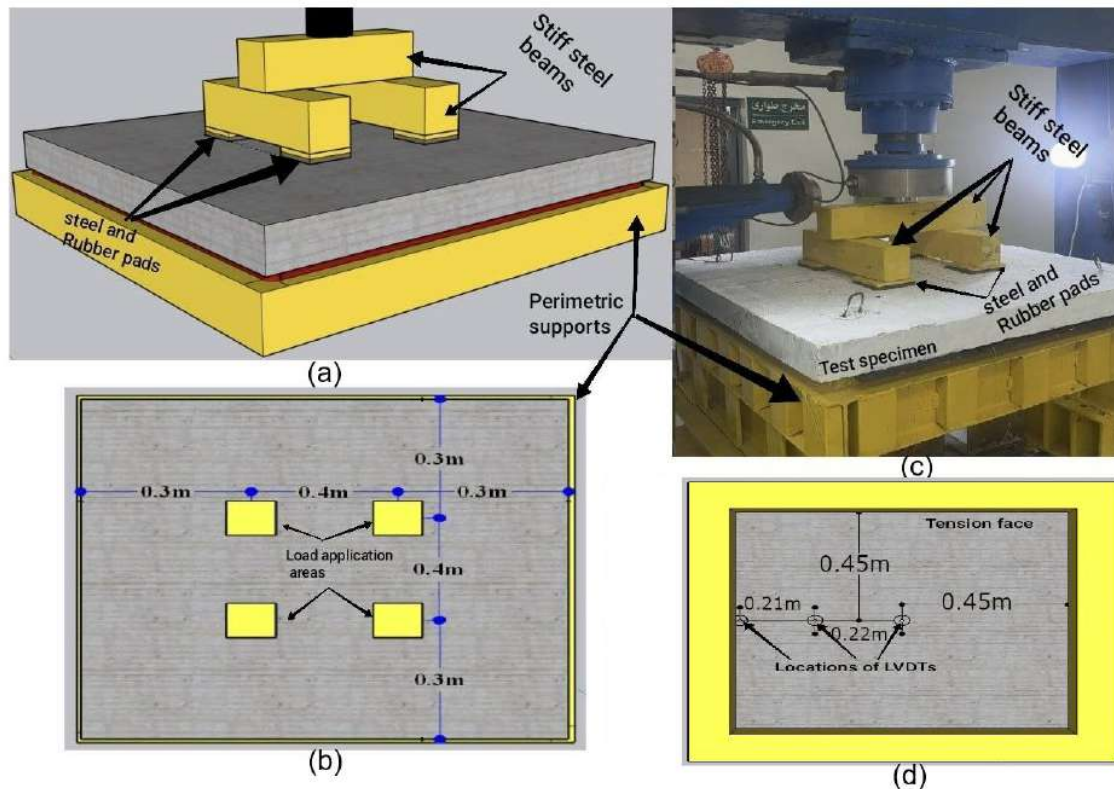


Fig. 5. Experimental setup (a) 3D schematic view; (b) top view; (c) the specimen positioning and the applied loading scheme; (d) Layout of LVDTs installed for deflection measurements

A four-point bending setup was used for the loading, as highlighted in Fig. 5(a and c) which applied the loading in a load-controlled manner. The applied load from the testing machine was initially transferred to the high-strength steel spreader beam and then to two transverse steel beams (one on each side of the primary beam) oriented perpendicular to the primary beam. Each transverse beam transmitted the load to the slab through two loading points at 300 mm from slab edges, leading to a distance of 400 mm between loading points centers. A square steel plate 100 x 100

mm comprising 12 mm thick was inserted between the steel beam and slab followed by a 100 × 100 × 10 mm rubber plate to reduce the local crushing of the concrete surface and allow uniform distribution of load at each loading location. All slabs retained an effective span of 960 mm.

In the test setup, the machine load cell recorded the applied load and three linear variable displacement transducers (LVDTs) were used to obtain slab deflections as shown in Fig. 5 (d). In the geometry, one LVDT was placed at the center of the slab where the maximum mid-span deflection would occur, and the other two were placed between the support and the center of the slab, one close to the support, to enable direct comparison of deflection measurements. Visual inspection for monitoring crack initiation, propagation, and failure patterns was conducted periodically during the tests, and crack widths were examined using a crack width gauge. However, these observations were used primarily for qualitative interpretation of crack development and failure behavior, and were not systematically recorded for a quantitative serviceability assessment based on crack-width criteria. All specimens were monotonically loaded to failure. For the retrofitted specimens (CD250 and CD2100), a pre-loading was applied up to 50% of the ultimate load of Specimen R, then followed by unloading, strengthening, and subsequent reloading until failure. The same test setup, instrumentation, and loading protocol were employed for all specimens to ensure uniformity and facilitate comparative evaluation of the flexural behavior and strengthening efficiency of the various test configurations.

3. Results and Discussion

In this section, the results related to flexural performance of the tested RC slabs are discussed in detail. It serves as an experimental study aiming at understanding the structural performance, load-carrying capacity and deformation behavior of the specimens. Particular attention is given to the serviceability behavior of the slabs in terms of stiffness development and deflection response, with reference to the general principles of the short-term serviceability limit state (SLS) described in Eurocode 2—Part 1 (CEN 2004). In the present study, the serviceability load level (PSLS) was determined based on the deflection limit equal to $l/250$, where l represents the effective span of the slab. For the tested specimens, this deflection corresponds to 3.6 mm, and the associated load values were obtained directly from the experimental load–deflection curves. This approach enables a comparative interpretation of slab behavior and identification of the load range corresponding to deflection-based serviceability performance within the scope of the present experimental study. In addition, the stiffness (k) of the slabs was evaluated quantitatively at various loading stages (both pre-cracking and post-cracking). Stiffness was determined from the slope of the load–deflection curves, where the initial stiffness (k_i) corresponds to the approximately linear portion of the response before first cracking, and the post-cracking stiffness (k_p) represents the response after crack initiation and before the final nonlinear softening stage, as defined by the following equation.

$$k = \Delta P / \Delta \delta \quad (1)$$

Where P represents the applied load and δ denotes the corresponding mid-span deflection.

In order to ensure a clear and structured comparison, each subsequent subsection contains:

- A table compilation providing the context of the primary mechanical characteristics and markers of functionality.
- Load-deflection curves to demonstrate the general trends in behavior.
- Photo Record of Failure Modes to see the crack patterns and failure modes of each specimen at ultimate state.

3.1 Strengthening Efficiency of FRCM System

The experimental observations summarized in Table 5 indicate the beneficial influence of the Carbon-FRCM system on the structural response of the tested slabs. The first crack load (P_{cr}) of both specimens C250 and C2100 was identical at 40 kN which corresponds to 166.7% higher than standard control slab (R). These results suggest that partial coverage (C250) can provide crack-delay behavior comparable to that of full coverage (C2100) under the investigated loading conditions. For the ultimate bearing capacity (P_u), as shown in the Load-Deflection diagram (Fig.

6), C 2100 recorded 163 kN compared to 153 kN for C250. Despite using approximately 50% less carbon textile reinforcement, specimen C250 achieved approximately 93.8% of the ultimate load capacity recorded for specimen C2100.

The experimental response indicates that the strengthening configuration significantly influenced the stiffness development and post-cracking deformation behavior of the tested slabs. The first stiffness (k_i) of specimen C250 appeared to be 17.05 kN/mm, a significant increase over that of the reference slab (10.6 kN/mm), and the maximum for the C2100 specimen was 25 kN/mm. A similar trend was observed for the post-cracking stiffness response (k_p), reflecting the influence of the strengthening configuration on stiffness retention after crack initiation, with C2100 (4.6 kN/mm) performing better than C250 (3 kN/mm). For serviceability, based on Eurocode 2 requirements, C250 improved the service load (PSLS) to 55 kN (77% improvement over R) and C2100 to 70 kN.

From a material efficiency perspective, specimen C250 may represent a practical strengthening option, as comparable structural improvement was achieved using a reduced amount of carbon textile. It should be noted that no detailed economic analysis was performed in this study, and the comparison is limited to relative material usage between strengthening configurations. The ultimate load for C250 was 6.5% lower than that of C2100; However, the doubling of the reinforcement area in C2100 did not result in a proportional increase in structural performance, as reflected in the corresponding deflection values (11.93 mm for C2100 compared to 15.55 mm for C250). While full-surface strengthening (C2100) provides modest gains in capacity and stiffness, the results indicate that localized strengthening (C250) can achieve efficient structural performance with reduced material usage, making it a practical option for engineering applications.

Table 5. Summary of experimental results for specimens: R, C2100, and C250

Specimen ID Configuration	R None	C2100 Continuous	C250 sheet
Layers	None	2	2
P_{cr} (kN)	15	40	40
P_u (kN)	120	163	153
Δ_u (mm)	21.34	11.93	15.55
P_{SLS}	31	70	55
k_i (kN/mm)	10.6	25	17.05
K_p (kN/mm)	1.13	4.6	3
Preloading level	None	None	None

Qualitative data from visual evidence of the failure modes in Fig. 7 supports these numerical data. The control slab (R) experienced wide, localized flexural cracks. In contrast, failure of the strengthened specimens showed the "multi-cracking" mode. Localized stress-bridging was observed in cracks concentrated within and along the edges of the reinforced patch in C250. In contrast C2100 showed its tension face filled with a densely distributed network of fine cracks (crack smearing). The extensive crack branching observed in specimen C2100 suggests that the continuous Carbon-FRCM configuration contributed to a more distributed transfer of tensile stresses across the tension zone. This behavior was associated with improved stiffness retention and lower deflection values prior to ultimate failure. The observed structural behavior can be further interpreted based on the interaction between the cementitious matrix, textile reinforcement, and cracked concrete. After the onset of cracking, the FRCM system contributes to tension stiffening by bridging flexural cracks and transferring tensile stresses across the cracked sections. In the localized configuration (C250), this mechanism is concentrated within the strengthened region, resulting in partial stress redistribution and localized crack control. In contrast, the continuous configuration (C2100) promotes a more uniform stress distribution over the entire tension zone, leading to improved stiffness retention and more distributed cracking patterns. This behavior may explain the enhanced stiffness retention and reduced deflection observed in the fully strengthened specimens under the investigated loading conditions. It should be noted that the presented results are based on a limited number of specimens and are intended

to reflect observed behavioral trends under controlled laboratory conditions rather than generalized structural performance.

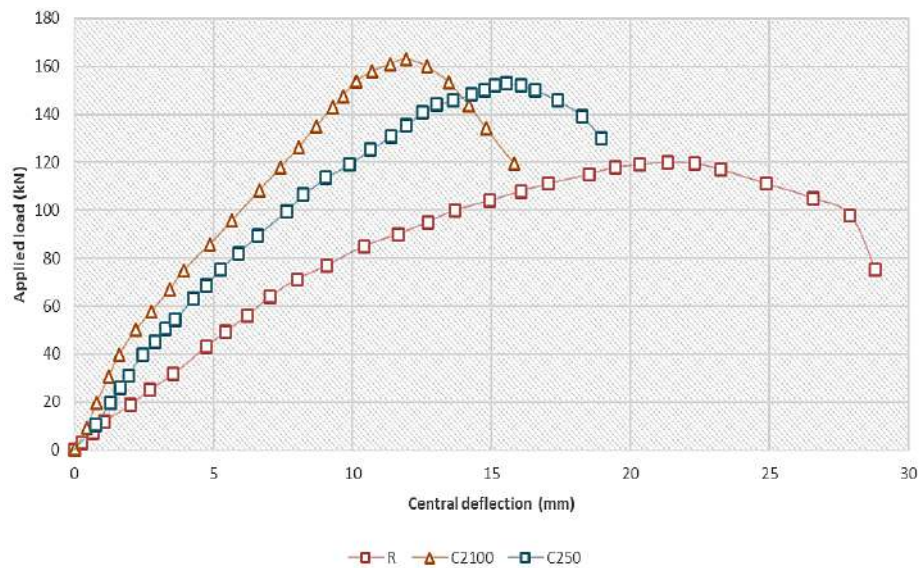


Fig. 6. Load-deflection response of specimens: R, C2100, and C250

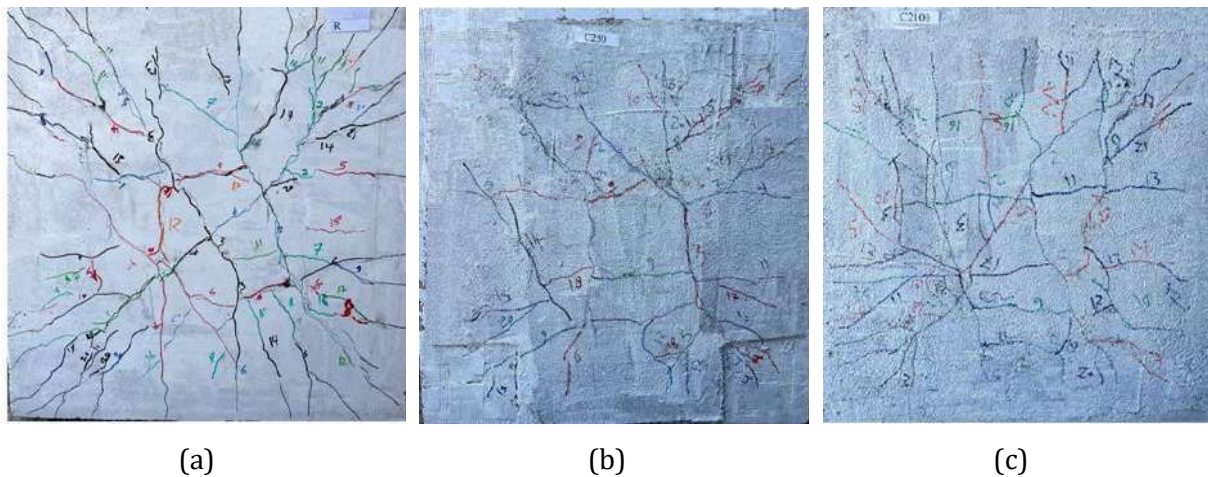


Fig. 7. Failure modes and crack patterns of specimens: (a) R, (b) C250, and (c) C2100

3.2 Repair Efficiency of FRCM System

The functionality of the Carbon-FRCM system in rehabilitating and upgrading the pre-damaged RC slabs is discussed here. The specimens CD250 and CD2100 were pre-loaded with 60 kN, i.e. 50% of control slab (R) ultimate capacity. The purpose of this pre-loading phase was to induce controlled flexural cracking representative of a moderate pre-damage condition prior to strengthening, defined in this study based on the applied preload level rather than on quantitative damage indices. This approach allows a consistent comparison of the effectiveness of different strengthening configurations while maintaining the structural integrity of the specimens. It should be noted that the pre-damage condition was defined solely based on the applied preload level corresponding to 50% of the ultimate capacity of the control specimen. Although this loading stage produced visible flexural cracking representative of a moderate damage condition, no quantitative measurements of crack width, residual deflection, or stiffness degradation were systematically recorded prior to strengthening. Consequently, the initial damage state could not be characterized using quantitative damage indices. Therefore, the repair efficiency discussed in this study should be interpreted as a comparative structural response under a controlled preloading condition rather than as a fully quantified assessment of damage recovery. As shown in Table 6, all the experimental results indicated that the damaged slabs were successfully repaired to their original capacity or even

greater than that using the FRCM system. In particular, specimen CD2100 attained an ultimate load of 156.5 kN whereas CD250 achieved 132.9 kN, which are greater than the undamaged control slab by 30.4% and 10.7% respectively.

The repaired specimens exhibited partial recovery of post-cracking stiffness following the application of the Carbon-FRCM system. The post-cracking stiffness (k_p) evaluated increased dramatically; k_p for specimen CD2100 was 3.5 kN/mm; k_p for specimen CD250 was 1.6 kN/mm, both values much greater than the 1.13 kN/mm for the cracked control specimen. Although there were already cracks in the concrete, the load-deflection curves of Fig. 8 show a stable transition and good load transfer from the damaged concrete substrate to the carbon fiber mesh via the cementitious matrix. The observed failure mode was primarily characterized by rupture of the carbon textile without significant visible debonding of the FRCM layer. This behavior suggests that an effective stress transfer mechanism developed between the cementitious matrix and the concrete substrate under the investigated loading conditions. It should be emphasized that no direct bond-slip tests, interface strain measurements, or detailed interfacial characterization were conducted in the present study. Consequently, bond behavior is interpreted indirectly based on the observed structural response and failure patterns, and should be considered as an observational assessment rather than a direct experimental verification.

Table 6. Summary of experimental results for specimens: R, CD250, and CD2100

Specimen ID	R	CD250	CD2100
Configuration	None	sheet	Continuous
Layers	None	2	2
P_{cr} (kN)	15	None	None
P_u (kN)	120	132.9	156.5
Δ_u (mm)	21.34	14.2	14.5
P_{SLS}	31	None	None
k_i (kN/mm)	10.6	None	None
K_p (kN/mm)	1.13	1.6	3.5
Preloading level	None	50%	50%

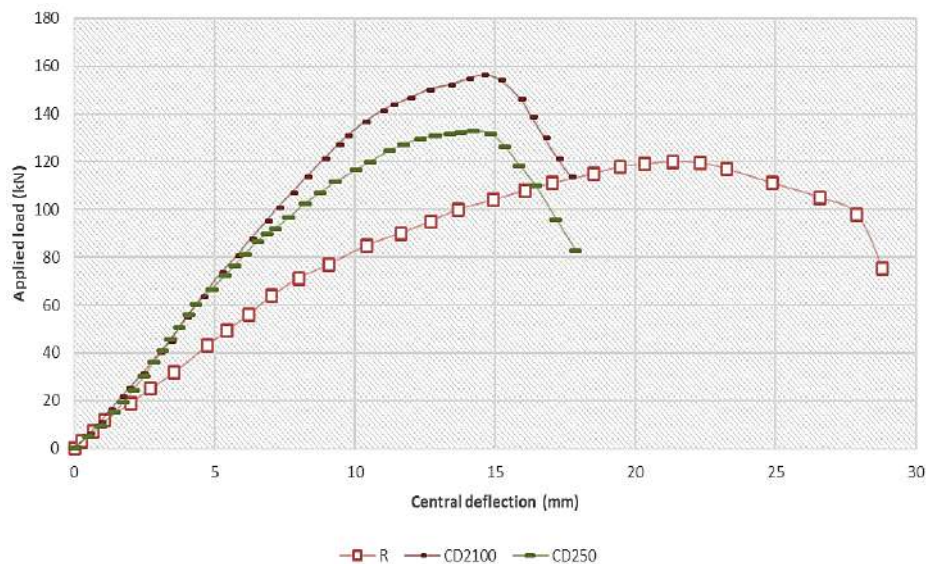


Fig. 8. Load-deflection response of specimens: R, CD2100, and CD250

From a structural performance perspective, the comparison between specimens CD2100 and CD250 provides useful insight into the influence of strengthening extent on slab behavior. Although the CD2100, with full surface coverage, demonstrated a 17.7% increase in ultimate load capacity compared to the localized configuration CD250, this improvement was associated with a significant increase in the amount of reinforcement used. This non-proportional relationship indicates that

while the continuous configuration enhances capacity and stiffness, the increase in reinforcement quantity is significantly higher relative to the achieved performance gain. On the other hand, specimen CD250 achieved full recovery of the pre-damaged slab, delivering a 10.7% increase in load capacity compared to the control specimen while using a reduced amount of reinforcement relative to CD2100. This highlights the ability of localized strengthening to provide effective structural improvement with lower material usage. This observation is further supported by visual inspection of the crack patterns shown in Fig. 9. While CD2100 exhibited a more distributed crack pattern, the localized reinforcement in CD250 effectively controlled major flexural cracks and prevented premature failure. These results suggest that strategic placement of FRCM in critical tension zones can provide efficient structural enhancement without the need for full surface coverage.

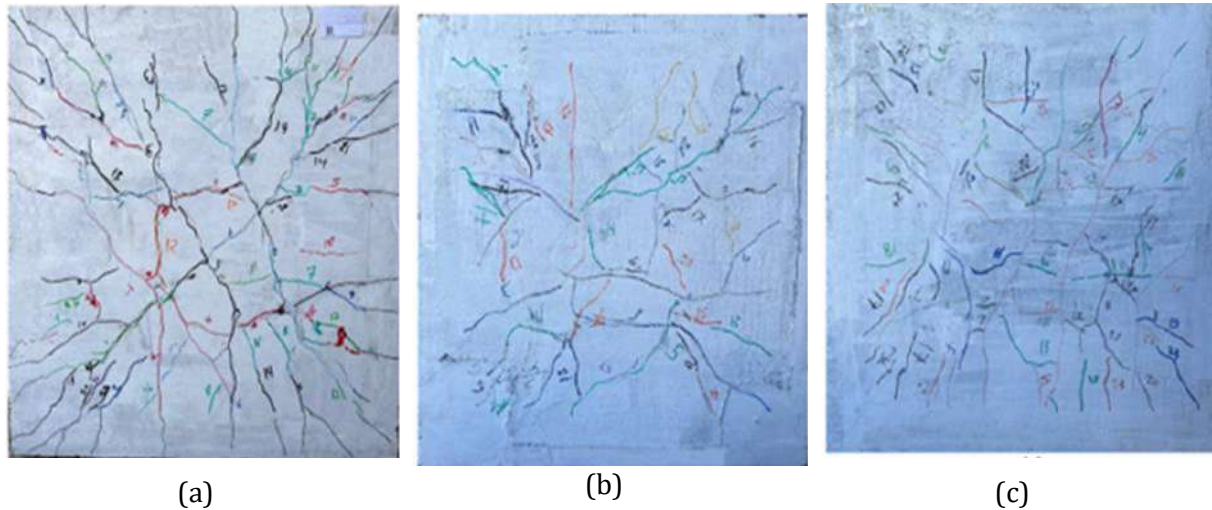


Fig. 9. Failure modes and crack patterns of specimens: (a) R, (b) CD250, and (c) CD2100

3.3 Comparative Structural Performance of Carbon-FRCM: Strengthening vs. Repair

The experimental outcomes indicate that the Carbon-FRCM system can provide effective strengthening and repair performance for reinforced concrete slabs under the investigated conditions, since all the rehabilitated specimens reached an ultimate capacity higher than the control slab ($R=120$ kN), as indicated in Table 7. The comparison of the ultimate load capacities (P_u) indicates that the repaired specimens (CD series) achieved structural responses comparable to or greater than that of the control specimen under the investigated experimental conditions. Specimen CD2100 reached an ultimate load of 156.5 kN, which is the equivalent of 96% of the strength capacity of its retrofitted equivalent, C2100 (163 kN)..

Table 7. Summary of experimental results for specimens: R, C250, C2100, CD250, and CD2100

Specimen ID	R	C250	C2100	CD250	CD2100
Configuration	None	sheet	Continuous	sheet	Continuous
Layers	None	2	2	2	2
P_{cr} (kN)	15	40	40	None	None
P_u (kN)	120	153	163	132.9	156.5
Δ_u (mm)	21.34	15.55	11.93	14.2	14.5
P_{SLS}	31	55	70	None	None
k_i (kN/mm)	10.6	17.05	25	None	None
K_p (kN/mm)	1.13	3	4.6	1.6	3.5
Preloading level	None	None	None	50%	50%

Although localized repair at CD250 attained 132.9 kN, which only lost 86.8% of the capacity for the strengthened specimen C250 (153 kN), It implies that because a continuous (full-face) reinforcement configuration is more likely to effectively contain and bridge existing cracks over the

entire length, it appears less sensitive to the influence of pre-existing cracking compared to the localized strengthening configuration investigated in this study

This highlights the influence of pre-damage on the post-cracking stiffness (k_p). For the 2100 series, the stiffness decreased from 4.6 kN/mm in the strengthened state (C2100) to 3.5 kN/mm in the repaired (CD2100) state. Hence the small reduction suggests the continuous FRCM layer "heals" the section by restoring a significant degree of flexural stiffness despite the 50% pre-loading damage. The 250 series, on the other hand, underwent a more drastic stiffness reduction from 3 kN/mm (C250) down to 1.6 kN/mm (CD250). These observations suggest that localized strengthening can effectively restore load-carrying capacity; however, its ability to control post-cracking deformation and stiffness degradation appears to be less pronounced compared to the continuous strengthening configuration. Counter intuitively the findings show that a full coverage repair to a damaged slab (CD2100) can give a greater ultimate capacity (156.5 kN) than localized strengthening of a healthy slab (C250 = 153 kN). This observation highlights the potential structural effectiveness of the Carbon-FRCM strengthening approach; rather than simply "repairing" an already compromised structure back to its original condition, it upgrades that structure up to a higher performance category. Overall, although zoned reinforcement (250 layout) is still the cheapest option for both applications, the continuous layout (2100) ensures better structural recovery, especially in terms of preserving normal section rigidity and crack control regarding prior major damage. It should be noted that the comparison presented in this section is limited to structural performance and relative material usage, and does not represent a comprehensive economic or cost analysis.

3.4 Influence of FRCM Coverage Ratio and Configuration

Coverage ratio of Carbon-FRCM system is crucial to finding a compromise between material saving and structural behavior. As shown in Table 7 and Fig. 3, this research focused on two principal configurations: a localized (250 series) sheet configuration that covers the high-moment region only and a continuous (2100 series) configuration that spans the entire face. Interestingly, subject to this change in the coverage area from localized patch to full surface reinforcement, the experimental data suggests only a marginal increase in ultimate load capacity; namely, C2100 was only 6.5% stronger than C250 (163 kN vs 153 kN), and CD2100 was only 17.7% superior to CD250 (156.5 kN vs 132.9 kN). Although the larger reinforcement coverage in the 2100 series provided an additional structural safety margin, this suggests the localized 250 configuration appears to provide efficient structural performance at capturing the main flexural stresses at their most crucial location, as illustrated by the load-deflection comparisons in Fig. 10.

In the control of deformations and distribution of cracks, the effect of the coverage ratio was present as the most significant factor. In Table 7, the ultimate deflection was limited to 11.93 mm for specimen C2100, which was 100% covered, and was 15.55 mm for specimen C250. This improvement in serviceability is related to the crack smearing effect identified in same full-coverage models (Fig. 7(c) and Fig. 9(c)) that the continuous mesh contributed to a more distributed tensile stress transfer across the slab tension face on the entire tension face of the slab. On the contrary, due to the unified FRCM patch boundaries, the 250 series showed almost localized cracking propagation, between the crack bands (Fig. 7; Fig. 9(b)), that could also be justified in terms of very low post cracking stiffness (k_p) values, equal to 3 kN/mm and 1.6 kN/mm for the C and CD series, respectively, compared to the higher stiffness kept by the 2100 series.

Ultimately, the choice of coverage ratio reflects a balance between structural performance and material usage. Although the 2100 configurations provide improved overall stiffness and more uniform crack control, the localized 250 configuration demonstrates effective strengthening and repair performance with reduced material usage. This trend is clearly illustrated in the integrated performance curves shown in Fig. 6. From a practical perspective, localized FRCM application can be considered an efficient approach for structural restoration, as it enables recovery of slab capacity beyond its pre-damage level while limiting the amount of reinforcement applied to critical regions. It should be noted that the experimental program was designed to provide a focused comparative assessment of strengthening configurations rather than a statistical evaluation of material variability.

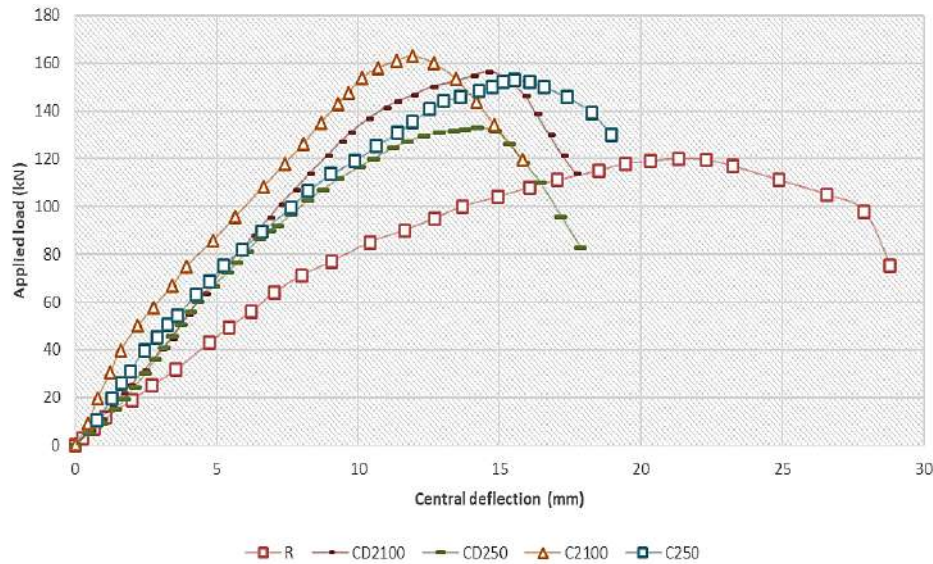


Fig. 10. Load-deflection response of specimens: R, CD2100, CD250, C2100, and C250

Although the number of specimens tested was limited, the observed behavioral trends in terms of stiffness evolution, cracking sequence, and load-carrying capacity showed consistent responses among the investigated strengthening arrangements. Under controlled laboratory conditions, the experimental findings provide meaningful insight into the structural performance of Carbon-FRCM strengthened slabs, particularly regarding the comparative behavior between localized and full-surface strengthening configurations. Nonetheless, the present study has a scope for further research with a greater number of specimens and loading conditions, which could possibly enhance the transferability of the results and a wider generalization for design-oriented purposes.

4. Conclusion

The following conclusions are drawn from the experimental results and comparative effectiveness of the Carbon-FRCM system for strengthening and repair of RC slabs:

- The Carbon-FRCM system was found to be effective within the scope of the experimental program in enhancing the structural performance of both intact and pre-damaged slabs. The increase in ultimate load capacity (P_u) of the strengthening specimens (C series) reached up to 35.8% for C2100 compared to the control slab.
- The experimental results indicate that the Carbon-FRCM system was capable of restoring and enhancing the load-carrying capacity of slabs (CD series) that were pre-loaded to 50% of the original strength. Specimen CD2100 reached 96% of the maximum capacity of its strengthened counterpart (C2100) and exceeded the original capacity of the control slab by 30.4%.
- FRCM resulted in noticeable increases in the initial stiffness (k_i) and service loads (P_{SLS}). In the first load stage, the initial stiffness of specimen C2100 was the highest at 25 kN/mm, which represents a 135.8% increase compared to the control slab (R), while C250 increased the service load to 55 kN [31 kN for reference specimen].
- The results suggest that localized strengthening (250 series) can provide effective structural improvement with reduced material usage compared to full-area coverage (2100 series) under the investigated conditions. Although a larger strengthening area was used in specimen C2100, only a 6.5% increase in ultimate load capacity was achieved compared to that of C250.
- Flexural cracks in the control slab were relatively wide, accompanied by early stiffness degradation, indicating a more brittle failure mode. In contrast, the specimens strengthened with Carbon-FRCM exhibited a more distributed cracking pattern ("crack smearing"), reflecting improved crack control and higher deformation capacity. The most uniform crack distribution and the greatest reduction in deflection were observed for the full-surface

coverage (2100 series), with a final deflection approximately 44% lower than that of the reference slab.

- No visible debonding or delamination of the FRCM layer from the concrete substrate was observed in the tested specimens. The dominant failure mode was rupture of the carbon textile, suggesting that bond transfer between the cementitious matrix and the concrete surface was effective under the applied loading conditions.

It should be noted that the conclusions presented are based on a limited experimental program and are intended to reflect observed behavior under controlled laboratory conditions rather than generalized structural performance.

5. Limitations and Future Work

This work represents an experimental study on the structural performance of Carbon-FRCM for strengthening and repair of reinforced concrete slabs. In addition, the experimental program was conducted using a limited number of specimens without repeated tests for each strengthening configuration; therefore, broader statistical validation of the observed structural trends requires further experimental investigation. The experimental program was carried out under monotonic loading, and a single pre-damage level corresponding to 50% of the ultimate capacity of the control specimen. This value represents moderate damage under controlled parameters; other damage levels or other deterioration mechanisms may affect structural response differently. Moreover, the tests were conducted in a laboratory environment and the long-term response of FRCM-concrete interface to cyclic loading or environmental effects calls for more studies.

Future research might aim to determine more optimal strengthening layouts to ensure a better efficiency of the structure with respect to the consumption of material, explore alternative types of textiles and subject the models to different loading conditions. Also, numerical modeling techniques such as finite element analysis (FEA) may be beneficial in providing a better understanding of the interfacial stress transfer mechanisms and predicting potential failure modes within FRCM-strengthened structural members.

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Multi-objective optimization algorithm for building energy optimization based on MOEA/D

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Article Info	Abstract
<p>Article History:</p> <p>Received 07 Aug 2026</p> <p>Accepted 14 Sep 2026</p> <p>Keywords:</p> <p>Building energy-saving; MOO; MOEA/D; Thermal comfort; Energy consumption control</p>	<p>Building energy-saving design needs to consider energy consumption and thermal comfort. The research aims to propose an efficient Multi-Objective Optimization (MOO) method adapted to building energy-saving design. The research first constructs an MOO model that minimizes the annual total energy consumption and maximizes indoor thermal comfort, and clarifies the decision-making elements and constraint boundaries. According to the decomposition idea of the multi-objective evolutionary algorithm, the weight vector generation, neighborhood update and mutation strategy are improved, and an improved MOO is constructed. The method is verified by residential building data in four typical climate zones in China. When this algorithm iterates for 500 generations, the Generational Distance (GD) is only 0.025, and the inverted GD reaches 0.045. The convergence and solution set distribution uniformity are significantly better than those of the comparison algorithm. The energy consumption of the solutions produced by this algorithm is 29.5%-38.6% lower than those of the comparison algorithm, with Guangzhou as low as 31.2kWh/m²·year and Harbin 42.3kWh/m²·year. Carbon emissions are reduced by 30.1%-39.8%, with Guangzhou down to 20.5kgCO₂/m²·year. The thermal comfort compliance time accounts for more than 92% with Guangzhou reaching a maximum of 96.8%. The initial investment costs are reduced by 11.7%-16.4%. This algorithm is superior to existing algorithms in convergence and uniformity of solution set distribution, which can adapt to the energy-saving design across climate zones, provide practical technical support for energy-saving and cost control throughout the building's life cycle, and has important engineering application value.</p>

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1. Introduction

Affected by the green transformation of the construction industry, energy consumption control and thermal comfort guarantee throughout the building life cycle have become core issues in the industry. Building energy-saving design needs to balance multiple objectives, covering multiple dimensions such as envelope optimization, energy system regulation, and cost control. It faces practical challenges such as differences in different climate zones and complex engineering constraints [1-2]. However, current industry practice still generally adopts single-objective or experience-oriented design methods, which are difficult to effectively handle the inherent conflicts between multiple objectives. As a result, design solutions often suffer from problems such as high energy consumption, insufficient comfort, or excessive initial investment in actual operation [3-4].

Currently, MOO technology has been widely used in the field of building energy-saving design. To solve the high uncertainty in building life cycle environmental impact assessment, Kiss B et al. used a MOO method that combined dynamic energy consumption simulation and life cycle assessment,

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using building envelope parameters as variables. The optimized building envelopes under the current power structure could reduce life cycle greenhouse gas emissions by an average of 18% [5]. To address the multi-objective, multi-constraint, and time-consuming issues in the renovation design of historical buildings, the study employs energy simulation and the Non-Dominated Sorting Genetic Algorithm II (NSGA-II) to achieve energy savings, carbon reduction, and net present value optimization. Wei H et al. combined the logarithmic additive decomposition method for optimization, and then obtained the optimal solution for multiple scenarios through the weighted sum method. When this method was applied to a courtyard building in Nanjing, energy consumption was reduced to 63.62%, net present value increased by 151.84%, and carbon emission reduction rate reached 60.48%, achieving MOO balance [6]. To balance energy efficiency, life cycle costs and thermal comfort in densely populated countries, Elsheikh A et al. proposed a MOO model for Egypt's three main climate types. Wall and roof types were design variables, incorporating local sustainable materials to optimize building envelope and HVAC system parameters. The optimal solutions in each climate zone achieved a balance between energy consumption and life cycle costs, but the proportion of discomfort time reduction in Mediterranean and semi-arid climate zones was lower than that in arid climate zones [7]. In response to the problem that the building energy efficiency certificate policy is difficult to promote stakeholder participation in renovation and the process is complicated, Araújo G R et al. used the building energy efficiency certificate database to integrate machine learning and MOO technology to provide optimal renovation plans, costs and investment returns. The coefficients of determination of energy consumption and energy efficiency certificate prediction models reached 0.84 and 0.79, respectively. The energy consumption was reduced to 60%. The return on investment in three years could be up to 7 times [8].

There are numerous studies that have developed various variants of Multi-objective Evolutionary Algorithm based on Decomposition (MOEA/D) to enhance the Multi-Objective Optimization (MOO) capabilities, but customized improvements for building optimization problems are still relatively limited. Agent-assisted optimization is often used to reduce the high computational cost of EnergyPlus simulations, but relevant practices of deep coupling between agent models and MOEA/D are still insufficient. A large number of works have combined meta-heuristic algorithms with EnergyPlus for building performance optimization, but most studies are limited to a single climate and building case. In the field of building renovation, there have been many studies on the trade-offs of energy consumption, cost, and thermal comfort, but the collaborative optimization of building envelope parameters for new residential buildings still needs to be supplemented. Uncertainty-aware optimization has gradually gained attention, but the existing solutions generally have high computational costs, and there is still a research gap for efficient frameworks suitable for residential scenarios. Therefore, the research focuses on multi-objective collaborative optimization of building energy-saving design to propose an improved optimization method based on MOEA/D to solve the performance defects and scene adaptation limitations of the existing algorithm. It is expected to provide optimization solutions that take into account energy-saving and carbon reduction, comfort experience and cost control for residential buildings, upgrade building energy-saving design technology, and implement projects. The improvement strategies of the research have clear differences in mathematical construction from the existing MOEA/D variants. In terms of weight vector generation, the existing methods mostly rely on uniform sampling or density feedback. This study proposes that the weight adjustment amplitude is directly coupled with the curvature estimation value of the Pareto front, rather than solely relying on the neighborhood density statistics. In terms of neighborhood update, traditional methods mostly adopt linear decreasing rules. This study constructs an adaptive criterion for neighborhood size based on the relative change rate of the aggregation function. In terms of mutation strategies, this study dynamically determines the combination coefficient of polynomial mutation and Gaussian mutation based on the non-dominated rank and aggregation density. The above improvements respectively address the limitations of traditional MOEA/D in building energy optimization, such as the difficulty of fixed weights in covering the real trade-off surface in different climate zones, insufficient diversity in the early stage of the neighborhood and decreased convergence accuracy in the later stage, and the tendency of single mutation to get stuck in local optima when dealing with high non-linear building parameter spaces. Its applicability is reflected in the significantly

different curvatures of the Pareto front for building energy and thermal comfort targets in different climate zones, and the adaptive weights can effectively cope with this. The thermal parameters have a clear engineering feasible domain and the calculation cost of the objective function is high. The neighborhood adaptation and mixed mutation significantly improve the search efficiency within a limited evaluation. Theoretically, the three strategies work together. The adaptive weights ensure the search direction covers the range, the dynamic neighborhood update controls the information transmission of sub-problems, the mixed mutation maintains the population diversity, and the algorithm's convergence and solution set uniformity are superior to the MOEA/D variants that adopt single or partial improvement strategies. The study systematically applies the aforementioned improved algorithm to the multi-objective energy-saving optimization of residential buildings in four typical climate zones in China, and establishes a complete optimization-simulation collaborative computing framework. The main scientific contributions are as follows: It revealed the collaborative mechanism of the three improvement strategies in the building energy optimization problem, and proved the complementary effectiveness of adaptive weight generation, dynamic neighborhood update and hybrid mutation in the trade-off between energy consumption and comfort. Moreover, it quantified the independent contributions of each strategy through ablation experiments, providing methodological support for the engineering application of MOEA/D type algorithms in building performance optimization.

2. Proposed Methodology

2.1 Construction of MOO Model for Building Energy-Saving Design

The study first deconstructs the building energy-saving design. The core lies in clarifying the internal logic of the target system, decision-making elements and constraint boundaries, and analyzing the core contradiction between energy consumption control and thermal comfort guarantee, while considering the engineering feasibility of the design plan, as shown in Figure 1.

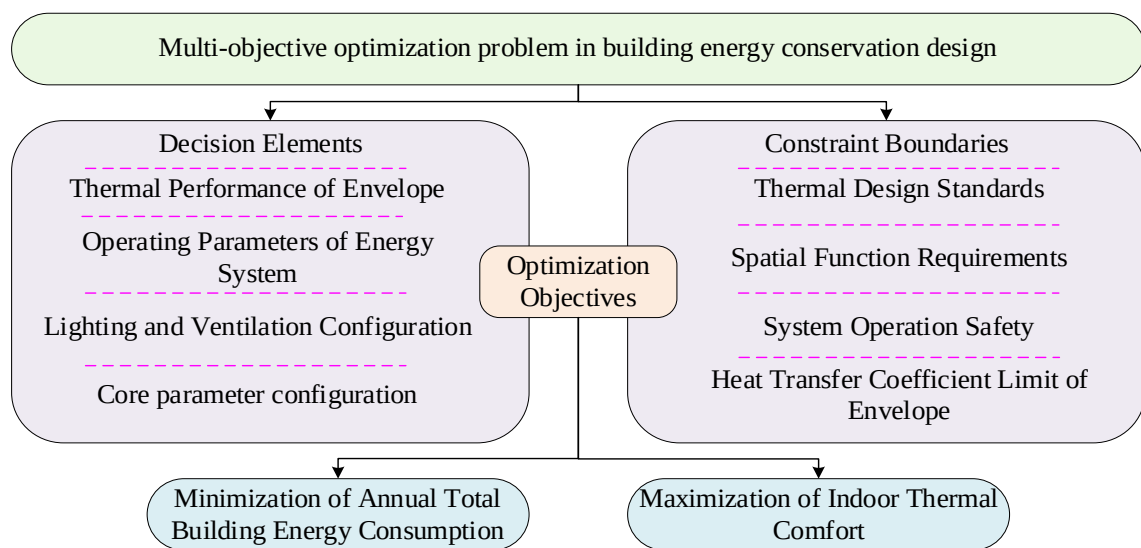


Fig. 1. MOO problem of building energy-saving design

In Figure 1, the decision-making elements are selected around three key dimensions: thermal performance of the building envelope, operating parameters of the energy system, and lighting and ventilation configuration. Core parameters that have obvious impacts on energy-saving effects and comfort experience are selected, including parameters related to the insulation layer, thermal performance parameters of doors and windows, air-conditioning system operation control parameters, and lighting system configuration parameters. All parameter values are based on current specifications to ensure the rationality of engineering applications. The constraint boundaries are formulated based on three core criteria: thermal design standards, space functional requirements and system operation safety. They cover key contents such as envelope heat transfer coefficient limits, door and window area ratio requirements, equipment operating parameter

intervals, etc., forming effective limitations on decision-making elements and ensuring that the optimization plan meets the basic requirements of architectural design [9-10]. The primary optimization goal is to minimize the total annual energy consumption. This goal comprehensively covers heating, cooling, ventilation and lighting. It takes into account the dynamic changes in climate conditions in different seasons and the time distribution characteristics of the building usage pattern, and uses the dynamic energy consumption calculation method to quantify it, given in equation (1).

$$f_1 = \sum_{t=1}^T (Q_h(t) + Q_c(t) + Q_v(t) + Q_l(t)) \cdot \Delta t \quad (1)$$

In equation (1), f_1 is the total annual energy consumption target of the building. T is the total number of calculation periods throughout the year. Δt is the time step (h) of each calculation period. $Q_h(t)$ is the heating load power (kW) in period t , which is determined by the heat transfer loss, the difference between heat transfer and radiation heat gain through doors and windows, and indoor heat load demand. $Q_c(t)$ is the cooling load power (kW) in period t , which is related to the indoor and outdoor temperature difference, solar radiation intensity, and the heat released by the indoor heat source. $Q_v(t)$ represents the ventilation energy consumption (kW) in period t , which depends on the ventilation volume and energy loss in the air treatment process. $Q_l(t)$ signifies the lighting energy consumption power (kW) in period t , which is affected by the power configuration and the efficiency of natural lighting utilization.

The second optimization goal is to maximize indoor thermal comfort. The predicted dissatisfaction percentage is used as a quantitative index. This index is based on the internationally accepted thermal comfort evaluation standard and comprehensively considers the synergistic effect of the four thermal environment parameters of indoor temperature, relative humidity, air flow velocity, and average radiant temperature. It comprehensively reflects the human body's subjective feelings, given in equation (2).

$$f_2 = 100 - 95 \cdot \exp(-0.03353 \cdot PMV^4 - 0.2179 \cdot PMV^2) \quad (2)$$

In equation (2), f_2 signifies the indoor thermal comfort evaluation target. PMV signifies the predicted average voting value, which is calculated through indoor thermal environment parameters, human metabolic rate, clothing thermal resistance, and other factors. The value range is [-3,3]. The closer the value is to 0, the better the thermal comfort state. The value of the objective function is [5,100]. The smaller the value, the better the overall comfort performance of the indoor thermal environment. When the value does not exceed 10%, it is determined to meet the basic requirements for human thermal comfort [11-12]. The actual optimization objective of this model is Predicted Percentage of Dissatisfied (PPD). ITCI is merely a derived evaluation indicator used in the experiment for result comparison and is not the algorithm optimization target. The fixed assumptions used in the calculation of thermal comfort are set as a metabolic rate of 1.0 met, a clothing thermal resistance of 0.5 clo, an indoor air velocity of 0.1 m/s, an indoor relative humidity of 50%, an average radiant temperature consistent with the indoor air temperature, and a personnel occupancy schedule following the typical daily routine of a residence. The personnel are present in the room from 8:00 to 22:00 each day, and the remaining time periods are considered as unoccupied. Based on building thermal engineering principles and energy balance equations, a coupled calculation model of building thermal environment and energy consumption is constructed to clarify the impact mechanism of decision-making elements on the two objective functions, as shown in Figure 2.

In Figure 2, the decision-making element input module covers parameters related to the envelope structure, energy system, and lighting and ventilation. The coupled calculation module realizes parameter transformation through the heat conduction equation, energy balance equation, and thermal comfort evaluation model. The objective function output module outputs the energy consumption and thermal comfort quantification results, respectively [13-14]. To collaboratively optimize the two conflicting goals, the MOO model is constructed, given in equation (3).

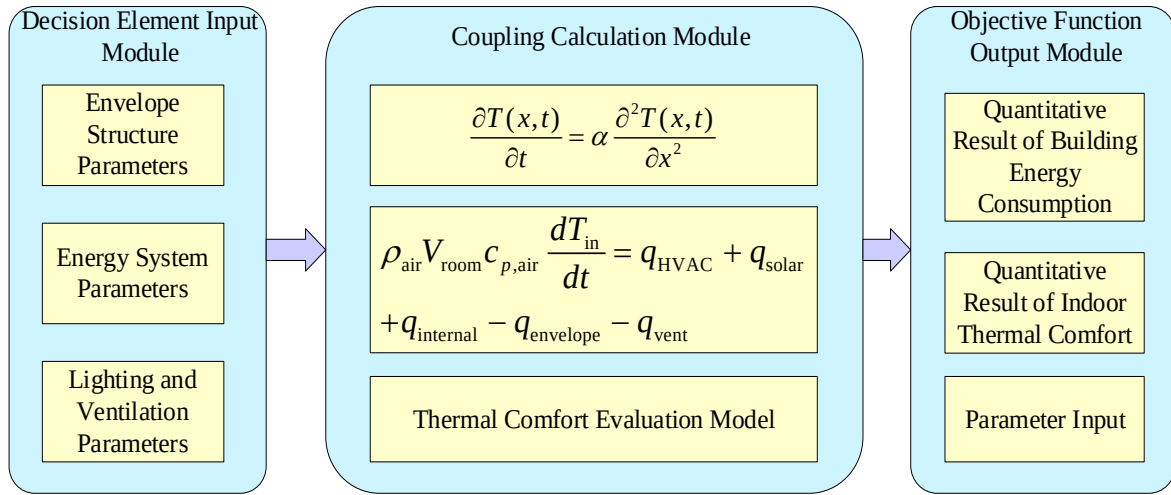


Fig. 2. Coupled calculation model of building thermal environment and energy consumption

$$\begin{cases} \min & F = [f_1, f_2]^T \\ \text{s.t.} & G_k \leq 0 \quad (k = 1, 2, \dots, K) \\ & a_j \leq x_j \leq b_j \quad (j = 1, 2, \dots, J) \end{cases} \quad (3)$$

In equation (3), F is a multi-objective function vector. There is a significant conflict between f_1 and f_2 , that is, the ultimate optimization of a single goal will degrade the performance of the other goal. G_k signifies the k -th constraint. K signifies the total constraints. x_j signifies the j -th decision parameter. J signifies the total decision parameters. a_j and b_j signify the lower limit and upper limit of the j , and jointly define the feasible region of the decision parameter. In this optimization model, the decision variables include the thermal parameters of the building envelope, the performance parameters of doors and windows, the operation control parameters of the air conditioning system, and the configuration parameters of the lighting system.

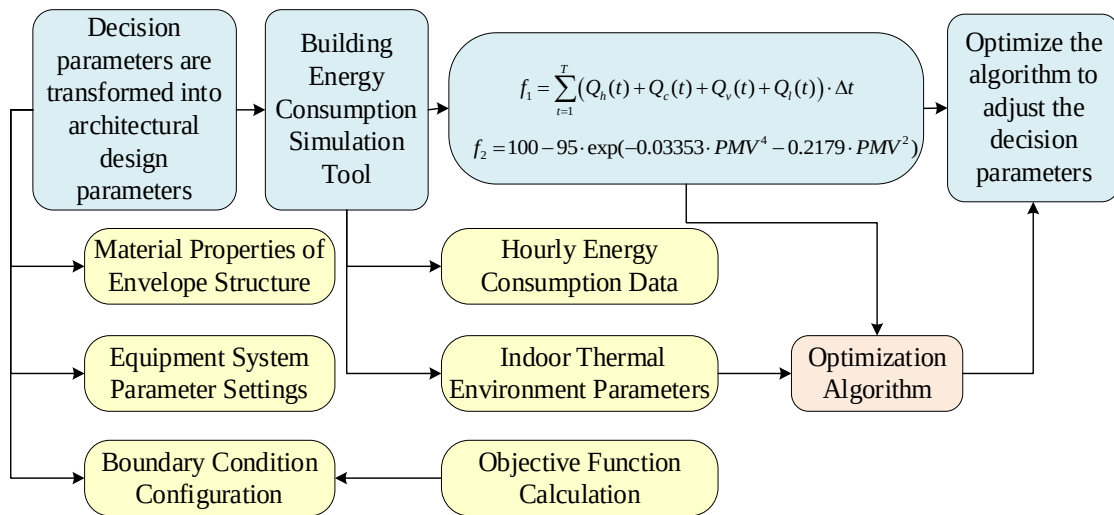


Fig. 3. Collaborative calculation mechanism of optimization model and building energy consumption simulation tool

The constraints include the limit values of the heat transfer coefficient of the building envelope, the requirements for the window-to-wall area ratio, and the parameter ranges for equipment operation. Carbon emissions, initial investment costs, and lighting satisfaction are not the optimization objectives or constraints. Instead, they are post-evaluation indicators for the optimization solution, used to comprehensively evaluate the engineering applicability of the obtained Pareto solutions from the economic and environmental perspectives. Among these, carbon emissions are calculated by multiplying the energy consumption values obtained from the optimization by the corresponding grid carbon emission factors for each climate zone. The initial

investment cost is estimated based on the market prices of building envelope and equipment selection. Moreover, lighting satisfaction is evaluated by combining natural lighting simulation and the power density of artificial lighting. These indicators do not participate in the calculation of the objective function or individual selection during the optimization iteration process. They are only used for multi-dimensional post-evaluation of the Pareto solution set after the optimization is completed.

To ensure the accuracy of model calculations and engineering applicability, a collaborative calculation mechanism between the optimization model and building energy consumption simulation tools is established, as shown in Figure 3.

From Figure 3, the decision-making parameters are first converted into building design parameters that can be identified by the simulation tool, including envelope material properties, equipment system parameter settings, boundary condition configuration, etc. Then, the building hourly energy consumption data and indoor thermal environment parameters are solved through the simulation tool. Two objective function values are calculated based on the simulation output results. Finally, the objective function value is fed back to the optimization algorithm to ensure that the objective function calculation can truly reflect the actual operating status of the building.

2.2 Design of Improved MOO Algorithm Based on MOEA/D

Based on the constructed MOO model of building energy-saving design, taking into account the conflict between energy consumption and thermal comfort goals and the engineering constraint characteristics of decision-making parameters, the study adopts the core decomposition idea of MOEA/D to design a MOO algorithm. It adapts to building energy-saving design by improving weight vector generation, neighborhood update, and mutation strategies to achieve efficient search for Pareto optimal solution sets. The core logic is to decompose the building energy-saving MOO problem into several single-objective sub-problems to be solved in parallel. Information collaboration between sub-problems improves the convergence and distribution uniformity of the solution set, ensuring that the optimization results are both consistent with engineering reality and have diverse trade-off solutions [15-16]. The study first designs an adaptive weight vector generation strategy based on the trade-off preferences of building energy-saving goals. The uniformly distributed weight vector corresponds to the trade-off ratio of different energy-saving and comfort goals. The weight vector satisfies the non-negativity and normalization conditions, given in equation (4).

$$\sum_{m=1}^M \lambda_{i,m} = 1, \quad \lambda_{i,m} \geq 0 \quad (4)$$

In equation (4), $\lambda_{i,m}$ signifies the component of the i -th weight vector on the m -th target. M signifies the number of objective functions. The initial weight vector is uniformly generated in the target space using the systematic method of Das and Dennis. The weight adaptive adjustment is triggered every 50 generations. The adjustment is based on the Euclidean distance distribution of the optimal solutions corresponding to adjacent weight vectors in the target space for the current population. If the standard deviation of the distance in a certain area exceeds 1.2 times the global standard deviation mean, it is determined that the representativeness of that area is insufficient. The weight modification adopts the redistribution strategy, transferring the weights from dense areas to areas with insufficient representativeness. The migration amplitude is proportional to the distance deviation between the regions, while maintaining all weights non-negative and normalized. This mechanism enables areas with larger front curvature to obtain more weights to refine the search, and areas with flat fronts to reduce the weight density to avoid redundant computations. Each weight vector corresponds to a potential optimization direction on the Pareto front. For example, when the weight of the energy consumption target increases, the search is more inclined to low energy consumption solutions. When the weight of the thermal comfort target increases, the search highlights improving comfort performance [17]. The overall framework of the improved MOO algorithm depending on MOEA/D is shown in Figure 4.

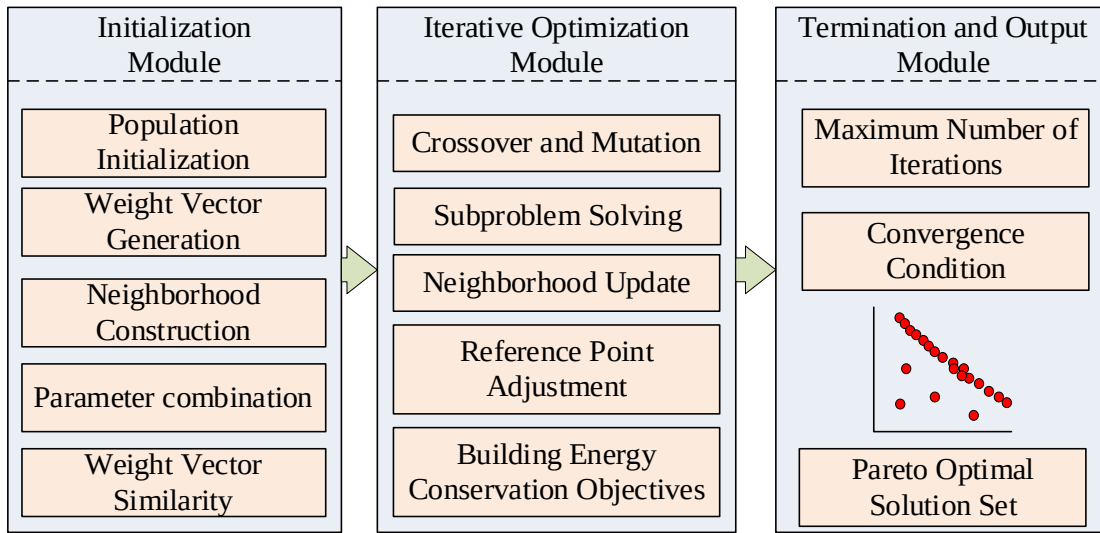


Fig. 4. Overall framework of improved MOO based on MOEA/D

In Figure 4, the initialization module includes population initialization, weight vector generation, and neighborhood construction. Population individuals correspond to building energy-saving design parameter combinations, and neighborhoods are determined based on weight vector similarity. The iterative optimization module covers cross-mutation, sub-problem solving, neighborhood update, and reference point adjustment. Each step of the operation is designed around the optimization needs of building energy-saving goals. Termination and output module: When the maximum number of iterations or convergence conditions are met, the Pareto optimal solution set is output. To efficiently decompose the building energy-saving MOO problem, the improved Chebyshev aggregation function is used in this study, fully considering the dimensional differences and project priorities of the building energy-saving goals, and converting each sub-problem into a single-objective optimization, given in equation (5).

$$g^t(X|\lambda_i, z^*) = \max_{1 \leq m \leq M} \lambda_{i,m} \cdot \frac{|f_m(X) - z_m^*|}{\Delta f_m} \quad (5)$$

In equation (5), $g^t(X|\lambda_i, z^*)$ is the aggregate function value of the i -th sub-problem. X is the building energy-saving design decision-making parameter vector. z^* is the ideal reference point. z_m^* is the current optimal value of the m -th target. Δf_m is the difference in the value range of the m -th target, which is used to eliminate the dimensional influence. This aggregation function balances the optimization priorities of different goals through the weight vector λ_i , so that the optimal solution to the sub-problem accurately corresponds to the trade-off requirements of the building energy-saving goals. In view of the continuity and engineering constraint characteristics of building energy-saving design parameters, a hybrid mutation strategy is designed, which combines polynomial mutation and adaptive Gaussian mutation to ensure population diversity and improve local search efficiency. The mutation operation is shown in equation (6).

$$x'_j = \begin{cases} x_j + \delta_j \cdot (b_j - a_j), & r < p_m \\ x_j, & r \geq p_m \end{cases} \quad (6)$$

In equation (6), x'_j signifies the j -th decision parameter after mutation. x_j is the parameter value before mutation. a_j and b_j are respectively the upper and lower boundaries of the engineering value of this parameter. δ_j signifies the variation step size, which is determined by the sensitivity of building parameters. Parameters that have obvious impacts on energy consumption are set with smaller step sizes to ensure search accuracy, and parameters that are sensitive to comfort impact are set with adaptive step sizes. r signifies a random number in $[0,1]$. p_m signifies the mutation probability (taken as 0.02). The distribution index for polynomial mutation is set to 20. The adaptive Gaussian mutation expression is as shown in equation (7).

$$\Delta_j = N(0, \sigma_j^2) \quad (7)$$

In equation (7), σ_j represents the standard deviation of the j -th decision parameter, and the variance σ_j^2 is dynamically adjusted according to the parameter sensitivity. Parameter sensitivity is defined as the amplitude of the local partial derivative of the objective function with respect to the decision parameters. It is calculated through finite difference method sampling. The higher the parameter sensitivity, the smaller the standard deviation of Gaussian variation. This is used to constrain the variation amplitude. The algorithm selects between polynomial variation and adaptive Gaussian variation with an equal probability of 0.5. Before the variation operation, the decision variables are normalized and mapped to the [0,1] interval. After the variation is completed, they are mapped back to the original physical interval. If the parameters after variation exceed the engineering upper and lower limits, the out-of-bound variables are directly truncated to the corresponding boundary values.

To improve the collaboration efficiency between sub-problems and the uniformity of solution set distribution, a dynamic neighborhood update mechanism is constructed to dynamically adjust the neighborhood size relying on the optimization progress. The neighborhood update mechanism is shown in Figure 5.

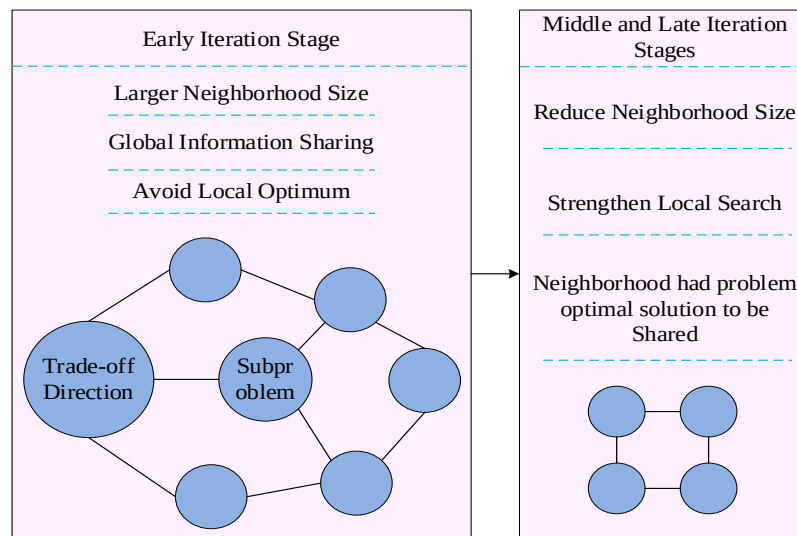


Fig. 5. Neighborhood update mechanism

In Figure 5, each node represents a sub-problem, corresponding to a certain trade-off direction of building energy-saving design. The connecting lines represent the neighborhood correlation between sub-problems. In the early stage of the iteration, a larger neighborhood size is used to promote global information sharing and avoid local optimality. In the middle and later stages, the neighborhood size is reduced to strengthen local search and enhance the convergence accuracy of the solution set. Through the sharing of optimal solutions to sub-problems in the neighborhood, the building design parameter combination with better performance can be quickly transferred and the search efficiency of the overall algorithm is improved. This mechanism sets the initial neighborhood size to 20, the minimum neighborhood size to 8, and the neighborhood size changes in a discrete stepwise manner. The update frequency is executed once every 50 generations, and the neighborhood update function is shown in equation (8).

$$T_{cur} = T - \frac{gen}{Gen_{max}min_{max_{max}}} \quad (8)$$

In equation (8), T_{cur} represents the current neighborhood size, T_{max} indicates the initial neighborhood size, T_{min} denotes the minimum neighborhood size, gen represents the current iteration number, and Gen_{max} is the maximum number of iterations. After each update of the neighborhood size, the neighborhood relationship between sub-problems will be recalculated based on the weight vector similarity. During the iteration process, if the neighborhood size does

not trigger an update, the original neighborhood association will be retained without any modification.

To efficiently update population and retain high-quality individuals, a selection strategy based on the priority of building energy-saving goals is designed, combining non-dominated sorting and aggregation function value sorting to screen the parent and offspring populations. The mathematical description of the selection strategy is shown in equation (9).

$$\text{select}(X) = \text{argmin}\{\alpha \cdot g^t(X|\lambda_i, z^*) + (1 - \alpha) \cdot \text{rank}(X)\} \quad (9)$$

In equation (9), $\text{select}(X)$ is the selected optimal individual, corresponding to the optimal parameter combination of building energy-saving design. α is the balance coefficient (taken as 0.6). $\text{rank}(X)$ signifies the non-dominated ranking. This strategy considers both the aggregation function value of the sub-problem and the non-dominated level, ensuring that the retained individuals not only meet the trade-off requirements of the building energy-saving goals, but also have good non-dominated characteristics [18-19]. The coupling and data interaction between the algorithm and the building energy-saving optimization model are shown in Figure 6.

In Figure 6, the algorithm first generates an initial building design parameter combination, and imports the parameters into the energy consumption simulation tool to calculate the objective function value. Then, the algorithm performs cross mutation, neighborhood update and other operations based on the target value to generate a new parameter combination. The new parameters are again imported into the simulation tool to calculate the target value, and the iterative cycle continues until the termination condition is satisfied. The algorithm is responsible for searching for the optimal parameter combination, and the simulation tool is responsible for accurately calculating the building energy-saving target value. The two work together to efficiently optimize building energy-saving design. The algorithm pseudocode is shown in Table 1.

Table 1. Algorithm pseudocode

Input: population size N, maximum iterations Gmax, neighborhood size T, mutation probability pm
Output: Pareto optimal solution set
1: Generate N uniform weight vectors, compute Euclidean distances among them, determine T neighbors for each subproblem
2: Initialize population randomly, evaluate objective function values, initialize ideal reference point z^*
3: for $g = 1$ to Gmax do
4: for $i = 1$ to N do
5: Randomly select two individuals from the neighborhood of the i -th subproblem, perform simulated binary crossover and hybrid mutation to generate new individual y
6: Evaluate objective function values of y , update reference point z^*
7: if y has better aggregation function value than individuals in the neighborhood then replace
8: end for
9: if $g \bmod 50 == 0$ then
10: Compute Euclidean distance distribution of optimal solutions corresponding to adjacent weights in objective space
11: Identify regions where distance standard deviation exceeds 1.2 times the global mean, mark them as underrepresented
12: Migrate weights from dense regions to underrepresented regions while maintaining non-negativity and normalization
13: end if
14: end for
15: Output non-dominated individuals in current population as Pareto optimal solution set

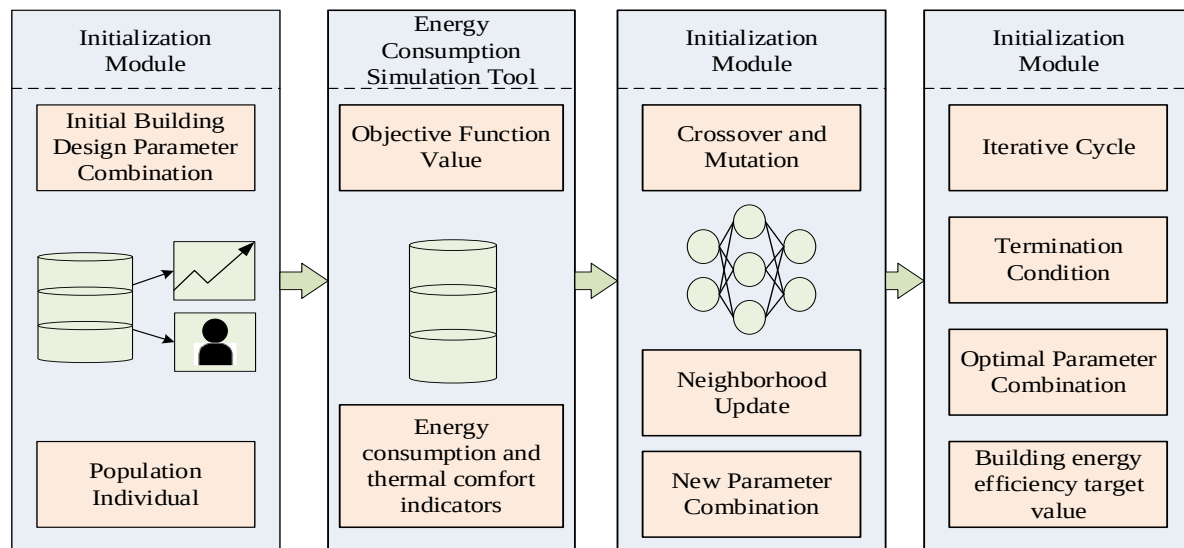


Fig. 6. Coupling and data interaction between algorithm and building energy-saving optimization model

3. Experiment Setup and Analysis

3.1 Datasets and Experiment Setup

The study utilized two types of data sources: (1) Self-made simulation data set: Based on the prototype of residential buildings in typical climate zones of China (Harbin, Beijing, Shanghai, Guangzhou), the parameters of the enclosure structure were set according to the "Code for Thermal Design of Civil Buildings" (GB 50176-2016) and the "Technical Standard for Near-Zero Energy Buildings" (GB/T 51350-2019). (2) Open-source meteorological data were taken from the typical meteorological year TMY data set provided by EnergyPlus, including hourly dry bulb temperature, relative humidity, solar radiation, wind speed, etc. This study did not conduct on-site measurement work; all building data were obtained through EnergyPlus simulation. The experimental hardware platform configuration is presented in Table 2.

Table 2. Experimental hardware platform configuration

Component category	Specific model/ specification
CPU	Intel Core i9-13900K (24 cores, 32 threads, maximum 5.8 GHz) 64 GB
Memory	DDR5 5600 MHz
Storage	2 TB NVMe PCIe 4.0 SSD
Operating System	Ubuntu 22.04 LTS (open source)
Energy consumption simulation engine	EnergyPlus v23.1.0 (open source)
Meteorological data source	Self-made: Four types of typical residential models constructed based on the "Energy Efficiency Design Standards for Residential Buildings" of China
Building prototype data	Specific model/ specification

The adopted residential prototype was a three-story rectangular slab-style house, with a total building area of 288 m². The building faces south, and the window-to-wall ratio was uniformly set at 0.45. The walls consisted of an exterior finish, insulation layer, concrete base, and internal plastering. The roof was equipped with an external insulation structure. The floor used a contact-ground reinforced concrete structure. The heat transfer coefficient of the exterior window glass ranged from 2.0 to 2.8 W/(m²·K), the building air infiltration rate was set at 0.5 air-change per hour, the personnel density was set at 0.04 people per square meter, the indoor lighting power density was 5 W/m², and the equipment power density was 3.5 W/m².

Table 3. Key input parameters of EnergyPlus building prototypes for four climate zones

Parameter category	Parameter name	Harbin (Severe cold)	Beijing (Cold)	Shanghai (Hot-summer cold-winter)	Guangzhou (Hot-summer warm-winter)
Building geometry	Building form	Three-story rectangular slab residential building	Three-story rectangular slab residential building	Three-story rectangular slab residential building	Three-story rectangular slab residential building
	Total floor area, m ²	288	288	288	288
	Building orientation	Due south	Due south	Due south	Due south
	Window-to-wall ratio	0.45	0.45	0.45	0.45
Envelope thermal performance	Wall heat transfer coefficient, W/(m ² ·K)	0.20~0.45	0.25~0.60	0.30~0.80	0.40~1.00
	Roof heat transfer coefficient, W/(m ² ·K)	0.18~0.40	0.22~0.50	0.28~0.65	0.35~0.75
	Exterior window heat transfer coefficient, W/(m ² ·K)	2.00~2.20	2.20~2.40	2.40~2.60	2.60~2.80
	Exterior window shading coefficient	0.30~0.80	0.30~0.80	0.30~0.80	0.30~0.80
Ventilation & infiltration	Air infiltration air change rate (ACH)	0.5	0.5	0.5	0.5
Internal loads	Occupancy density, person/m ²	0.04	0.04	0.04	0.04
	Lighting power density, W/m ²	5.0	5.0	5.0	5.0
	Equipment power density, W/m ²	3.5	3.5	3.5	3.5
HVAC system	Air-conditioning type	Split-type air conditioner	Split-type air conditioner	Split-type air conditioner	Split-type air conditioner
	Heating COP	2.8	2.8	2.8	2.8
	Cooling COP	3.0	3.0	3.0	3.0
	Heating set-point temperature, °C	20	20	20	20
	Cooling set-point temperature, °C	26	26	26	26
Simulation assumptions	Occupancy & load schedule	Activated: 8:00-22:00, inactive for remaining hours	Activated: 8:00-22:00, inactive for remaining hours	Activated: 8:00-22:00, inactive for remaining hours	Activated: 8:00-22:00, inactive for remaining hours
	Meteorological data source	EnergyPlus TMY3	EnergyPlus TMY3	EnergyPlus TMY3	EnergyPlus TMY3
	Simulation duration, h	8760	8760	8760	8760

The HVAC system adopted a split air conditioning system. The heating and cooling energy efficiency ratios were 2.8 and 3.0, respectively. The heating set temperature was 20 °C, and the cooling set temperature was 26 °C. The loads of people, lighting, and equipment followed the residential occupancy schedule. The indoor loads were enabled from 8:00 to 22:00, and the loads were turned

off during other periods. All models used the corresponding city TMY3 hourly meteorological files to conduct 8760-hour annual simulations. The key EnergyPlus input parameters for the residential prototypes corresponding to the four climate zones are summarized in Table 3.

3.2 Performance Verification of Improved MOO Algorithm Based On MOEA/D

To comprehensively verify the comprehensive performance of the Improved MOEA/D (IMOEAD), three comparison algorithms that were widely used and had excellent performance in the current MOO field were selected, namely: NSGA-II, Improved Firefly Algorithm (IFA), Hybrid-Integer MOO Algorithm (HIMO). The parameter settings for each comparison algorithm were uniformly listed in Table 4. The running environment and hardware configuration of all algorithms were kept consistent. Each algorithm was run independently 30 times, and the average values of each indicator were used for performance comparison.

Table 4. Parameter settings of each comparative algorithm

Algorithm	Parameter Name	Set value
IMOEAD	Population Size	100
	Maximum Iteration Number	500
	Initial Neighborhood Size	20
	Minimum Neighborhood Size	5
	Number of Weight Vectors	100
	Mutation Probability	0.02
	Polynomial Mutation Distribution Index	20
NSGA-II	Weight Adaptive Trigger Interval	50 generations
	Population Size	100
	Maximum Iteration Number	500
	Crossing Distribution Index	20
	Mutation Distribution Index	20
	Crossing Probability	0.9
	Mutation Probability	0.1
IFA	Population Size	100
	Maximum Iteration Number	500
	Absorption Coefficient of Light	1.0
	Step Size Factor	0.25
HIMO	Maximum Attractiveness	1.0
	Population Size	100
	Maximum Iteration Number	500
	Crossing Probability	0.85
	Mutation Probability	0.15
	Number of Local Search Iterations	20

The population size of the algorithm was uniformly set to 100, and the maximum number of iterations was 500. The complete operation of a single optimization run included 50,000 EnergyPlus simulation calls. The average time consumed for a single annual hourly simulation of EnergyPlus under the hardware configuration (Intel Core i9-13900K, 64GB DDR5 memory) was approximately 4.2 seconds. The total computing time for a single complete optimization run was approximately 238 seconds (including the internal operation overhead of the algorithm). Under the same population size and iteration conditions, the total optimization time of NSGA-II, IFA, and HIMO was approximately 215 seconds, 267 seconds, and 241 seconds respectively, which were at the same order of magnitude as IMOEAD. The Pareto frontier solutions of different algorithms are shown in Figure 7. X-axis: f_1 (Normalized total annual operating energy consumption, dimensionless, value range [0, 1], obtained by dividing the actual energy consumption of each solution by the maximum energy consumption value obtained by the algorithm). Y-axis: f_2 (Normalized indoor thermal comfort index ITCI, dimensionless, value range [0, 1], obtained by dividing the actual ITCI value of each solution by the maximum ITCI value obtained by the algorithm. Here, the ITCI value is numerically equal to PPD).

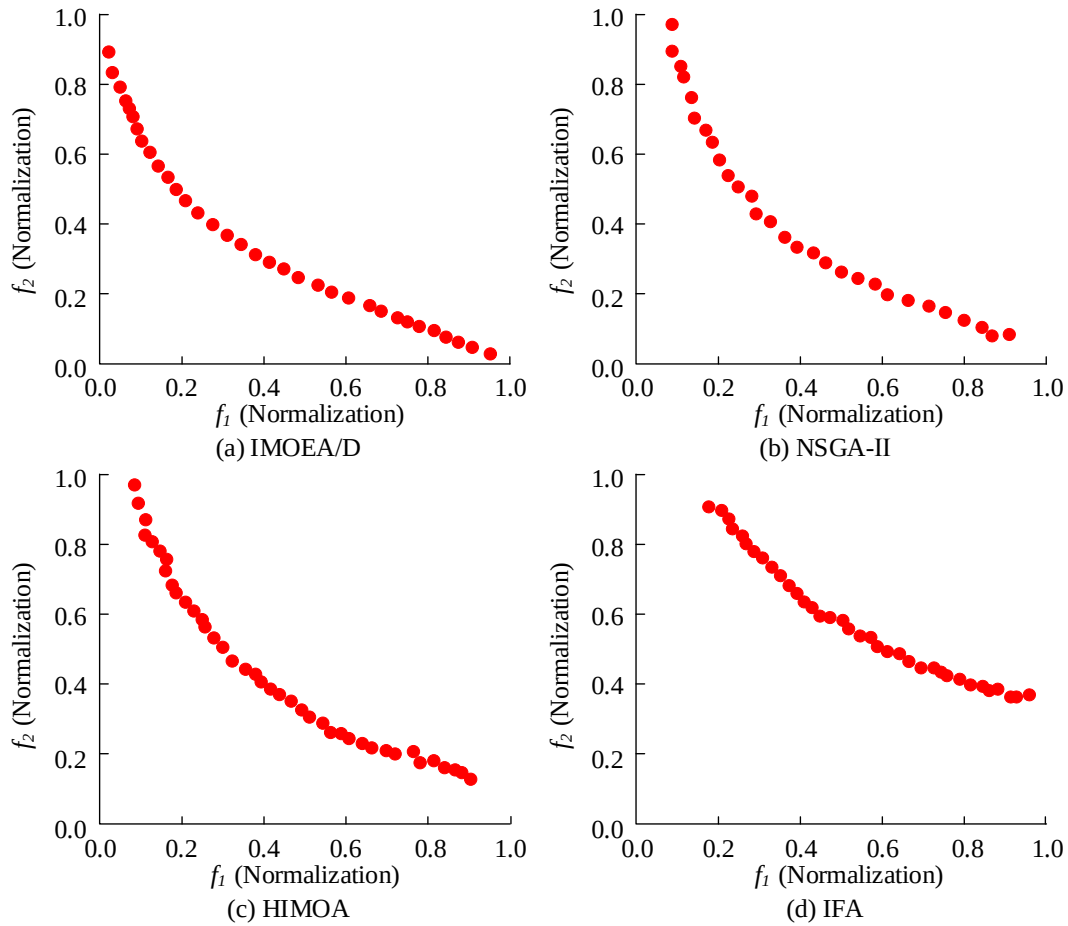


Fig. 7. Pareto front solutions of different algorithms

Figure 7(a) shows the Pareto front curve of IMOEA/D. f_2 continued to decrease from about 0.95 to below 0.02 as f_1 increased, and all solution points were distributed continuously and uniformly without obvious aggregation or empty areas. The solution set of this algorithm not only covered the full trade-off interval of f_1 and f_2 , but also ensured that the distribution density of different trade-off points was consistent. Figure 7(b) is the Pareto front of NSGA-II. The solution points were slightly clustered near $f_1=0.8$, and the distribution uniformity was weaker than IMOEA/D. Figure 7(c) shows the Pareto front of HIMOEA. The solution points were obviously clustered near $f_1=0.75$, and the solution point density in some trade-off intervals was low. Figure 7(d) is the Pareto front of IFA. The solution points were densely clustered in the $f_1=0.4\sim 0.6$ interval, and the solution points in the $f_1>0.8$ interval are sparsely distributed. The distribution uniformity and continuity of the solution set were also better than NSGA-II, HIMOEA and IFA, indicating that this algorithm had significant advantages in the trade-off ability. Figure 8 presents the convergence.

In Figure 8(a), after 200 iterations, the average objective function value of IMOEA/D stabilized below 0.10 ± 0.009 , and there was no obvious decrease in subsequent iterations, indicating that IMOEA/D could quickly reduce the average objective function value, and the convergence speed was significantly faster than other algorithms. In Figure 8(b), after 200 iterations, the optimal objective function value of IMOEA/D remained below 0.10 ± 0.007 , and the optimal objective function values of NSGA-II, HIMOEA, and IFA were stable at around 0.19 ± 0.018 , 0.28 ± 0.025 , and 0.32 ± 0.028 , respectively. IMOEA/D not only had fast convergence speed and low final objective function value, but also had more uniform solution quality within the population. It was better than NSGA-II, HIMOEA and IFA in convergence performance and solution set stability of MOO. The comprehensive comparison of the solution quality of different algorithms is shown in Figure 9. GD evaluated the closeness of the algorithm solution set to the real Pareto front. The smaller the GD, the better the convergence of the solution set. The Inverted Generational Distance (IGD) reflected the convergence and distribution uniformity of the solution set. The smaller the IGD, the solution set was both close to the Pareto front and evenly distributed.

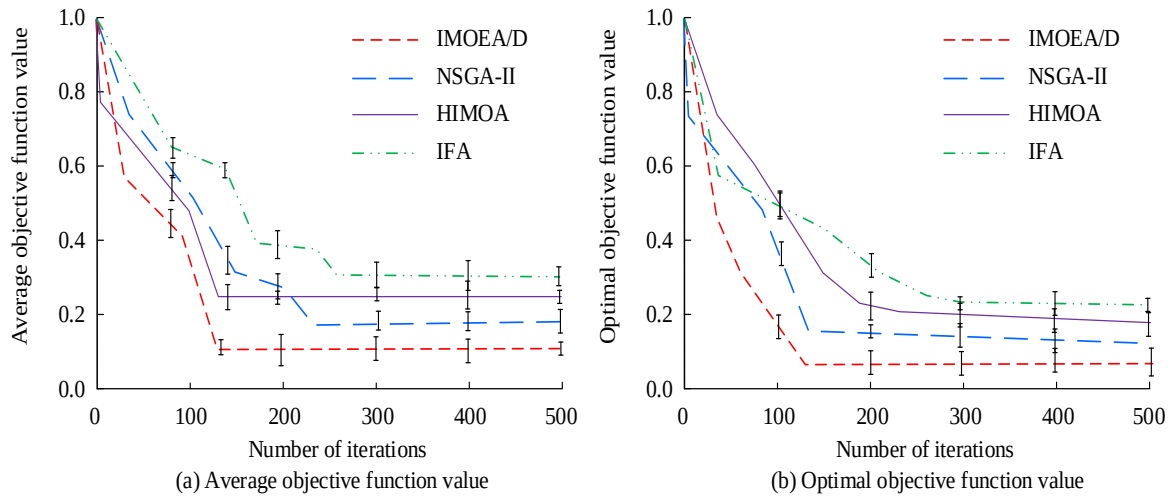


Fig. 8. Convergence of different algorithms

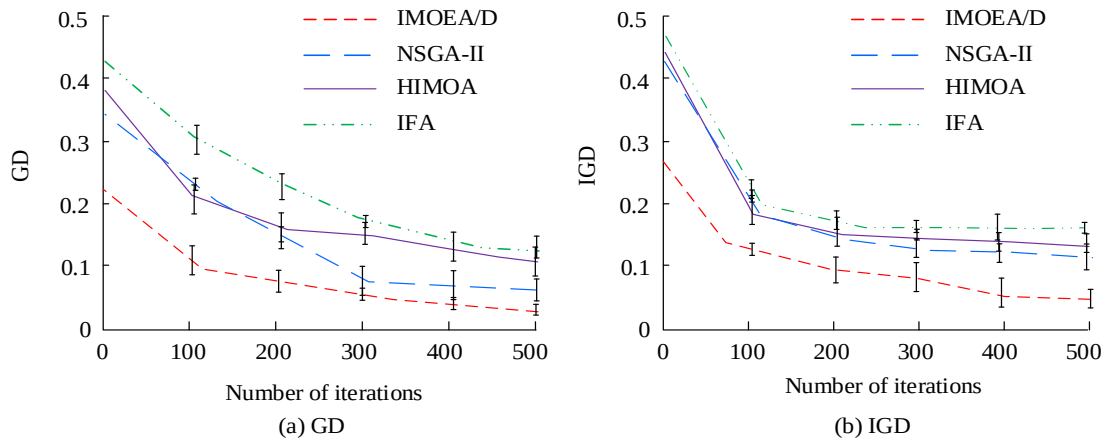


Fig. 9. Comprehensive comparison of solution quality of different algorithms

In Figure 9(a), when iteration reached 300 generations, the GD value of IMOEA/D dropped to 0.078 ± 0.012 , which was close to the ideal convergence state. At this time, the GD values of NSGA-II, IFA, and HIMOEA were still as high as 0.169 ± 0.015 , 0.215 ± 0.021 , and 0.198 ± 0.019 , respectively. After 500 iterations, the GD value of IMOEA/D was only 0.025 ± 0.004 . The improved Chebyshev aggregation function and dynamic neighborhood update mechanism adopted by IMOEA/D could more accurately guide the population to converge, effectively improving the convergence performance. In Figure 9(b), after 300 iterations, the IMOEA/D IGD value was 0.103 ± 0.014 , reflecting its significant advantage in the uniformity of solution set distribution. The adaptive weight vector generation strategy designed by IMOEA/D could ensure the uniformity of weight distribution. Combined with the mixed mutation strategy, the population diversity was maintained, allowing the algorithm to maintain a uniform distribution of solution sets during the convergence process and avoid local aggregation.

The MOO algorithm had inherent randomness. The result of a single run was not sufficient to fully demonstrate its superior performance. Therefore, all comparative experiments were independently repeated 30 times. The evaluation indicators were compared among the algorithms based on the average values of the 30 runs, and the standard deviation, minimum value, and maximum value were also recorded. When the iteration reached 500 generations, the GD mean of IMOEA/D was 0.025 (standard deviation 0.004, minimum 0.019, maximum 0.032), the IGD mean was 0.045 (standard deviation 0.006, minimum 0.037, maximum 0.055), the GD mean of NSGA-II was 0.169 (standard deviation 0.015, minimum 0.148, maximum 0.191), and the IGD mean was 0.187 (standard deviation 0.018, minimum 0.162, maximum 0.215). The mean values of all indicators of IFA and HIMOEA were between those of NSGA-II and IMOEA/D. The Wilcoxon signed-rank test was used to statistically test the differences in each indicator between IMOEA/D and each comparison algorithm at a significance level of 0.05. The results showed that all p -values were less

than 0.05, indicating that IMOEAD was significantly superior to NSGA-II, IFA, and HIMOAA in the GD, IGD, and Hypervolume (HV) indicators.

3.3 Validation of MOO Algorithms in Different Scenarios

To verify the adaptability and optimization effect of the IMOEAD algorithm proposed in the study in actual building scenarios, residential buildings were selected as the research object, with building energy consumption and living comfort as the core optimization objectives. Building Comprehensive Energy Consumption (BCEC) covers all-dimensional energy consumption such as heating, cooling, lighting, and ventilation. A lower value signifies the better energy consumption optimization effect. The Indoor Thermal Comfort Index (ITCI) adopts the Predicted Mean Vote (PMV) evaluation system. The value range is -3 (extremely cold) to +3 (extremely hot). The closer to 0, the higher the living comfort. In the experiment, $|ITCI|$ is used as the measurement standard. A lower value signifies the better comfort. Figure 10 presents the optimization effects.

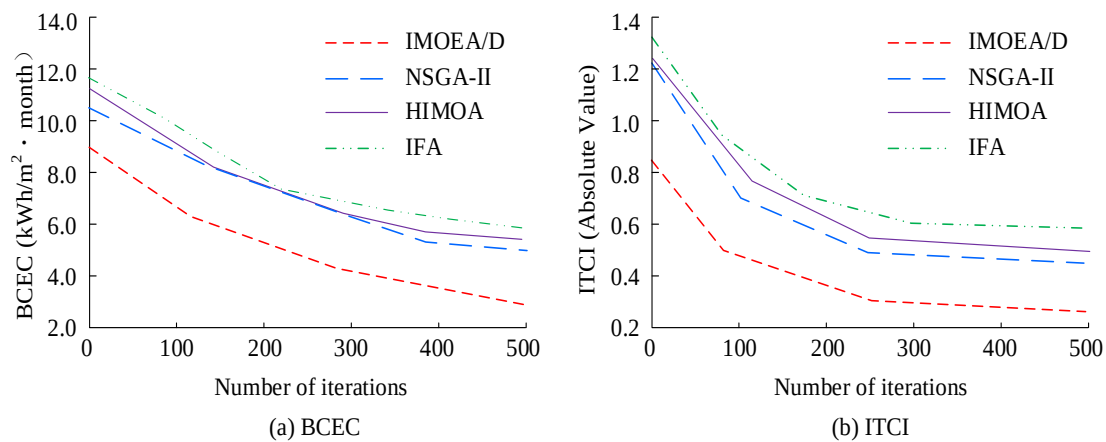


Fig. 10. Optimization effects of energy consumption and comfort of residential buildings using different algorithms

In Figure 10(a), after 100 iterations, the BCEC of IMOEAD was 6.4kWh/m²·month, which initially reflected the advantages of energy consumption optimization. After 300 iterations, the BCEC of IMOEAD dropped to 4.3kWh/m²·month, and the range of energy consumption optimization was further expanded. After 500 iterations, the BCEC of IMOEAD reached 3.2kWh/m²·month, which was 38.5% lower than that of NSGA-II. In Figure 10(b), IMOEAD also showed significant advantages in comfort optimization. After 500 iterations, the $|ITCI|$ of IMOEAD was only 0.22, and it always remained within the optimal comfort zone. IMOEAD could effectively reduce building energy consumption and significantly improve living comfort simultaneously. The optimization effect was far better than the comparison algorithm, and it had the feasibility of being applied in actual residential building energy-saving design. There were complex trade-off relationships among various optimization indicators.

Reducing annual operating energy consumption usually brought the potential risk of decreased thermal comfort, while improving the rate of meeting thermal comfort standards often led to an increase in energy consumption and initial investment costs. Carbon emissions were positively correlated with building operating energy consumption. Although high-performance selection of the enclosure structure was beneficial for energy conservation and thermal comfort, it would increase the initial investment. IMOEAD optimized and screened parameter combinations through Pareto solution sets. It coordinated the parameters of the enclosure and air conditioning system within the constraints of the construction project, avoiding over-compressing the indoor thermal environment conditions in pursuit of energy conservation. Therefore, it could achieve energy consumption reduction while maintaining a thermal comfort rate of over 92%. Some candidate solutions could balance lower carbon emissions and controllable initial investment. Different Pareto solutions corresponded to different trade-offs in each indicator. The multi-dimensional optimization effects are presented in Table 5.

Table 5. Comprehensive comparison of multi-dimensional optimization effects

Climate zone	Optimization indicators	Unit	IMOEA/D	NSGA- II	IFA	HIMOAA
Cold climate zone (Harbin)	Annual operational energy consumption	kWh/m ² ·year	42.3	65.7	72.1	68.9
	Carbon emissions	kgCO ₂ /m ² ·year	28.6	43.2	47.5	45.1
	Initial investment cost	Yuan/square meter	1280.0	1450.0	1520.0	1480.0
	Proportion of thermal comfort compliance time	%	92.7	81.5	78.3	80.2
	Lighting satisfaction	%	89.4	82.6	79.8	81.3
Cold climate zone (Beijing)	Annual operational energy consumption	kWh/m ² ·year	38.6	57.9	63.4	60.5
	Carbon emissions	kgCO ₂ /m ² ·year	25.3	37.8	41.2	39.4
	Initial investment cost	Yuan/square meter	1150.0	1320.0	1380.0	1340.0
	Proportion of thermal comfort compliance time	%	94.2	83.8	80.5	82.6
	Lighting satisfaction	%	91.6	84.3	81.7	83.5
Hot summer and cold winter climate zone (Shanghai)	Annual operational energy consumption	kWh/m ² ·year	35.7	52.4	57.8	54.6
	Carbon emissions	kgCO ₂ /m ² ·year	23.1	34.5	37.6	35.8
	Initial investment cost	Yuan/square meter	1080.0	1240.0	1290.0	1260.0
	Proportion of thermal comfort compliance time	%	95.5	85.3	82.1	84.4
	Lighting satisfaction	%	93.2	86.7	83.9	85.8
Hot summer and warm winter climate zone (Guangzhou)	Annual operational energy consumption	kWh/m ² ·year	31.2	46.8	51.3	48.7
	Carbon emissions	kgCO ₂ /m ² ·year	20.5	30.7	33.4	31.9
	Initial investment cost	Yuan/square meter	1020.0	1180.0	1230.0	1200.0
	Proportion of thermal comfort compliance time	%	96.8	87.6	84.2	86.5
	Lighting satisfaction	%	94.7	88.5	85.6	87.4

In Table 5, after IMOEA/D optimization, the energy consumption of the whole life cycle of the building was reduced by 29.5% to 38.6%, and the carbon emission was reduced by 30.1% to 39.8% compared with the comparison algorithm in each climate zone. The energy consumption and carbon emission reduction advantages in the severe cold climate zone were the most significant. The initial investment cost of IMOEA/D was 11.7%~16.4% lower than that of the comparison algorithm. The proportion of thermal comfort compliance time after IMOEA/D optimization exceeded 92%, up to 96.8%, and lighting satisfaction exceeded 89%. Although it faced higher energy consumption control pressure in severe cold climate zones, it still maintained significant optimization advantages. It proved that it had wide adaptability across climate zones and could meet the MOO needs.

3.4 Ablation Experiment

To quantify the independent contributions of each improvement strategy, additional ablation experiments were conducted under the same experimental conditions. The comparison groups included traditional MOEA/D, MOEA/D with only adaptive weight vectors (MOEA/D-AW), MOEA/D with only dynamic neighborhood (MOEA/D-DN), MOEA/D with only mixed mutation (MOEA/D-HM), and the complete IMOEA/D. The evaluation results are shown in Table 6.

Table 6. Performance comparison of various algorithms in ablation experiments

Algorithm	GD	IGD	HV	Convergence rate (%)	Calculation time (s)	Diversity index (Δ)
MOEA/D	0.112	0.168	0.742	62.4	186	0.387
MOEA/D-AW	0.071	0.102	0.803	74.1	201	0.215
MOEA/D-DN	0.065	0.098	0.812	76.8	209	0.228
MOEA/D-HM	0.083	0.121	0.785	69.5	194	0.251
IMOEA/D	0.025	0.045	0.892	91.3	238	0.163

The results in Table 6 show that the dynamic neighborhood had the most significant improvement on GD (a reduction of 42.0%), the adaptive weights had the most obvious enhancement on the diversity index (a reduction of 44.4%), and the hybrid mutation contributed the middle to the convergence rate (an increase of 7.1 percentage points). When all three strategies were enabled simultaneously, the GD and IGD of IMOEA/D dropped to 0.025 and 0.045 respectively, while HV reached 0.892 and the convergence rate increased to 91.3%. All these indicators were superior to any single improvement strategy, indicating that the three mechanisms had a synergistic effect. In terms of computational time, the complete IMOEA/D took 238 seconds, which was 28.0% more than traditional MOEA/D, but the performance improvement obtained was within the acceptable range for engineering.

4. Conclusions

This study addressed the multi-objective conflicts in building energy-saving design regarding energy consumption and thermal comfort, as well as the uneven distribution of solution sets and slow convergence speed of existing MOO algorithms. A MOO scheme based on the IMOEA/D was proposed. Based on the residential building prototypes of four typical climate zones in China (extremely cold, cold, hot summer cold winter, and hot summer warm winter), the EnergyPlus simulation model was established. The IMOEA/D was used to conduct collaborative optimization of the thermal parameters of the envelope structure and the equipment operation parameters for each climate zone. The experimental results showed that under 500 iterations, the GD index of IMOEA/D was 0.025, and the IGD index was 0.045. The convergence performance and the uniformity of the solution set distribution were significantly better than those of NSGA-II, IFA, and HIMOEA. In the residential building scenarios of the four climate zones, the annual operating energy consumption after optimization by IMOEA/D was 42.3, 38.6, 35.7, and 31.2 kWh/m²-year, respectively, and the corresponding carbon emissions were 28.6, 25.3, 23.1, and 20.5 kgCO₂/m²-year, which were reduced by 30.1% to 39.8% compared to the comparison algorithms. The time for thermal comfort to be met was over 92%, with the highest reaching 96.8%. The initial investment cost was reduced by 11.7% to 16.4%. These results were attributed to the performance

of the IMOE/D algorithm in residential buildings, the four climate zones, and the set parameter range. The conclusion was limited to the multi-story residential building prototype and the corresponding climate conditions, and its applicability could not be extrapolated without adjustment to public buildings, high-rise residences, or industrial buildings and other types. In addition, the thermal comfort evaluation in this study adopted a deterministic occupancy mode assumption and did not consider the random factors such as residents' opening windows behavior, shading adjustment, and air conditioning usage habits on the dynamic impact on actual energy consumption and comfort. Further research can introduce a random simulation module for human behavior based on proxy models and couple it with the MOO framework to enhance the algorithm's robustness in real living scenarios.

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Experimental investigation of SnS:Cu₂O nanophotonic thin films for enhancing voltage output of solar cell

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Abstract

This paper aimed to find a way that the light could be allowed to react with nano photonic material and enhance the output voltage of the photo cell. SnS:Cu₂O (1:1) was used to produce a thin film nanophotonic material. It was heated at 200 degrees Celsius to create a quantum effect. The results from an X-ray diffraction (XRD) showed the sample had changed from its normal form to become polycrystalline. The size decreased from an average size of 16nm to 12nm and the dislocation density (δ) increased from 0.003906 to 0.010000. Results showed that this material has a high average of optical transmittance greater than 80%. When annealed the optical transmittance was shifted towards lower wavelength. By implementing the prepared nanophotonic thin film into a silicon based solar cell system, a clear increase in produced voltage was observed to increase from 0.0 volts to 2.34 volts when exposed to sunlight. These results show how light and nano particles have a great positive effect on each other when used together. Such results would be considered in the field of optics, solar cells and communications, leading to more advanced technology in the future.

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1. Introduction

As light is one of the fundamental elements of nature, it has played a crucial role in the development of science and technology. With scientific advancements in recent decades, new fields have emerged that rely on exploiting the properties of light at extremely small dimensions, leading to the emergence of nanophotonics. This science represents an intersection of physics, engineering, and applied sciences, where light is used at the nanometer scale to achieve innovative applications in communications, computing, energy, and biomedicine. Nanophotonics is a relatively new but already well-defined field of science and technology that aims to establish and utilize the unusual properties of light and its interaction with matter in various nanostructures. Nanophotonics involves the confinement of light in complex media and electron waves in nano-solids, which determine the various physical phenomena in nanophotonics [1].

The challenge is to control light and its interaction with matter at the nanoscale to obtain new optical properties and functions that go beyond the limits of conventional physics and natural materials. An encouraging approach to these difficulties is to employ quantum dots. Quantum dots can respond precisely to light and by controlling their size and shape their optical properties can be tuned. Because quantum dots are very small, containing only a few hundred atoms, they are surrounded by materials with a larger band gap. Here the gap energy is the energy difference between the top of the valence band and the bottom of the conduction band (the minimum energy required to excite an electron). According to quantum theory, the electrons in quantum dots are confined in three dimensions – length, width and height. This three-dimensional confinement gives them a special property of interacting with light with remarkable efficiency. This effect results in drastic changes in the absorption and emission properties of light, depending on the size of the

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quantum dots. In the quantum dots the electrons are confined to a small volume. The energy levels go up. The energy gap between the ground and excited states goes up. So, when illuminated the quantum dots absorb the light energy and the electrons jump to a higher energy level then fall back to their original energy level by emitting photons. The emitted light's wavelengths depend on the size of the quantum dot, so the same dot can be different colors depending on its size. The rapid progress of modern technology and the huge potential to control light and its interaction with matter at nanometer scale has made nanophotonics an important area of research. Research in this field aims to realize new optical properties and functions beyond the limits of traditional physical materials. Considering the difficulties of this field, like the overcoming of the limits of traditional light diffraction and the design of advanced optics, several solutions have been developed, including the use of quantum dots and their integration with conventional devices. This work explores the essential role of light in nanophotonics, from the theoretical principles that govern the interaction of light at the nanoscale to the practical applications that are revolutionizing fields such as communications and renewable energy. The research also addresses the latest developments in metamaterial design and mechanisms for integrating them with light to achieve unconventional control over optical properties. This research aims to study the role of light in nanophotonics, focusing on the fundamental physical mechanisms and promising applications [2].

Fig. 1 shows the photonic crystals or optical crystals as a visible representation. The color bar on the top of the image represents the square magnitude of the E-field normalized to an arbitrary number. It will be dynamically normalized to values between 0 and 1 where 0 is no intensity of electric field or dark black/ red and 1 would be high intensity of electric field bright white/yellow. y-Axis in microns (μm) from 0-12. X-Axis is on the same space scale as the y-axis, so, also in (μm) from 0-20 and over.

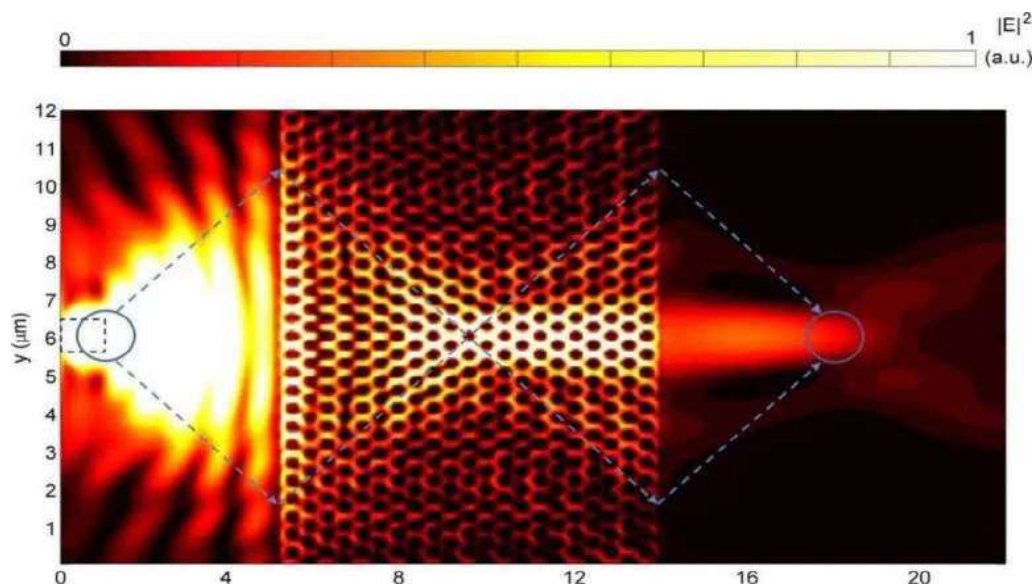


Fig. 1. Photonic crystals or optical crystals [3]

1.1 Nature of Nanophotonics

Nanophotonics can be defined as the science and technology of confining the light waves and electrons in the nanophotonic scale. Electron confinement effect on the optical properties of matter especially on semiconductors and insulators has been a matter of concern for study. These effects phenomena are generally called quantum confinement phenomena, because they are a manifestation of the wave properties of the electrons. These effects show up optically in size dependent optical absorption spectra, emission spectra and transition probabilities of structurally engineered solids of the order of a few nanometers. Their possible applications are various optical elements with certain shapes and sizes can be changed (emitters, filters, lasers, optical elements and switches, photoelectric modulators, and more). This area of nanophotonics has been studied in depth since the 1970s, and several study books have been written about it.

In a different area of nanophotonics, light waves are limited in structurally designed insulating materials. This includes the idea of photonic solids, which control the movement of light waves in the same way that solids control the movement of electron waves [4]. To put it simply, this area of nanophotonics is very traditional. It only uses wave optics and does not go beyond Maxwell's equations. The main thing that is done in this field is making new photonic parts, such as high-quality optical cavities and cavities in superintegrated and other devices. The study of how light and matter combine in nanostructures is a third area of nanophotonics. This includes changing the spontaneous emission and scattering of light. Optical engineering of photonic frequencies can make spontaneous photon emission and scattering stronger or weaker. Recently, in nanophotonics [5], it has become possible to trap light and matter states in microcavities and optical cavities. If you look at the basics of nano-optics, you can see that optical effects at the nanoscale level are possible and not limited by any basic rules. The way light moves through space is controlled by the dispersion relation $\omega = c \cdot k$. This relation links a photon's wave vector $k = \sqrt{k^2x + k^2y + k^2z}$ to its angular frequency ω through the speed of propagation c . For photons, Heisenberg's uncertainty principle says that the product of the uncertainty in where a tiny particle is in space in a certain direction and the uncertainty in its momentum component in that direction cannot be less than h^* . This leads to the following connection:

$$\Delta(hk_x) \cdot \Delta x \geq \frac{h}{2} \quad (1)$$

which can be rewritten as

$$\Delta x \geq \frac{1}{2\Delta k_x} \quad (2)$$

The interpretation of this result is as follows: The constraint depends on the spatial constraint achievable for photons is inversely proportional to the extent of the wave vector components' propagation in the opposite spatial direction, x . Such propagation of wave vector components occurs, for example, in a light field converging towards a focal point, such as behind a lens. The field can be represented by a local superposition of plane waves propagating at different angles, where the wave vector in the opposite direction is k_x . The total length of the wave vector in free space is $k = 2\pi/\lambda$, and this leads to:

$$\Delta x \geq \frac{\lambda}{4\pi} \quad (3)$$

which is consistent with the widely recognized expression for the Rayleigh diffraction limit. Note that the achievable spatial constraint is limited only to the extent of the wave vector components' propagation in a given direction. To increase the extent of the wave vector components' propagation, a mathematical stratagem can be employed. If two perpendicular orientations in space that are arbitrarily chosen for example, x and z , one of the wave vector components can be increased to values exceeding the total length of the wave vector, provided that the wave vector in the perpendicular direction becomes purely imaginary. If this is the case, then the requirement for the total wavevector length can be satisfied [6].

$k = \sqrt{(k^2x + k^2y + k^2z)}$ to be $k^2\pi$. The range of wavevectors that can be used in the x -direction grows as the wavevector gets bigger. And Eq (3) is no longer the only thing that limits light. The rise in the wavevector component in the opposite direction, on the other hand, must be completely imaginary. Putting this wavevector that is only imaginary into the formula for a plane wave gives us $\exp(ik_z z) = \exp(-|k_z|z)$. In one direction, this leads to a field that shrinks exponentially, which is called a declining wave. In the other direction, however, the field grows exponentially. Since fields that grow exponentially don't matter in real life, the method described can be safely ignored in order to find a solution. Also, keep in mind that Eq (3) is always true in empty space. Still, this argument only works when there is an infinite amount of empty room. If you split unlimited free space in half and give each half a different refractive index, the field that is exponentially decreasing can exist in one half of space without the field that is exponentially increasing in the other half. In the other half of space, there may be a different answer that meets the border conditions for the fields at the interface. These simple reasons show that Rayleigh's limit for light confinement is no longer strictly

true when there is spatial variation. However, it is possible for light to be confined indefinitely, at least in theory. This insight is what nano-optics is built on. One important question in nano-optics is how to change the structure of materials to get the best optical confinement that is possibly possible. Another question is what the physical effects are of fields that are both exponentially decaying and tightly contained [7].

1.2 Properties of Nanophotonics

One of the most prominent properties of nanophotonics is the ability to utilize photonic crystals and plasmons to guide and control light propagation in innovative ways. For example, this technology can be used to develop optical materials with unique properties such as a negative refractive index, which opens the door to the design of advanced applications such as optical devices that transcend the limitations of conventional photonics, such as perfect lenses and cloaking devices [8].

1.3 Nanoscale and Its Effects

Nanotechnology is a contemporary technology with numerous applications in the production of molecules or particles at the nanoscale. It is a sophisticated domain of science and engineering that concerns the production, processing, and application of minuscule units, structures, devices, and systems. Nanoparticles are characterized as individual particles with size not exceeding 100 nanometers. The surface structure, size and chemical nature are related to the unique features and properties of nanoparticles. This has led to a substantial increase of medical and industrial applications. Hence, the unique properties and physical modifications of different materials at the nanoscale have facilitated the development of industrial products with enhanced characteristics. The widespread use of nanotechnology products and nanomaterials has resulted in the presence of nanoparticles in the environment. [9]

1.4 Photonic Interaction at the Nanoscale

Photonics is the science of generation, absorption, emission, collection and control of light, and of its application in technology. Light is the visible portion of the electromagnetic spectrum to the human eye with wavelengths from about 400 to 700 nanometers. Moreover, ultraviolet and near-infrared extensions are usually added to reproduce the electromagnetic radiation spectrum, from 100 nm to 1-2 μm , which is the domain of photonics [10]. The propagation of light waves in a medium with inhomogeneities of the order of the wavelength of light is modified by many interferences and scattering effects. Vision is enabled by light scattering and interference, phenomena also observed in soap bubbles and the iridescent film of gasoline on a wet road after it rains, producing bright colors. The changes in space due to the changes in the conditions of the light propagation are small with respect to the wavelength of the light. They are particularly important in the size range from 10 to 100 nm to several micrometers. The space variations of light waves are connected with the differences of the dielectric permittivity. All matter is made of atoms. Nuclei and electrons make up atoms. The first electron shell of the basic hydrogen atom is at a radius of 0.053 nanometers. Atoms can bond to form molecules and solids. A typical molecule is on the order of 1 nm across. Typical crystalline solids have a lattice period on the order of 0.5 nm. There is a decrease in the light-matter interaction of processes directly related to the electron system in molecules and solids. Hence, a detailed study of the properties of electrons is required to understand the interaction of light with matter. Electrons have wave-like properties. They possess wave characteristics, like wavelength, as well as particle attributes, such as mass and charge. The kinetic energy of an electron confined by a potential energy due to its mobility in an electric field produced by an atomic nucleus is equivalent to the typical value of $k_B T = 27 \text{ meV}$ at room temperature. Dependent upon the temperature (T) and the Boltzmann constant (k_B), the characteristic thermal de Broglie wavelength of electrons in solids is approximately 10 nanometers under these conditions [12].

1.5 Quantum Confinement and its Effects on Quantum Dots

Quantum dots are one of the most well-known forms of nanomaterials. They are extremely small particles or three-dimensional, semiconducting nanostructures with dimensions ranging from 2 to

10 nanometers (10 to 50 atoms in diameter), and a volume ranging from 100 to 100,000 atoms in a single quantum dot. They are called quantum dots because they are very small particles of matter effectively concentrated in a single point with zero dimensions [13]. Fig. 2 shows a schematic of the thermal evaporation system working with the quantum point basis.

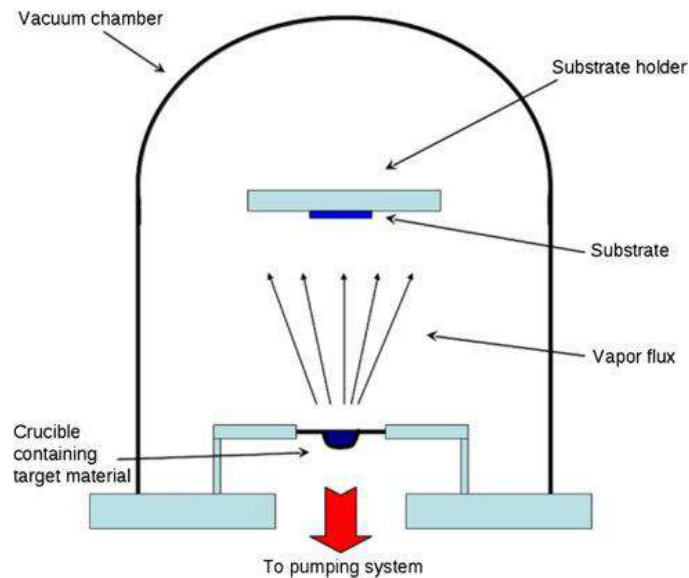


Fig. 2. The phenomenon of thermal evaporation in a schematic [14]

In quantum physics, quantum confinement occurs when electrons in a material are confined to such narrow boundaries that the effects of quantum mechanics become pronounced. The quantization of energy levels is brought about by this confinement in quantum dots, which means that electrons can only occupy certain energy values, instead of moving freely across a wide range of energies as in larger materials. This results in a phenomenon known as the quantum size effect.

These special properties make quantum dots ideal candidates for building solar cells. Solar cells are devices that convert light energy into electrical energy by the interaction between photons and electrons. Photons are absorbed, electrons are released, and a flow of electrons is created, which we call an electric current. The efficiency of light absorption and energy conversion can be optimized by designing quantum dots to a specific size [15]. Size dependence of electronic and optical properties of quantum dots This is dictated by quantum mechanics. They are under quantum confinement. Quantum confinement constrains the conduction band electrons and valence band holes, or excitons (bonded pairs of conduction electrons and valence holes). When exposed to an external light source, such as ultraviolet radiation, these dots cause their atoms to absorb this light, which is then emitted as different colors of different wavelengths [16].

The quantum confinement in quantum dots leads to novel phenomena in light-matter interactions, and new possibilities for innovative technological applications in the fields of energy, electronics and medicine. For example, quantum dots can be used in medical imaging applications because they can emit light of different wavelengths when treated with certain light sources. These properties can be used to develop more precise imaging devices and less invasive imaging devices. Moreover, quantum dots can be employed in the fabrication of optical sensors. Quantum dots can be tuned to respond to specific wavelengths, which gives them high sensitivity to changes in their environment. This opens up possibilities for applications such as pollution detection or monitoring biological processes in living organisms. Consequently, nanoscale interactions between light and matter play a prominent role in technological progress and quality of life with a multitude of applications in different fields. leading to a better understanding of quantum physics and its practical utilization [17].

1.6 Manipulating Light at the Nanoscale

For more than half a century, scientists have endeavored to guide and affect light at the nanoscale. Early theoretical investigations during the 1950s and 1960s at first intended to confine

electromagnetic radiation inside nanometer-scale structures. However, intensive experiments in the 1970s and 1980s ended in frustration as researchers endeavored to effectively detect and manipulate these optical interactions at the nanoscale utilizing the manufacturing capabilities accessible at that period. Persistent setbacks in this field have led some to question the relevance of nanooptics. The trend began to shift in the 1990s with the development of new nanoprinting techniques that enabled the characterization of subatomic features at wavelengths of light between 400 and 700 nanometers. Furthermore, new scanning microscopes allowed for the characterization of optical properties in real space at the nanometer level. Researchers were finally able to access the nanoscale to elucidate the behavior of light and the optical properties of materials at these minuscule length scales [18].

More fast progress has been made thanks to new ideas in related fields like making semiconductors, electron imaging, and quantum materials. New things like surface plasmon resonances, photonic band gaps, and metamaterials have been found that show that optical devices can work at the nanometer scale. Nanophotonics uses ideas from optics, quantum physics, and materials science to change how light and matter interact at the nanoscale level. Nanophotonics is a very active area of study thanks to advances in fabrication techniques and a deep understanding that has grown over many years. Nanophotonic elements, like photonic crystals, plasmonic surfaces, and metamaterials, are being used by scientists to get more precise control over how light is absorbed, scattered, emitted, and spread. Revolutions in a variety of application domains are being driven by the nanoengineering of light-matter interactions. Transformative capabilities, including invisible cloaking and single-molecule recognition, are now facilitated by nanophotonic components. As researchers keep improving light control at the nanoscale, nanophotonics anticipates a novel future in the renewable energy, fields of healthcare, communications, and information processing [19], [20].

1.7 Applications and Technologies

The unparalleled manipulation of light enabled by nanophotonics is resulting in advancements across several critical technologies and applications. Optical communications consist of the following:

1.7.1 Nanophotonic Components Such as Photonic Crystals

The optical functions can be integrated via optical modulators and plasmonic guides onto the chips to develop the nanophotonic integrated circuit. This makes it possible to process and transmit information more quickly by utilizing light rather than electrons. Nanophotonics is essential for the development of optical computers, communication technologies and optical-on-chips [21].

1.7.2 Renewable Energy

Nanodesigns markedly enhance the light absorption and emission efficiency of photovoltaic cells and illumination devices. Nanophotonic solar cells, LEDs, and lasers offer enhanced sustainability and cost-effectiveness in energy solutions. Quantum dots and luminous nanomaterials promote wavelength conversion for improved light harvesting.

1.7.3 Biomedicine

Nanoparticles and plasmonic surfaces facilitate the precise detection and quantification of biomarkers, including individual molecules. Nanophotonic technologies are propelling advancements in quick diagnostics, precise medicine, and the imaging of diminutive biological structures. Nanoparticle agents facilitate targeted phototherapies [22].

1.7.4 Quantum Technology

Confining and manipulating light at the nanoscale results in exceptional control over quantum states. This results in novel quantum light sources, modeling methodologies, and sensing materials. Nanophotonics will play a crucial role in the progression of quantum computing, information technology, and communications [23].

1.7.5 Right Beam

The propagation of light within the photonic crystal is obstructed by the propagation gap; hence, typical plane waves are reflected and do not readily infiltrate the crystal. The right beam directs incoming waves deep into the crystal by bending them, allowing for the concentrating of light at any specified depth within the obstructed gap [24].

1.7.6 Recent Developments

Nanophotonics is a rapidly evolving field, with new discoveries constantly expanding the possibilities for controlling light at the nanoscale. Among the most notable recent discoveries are:

- **Photonic Crystals:** Photonic crystals allow for the customization of light flux through dielectric structures with nano-patterns. Recent advancements have enabled the dynamic management of reconfigurable circuitry and the direction of propagation. Chip-sized photonic dielectrics based on photonic crystals can reduce the device footprint [25].
- **Nanolasers:** Minuscule nanolasers with wavelengths less than that of visible light can facilitate ultra-dense optoelectronic integration. Researchers have recently showcased room-temperature nanolasers constructed from two-dimensional semiconductors like molybdenum disulfide, which are optimal for on-chip applications [26].
- **Quantum Dots:** These nanocrystals could produce single photons for quantum technologies. New approaches are being developed to fabricate improved quantum dots that are deterministically placed in photonic crystal cavities, paving the way for scalable quantum photonics [27].
- **Nanoplasma:** With surface plasmons on metallic nanoparticles, light can be focused beyond the limit of diffraction at the nanoscale. Biosensing, high-resolution microscopy, and next-generation data store densities are all getting better thanks to progress in this area. This field is moving forward very quickly as new nanophotonic materials and production methods are created. Because of these improvements, it is now possible to get visual effects and qualities that were not possible before. Nanophotonics has a bright future ahead of it, with big changes coming to technology and health care [28].

2. Materials and Methods

A thin film of SnS:Cu₂O Nanophotonics was prepared by mixing an equal ratio (1:1) of tin sulfide (SnS) and Cuprous Oxide (Cu₂O) with (0.01) gm for each compound and evaporated onto glass and silicon substrates [29]. Preparation on glass substrates was carried out using a physical thermal evaporation technique with an Edwards Auto 306 coating plant. The glass substrates were ultrasonically cleaned with acetone, ethanol and distilled water. Deposition was performed under high vacuum of about 1×10^{-7} mbar with deposition rate of about 0.3 nm/s for desired thickness of 50nm. After deposition the films were annealed at 200°C for 2 hours. The XRD analysis of samples was done with Shimadzu-6000 X ray diffractometer. Optical properties were studied using SCINCO MEGA2100-SCINCO UV-Vis spectrophotometer from 300 to 1200 nm. Fig. 3 shows the Cuprous Oxide Cu₂O: SnS Nanophotonics glass thin film. This thin film was used to demonstrate the interaction between light and nanophotonic materials. The film was placed in front of a silicon solar cell under direct light of an incandescent 20-Watt CFL lamp with 1,200 lumens situated at a 15 cm distance to produce 1.21 mW/cm². The solar cell was connected to a voltmeter to observe the interaction between the film material and light considering I-V characterization for standard performance analysis of the solar cell. Measuring the pattern of X-ray diffraction for a Cu₂O:SnS Nanophotonic film was carried out with Cu-K α radiation ($\lambda \approx 0.15418$ nm), and a scanning range (2θ) of 20° to 40° adopted

3. Results and Discussion

For the film grown on a glass substrate, as depicted in Fig. 4, one peak can be noticed by referring to the (020) plane at angles of $2\theta = 31.169^\circ$ as a preferred orientation as compared to values of the card (JCPDS Card No. 01-079-2193). After the annealing process, it is possible to observe three

main peaks at angles of $2\theta = 19.673^\circ$, 31.673° and 35.637° . So, the preferred orientation at $2\theta = 19.673^\circ$ where the polycrystalline structure of the thin films is displayed in Fig. 4.

The lowering in the peak intensity of Cu₂O:SnS exhibits similar behavior to that obtained by Devika et al (2006) [30]. Scherrer's formula Eq. (4) was utilized to determine the crystallite size of the Cu₂O:SnS thin films.

$$D = \frac{0.9 \lambda}{\beta \cos \theta} \quad (4)$$

where λ stands for the wavelength, β is the FWHM and θ represents Bragg angle. From Table 1, the average crystal size of Cu₂O:SnS films are decreased after the annealing process at 200°C from 16 nm to 12 nm. The observed reduction in crystallite size down to the nanoscale restricts the exciton Bohr radius, thereby inducing quantum confinement effects. The apparent reduction in crystallite size following thermal annealing at 200 °C cannot be attributed to crystal splitting. Instead, it is highly likely driven by severe thermal mismatch stress between the Cu₂O:SnS film and the glass substrate. This induces severe lattice microstrain and structural defects, which manifests as diffraction peak broadening in the XRD spectra, artificially reducing the calculated Scherrer crystallite size. When the temperature of annealing offers the least amount of thermal energy needed to move the particles or atoms, the particles undergo diffusive motion, which reduces their size.

Using the Williamson-Hall equations, the density of dislocation (δ) Eq. (5) and micro-strain (ε), Eq. (6) values of each sample were determined where they are inversely proportional to the variation in crystalline size.

$$\delta = \frac{1}{D^2} \quad (5)$$

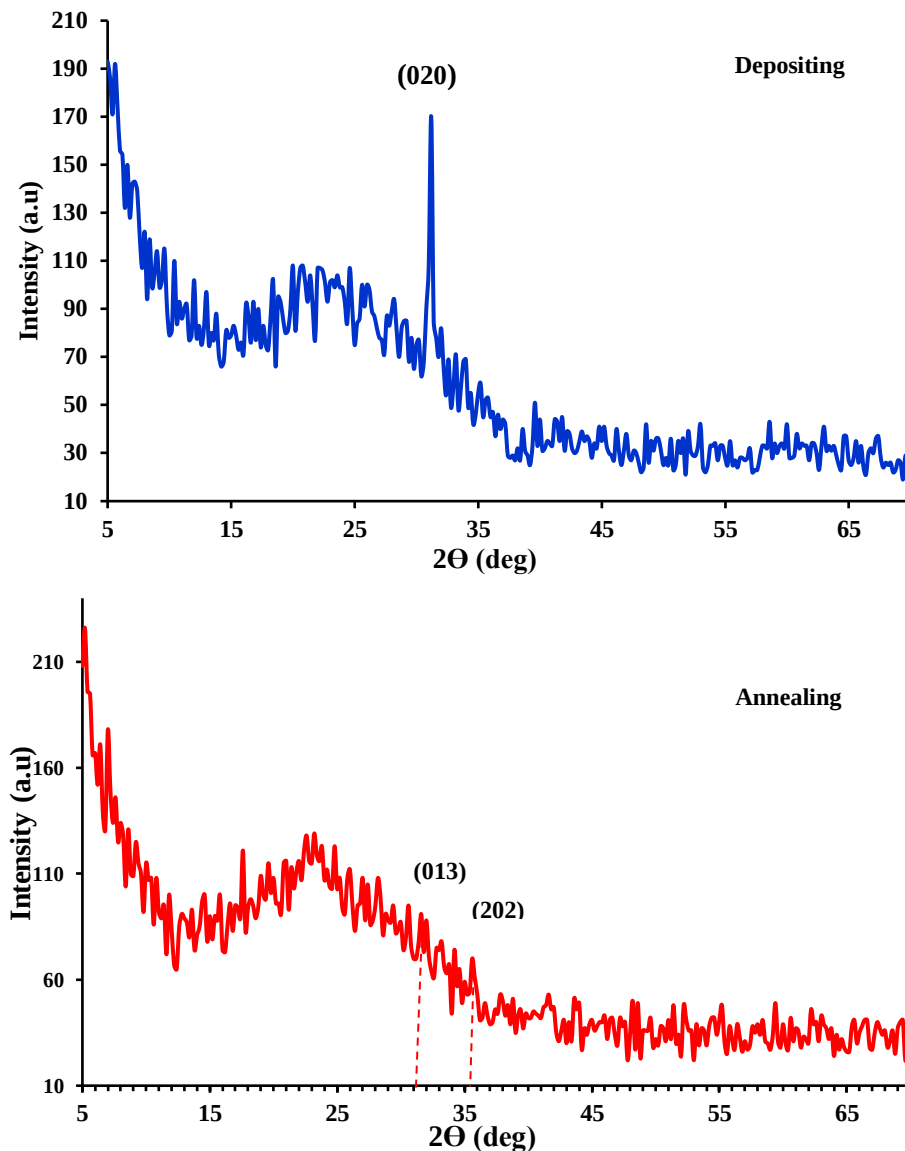
$$\varepsilon = \frac{\beta \cos \theta}{4} \quad (6)$$

As indicated in Table 1, an increase in the dislocation density of Cu₂O:SnS nanostructure thin films may be ascribed to a rise in the grain boundaries as a result of a change in the crystallite size. This may have been caused by the annealing process, which included raising the temperature [29, 31].



Fig. 3. Cuprous oxide Cu₂O: SnS nanophotonics glass thin film prepared

The measurement spectrum of optical transmission (T%) of Cu₂O: SnS Nanophotonic films in the 300-1100 nm range, the average of T% in the visible region for deposited films is roughly more than 80%, for annealed samples. It is clear that the process of annealing influences optical transmittance and shifts it toward shorter wavelengths (blue shift). Due to the effect of the quantum confinement brought on by the reduction in the size of nanoparticles and dimensional structure, a strong blue shift can be seen as displayed in Fig. 5.

Fig. 4. The X-ray diffraction for a Cu₂O:SnS nanophotonic of thin filmsTable 1. Obtained X-ray results of Cu₂O: SnS nanophotonics samples as growth and annealed

Samples	2θ °	FWHM	(hkl)	Average Crystal Size (nm)	$\delta=1/D^2$ (nm ⁻²)
Cu ₂ O NPs:SnS as deposited	31.169	0.514	(020)	16.05 ~16	0.003906
Cu ₂ O NPs:SnS annealed at 200° C	31.673	0.858	(013)	9.62~10	0.010000
	35.637	0.752	(202)	11.10 ~11	0.008264

Through using the system of nanophotonic thin films with solar cell and voltmeter, when an electric light was directed on the thin nanomaterial film after placing it on a silicon solar cell, it becomes evident from the voltmeter reading (output voltage increased from 0.00 to 2.34 V) that the efficiency of the solar cell is changed due to the influence of the nanomaterial used to manufacture the films. Fig. 6 shows output voltage increase of the solar cell due to the effect of nanophotonics thin films and light.

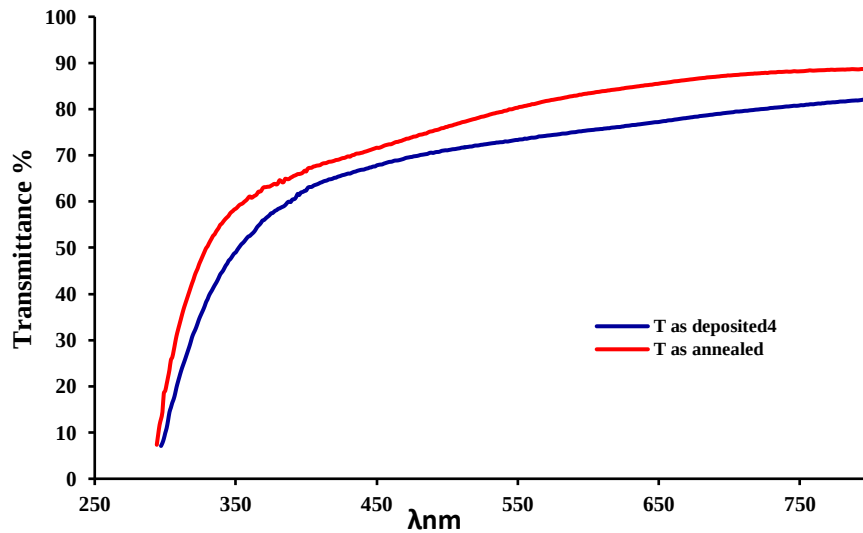


Fig. 5. Transmittance of Cu₂O NPs: SnS nanophotonics thin films before and after annealed at 200°C



Fig. 6. Output voltage increase of a solar cell due to the effect of nanophotonics thin films and light

4. Conclusions

In this paper it has been shown how Cu₂O: SnS nano photonic thin films can be fine-tuned and how they influence light matter interaction. When studying samples prepared through the study, it can be found that the heat treatment of the films at 200°C played a major role in determining the characteristics of the film used through quantum confinement. By using quantum confinement, it is clear to observe changes to the nanostructure of the thin film. Upon annealing it has been observed a shift from having one peak at $2\theta = 31.169^\circ$ (020) to three different peaks which shows that the sample used now has a polycrystalline structure and resulted average size decreased from 16 nm to 10 nm. The splitting that occurred at the grain boundary showed an increased value of dislocation density (δ) going from 0.003906 to 0.010000. It was also possible to observe changes to the optical properties of the thin films used, with a high average of optical transmittance (T%) greater than 80%. When annealed the optical transmittance was shifted towards lower wavelength. By implementing the prepared nanophotonic thin film into a silicon based solar cell system, it was clear to see an increase in voltage. This was shown when the voltmeter reading jumped from 0.0 volts to 2.34 volts under direct light.

According to these findings it can be concluded that there is a way to manipulate nanoparticles to convert more energy from sunlight. With high transparencies and quantum confinement there is great potential for Cu₂O: SnS nanophotonic thin films in the field of optoelectronics, communication and photovoltaic. As the results of this research illustrate the emerging integration between quantum dots and nanophotonics in the design of optical devices, practical applications of such research include optical communications, enhanced solar cells, and precise medical diagnostics using nanoparticles, along with promising potential in quantum computing and communications.

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Explainable remaining useful life prediction of rolling bearings using open-source vibration data

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Abstract

Remaining useful life (RUL) prediction is more useful for maintenance decision-making when the basis of a warning can also be inspected. This study develops an explainable RUL prediction framework for rolling bearings using the open-source NASA IMS 1st_test run-to-failure dataset. Each one-second vibration record is divided into 1024-sample windows, from which RMS, Peak-to-Peak, Kurtosis, and Crest Factor are extracted and aggregated as a four-feature observation. Ten consecutive observations form the input to a lightweight gated recurrent unit (GRU) regressor. To avoid imposing an arbitrary degradation-onset boundary on the prediction target, normalized RUL is defined directly from elapsed life and decreases continuously from 1 to 0. Using the first 70% of the run for training and the final 30% for chronological testing, five random initializations give an MAE of 0.329 ± 0.084 and an RMSE of 0.351 ± 0.098 for the GRU model, compared with MAE/RMSE values of $0.348 \pm 0.128/0.393 \pm 0.149$ for the 1D-CNN and $0.363 \pm 0.011/0.386 \pm 0.013$ for the LSTM. Gradient sensitivity, Integrated Gradients, SHAP, and permutation analysis consistently identify Kurtosis as the most influential late-life feature, while a feature-deletion test produces the largest average MAE increase when Kurtosis is removed. The study is intentionally limited to one fixed-speed, fixed-load IMS run and should therefore be interpreted as controlled benchmark validation rather than vehicle-level validation.

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1. Introduction

Background and Problem Formulation

Rolling bearings in vehicle drivetrains and industrial rotating machinery operate under cyclic loading, friction, and speed variations. Progressive damage such as pitting or spalling can increase vibration and, if undetected, lead to unplanned downtime or secondary damage. Remaining useful life (RUL) prediction seeks to estimate the useful operating time remaining before a defined end-of-life condition and is therefore an important element of condition-based and predictive maintenance [1-3]. Vibration measurements are widely used for this purpose because they contain both slowly evolving energy changes and impulsive signatures associated with bearing degradation.

A numerical RUL estimate alone, however, provides limited information about why a warning has been issued. For maintenance personnel, an inspectable link between the prediction and degradation-related vibration indicators can improve traceability and support engineering review. This work therefore considers RUL prediction together with feature-level explanation, while deliberately limiting its experimental claims to a controlled open-source bearing benchmark.

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1.1 Related Works and Literature Gap

With the rapid development of data acquisition and intelligent computing, data-driven vibration analysis has become a major direction in RUL research. Hybrid architectures combining convolutional neural networks (CNNs) with recurrent models such as LSTM, BLSTM, and GRU are widely used because convolution can capture local patterns while recurrent units' model temporal dependence [4,5]. To improve generalization under limited labelled data or operating-condition shifts, transfer-learning and semi-supervised strategies have been explored [6-8]. Graph-based models provide another route to learning coupled degradation representations [9]. Open benchmark data such as NASA IMS remain important for controlled method comparison and reproducibility [10-12].

Recent RUL research increasingly combines prediction with uncertainty quantification, operational decision-making, and physical knowledge. Deep learning-stochastic ensembles have been linked to predictive-maintenance scheduling and dynamic mission-abort policies [13], while RUL-aware physics-informed Bayesian digital twins connect probabilistic prognostics with safety-constrained control [14]. Deep ensembles have also been used for dynamic burn-in optimization with precise lithium-ion battery RUL prediction [15]. In parallel, explainable RUL frameworks increasingly incorporate degradation physics and attribution mechanisms: Kim et al. developed a physics-informed explainable RUL framework [16], and Zhang and Wang combined rolling-bearing RUL uncertainty prediction with SHAP-based interpretation [17]. These developments show that explain ability is no longer limited to post-hoc visualization. Against this background, the present work addresses a narrower and deliberately transparent question: whether a compact set of physically interpretable time-domain vibration indicators can support temporal RUL prediction while providing a directly inspectable feature-level explanation on an open bearing benchmark.

Related studies also address interpretable intelligent fault diagnosis in rotating machinery [18], explainable predictive maintenance [19], general condition-monitoring RUL modelling [20], predictive maintenance for multi-rotor systems [21], explainable Industry 4.0 fault detection [22], and explainable aero-engine RUL estimation [23]. Work on tool-wear RUL prediction [24] and degradation-start detection [25] further illustrates the breadth of prognostics and interpretability research. Together with the recent studies above, this literature motivates transparent prognostic modelling while leaving room for a compact, feature-level analysis on an open rolling-bearing run-to-failure benchmark.

1.2 Research Objectives and Primary Contributions

The study is intentionally confined to the NASA IMS 1st_test run-to-failure bearing dataset and does not claim a universally superior neural architecture or direct validation for vehicle duty cycles. Its contribution is a compact and reproducible workflow that links physically interpretable vibration indicators, temporal RUL regression, and feature-attribution checks. The central question is whether a small set of established degradation indicators can provide useful RUL information while retaining an inspectable basis for late-life warnings.

Data and features. The analysis uses the IMS 1st_test records at their native acquisition resolution; no artificial up sampling is applied. Bearing 3 is selected because an inner-race defect was reported at the end of this run, and Channel 5 is used as the primary vibration measurement. Each one-second record contains 20,480 samples. It is divided into twenty non-overlapping 1024-sample windows, and four time-domain indicators—RMS, Peak-to-Peak, Kurtosis, and Crest Factor—are computed for each window and averaged to form one chronological feature observation.

Prediction model and baselines. Ten consecutive four-feature observations are organized as a temporal input sequence. A lightweight GRU regressor is used as the proposed sequence predictor and is compared with 1D-CNN and LSTM reference models under the same chronological 70%/30% split. All three models use the same feature sequences, training portion, RUL targets, optimization budget, and evaluation metrics. Results are repeated across five random initializations to expose sensitivity to model initialization.

Explain ability. The numerical RUL estimate is complemented by feature-level attribution. Gradient sensitivity is used as the primary local explanation and is cross-checked with Integrated Gradients, SHAP, permutation sensitivity, and a feature-deletion test. Explanations are evaluated across early-middle-, and late-life stages and across five random initializations. The attribution scores are interpreted as predictive sensitivity of the trained model rather than as causal proof of a physical damage mechanism.

1.3 Chapter Arrangement

Section 2 describes the IMS data, signal preprocessing, RUL target and sequence construction, prediction models, attribution methods, and chronological evaluation protocol. Section 3 reports predictive performance, a data-driven degradation-onset check, feature-deletion results, and attribution robustness. Section 4 discusses engineering interpretation and generalization limitations, and Section 5 concludes the study.

2. Materials and Methods

2.1 Data Source and Study Scope

RUL model development requires observations spanning healthy operation through late-life degradation. The NASA IMS bearing run-to-failure dataset provides a controlled public benchmark with complete vibration histories and is therefore used for reproducible method evaluation. The present study does not treat IMS as a surrogate for vehicle duty cycles. Instead, it uses the dataset to examine the prediction-and-explanation workflow under fixed operating conditions; transfer to variable-speed or vehicle environments remains outside the validated scope of this work.

In IMS Test 1, four Rexnord ZA-2115 double-row roller bearings are mounted on a common shaft and operated at 2000 RPM under a 6000 lb radial load. Each vibration file contains 20480 samples acquired over approximately one second, and 2156 chronological files are available from the run. Bearing 3 developed an inner-race defect by the end of Test 1; therefore, Channel 5, one of the two vibration channels associated with Bearing 3, is used as the primary signal in this study. The first 70% of the chronological observations are reserved for model training and the final 30% are held out for testing. The experimental configuration and data-acquisition procedure are summarized in Fig. 1.

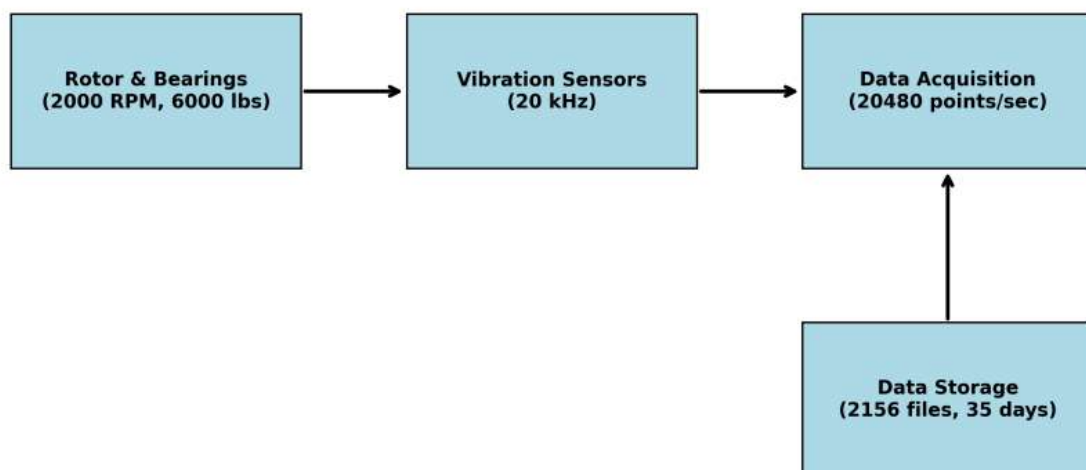


Fig. 1. Schematic diagram of the experimental test rig and data acquisition system

2.2 Signal Preprocessing and Degradation Feature Engineering

The 2156 IMS files are ordered by acquisition time. No temporal resampling or artificial up sampling is performed. For each file, Channel 5 is divided into twenty non-overlapping windows of 1024 samples. Four time-domain indicators are calculated in every window and then averaged across the twenty windows to obtain one four-dimensional feature observation for that acquisition

time. This preserves the chronological degradation trend while keeping the sequence model compact. Four features are computed for each 1024-sample window:

- Root Mean Square (RMS) measures the overall vibration-energy level in the analyzed window. Progressive wear, looseness, or increasing dynamic interaction can raise the RMS level, although RMS is also sensitive to operating-condition changes.
- Peak-to-Peak amplitude is the difference between the maximum and minimum signal values within the window. It is sensitive to extreme excursions and therefore provides complementary information on intermittent impacts and transient vibration events.
- Kurtosis quantifies the impulsiveness and non-Gaussianity of the amplitude distribution. High kurtosis is commonly associated with isolated high-amplitude events and can therefore respond to localized contact or surface damage.
- Crest Factor (CF) is the ratio between the absolute peak amplitude and RMS. It indicates how large the strongest peak is relative to the overall vibration-energy level and therefore complements the scale-sensitive RMS indicator.
- The four indicators jointly represent vibration energy (RMS), amplitude range (Peak-to-Peak), impulsiveness (Kurtosis), and peak-to-energy ratio (Crest Factor). Their chronological evolution is shown in Fig. 2. A data-driven degradation-onset check is superimposed for descriptive analysis; it is not used to construct the RUL target.

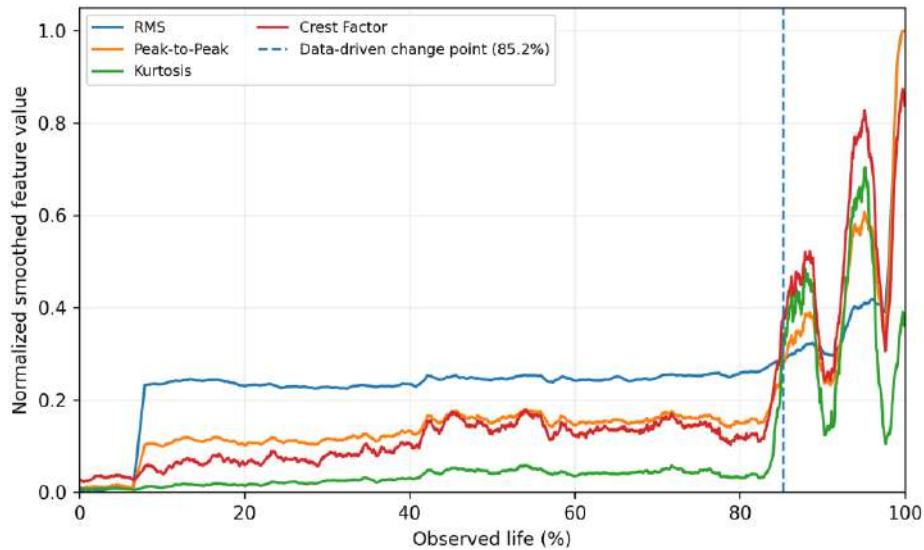


Fig. 2. Normalized degradation-feature trends and data-driven change point for IMS Test 1, Bearing 3, Channel 5

2.3 RUL Target and Sequence Construction

To remove dependence on an arbitrarily selected first-prediction-time boundary, the revised RUL target is defined directly from elapsed life rather than by imposing a healthy plateau followed by a manually selected decay stage. Let t denote the chronological observation index and T the final observation index. The normalized RUL target is

$$y_t = \frac{T - t}{T}, 0 \leq t \leq T \quad (1)$$

This definition gives $y_t=1$ at the beginning of the observed run and $y_t=0$ at its end. The former fixed 70% FPT label rule is therefore no longer required. A separate data-driven change-point analysis is used only to characterize when the vibration health trajectory changes most strongly; it does not alter the RUL target.

At each observation t , the four-feature vector is $x_t = [RMS_t, P2P_t, Kurtosis_t, CF_t]^T$. To include recent degradation history, ten consecutive feature vectors are concatenated as $X_t = [X_{t-9}, \dots, X_t] \in \mathbb{R}^{10 \times 4}$.

Feature-wise Min-Max scaling is fitted using only the first 70% training portion and then applied unchanged to the held-out 30% test portion. This prevents future test information from entering the preprocessing stage.

2.4 Sequence-Based RUL Prediction Model

The proposed sequence predictor is a lightweight gated recurrent unit (GRU) regression network. Each normalized 10×4 input sequence is passed to a single GRU layer with 32 hidden units. The final hidden state is mapped through a fully connected layer with 16 ReLU units and a single sigmoid output neuron that produces the normalized RUL estimate $\hat{y}_t \in [0,1]$.

$$h_t = GRU(X_t; \theta_g) \quad (2)$$

$$z_t = ReLU(W_1 h_t + b_1) \quad (3)$$

$$\hat{y}_t = \sigma(W_2 z_t + b_2) \quad (4)$$

The model is trained by minimizing mean squared error, $\mathcal{L}_{RUL} = \frac{1}{N} \sum_{i=1}^N (y_i - \hat{y}_i)^2$. Adam optimization is used with an initial learning rate of 1×10^{-3} , a batch size of 128, and 150 training epochs. To quantify initialization sensitivity, all reported predictive results are repeated over five random initializations. For comparison, the LSTM baseline contains one 32-unit LSTM layer followed by a sigmoid regression output. The 1D-CNN baseline uses two temporal convolution layers with 16 and 32 channels (kernel size 3), ReLU activations, adaptive average pooling, and a sigmoid regression output. The baselines use the same four-feature sequences, optimizer, training/test split, and RUL target.

2.5 Feature Attribution and Robustness Validation

The primary local explanation uses input-gradient sensitivity. For feature j , sensitivity over a sequence of length L is defined as $A_j = \frac{1}{L} \sum_{\tau=1}^L \left| \frac{\partial \hat{y}_\tau}{\partial x_{\tau,j}} \right|$. The four sensitivities are normalized as $\omega_j = \frac{A_j}{\sum_{k=1}^4 A_k}$, so that $\sum_{j=1}^4 \omega_j = 1$. These weights quantify model-level predictive sensitivity and are not interpreted as causal contributions to physical bearing damage. Gradient sensitivity is cross-checked using three complementary methods: Integrated Gradients with a zero baseline in the scaled feature space, SHAP expected-gradient attribution using background sequences from the training portion, and permutation sensitivity obtained by permuting one feature trajectory across sequences while retaining the others. A deletion check additionally replaces each feature by its training-set mean and records the resulting change in test MAE. Attribution results are aggregated over early life ($RUL > 0.66$), middle life ($0.33 < RUL \leq 0.66$), and late life ($RUL \leq 0.33$), and over five random model initializations. The complete prediction and feature-attribution framework is summarized in Fig. 3.

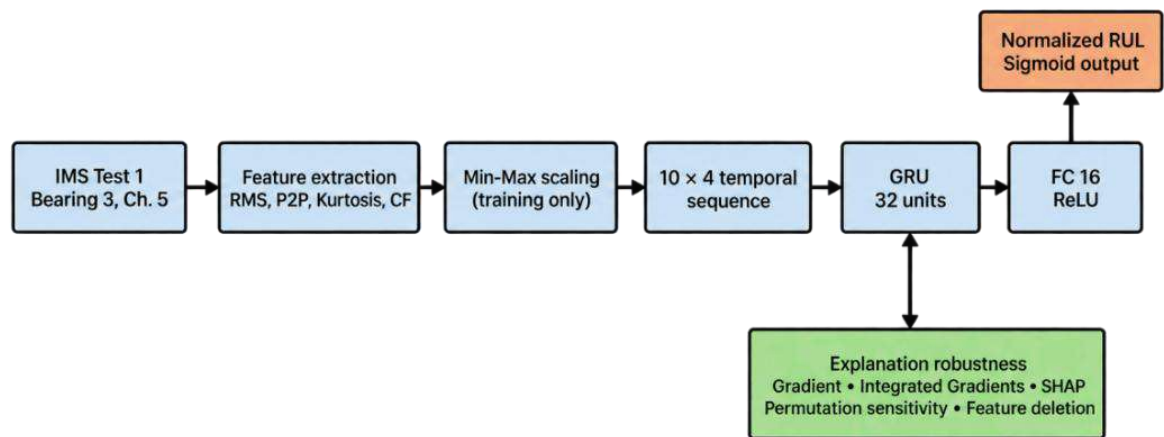


Fig. 3. Architecture of the proposed explainable remaining useful life prediction framework

2.6 Experimental Protocol and Evaluation Metrics

To avoid future-information leakage, the 2156 chronological observations are split without random shuffling: the first 70% form the training set and the final 30% form the held-out test set. Min-Max scaling is fitted only on the training portion. The GRU, 1D-CNN, and LSTM models are trained under the same split and repeated across five random initializations. The test portion is not used for parameter fitting.

Prediction performance is evaluated using Mean Absolute Error (MAE) and Root Mean Squared Error (RMSE) on normalized RUL. MAE reports the average absolute prediction error, whereas RMSE penalizes larger errors more strongly. Results are reported as mean $\pm$ standard deviation across five initializations.

$$MAE = \frac{1}{N} \sum_{i=1}^N |y_i - \hat{y}_i| \quad (5)$$

$$RMSE = \sqrt{\frac{1}{N} \sum_{i=1}^N (y_i - \hat{y}_i)^2} \quad (6)$$

For the degradation-onset robustness check, a 31-observation rolling mean is applied to the four feature trajectories, followed by robust scaling and one-dimensional principal-component fusion. A two-segment least-squares change point is then selected by minimizing within-segment squared error over candidate locations between 40% and 90% of observed life. This change point is reported descriptively and is not used to create the RUL labels.

3. Results and Discussion

3.1 Model Predictive Performance Baseline Comparison

All predictive results use the held-out final 30% of IMS Test 1. Across five random initializations, the proposed GRU model obtains an MAE of 0.329 ± 0.084 and an RMSE of 0.351 ± 0.098 . The 1D-CNN baseline obtains 0.348 ± 0.128 MAE and 0.393 ± 0.149 RMSE, while the LSTM obtains 0.363 ± 0.011 MAE and 0.386 ± 0.013 RMSE. Thus, the GRU gives the lowest mean error under the common protocol, although the standard deviations also show that initialization affects the GRU and 1D-CNN more strongly than the LSTM.

Table 1. RUL prediction performance over five random initializations

Model	Mean Absolute Error (MAE)	Root Mean Squared Error (RMSE)
1D CNN	0.348 ± 0.128	0.393 ± 0.149
LSTM	0.363 ± 0.011	0.386 ± 0.013
Proposed GRU explainable model	0.329 ± 0.084	0.351 ± 0.098

The comparison is intended to establish that the compact sequence model is competitive on the controlled benchmark rather than to claim universal architectural superiority. The difference between the GRU and the two baselines should be interpreted together with the single-run experimental scope and the observed variation across random initializations.

Late-life prediction remains the most demanding region because the test set contains RUL values below those observed during much of training. RMSE is therefore reported together with MAE so that occasional large late-life errors are not hidden by average absolute error. No conclusion is drawn from these results regarding performance under variable speed, load, temperature, or vehicle duty cycles.

3.2 Degradation-Onset and Feature-Deletion Checks

The former manuscript imposed a 70% first-prediction-time boundary on the RUL target. In the revised protocol this manual boundary is removed: normalized RUL decreases continuously with elapsed life, while degradation onset is examined independently. The data-driven two-segment change-point analysis identifies the strongest transition at approximately 85.2% of the observed life (file timestamp 2003.11.22.17.16.56). The difference between this data-driven location and the former 70% assumption confirms that a fixed FPT should not be treated as a known physical degradation onset.

Feature deletion provides an additional check on whether the attribution ranking corresponds to predictive dependence. Averaged over five GRU initializations, replacing Kurtosis by its training-set mean increases test MAE by 0.0249, compared with increases of 0.0045 for Peak-to-Peak and 0.0015 for RMS; replacing Crest Factor changes MAE by -0.0019 on average. Kurtosis therefore produces the largest degradation in predictive performance when removed, consistent with its top ranking in the attribution analyses.

The deletion result should not be interpreted causally: a feature can be important to the trained predictor because it is correlated with the degradation trajectory. Nevertheless, agreement between feature deletion and multiple attribution methods provides a more robust basis for interpreting model dependence than gradient magnitude alone.

Because the RUL label is now independent of a manually specified FPT, model accuracy is no longer conditioned on choosing 60%, 70%, or 80% as a label breakpoint. The change-point result is retained only as an independent description of when the measured vibration indicators exhibit their strongest distributional transition.

3.3 Explainable Attribution Analysis

Feature attribution is evaluated using Gradient sensitivity, Integrated Gradients, SHAP, and permutation sensitivity. Across the early-, middle-, and late-life stages, all four methods rank Kurtosis as the most influential feature on average. Inter-method Spearman rank agreement is 0.77 in early life and 0.90 in both middle and late life. Across the five random initializations, mean pairwise rank correlations range from 0.64 to 1.00 depending on the method and life stage, indicating moderate-to-high explanation stability rather than a single-seed result.

In late life ($RUL \leq 0.33$), the mean normalized Kurtosis importance is 0.762 for Gradient sensitivity, 0.723 for Integrated Gradients, 0.731 for SHAP, and 0.536 for permutation sensitivity. Peak-to-Peak is generally the second most influential feature, especially under permutation sensitivity. Figure 4 summarizes the late-life attributions, and Table 2 reports the corresponding values.

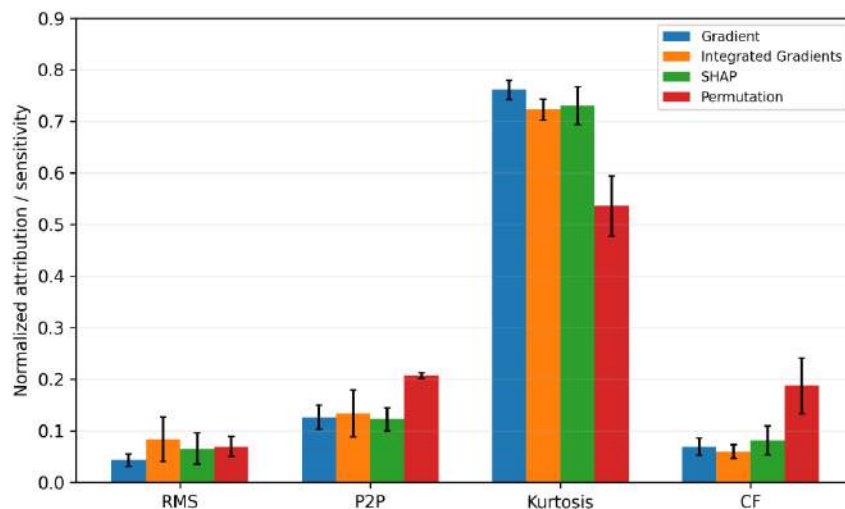


Fig. 4. Late-life feature attribution/sensitivity comparison across four explanation methods; error bars show variation across five GRU initializations

The attribution pattern is physically plausible but should be interpreted as predictive association. Kurtosis measures impulsiveness and departure from a Gaussian amplitude distribution; its dominant late-life ranking is consistent with increasingly pronounced impact-like events as localized surface damage develops. Peak-to-Peak captures extreme amplitude excursions and therefore complements Kurtosis when intermittent impacts become stronger. RMS represents overall vibration energy and is useful for monitoring broad energy growth, while Crest Factor expresses the peak amplitude relative to RMS and can distinguish isolated peaks from a general rise in vibration level.

The result differs from the original single-gradient interpretation in which RMS appeared dominant. The revised analysis averages explanations over multiple observations and random initializations and cross-checks them with independent attribution mechanisms. The consistent ranking of Kurtosis, together with the feature-deletion result, indicates that the trained model relies strongly on impulsive information in the selected IMS channel. This does not establish that Kurtosis is a universal bearing-health indicator or a causal damage mechanism.

For predictive-maintenance use, these explanations provide context for a low-RUL estimate: an impulse-dominated warning suggests a different signal pattern from one driven primarily by broadband energy growth. Such context can support engineering review, but the attribution does not identify a specific fault location by itself. Frequency-domain or envelope features would be required for more direct differentiation of inner-race, outer-race, and rolling-element fault mechanisms.

Table 2. Mean late-life normalized feature attribution/sensitivity over five GRU initializations

Method	RMS	Peak-to-Peak	Kurtosis	Crest Factor
Gradient	0.043	0.126	0.762	0.069
Integrated Gradients	0.083	0.134	0.723	0.060
SHAP	0.065	0.123	0.731	0.081
Permutation	0.069	0.207	0.536	0.187

4. Discussion and Engineering Generalization Limitations

The revised results support two conclusions within the controlled IMS benchmark. First, compact time-domain features retain sufficient information for a sequence model to follow the observed life trajectory, although initialization variability is non-negligible. Second, explanation robustness improves when gradient sensitivity is considered together with Integrated Gradients, SHAP, permutation sensitivity, and feature deletion rather than being treated as a stand-alone causal measure.

Generalization beyond this experiment has not been established. IMS Test 1 operates at a constant 2000 RPM and a constant 6000 lb radial load, whereas automotive bearings experience speed and torque transients, start-stop operation, temperature variation, lubrication changes, structural transmission paths, and road-induced disturbances. Accordingly, the title and claims of the revised manuscript refer to rolling-bearing benchmark validation rather than validated vehicle-bearing prognosis. Additional IMS runs and datasets with variable speed/load conditions are required before deployment-oriented conclusions can be made.

The explanation is also limited by the selected feature space. RMS, Peak-to-Peak, Kurtosis, and Crest Factor distinguish broad energy and impulsive behaviour but do not directly identify the physical fault location. Attribution values therefore indicate which measured statistics the predictor relies on, not why a particular physical defect formed. More complete diagnostic interpretation would require frequency-domain, envelope, or order-related information together with validation across naturally occurring faults and multiple operating conditions.

5. Conclusions

This study develops a compact explainable RUL workflow for the open-source NASA IMS rolling-bearing benchmark. Vibration records from Bearing 3, Channel 5 are represented using RMS, Peak-to-Peak, Kurtosis, and Crest Factor, arranged into ten-step temporal sequences, and mapped to normalized RUL using a lightweight GRU regressor. The arbitrary 70% FPT label rule used in the previous manuscript has been removed; normalized RUL is defined directly from elapsed life, while a separate data-driven change-point analysis characterizes degradation onset without controlling the target labels.

Under a chronological 70%/30% train-test split and five random initializations, the GRU achieves an MAE of 0.329 ± 0.084 and an RMSE of 0.351 ± 0.098 . The corresponding 1D-CNN values are 0.348 ± 0.128 and 0.393 ± 0.149 , and the LSTM values are 0.363 ± 0.011 and 0.386 ± 0.013 . These results indicate competitive predictive performance on the selected run but do not establish architectural superiority beyond this benchmark.

Explainability is evaluated using four complementary approaches rather than gradient sensitivity alone. Gradient, Integrated Gradients, SHAP, and permutation sensitivity all rank Kurtosis as the most influential feature on average in late life, and the deletion test produces the largest average MAE increase when Kurtosis is removed. The attribution pattern is physically consistent with stronger impulsive activity during advanced degradation, but it is interpreted as model-level predictive dependence rather than physical causality.

The present evidence is limited to a single fixed-speed, fixed-load IMS run. The framework should therefore be regarded as a transparent benchmark-oriented RUL workflow, not a field-validated vehicle prognostic system. Validation on additional run-to-failure bearings and under varying speed, load, temperature, lubrication, and vehicle duty cycles is required before deployment. Future work may also add frequency-domain and envelope indicators to connect RUL explanations more directly to specific bearing fault mechanisms.

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Multi-mode material response of basalt- and glass-fiber-reinforced dense-graded asphalt

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Abstract

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Short mineral fibers can modify load transfer across several asphalt damage modes, but direct comparisons within one controlled mixture matrix remain limited. This study compared 6 mm basalt fiber (BF) and glass fiber (GF), incorporated by the dry process at 0.2%, 0.3%, and 0.4% of the total mixture mass, in dense-graded hot-mix asphalt prepared with a 60/70 penetration-grade binder at a fixed asphalt content of 5.0% by total mixture mass. Marshall response along with dry and moisture-conditioned indirect tensile strength was first used to examine dose dependence and select a common dosage for further testing. The 0.3% mixtures were then evaluated by wheel tracking at 60 °C, constant-strain four-point bending fatigue, and an adapted Cantabro abrasion procedure before and after thermal aging. Both fibers produced a non-monotonic load-capacity response, and 0.3% was selected for the extended test program based on the combined Marshall and tensile response within the evaluated dosage range. At this dosage, BF and GF increased Marshall stability by 27.2% and 17.5%, increased dry tensile strength by 15.5% and 11.9%, and reduced rut depth after 10,000 wheel passes by 30.0% and 22.9%, respectively. Fatigue life increased by 49.1% and 46.3%. After seven days of aging at 60 °C, Cantabro mass loss decreased from 27.0% for the control to 15.1% for BF and 18.3% for GF. Across the evaluated laboratory response modes, BF showed larger numerical improvements in several indicators, particularly rut depth and aged particle loss.

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1. Introduction

Hot mix asphalt (HMA) is a heterogeneous, temperature- and loading-rate-dependent composite in which a mineral skeleton is embedded in a viscoelastic binder in a rich mastic and interrupted by air voids. Its macroscopic response emerges from aggregate contact forces, binder rheology, mastic cohesion, aggregate-binder adhesion, and void structure. Temperature, loading duration, water, and oxidative aging all affect how stress and strain are partitioned among these phases. Rutting, fatigue cracking, moisture-related damage, and raveling are therefore coupled manifestations of deformation and damage within the same multiphase material [1,2].

No single strength or specification index can describe resistance to all forms of asphalt distress. At high temperature, the softened mastic allows greater aggregate movement and permanent deformation. Repeated tensile strain can initiate and develop microcracks as mixture stiffness decreases. Moisture can weaken mastic cohesion and aggregate-binder adhesion, whereas aging can increase stiffness and reduce the ability of the mixture to relax stress. For this reason, the effect of a modifier should be examined under different loading and conditioning conditions rather than through one test alone.

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Discrete fibers can change the internal response of an asphalt mixture rather than acting only through the binder. When they are well dispersed, fibers may redistribute local stress, restrain aggregate movement, and stabilize binder-rich regions. Their effect depends on fiber geometry, dosage, dispersion, thermal stability, surface interaction, and mixture volumetrics [3]–[5]. Recent studies also show that the fiber–asphalt interface can influence mixture behavior. Pull-out strength and adhesion work have been used to study basalt-fiber/asphalt-mastic interaction [6], while comparative work on basalt, glass, and polyester fibers has related interfacial characteristics to cracking resistance [7]. These studies help explain the behavior of fiber-modified asphalt. However, surface energy, pull-out strength, and microstructure were not measured in the present study. The mechanisms discussed here are therefore used only to interpret the laboratory results.

Fiber content is therefore expected to have an optimum range rather than a continuously increasing effect. At low content, the number of fibers may be insufficient to influence many potential damage zones. At higher content, fiber–fiber contact, binder absorption or immobilization, and agglomeration may reduce coating uniformity or create nonuniform regions [5,8-11]. The 0.2%, 0.3%, and 0.4% dosages were selected from the low dosage range commonly investigated for mineral-fiber asphalt mixtures. Recent work has also examined low fiber contents and reported optimum behavior around 0.2–0.3% for its specific material system [7]. The optimum fiber content depends on the mixture and should not be considered a fixed property of the fiber.

The mixture was therefore examined under several conditions: high temperature, moisture conditioning, repeated strain, and thermal aging. Wheel tracking was used to measure permanent deformation at high temperature. Dry and conditioned tensile tests were used to compare tensile strength before and after moisture conditioning. Four-point bending was used to follow stiffness loss during repeated loading, while the Cantabro procedure was used to assess particle loss before and after thermal aging. Previous studies have also shown that the response of fiber-modified asphalt varies with temperature, moisture exposure, mixture layer, and composition [12-15].

Basalt fiber (BF) and glass fiber (GF) can be compared within the same asphalt mixture because both are mineral fibers that can tolerate normal HMA production temperatures. However, their physical and surface properties are not identical. Previous studies on BF have reported changes in Marshall properties, tensile response, moisture resistance, rutting, fracture, and fatigue at appropriate dosages [8,9,15-20]. Studies on GF have also reported improvements in cracking, rutting, and moisture-related performance [21-24]. In many published studies, however, fibers were used together with other mixture changes, such as reclaimed asphalt, diatomite, or warm-mix additives. This makes the effect of fiber type alone harder to distinguish.

Direct BF-GF comparisons made in one conventional mixture are still limited. Comparing results from separate studies is difficult because binder grade, aggregate source, gradation, volumetric properties, fiber dosage, and compaction may all be different. A fiber may also perform better in one test but show little advantage in another. For that reason, this study compares the response of the two fibers across several tests instead of judging them from one result.

This study compares 6 mm BF and GF at 0.2%, 0.3%, and 0.4% of total mixture mass in the same dense-graded HMA. The binder, aggregate, gradation, mixing procedure, and conditioning methods were kept the same for both fiber types. The mixture followed the relevant Iraqi specification for materials and gradation [25]. The study first examines the effect of dosage on Marshall and tensile response. It then compares the selected BF and GF mixtures in terms of rutting, tensile response, fatigue behavior, and particle loss.

2. Materials and Methods

2.1 Asphalt Binder

The asphalt binder was obtained from the Al-Daurah refinery and had a penetration grade of 60/70. Table 1 lists its measured physical properties.

Table 1. Physical properties of the 60/70 penetration-grade asphalt binder

Property (unit)	Standard	Result	SCRB limit
Penetration, 25 °C (0.1 mm)	ASTM D5	65	60–70
Ductility, 25 °C (cm)	ASTM D113	128	>100
Flash point (°C)	ASTM D92	284	>232
Fire point (°C)	ASTM D92	300	–
Viscosity, 135 °C (Pa·s)	ASTM D4402	0.375	–
Viscosity, 165 °C (Pa·s)	ASTM D4402	0.125	–
Softening point (°C)	ASTM D36	49	–

2.2 Aggregate and Mineral Filler

Crushed coarse aggregate and sand were combined with limestone mineral filler. The aggregate properties are presented in Table 2. The selected gradation followed SCRB R/9 Type IIIA for dense-graded HMA [25] and remained within the specified envelope (Figure 1). Based on the sieve series, the maximum aggregate size was 19 mm and the nominal maximum aggregate size was 12.5 mm. Los Angeles abrasion was determined in accordance with ASTM C131/C131M [26].

Table 2. Physical properties of the coarse and fine aggregates

Fraction / property	Standard	Result	Limit
Coarse: Los Angeles abrasion (%)	ASTM C131	21.2	30 max.
Coarse: Na ₂ SO ₄ soundness loss (%)	ASTM C88	3.3	12 max.
Coarse: angularity (%)	ASTM D5821	98	95 min.
Coarse: flakiness index (%)	ASTM D4791	0.91	10 max.
Coarse: elongated index (%)	ASTM D4791	2.50	5 max.
Fine: bulk specific gravity	ASTM C128	2.618	–
Fine: apparent specific gravity	ASTM C128	2.625	–
Fine: water absorption (%)	ASTM C128	0.19	–
Fine: sand equivalent (%)	ASTM D2419	89.7	45 min.
Fine: deleterious material (%)	ASTM C142	1.11	3 max.

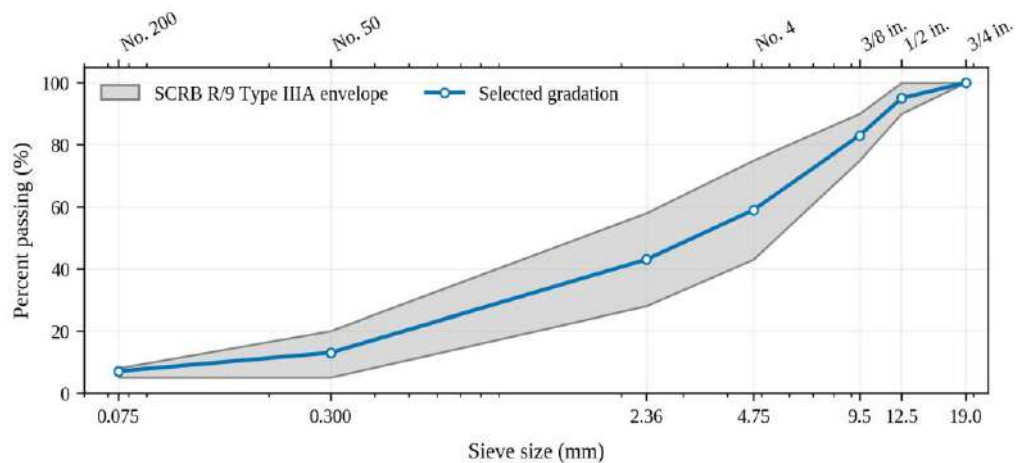


Fig. 1. Selected aggregate gradation within the SCRB R/9 Type IIIA envelope

2.3 Fibers

Basalt and glass fibers with a nominal length of 6 mm were used as received after oven drying; no additional surface treatment was applied. Their supplier-reported properties are listed in Table 3, and their physical appearance is shown in Figure 2. These values are treated as nominal constituent properties rather than measurements made in this study.

Table 3. Nominal physical and mechanical properties of the fibers

Property	Basalt fiber	Glass fiber
Length (mm)	6	6
Diameter (μm)	16	12
Density (g/cm^3)	2.77	2.60
Moisture content (%)	0.32	0.50
Elongation (%)	1.4	3.0
Tensile strength (MPa)	2,200–2,500	3,100
Elastic modulus (GPa)	87	81

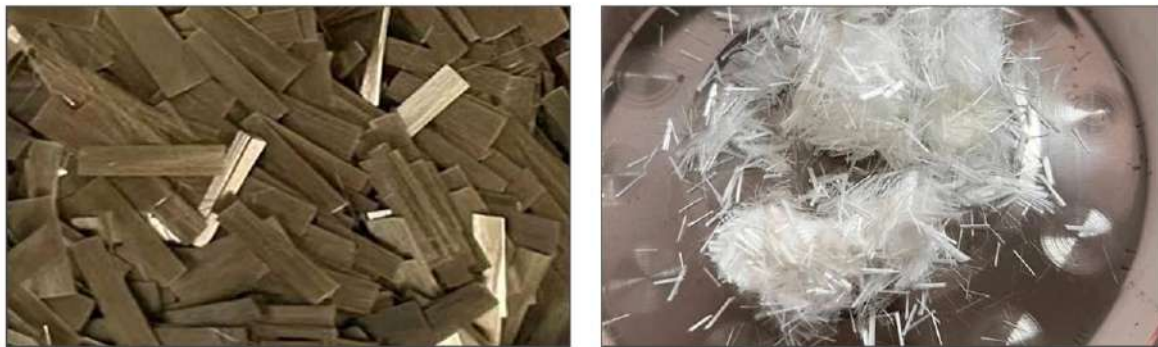


Fig. 2. Basalt fiber (left) and glass fiber (right) used in the study

2.4 Mixture Design and Specimen Preparation

The control and fiber-modified mixtures were prepared at the same asphalt content of 5.0% by total mixture mass so that fiber type and dosage could be compared under a common mixture condition. Marshall specimens were compacted with 75 blows per face in accordance with ASTM D6926 [27]. Both fibers were evaluated at 0.2%, 0.3%, and 0.4% by total mixture mass.

Before mixing, the fibers were dried at 105 °C for 24 h. A dry mixing procedure was used for both fiber types, consistent with the general approach reported for fiber-reinforced asphalt mixtures [15], [28]. For the fiber mixtures, the fibers were added first to the heated aggregate and spread through the aggregate to reduce clumping. The asphalt binder was then introduced and mixing continued. The entire mixing process took approximately 5 minutes. Mixing and compaction were carried out at approximately 153 °C and 146 °C, respectively, according to the binder viscosity–temperature relationship. Fiber distribution was checked during mixing by visual inspection, and no visible fiber balls or clumps were found. The control mixture was prepared using the same temperature and compaction sequence but without fiber.

Table 4 gives the measured volumetric properties for the control and fiber mixtures. As fiber content increased, Gmb showed a small decrease, while Va and VMA increased and VFA decreased. At the same dosage, these changes were slightly greater for BF than for GF.

Table 4. Volumetric properties of the control and fiber-modified mixtures

Mixture	Gmb (g/cm ³)	Va (%)	VMA (%)	VFA (%)
Control	2.350	4.2	15.0	72.0
0.2% BF	2.345	4.4	15.3	71.2
0.3% BF	2.340	4.6	15.6	70.5
0.4% BF	2.333	4.9	16.0	69.4
0.2% GF	2.347	4.3	15.2	71.7
0.3% GF	2.343	4.5	15.4	70.8
0.4% GF	2.338	4.7	15.7	70.1

2.5 Marshall Stability and Flow

Marshall stability and flow were measured according to ASTM D6927 [29] at a loading rate of 50.8 mm/min. Marshall quotient (MQ) was calculated using Eq. (1) and used as an empirical load-to-deformation index.

$$MQ = \text{Marshall stability (kN)} / \text{Marshall flow (mm)} \quad (1)$$

Where: MQ is the Marshall quotient (kN/mm), Marshall stability is the maximum measured load (kN), and Marshall flow is the corresponding deformation (mm).

2.6 Indirect Tensile Strength and Moisture Conditioning

Indirect tensile strength (ITS) was determined at 25 °C using ASTM D6931 [30] at a diametral loading rate of 50.8 mm/min. Moisture susceptibility was evaluated using AASHTO T283 [31]. Six specimens were prepared per mixture at $7 \pm 1\%$ air voids: three were tested unconditioned and three were vacuum-saturated, frozen at -18 ± 3 °C for 16 h, conditioned in water at 60 °C for 24 h, and equilibrated at 25 °C for 1 h. ITS was calculated using Eq. (2). Tensile strength ratio (TSR) was calculated using Eq. (3).

$$ITS = 2000P / (\pi Dt) \quad (2)$$

Where: P is peak load (N), D is specimen diameter (mm), and t is specimen thickness (mm).

$$TSR (\%) = (ITS_{\text{conditioned}} / ITS_{\text{unconditioned}}) \times 100 \quad (3)$$

Where: $ITS_{\text{conditioned}}$ is the indirect tensile strength of the conditioned specimens (kPa), and $ITS_{\text{unconditioned}}$ is the indirect tensile strength of the unconditioned specimens (kPa).

2.7 Wheel Tracking

The control, 0.3% BF, and 0.3% GF mixtures were tested according to BS EN 12697-22 [32]. Three roller-compacted slabs ($400 \times 300 \times 50$ mm) were prepared per mixture ($n = 3$), conditioned at 60 °C for 2 h, and tested in air at 60 °C under an approximately 700-N wheel load for 10,000 passes or until 20-mm rut depth. Rut depth was recorded with loading, and the wheel-tracking response was reported as a function of the number of passes.

2.8 Four-Point Bending Fatigue

Repeated four-point bending followed AASHTO T321 [33]. Three roller-compacted beams were tested per mixture ($n = 3$). The beams had nominal dimensions of $380 \times 63 \times 50$ mm (reported actual dimensions approximately $378 \times 62 \times 50$ mm). Testing was performed in constant-strain mode at 20 °C, 5 Hz, and 400 $\mu\epsilon$. Initial flexural stiffness was defined at cycle 100; fatigue life was the number of cycles required for stiffness to decline to 50% of its initial value.

2.9 Adapted Cantabro Abrasion

An adapted procedure based on ASTM D7064/D7064M [34] was used for the control and the two 0.3% mixtures. Although ASTM D7064/D7064M was developed for open-graded friction-course mix design, Doyle and Howard [35] evaluated the Cantabro durability test across eight dense-graded asphalt mixtures and identified mass loss as a repeatable and potentially useful comparative durability measure. Accordingly, the present application was deliberately limited to a controlled within-study comparison of cohesion and particle-loss resistance; it was not used as an OGFC acceptance criterion or as a direct predictor of field raveling. Cylindrical Marshall specimens (101.6-mm nominal diameter and 63.5-mm nominal height; reported actual height approximately 62.8 mm) were tested at 25 °C in the Los Angeles abrasion machine without steel balls for 300 revolutions at 30 rpm. Aged specimens were conditioned at 60 °C for seven days (168 h). Mass loss was calculated from the initial and final specimen masses using Eq. (4).

$$\text{Cantabro mass loss (\%)} = [(A_i - A_e) / A_i] \times 100 \quad (4)$$

Where: A_i is the initial specimen mass before testing (g), and A_e is the final specimen mass after testing (g).

2.10 Data and Statistical Analysis

Changes relative to the control mixture were calculated using Eqs. (5) and (6). Equation (5) was used when an increase represented improvement, whereas Eq. (6) was used when a decrease represented improvement. TSR differences were expressed in percentage points.

$$\text{Gain (\%)} = 100(X_{\text{fiber}} - X_{\text{control}}) / X_{\text{control}} \quad (5)$$

$$\text{Reduction (\%)} = 100(X_{\text{control}} - X_{\text{fiber}}) / X_{\text{control}} \quad (6)$$

Where: X_{fiber} is the measured response of the fiber-modified mixture, and X_{control} is the corresponding response of the control mixture. Both variables are expressed in the same units as the response being evaluated.

Results from three replicate measurements per mixture are presented as mean $\pm$ standard deviation (SD). One-way analysis of variance (ANOVA) was used to evaluate overall differences among mixture groups, followed by Tukey's honestly significant difference (HSD) test when ANOVA was significant. Statistical significance was assessed at $\alpha = 0.05$. TSR was interpreted descriptively because it was calculated from conditioned and unconditioned ITS results.

3. Results and Discussion

3.1 Dosage-Dependent Load-Flow Response

Marshall stability displayed a non-monotonic dependence on fiber content (Figure 3). Stability increased from 10.3 kN for the control to 13.1 kN for 0.3% BF and 12.1 kN for 0.3% GF, gains of 27.2% and 17.5%. At 0.4%, the corresponding mean values declined to 12.7 and 11.9 kN. Within the tested dosage range, 0.3% therefore gave the highest mean Marshall stability for both fiber types.

Flow decreased as fiber content increased. At 0.3%, mean flow decreased from 3.4 mm for the control to 2.9 mm for BF and 3.0 mm for GF, while MQ increased from 3.03 kN/mm to 4.52 and 4.03 kN/mm. MQ increased further at 0.4% because flow continued to decrease even though mean stability had started to decline. Similar non-monotonic responses have been discussed in relation to fiber restraint, binder availability, fiber-fiber interaction, and mixture homogeneity [5], [8]–[10], [16]. These mechanisms were not measured directly in this study. Based on the measured Marshall response, 0.3% provided the best balance between stability and flow within the tested range. The overall mixture-group effects were statistically significant for Marshall stability, flow, and Marshall quotient (one-way ANOVA, $p < 0.001$ for each response).

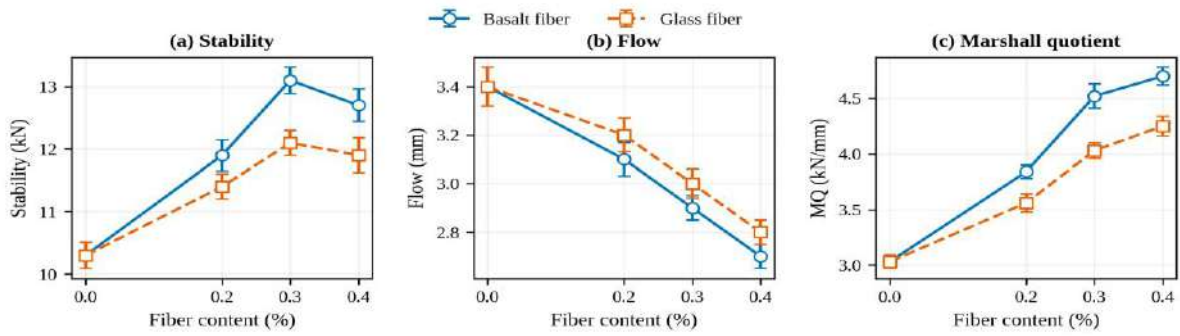


Fig. 3. Effects of basalt- and glass-fiber content on (a) Marshall stability, (b) flow, and (c) Marshall quotient. Values are means $\pm$ SD ($n = 3$)

3.2 Tensile Response and Moisture Conditioning

Dry and conditioned ITS followed the same non-monotonic numerical trend (Figure 4). At 0.3%, dry ITS increased from 850 kPa to 982 kPa for BF and 951 kPa for GF, whereas conditioned ITS increased from 678 kPa to 811 and 784 kPa. The corresponding gains were 15.5% and 11.9% in the dry state and 19.6% and 15.6% after conditioning. Within the tested dosage range, the highest mean dry and conditioned ITS values for both fiber types occurred at 0.3%; increasing the fiber content to 0.4% did not produce a further increase in these mean values.

TSR changed only slightly, from 80.0% for the control to 82.5% for BF and 82.5% for GF at 0.3%. The BF–GF difference was 0.0 percentage points. Thus, the two fiber mixtures had nearly the same proportional strength retention after moisture conditioning, although both had higher absolute dry and conditioned ITS values than the control. The overall mixture-group effects were statistically significant for both dry ITS and conditioned ITS (one-way ANOVA, $p < 0.001$ for each response). TSR was retained as a descriptive ratio, as specified in Section 2.10.

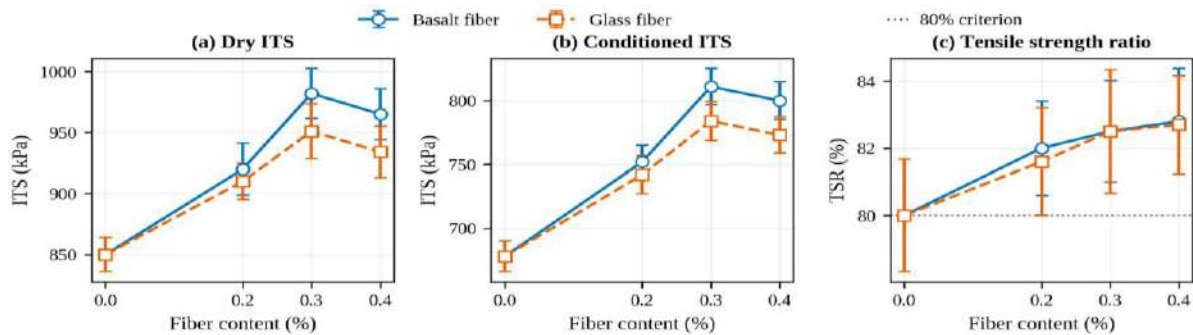


Fig. 4. Effects of fiber type and content on (a) dry ITS, (b) conditioned ITS, and (c) tensile strength ratio. Values are means $\pm$ SD ($n = 3$)

3.3 High-Temperature Permanent Deformation

All mixtures accumulated rut depth at 60 °C, but both fiber-modified curves remained below the control throughout the recorded loading range (Figure 5). After 10,000 passes, rut depth was 7.0 mm for the control, 4.9 mm for 0.3% BF, and 5.4 mm for 0.3% GF, corresponding to reductions of 30.0% and 22.9%, respectively (Table 5). These results show lower accumulated rut depth for the fiber-modified mixtures under the stated laboratory wheel-tracking conditions.

To characterize late-stage rut accumulation, the wheel-tracking slope (WTS) was calculated from the rut depths at 5,000 and 10,000 wheel passes shown in Figure 5 using Eq. (7). The resulting WTS values were 0.60, 0.42, and 0.46 mm/1,000 passes for the control, 0.3% BF, and 0.3% GF mixtures, respectively. These values are laboratory comparative indicators and are not field rutting-rate predictions.

$$WTS = (RD_{10,000} - RD_{5,000}) / 5 \quad (7)$$

Where: WTS is the wheel-tracking slope (mm/1,000 passes), and RD5,000 and RD10,000 are the rut depths (mm) at 5,000 and 10,000 wheel passes, respectively. The BF mixture also had a higher mean MQ and initial flexural stiffness than the GF mixture. According to the supplier-reported fiber properties listed in Table 3, BF had a higher nominal modulus and a larger filament diameter than GF. These observations may help explain the lower mean rut depth of the BF mixture, but the present tests cannot separate the effects of fiber geometry, dispersion, binder absorption, and interfacial interaction. The BF–GF difference is therefore reported as a laboratory response rather than evidence of a specific microstructural mechanism [8], [15], [6], [7].

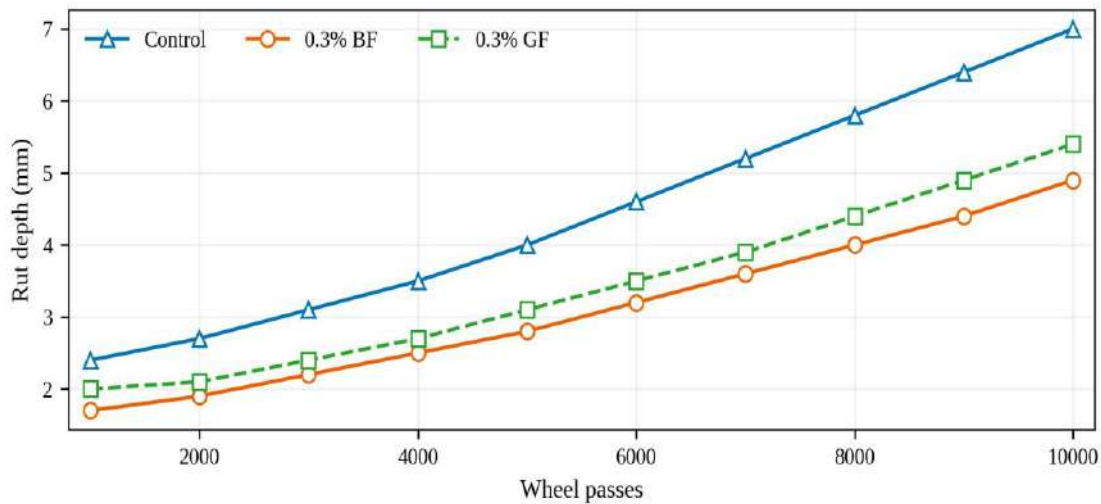


Fig. 5. Wheel-tracking response at 60 °C (n = 3 slabs per mixture)

Table 5. Wheel-tracking results and late-stage wheel-tracking slope at 60 °C

Mixture	Rut depth at 10,000 passes (mm)	Reduction vs. control (%)	WTS (mm/1,000 passes)
Control	7.0	0.0	0.60
0.3% BF	4.9	30.0	0.42
0.3% GF	5.4	22.9	0.46

3.4 Stiffness Evolution and Fatigue Damage

Initial flexural stiffness increased from 4,850 MPa for the control to 5,661 MPa for 0.3% BF and 5,431 MPa for 0.3% GF; fatigue life increased from 3,194 cycles to 4,762 and 4,674 cycles, respectively (Figure 6). These changes equal +16.7% and +12.0% in stiffness and +49.1% and +46.3% in fatigue life.

Under constant-strain loading, higher stiffness can increase the stress developed in a beam. In this study, however, both fiber mixtures showed higher mean initial stiffness and longer mean fatigue life than the control. This describes the measured response but does not identify the mechanism responsible for it. Crack bridging and interfacial effects were not measured directly and would require separate microstructural or interface testing. The mean fatigue lives of BF and GF differed by only 88 cycles, or 1.9% of the GF value. The fatigue test was conducted at one controlled strain level, 400 $\mu\epsilon$, to compare the three mixtures under the same loading condition. The results therefore describe relative fatigue performance at this strain level only. They do not define a complete fatigue relationship or allow calibration of strain-dependent coefficients such as k_1 and k_2 . Testing at several strain levels is needed for that purpose. The overall mixture-group effects were significant for initial flexural stiffness (ANOVA $p = 0.000362$) and fatigue life ($p < 0.001$). Tukey HSD showed that both fiber mixtures differed significantly from the control for both responses, whereas BF and GF did not differ significantly from each other in initial stiffness ($p = 0.1113$) or fatigue life ($p = 0.3964$).

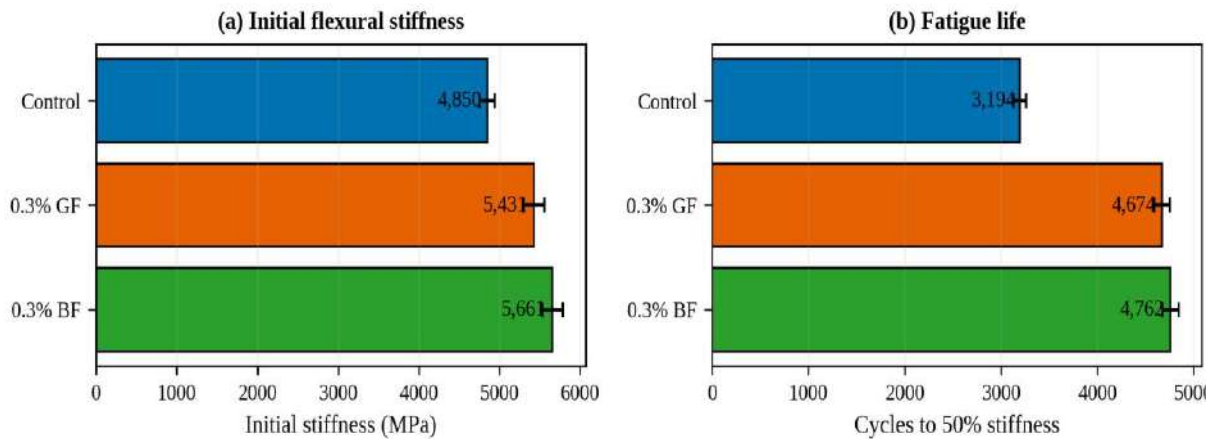


Fig. 6. Four-point bending results: (a) initial flexural stiffness and (b) fatigue life at 20 °C, 5 Hz, and 400 µε. Values are means ± SD (n = 3)

3.5 Particle-Loss Resistance Before and After Aging

In the adapted Cantabro test, mean unaged mass loss decreased from 22.0% for the control to 11.3% for BF and 12.3% for GF, numerical reductions of 48.6% and 44.1% (Figure 7). After seven-day aging at 60 °C, the respective mean losses were 27.0%, 15.1%, and 18.3%; relative to the control mean, BF and GF showed reductions of 44.1% and 32.2%. Thus, under the stated adapted procedure, both fiber mixtures showed lower mean particle loss than the control, with BF giving the lowest mean loss in both conditions. Thermal aging increased mean mass loss by 5.0 percentage points for the control, 3.8 points for BF, and 6.0 points for GF. Doyle and Howard [35] also reported that the Cantabro test can differentiate among dense-graded mixtures. The relatively high absolute losses of the control (22.0% unaged and 27.0% aged) should be viewed within the conditions of this adapted test, including the fixed 5.0% binder content and the dense-graded mixture structure. Void structure and effective binder availability may affect particle retention, but their individual effects were not isolated here. For this reason, the Cantabro results are used only to compare the mixtures tested in this study and are not interpreted using OGFC acceptance limits or as direct predictions of field raveling. The overall mixture-group effects were statistically significant for Cantabro mass loss in both the unaged and aged conditions (one-way ANOVA, $p < 0.001$ for each condition).

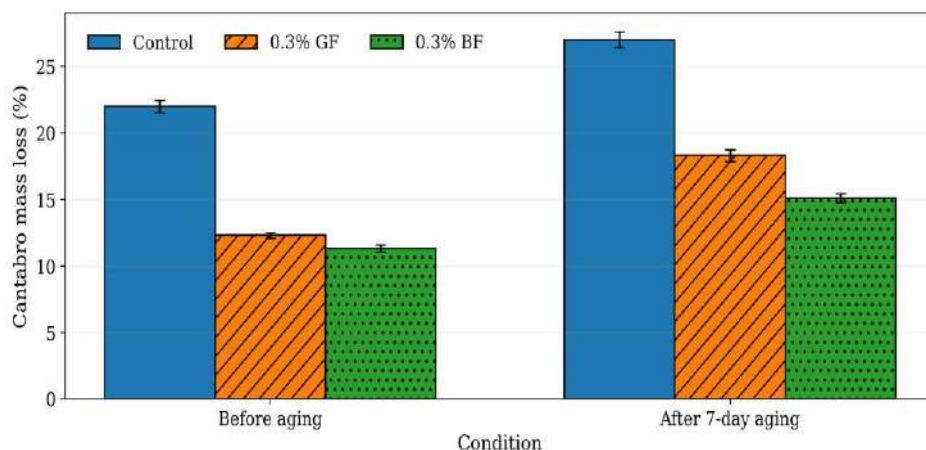


Fig. 7. Cantabro mass loss before and after seven-day aging at 60 °C under the adapted procedure. Values are means ± SD (n = 3)

3.6 Cross-Test Material-Response Synthesis

Across the measured responses, 0.3% gave the most balanced result within the tested dosage range. At 0.4%, MQ and TSR increased slightly, but mean stability and dry ITS decreased from their

0.3% values. For this reason, the selected dosage was based on the combined Marshall and tensile response rather than on the highest value of a single index.

Table 6. Descriptive effect magnitudes for the selected 0.3% mixtures relative to the control

Performance indicator	0.3% BF	0.3% GF	BF–GF contrast
Marshall stability increase (%)	+27.2	+17.5	+9.7
Marshall flow reduction (%)	+14.7	+11.8	+2.9
Dry ITS increase (%)	+15.5	+11.9	+3.6
Conditioned ITS increase (%)	+19.6	+15.6	+4.0
TSR gain (percentage points)	+2.5	+2.5	0.0
Final rut-depth reduction (%)	+30.0	+22.9	+7.1
Initial stiffness increase (%)	+16.7	+12.0	+4.7
Fatigue-life increase (%)	+49.1	+46.3	+2.8
Aged Cantabro-loss reduction (%)	+44.1	+32.2	+11.9
Unaged Cantabro-loss reduction (%)	+48.6	+44.1	+4.5

Note: Values are percentage changes unless identified as percentage points. A positive BF–GF contrast denotes the larger favorable numerical response for BF.

In this study, “performance fingerprint” means the pattern of results obtained from the different laboratory tests. It is not a weighted index or a combined score. Table 6 lists the response of each test separately. From the group means, BF had larger numerical changes than GF for several indicators, although the size of the difference depended on the test. The largest BF–GF differences were found for aged particle loss, Marshall stability, and rut depth. In contrast, TSR was the same for both fibers and the fatigue-life results were close. The term is therefore used only to describe the pattern across the tests and not to rank the two fibers statistically.

3.7 Indicative Economic Consideration

At the selected fiber content of 0.3% by total mixture mass, one ton of asphalt mixture requires approximately 3 kg of fiber. The direct additional fiber-material cost can therefore be expressed using Eq. (8):

$$\Delta C = (3 \text{ kg/tonne}) C_f \quad (8)$$

Where: ΔC is the additional fiber-material cost per ton of asphalt mixture and C_f is the local fiber price per kilogram. An indicative economic framework is presented because the economic value of fiber modification depends on local material, construction, maintenance, and long-term performance conditions [36]. At the present laboratory stage, the measured performance changes provide the technical basis for a future field-calibrated life-cycle evaluation. Field sections can subsequently be used to relate the additional fiber cost to maintenance demand and rehabilitation intervals under local conditions.

4. Conclusions

The tests in this study were carried out on one dense-graded HMA with 5.0% asphalt binder. Both 6 mm BF and 6 mm GF were tested at 0.2%, 0.3%, and 0.4% of total mixture mass. The Marshall and tensile results showed that 0.3% provided the most balanced response within the tested range, so this dosage was used for the additional tests. At 0.3%, mean Marshall stability increased by 27.2% for BF and 17.5% for GF. Mean dry ITS increased by 15.5% and 11.9%, respectively, and conditioned ITS also increased. The TSR value was 82.5% for both mixtures, so there was practically no numerical difference between BF and GF in moisture-strength retention.

The wheel-tracking results also showed lower rut depths for the fiber mixtures. After 10,000 wheel passes at 60 °C, the mean rut depth was 7.0 mm for the control, 4.9 mm for BF, and 5.4 mm for GF. In the four-point bending test at 20 °C, 5 Hz, and 400 µε, both fiber mixtures had higher mean initial stiffness and longer mean fatigue life than the control. The results at 400 µε provide a direct comparison of the three mixtures under the selected fatigue condition, while testing at additional strain levels would extend the fatigue characterization. Both fibers also reduced mean Cantabro particle loss before and after seven days of thermal aging, with BF giving the lowest mean aged loss. The adapted Cantabro procedure therefore provides a consistent comparative measure of particle-loss resistance for the dense-graded mixtures evaluated here.

Taken together, the results show that the effect of the fibers was not the same in every test. BF produced larger numerical changes in rut depth and aged particle loss, whereas BF and GF were very close in TSR and fatigue life. The two fibers should therefore be compared according to the response of interest rather than by one overall ranking. The observed laboratory improvements provide a useful basis for material selection and for subsequent field evaluation of long-term pavement performance. The present investigation focused on one 60/70 penetration-grade binder, one dense gradation, 5.0% asphalt content, laboratory conditioning, and one fatigue strain level. Future work can broaden this experimental scope through field sections, fatigue testing at several strain levels, low-temperature cracking tests, microstructural and interface characterization, and life-cycle cost analysis using documented local costs and field performance data.

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Intelligent prediction model of durability of shoe material foam structure integrating transformer attention mechanism and microscopic representation

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Abstract

The durability evaluation of modern shoe material foaming media has long relied on physical testing and faces technical bottlenecks such as long cycles and difficulty in revealing microscopic failure mechanisms. In response to this problem, the research aims to explore an evaluation path that takes into account local morphological quantification and long-range force dependence analysis and constructs an intelligent durability prediction model that integrates the window transformer attention mechanism and micro topological dimensionality reduction representation. The results show that in the application of anti-fatigue topology optimization, the model successfully suppressed the median stress of the foam matrix from 8.7MPa to 3.4MPa and strictly controlled the local extreme value at 6.1 MPa, reducing simulated stress concentration. Facing a severely distorted scene with a porosity of 70%, the mean absolute error was stably suppressed at 5.1k cycles, and network-only inference required 32.6 ms, demonstrating excellent fault tolerance and computational economy. In addition, in the long-term dynamic variable load verification, the predicted deformation rate at the end of life (9.80%) was highly consistent with the FEA reference level (9.90%), showing close agreement with the simulated loading trajectory. The research has opened up the cross-scale entity mapping from porous physical network to macroscopic mechanics and established an intelligent digital template with great transformation value for high-throughput virtual concurrent screening of shoe material formulas and anti-fatigue reverse auxiliary engineering.

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1. Introduction

1.1 Motivation for The Proposed Research

With the evolution of modern shoe engineering, foamed Porous Media (PM) has been widely used due to its lightweight and high resilience properties. However, cell geometry and cyclic loading conditions influence the fatigue response of cellular structures [1]. Machine-learning-assisted formulation design can reduce experimental effort in developing polyurethane foams with targeted density and mechanical strength [2]. In recent years, deep learning methods based on conventional Convolutional Neural Networks (CNNs) have gradually been introduced into this intersection field. However, predicting the response of heterogeneous materials requires linking microstructural representations to macroscopic mechanical behavior [3]. Therefore, there is an urgent need to explore an intelligent regression mechanism that combines efficient microscopic feature extraction and global spatial dependence modeling capabilities to break through the existing technical bottleneck of durability assessment.

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1.2 Related Work

To break through the above bottlenecks, existing research focuses on PM microscopic characterization and mechanical property prediction, exploring dimensions such as image analysis, feature extraction, and intelligent regression model construction, and has accumulated rich theoretical results. In terms of microscopic characterization, Nguyen and others reviewed image processing techniques for characterizing PM and examining pore morphology, liquid transport, and mechanical damage [4]. Furthermore, Zhuang et al. introduced deep learning and used CNNs to construct a bidirectional prediction framework of PM features and compression mechanical response, which significantly improved the evaluation accuracy [5]. In addition, Kadulkar et al. reviewed the use of machine learning in dimensionality reduction and accelerated material design [6]. However, when existing visual networks deal with complex pores, they are often limited by local receptive fields, making it difficult to capture long-range spatial dependencies such as stress concentration on pore walls. In response to this limitation, the study proposes an intelligent regression model that integrates Transformer Attention Mechanism (TAM) and Microscopic Characterization (MC) to break through the gap between global topology modeling and durability prediction.

As a cutting-edge architecture, TAM has been widely used in many fields such as vision, especially in MC. It has significant advantages in resolving long-range dependencies through self-attention, thus showing strong modeling potential. Khan et al. reviewed vision transformers and CNN-transformer hybrids, highlighting their approaches to visual representation learning [7]. Subsequently, in the field of microscopic image analysis, Alrfou and others reviewed deep learning methods, including CNNs and Transformers, for extracting microscopic features and modeling structure–property relationships [8]. Furthermore, in response to the pain points of limited local receptive fields of classic networks, Qin's team proposed a factorization architecture to reconstruct the TAM paradigm, which successfully captured long-range dependencies and enhanced robustness while maintaining low computational overhead [9]. In view of this, introducing TAM across fields into an intelligent prediction model for the durability of shoe material foam structures is highly feasible for fitting the global topological characteristics of complex pores.

MC has significant advantages in solving structure-activity mapping problems through quantitative analysis of topological structures. It has been widely used in materials science and other fields, especially in PM research, and has important value in macroscopic performance prediction. Sahoo and Mandal constructed three-dimensional carbon-foam models from observed microstructures and used finite element analysis to evaluate effective mechanical properties [10]. In porous materials, the Andrä H team combined three-dimensional images to simulate fiber networks, revealing the decisive influence of pore characteristics on macroscopic thermal conductivity [11]. Furthermore, Prifling et al. combined three-dimensional pore-space descriptors with regression models to predict gas diffusion in loam and sand [12]. Based on the above research logic, using MC to accurately extract morphological prior information is the only way to drive the intelligent prediction model of shoe material foam structure durability to approach the true mechanical limit.

1.3 Research Objective and Highlights

In summary, substantial progress has been made in PM morphology quantification and basic performance mapping. However, traditional CNNs are limited by their local receptive fields and have difficulty capturing the long-range dependence of pore wall stress concentration, resulting in insufficient model generalization prediction accuracy in complex scenarios. Therefore, the study proposes an Intelligent Prediction Model for Durability of Footwear Foam Structures (IPM-FMD) that combines TAM and MC. This model uses CNNs to extract morphological features from orthogonal distance-map channels and couples the resulting spatial feature maps with a self-attention network for fatigue-life regression. The contribution lies in preserving the spatial arrangement of distance-encoded pore morphology throughout CNN feature extraction and window-attention processing, followed by channel–spatial reweighting for fatigue-life regression. The role of the Swin Transformer (ST) is to model interactions among image features, while multi-dimensional attention adjusts their contributions to the regression output. Component

contributions are examined through the existing ablation variants, and engineering applications are evaluated through numerical simulations.

2. Methods and Materials

2.1 Overall Architecture of IPM-FMD Model

Efficient representations of heterogeneous microstructures should retain morphological information relevant to their effective mechanical properties [13]. To this end, a new morphological dimensionality reduction representation strategy is constructed through orthogonal cross-section sampling and coupled distance transformation. Its complete intelligent regression prediction link architecture is shown in Figure 1.

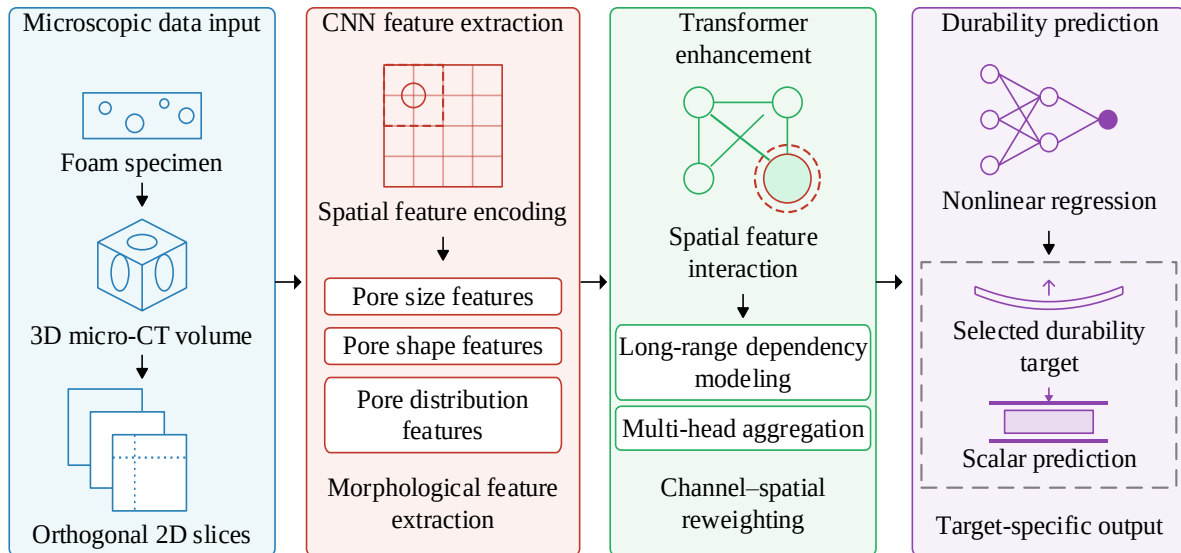


Fig. 1. Overall architecture diagram of IPM-FMD model

Note: One scalar is predicted for the selected durability target.

In Figure 1, IPM-FMD sequentially extracts three-dimensional orthogonal slices, uses CNNs to extract spatial morphological features, and relies on Swin Transformer blocks and multi-dimensional attention to model spatial interactions and reweight the learned features. Finally, the nonlinear regression network outputs a scalar prediction for the selected durability target.

2.2 Dimensionality Reduction Characterization of Microscopic Characteristics of Shoe Foam Pores Based on CNNs

2.2.1 Two-Dimensional Dimensionality Reduction and Pore Morphology Reconstruction of Shoe Material Foam Images

The fatigue performance of cellular lattice materials depends on unit-cell geometry and local stress concentration [14-15]. To reduce the computational cost of three-dimensional processing while retaining selected microscopic morphological priors, the model maps three-dimensional foaming pores to two-dimensional pseudo-color features, as shown in Figure 2.

In Figure 2, the model first receives a reconstructed three-dimensional tomographic volume, and achieves preliminary dimensionality reduction from three-dimensional voxels to a two-dimensional plane by extracting orthogonal slices through the internal pore space. For sliced images, intensity normalization and local contrast enhancement precede Li's minimum cross-entropy thresholding to separate the pore and solid phases. Subsequently, the Euclidean distance transformation matrix of this binary segmentation map is calculated to encode the distance from solid-phase pixels to the nearest segmented pore pixels. This representation describes local distance-to-pore information within the sampled planes. The distance field is calculated as shown in Equation (1).

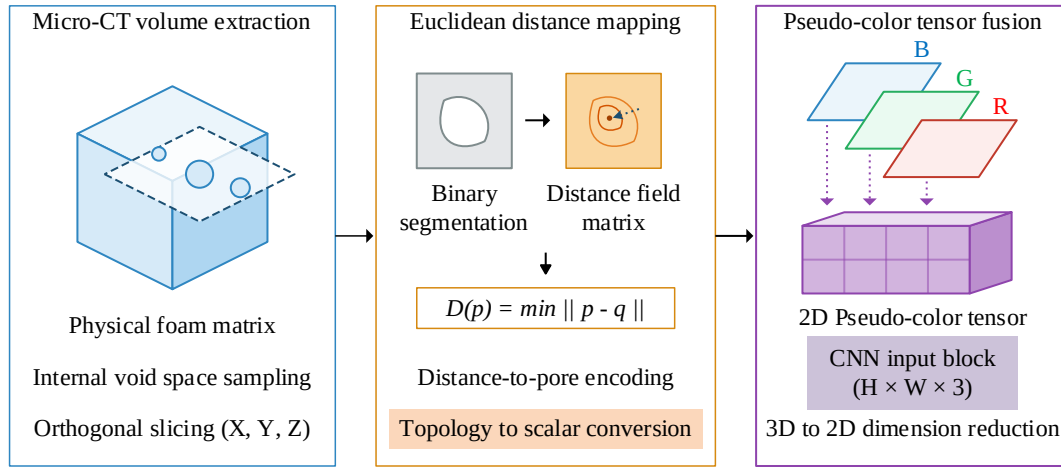


Fig. 2. Dimensionality reduction mapping from three-dimensional foaming pores to two-dimensional pseudo-color features

$$D_{map}(u, v) = \min_{(x, y) \in \Omega_{void}} \sqrt{(u - x)^2 + (v - y)^2} \quad (1)$$

In Equation (1), $D_{map}(u, v)$ is the Euclidean distance field value from the matrix pixel point with coordinate (u, v) on the orthogonal slice to the nearest pore boundary. Ω_{void} represents the microscopic pore physical space set obtained after segmentation by cross entropy threshold. (x, y) is the target pixel coordinate belonging to the pore set. Finally, the distance field matrix of multi-directional slices is channel-joined and reconstructed into a two-dimensional tensor in pseudo-color format, which is used as an efficient input feature for subsequent CNNs.

2.2.2 CNNs Convolution Extraction for Microscopic Pore Topological Features

Mechanical characterization of polymeric foams is complicated by their cellular morphology and deformation behavior [16]. To accurately capture these key anti-deformation features, the model introduces CNNs to perform deep down-sampling extraction. The specific network architecture is shown in Figure 3.

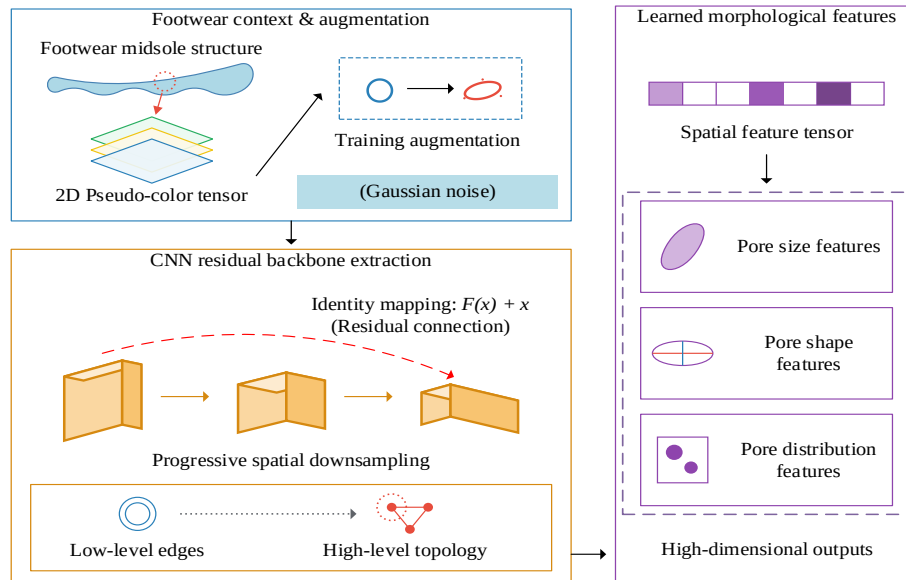


Fig. 3. CNNs downsampling extraction network architecture diagram for micro topological features

Note: The footwear sampling stage illustrates the intended application of the framework.

In Figure 3, Gaussian noise is added to the distance-map inputs during training to represent input perturbations. Subsequently, the two-dimensional pseudo-color tensor enters the base of the

residual network. Relying on the identity mapping and step-by-step spatial down-sampling mechanism, the network deeply abstracts shallow edge features into high-order topological semantics while filtering out matrix noise. The residual identity mapping of micro topological features is shown in Equation (2).

$$Y_{topo} = F(X_{in}, \{W_k\}) + X_{in} \quad (2)$$

In Equation (2), Y_{topo} is the learned spatial feature tensor output by CNNs. X_{in} is the input two-dimensional pseudo-color dimensionality reduction feature matrix. F is the multi-layer nonlinear mapping function in the residual network base. W_k represents the adaptive weight parameter used to extract the edge features of the foaming structure in each convolution layer. The CNN output is retained as a learned spatial feature tensor and passed to the subsequent window-attention blocks for modeling interactions among local morphological features.

2.3 Construction of Durability Prediction Network Integrating Swin Transformer

2.3.1 Modeling of Long-Range Dependence of Pores Based on Self-Attention

Although CNNs have efficiently quantified microscopic local features, the foamed matrix is a highly cross-linked porous skeleton. Cyclic loading can drive microscopic crack propagation and structural failure in heterogeneous materials [17]. Traditional CNNs are limited by their local receptive fields, making it difficult to capture the long-range dependence of stress transfer across space. To break through this bottleneck, a study has been carried out on the interaction center that introduces the self-attention mechanism to reconstruct the model. In terms of algorithm deployment, the model divides the high-dimensional feature tensor output by the CNNs into non-overlapping local windows, and builds a multi-head self-attention calculation graph internally. Successive blocks alternate between regular and shifted window partitions, enabling information exchange across neighboring windows. The spatial weight solution for the long-range dependence of pores is shown in Equation (3).

$$A_{swin} = \text{Softmax} \left(\frac{QK^T}{\sqrt{d_s}} + B_{pos} \right) V \quad (3)$$

In Equation (3), A_{swin} is the attention feature map output by Swin Transformer encoding interactions among image features. Q , K , and V respectively represent the query matrix, key matrix and value matrix generated by linear mapping of local window features. d_s is the dimension of the query and key vectors in each attention head, and its square root scales the dot products before SoftMax normalization. B_{pos} is the learnable relative position bias representing token offsets within each window. Alternating regular and shifted windows enables information exchange across neighboring spatial regions.

2.3.2 Microscopic Pore Feature Enhancement Network Integrating Multi-Dimensional Attention

Although the model has captured long-range dependence, local cell morphology also influences the compressive response of foams [18]. Attention mechanisms can emphasize task-relevant features through adaptive weighting [19]. The study implements three-line parallel analysis of channel and spatial dimensions on the feature tensor, extracts and aggregates complementary weights to reweight the learned morphological features, and the architecture is shown in Figure 4.

In Figure 4, the network uses three-way orthogonal decomposition to independently calculate the self-attention thermal distribution of channels and spatial height and width dimensions. Each complementary weight is then aggregated and non-linearly activated to generate a global mask, which is element-wise multiplied with the original features. The multi-dimensional feature-reweighting mechanism is shown in Equation (4).

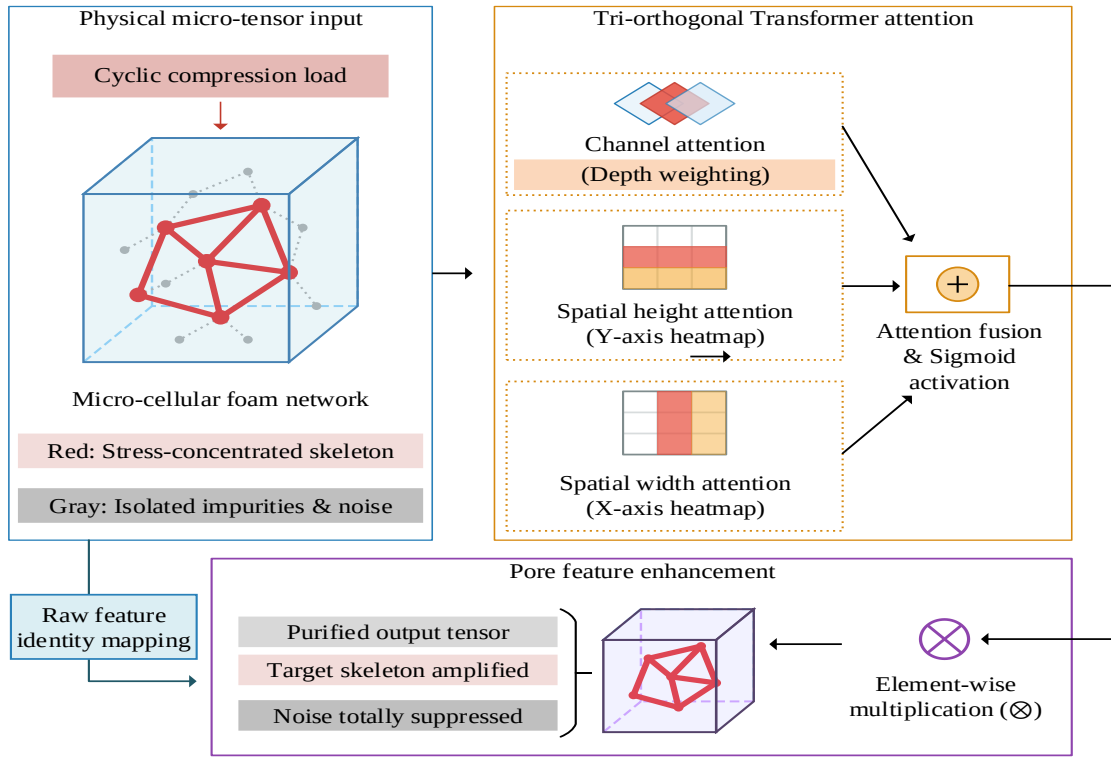


Fig. 4. Pore feature enhancement network diagram fused with multi-dimensional Transformer

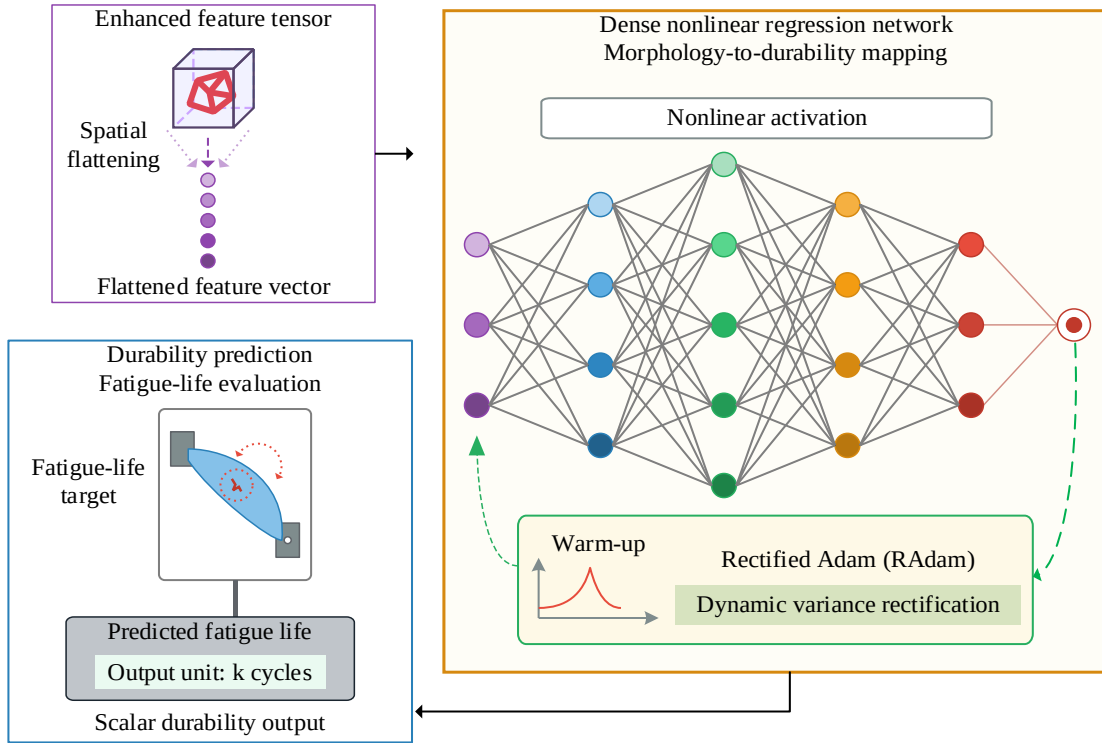


Fig. 5. Fully connected regression network diagram from microscopic pore characteristics to macroscopic durability

$$F_{out} = F_{in} \otimes \sigma(\lambda_c A_c \oplus \lambda_h A_h \oplus \lambda_w A_w) \quad (4)$$

In Equation (4), F_{out} is the enhanced feature tensor output after attention-based reweighting. F_{in} is the original topological feature of the input attention center. $\otimes$ represents the element-wise multiplication operation of the matrix. σ is the *Sigmoid* nonlinear activation function, used to generate a smooth global feature mask. A_c , A_h , and A_w represent the orthogonal attention heat

distribution map of the three dimensions of channel, height, and width respectively. λ is the learnable balance coefficient of the corresponding dimension. $\oplus$ represents the aggregation and addition of multi-dimensional features. The resulting mask adjusts the contributions of learned features before nonlinear regression.

2.3.3 Nonlinear Regression Mapping for Macroscopic Durability Prediction

Neural network models of nonlinear material behavior require suitable architectures and training conditions [20]. Deep neural networks can learn mappings from microstructural images to homogenized mechanical responses [21]. The nonlinear regression network is trained using the Rectified Adam (RAdam) optimization algorithm. The network is shown in Figure 5.

In Figure 5, to bridge the physical gap, the enhanced feature tensor is spatially flattened into a one-dimensional feature vector and directly input into the fully connected regression network. The network uses multiple hidden layers and nonlinear activation functions to fit the mapping from micro topology to the selected durability target. The regression mapping for macro durability indicators is shown in Equation (5).

$$\begin{cases} Z_{topo}^i = \psi_i(\omega_i^T Z_{topo}^{i-1} + b_i), & i = 1, \dots, L \\ P_{life} = \psi(\omega_{out}^T Z_{topo}^L + b_{out}) \end{cases} \quad (5)$$

In Equation (5), Z_{topo}^i is the input vector obtained by flattening the enhanced feature tensor, and P_{life} is the output of hidden layer, where i is the hidden-layer index and L is the total number of hidden layers. The weight matrix, bias vector, and activation function of hidden layer i are denoted by ω_i , b_i , and ψ_i , respectively. The output-layer weight vector and scalar bias are denoted by ω_{out} and b_{out} , and ψ is the output activation function. The superscript T denotes transposition. P_{life} is the scalar prediction for the selected durability target and has the same unit as its reference label. To overcome the gradient oscillation in the early stage of training, the RAdam algorithm is deployed at the bottom layer. Relying on the warm-up and dynamic variance correction mechanism, it effectively ensures the smooth optimization of weights and efficient convergence of the network. The predicted scalar output represents fatigue life under the material and loading conditions assigned to the corresponding numerical case.

3. Results

3.1 Experimental Settings and Configuration

3.1.1 Data Sources and Reference Labels

The experiment relied on the Foamed Cement API RP 10B-4 dataset from Digital Rocks Portal (project DRP-154; DOI: 10.17612/P7GD4Q) and the RealPUFoam volume archived in Zenodo (record 17590658; DOI: 10.5281/zenodo.17590658) for evaluation. The cement dataset contains two formulations prepared with nominal foam qualities of 10%, 20%, 30%, and 40%. The polyurethane foam volume comprises $1300 \times 951 \times 960$ voxels with an isotropic voxel spacing of $2.75 \mu\text{m}$. The selected tomograms provide pore–solid interfaces and spatial variations in cell morphology for evaluating the distance-map representation. Application to footwear foams requires correspondence in pore morphology, connectivity, physical pore scale, bulk density, and material response under the specified loading conditions.

The two sources were therefore used as morphology-oriented porous-media benchmarks rather than as direct substitutes for production footwear-foam datasets. Their relevance lies in providing resolved pore–solid interfaces, connectivity variations, and porosity ranges required to evaluate the distance-map representation and morphology-to-durability mapping. Differences may nevertheless remain relative to industrial EVA or PU foams in polymer formulation, cell-size distribution, anisotropy, bulk density, and processing history. Transfer to footwear materials therefore requires material-specific calibration and independent validation.

Reference fatigue-life labels were obtained from numerical analyses of the reconstructed geometries. Each label was linked to its source volume, derived geometry, material-parameter record, and loading history through persistent identifiers.

3.1.2 Specimen Partitioning and Image Preprocessing

Parent specimens were assigned to the training, validation, and test subsets in a 70:15:15 ratio using a fixed split seed of 2026. All slices, crops, augmented inputs, and derived geometries originating from the same parent specimen were retained in the same subset. Model selection used the validation subset, and the test subset was reserved for final evaluation.

For each labeled geometry, three orthogonal slice triplets were extracted at relative positions of 25%, 50%, and 75% along each axis. Each slice was centrally cropped to a square and resized to 224×224 pixels using bilinear interpolation. Intensities were clipped at the slice-specific first and ninety-ninth percentiles and rescaled to $[0, 1]$. Contrast-limited adaptive histogram equalization used 32×32 -pixel regions, a clip limit of 0.01, and 256 bins. Li's minimum cross-entropy thresholding used the mean intensity for initialization and a tolerance of 10^{-5} ; pixels below the threshold represented pores.

Euclidean distance maps, with pore pixels set to zero, were divided by the image diagonal (315.37 pixels) and stacked into three-channel inputs. Training augmentation used zero-mean Gaussian noise with a standard deviation of 0.01 and clipping to $[0, 1]$. Validation and test inputs were not augmented. Predictions from the three slice triplets were averaged to obtain one prediction for each labeled geometry.

3.1.3 Model Configuration and Training

To objectively measure the regression accuracy of the proposed method, the experiment selected four visual architectures as comparison benchmarks: ResNet-50, ViT-Base, ConvNeXt-V2, and FaViT-B2 [9]. FaViT-B2 was included as a recent Transformer-based reference for long-range dependency modeling. Each architecture was adapted to scalar fatigue-life regression and evaluated under the same specimen partition, input representation, training budget, and random-seed protocol. All experiments were executed independently in the PyTorch environment on an Ubuntu workstation equipped with an RTX 4090 graphics card.

The CNN comprised two stages with 48 and 96 channels, respectively. Each stage used a 3×3 stride-2 convolution, batch normalization, and ReLU, followed by two basic residual blocks with two 3×3 stride-1 convolutions per block. All convolutions used padding of one pixel. The output tensor was $96 \times 56 \times 56$. The Swin component used four stages with channel widths of 96, 192, 384, and 768; depths of 2, 2, 6, and 2; and attention-head counts of 3, 6, 12, and 24, respectively. The CNN output entered the first Swin stage directly. Patch merging halved spatial resolution and doubled channel width between stages. The 7×7 windows alternated between shifts of zero and three tokens, with no shifting in the final 7×7 stage. The feed-forward expansion ratio was 4, and the maximum stochastic-depth probability was 0.2.

The enhanced $768 \times 7 \times 7$ tensor was flattened into a 37,632-dimensional vector. Four hidden layers with 256, 128, 64, and 32 units each used ReLU and dropout of 0.1, followed by a linear scalar output.

Training used RAdam with $\beta_1 = 0.9$, $\beta_2 = 0.999$, $\epsilon = 10^{-8}$, and a coupled L2 weight-decay coefficient of 10^{-4} . The learning rate increased linearly from 10^{-6} to 10^{-4} during the first five epochs and subsequently followed cosine decay to 10^{-6} at epoch 100. The batch size was 32. Fatigue-life labels were converted from cycles to k cycles by division by 1,000, without additional target standardization. The training objective was mean squared error in the converted target unit. Training continued for 100 epochs, and the checkpoint with the lowest validation RMSE was retained for evaluation.

3.1.4 Numerical Simulation and Loading Conditions

Fatigue life was defined as the first cycle at which the accumulated fatigue damage reached 1.0. Damage accumulation followed the Palmgren–Miner rule using a calibrated stress–life relationship for the assigned solid-phase material. The six numerical screening cases had porosities of 52.4%, 58.7%, 64.1%, 71.5%, 76.8%, and 82.3%, respectively. Prediction errors were evaluated at impact energies of 2, 4, 6, and 8 J. The reported compression-response trajectory covered 100,000 loading cycles.

3.1.5 Statistical Analysis and Inference Timing

Each model was evaluated over ten training runs using random seeds from 2026 to 2035 and the same parent-specimen partition. Absolute and squared errors were first averaged within each parent specimen; MAE was the mean parent-level absolute error, and RMSE was the square root of the mean parent-level squared error. Performance across training runs was summarized as mean $\pm$ sample standard deviation.

Pointwise 95% confidence intervals for the run-averaged RMSE and MAE used 10,000 bootstrap resamples of test parent specimens, retaining all geometries of each selected parent and using identical indices across models. Metrics were recalculated per run and then averaged across runs; the 2.5th and 97.5th percentiles defined intervals conditional on the fitted models.

MAE comparisons used two-sided paired permutation tests on parent-level absolute errors averaged across runs, with 9,999 random swaps or exact enumeration when fewer distinct swaps existed. Holm correction covered four planned comparisons between IPM-FMD and the four baseline models at a family-wise significance level of 0.05.

Network-only latency was measured in evaluation mode without gradients, using GPU-resident FP32 inputs and a batch size of one. Each of the ten checkpoints underwent 100 warm-up passes and 1,000 timed forward passes using CUDA events and synchronization. Latency was averaged per checkpoint and summarized across checkpoints. Preprocessing, host-to-device transfer, and multi-slice aggregation were excluded and timed separately for complete-pipeline evaluation.

3.2 Performance Comparison Verification of IPM-FMD

3.2.1 High-Precision Regression Verification of The Fatigue Bending Resistance Limit of Foaming Media

To quantitatively evaluate the error interval and feature mapping accuracy of the architectures visualized in Figure 6 for predicting the fatigue bending life of foaming media, the experiment comprehensively examined the regression accuracy and computing power overhead of the model, as shown in Figure 6.

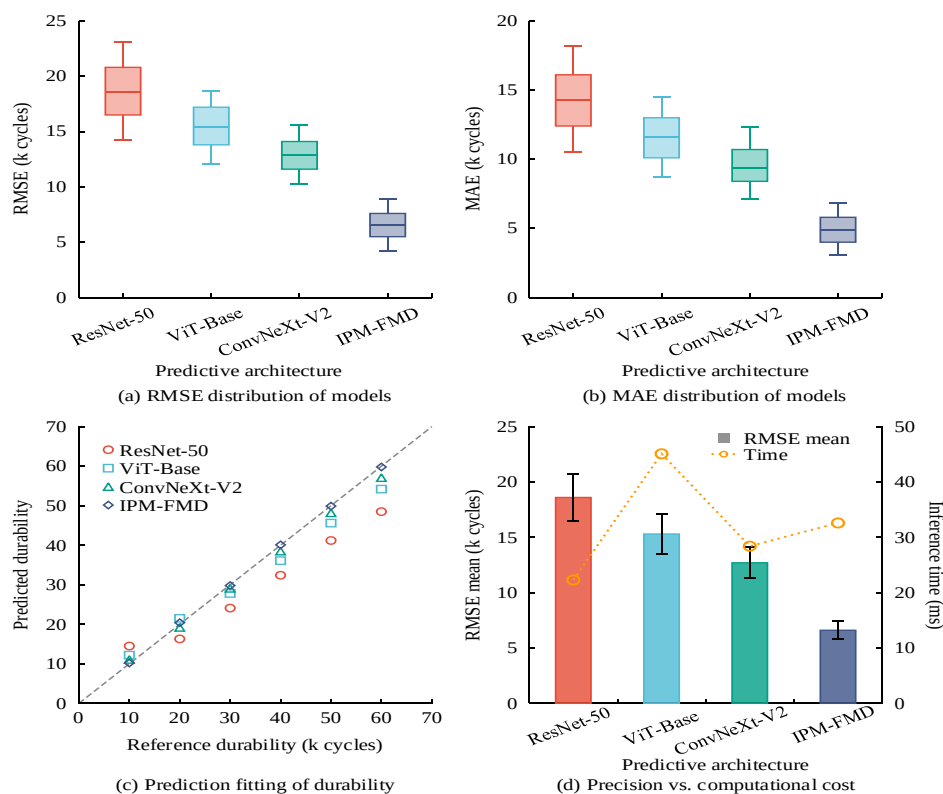


Fig. 6. Comprehensive performance comparison of various prediction architectures in the foaming medium durability regression task

In Figures 6(a) and 6(b), IPM-FMD shows lower and more compact RMSE and MAE distributions than the baseline architectures visualized in the figure. Its RMSE distribution is concentrated around 6.6 k cycles, reflecting stable prediction performance. Figure 6(c) further shows that the predicted values of IPM-FMD remain close to the ideal diagonal across the evaluated fatigue-life range. In contrast, ResNet-50 underestimates the reference lifetime at 60 k cycles, with an absolute prediction error of up to 11.5 k cycles. From Figure 6(d), IPM-FMD reduces the mean RMSE to 6.60 k cycles, while its network-only inference time of 32.6 ms remains lower than the 45.1 ms of ViT-Base, providing a favorable balance between regression accuracy and computational cost.

To further quantify the performance variability of different prediction architectures under repeated training, ResNet-50, ViT-Base, ConvNeXt-V2, FaViT-B2, and IPM-FMD were evaluated over ten independent training runs under the same Digital Rocks Portal and Zenodo data protocol, specimen partition, and training settings. Table 1 summarizes RMSE and MAE as mean $\pm$ standard deviation with 95% confidence intervals, together with network inference time and Holm-adjusted paired-comparison results.

Table 1. Statistical comparison of IPM-FMD and baseline models over ten independent runs

Model	RMSE, mean $\pm$ SD (95% CI), k cycles ↓	MAE, mean $\pm$ SD (95% CI), k cycles ↓	Inference time, mean $\pm$ SD (ms) ↓	Holm-adjusted <i>p</i> - value
ResNet-50	18.60 $\pm$ 2.72 (16.50–20.70)	14.31 $\pm$ 2.48 (12.53–16.09)	22.3 $\pm$ 0.6	0.004
ViT-Base	15.30 $\pm$ 2.12 (13.50–17.10)	11.59 $\pm$ 1.88 (10.25–12.93)	45.1 $\pm$ 1.4	0.007
ConvNeXt-V2	12.70 $\pm$ 1.72 (11.30–14.10)	9.55 $\pm$ 1.63 (8.38–10.72)	28.4 $\pm$ 0.8	0.016
FaViT-B2	10.85 $\pm$ 1.70 (9.63–12.07)	8.16 $\pm$ 1.28 (7.25–9.07)	36.8 $\pm$ 1.4	0.031
IPM-FMD	6.60 $\pm$ 1.53 (5.80–7.40)	4.92 $\pm$ 1.18 (4.08–5.76)	32.6 $\pm$ 1.0	/

Note: Values are reported as mean $\pm$ standard deviation over ten independent training runs; values in parentheses denote 95% confidence intervals. Adjusted *p*-values refer to paired MAE comparisons with IPM-FMD after Holm correction. Lower RMSE, MAE, and inference time indicate better performance.

As shown in Table 1, IPM-FMD achieved the lowest mean RMSE and MAE, reaching 6.60 $\pm$ 1.53 k cycles and 4.92 $\pm$ 1.18 k cycles, respectively. Compared with FaViT-B2, these errors were reduced by approximately 39.2% and 39.7%. The Holm-adjusted *p*-values for the paired MAE comparisons between IPM-FMD and the four baseline models were all below the predefined 0.05 significance level. In terms of computational efficiency, ResNet-50 and ConvNeXt-V2 required shorter inference times of 22.3 and 28.4 ms, respectively, whereas their prediction errors were substantially higher. IPM-FMD required 32.6 ms per network forward pass, which remained lower than FaViT-B2 at 36.8 ms and ViT-Base at 45.1 ms. Therefore, the proposed model mainly provides a favorable balance between prediction accuracy and computational cost rather than the lowest latency alone. Inference time represents network-only batch-1 latency on the RTX 4090 GPU; the detailed timing and statistical procedures are described in Section 3.1.5.

3.2.2 Engineering Generalization and Fault Tolerance Under Extreme Pore Distortion Scenarios

To examine the fault-tolerance performance and engineering generalization robustness of the IPM-FMD model when dealing with severe distortion of the pores of the foaming medium, the experiment further dynamically tracked the training convergence process of the architectures visualized in Figure 7, and objectively analyzed its error fluctuation trend within the differentiated porosity interval, as shown in Figure 7.

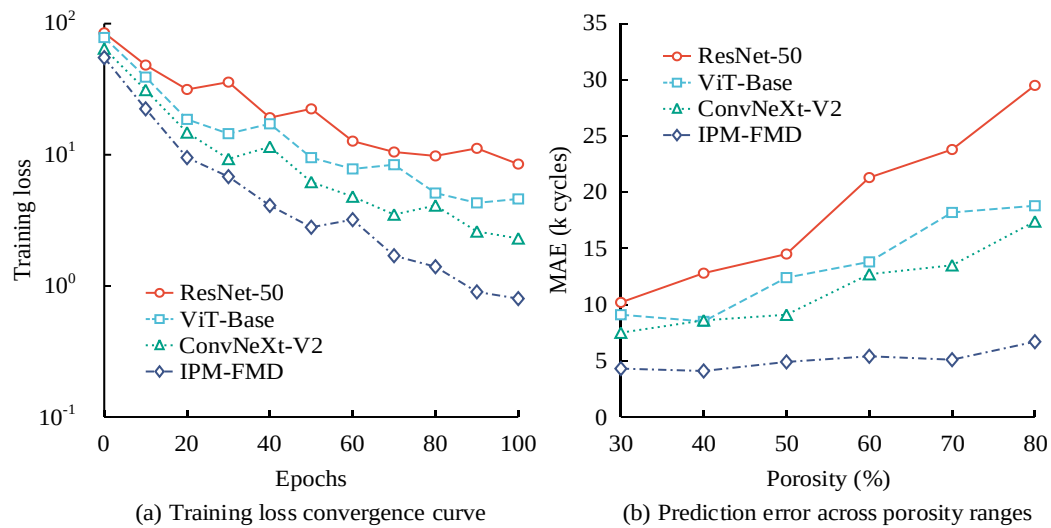


Fig. 7. Convergence stability and pore distortion tolerance performance evaluation of the prediction model under extreme working conditions

In Figure 7(a), affected by stochastic gradient perturbation, the benchmark model experienced significant gradient jumps during the convergence process. Although the IPM-FMD model produced slight oscillations at 60 rounds, the final Loss value dropped steadily to 0.8, demonstrating extremely strong training stability. As shown in Figure 7(b), as the porosity evolves to the extreme distortion interval of 80%, the prediction error MAE of ResNet-50 sharply soars to 29.5 k cycles, showing a serious generalization bottleneck. In contrast, the IPM-FMD model maintains lower prediction errors across the evaluated porosity range, with an MAE of 5.1 k cycles at 70% porosity. These results indicate improved robustness to pore-distortion variation under the benchmark conditions, while transfer to real footwear foams requires independent validation.

3.3 Ablation Experimental Analysis of IPM-FMD Model

To further verify the independent contribution of feature enhancement and other modules in the IPM-FMD model to durability prediction, the experiment further established the ablation scheme. Three degenerate variants are studied: No Multi-Attention (w/o MDA), No Swin-Transformer (w/o ST) and Standard Adam (Adam). The test environment and data follow the configuration of Digital Rocks Portal and Zenodo data set. The specific ablation experiment variable control and configuration details are shown in Table 2.

In Figure 8(a), the loss value of the complete IPM-FMD model quickly dropped to 1.15 after the 40th round, and finally converged to 0.81, while the w/o ST and Adam variants oscillated significantly due to the influence of topological noise. As shown in Figure 8(b), after stripping off the multidimensional attention or window transformer, the RMSE of its fatigue life prediction climbs to 8.8 k cycles and 12.5 k cycles respectively, indicating that long-range modeling contributes to controlling extreme deviations. As shown in Figure 8(c), the median absolute error of the complete model is only 5.4 k cycles and the interquartile range is the narrowest, supporting the combined contribution of the core mechanisms to system robustness when coping with extreme deformation.

Because each input channel encodes normalized distance-to-pore information, the learned spatial features remain directly related to pore proximity and local skeleton geometry. The increased prediction errors after removing the multi-dimensional attention or Swin Transformer indicate that spatial reweighting and non-local feature interaction both contribute to fatigue-life regression. However, these ablations do not localize the contribution of individual pore regions, and no dedicated post-hoc saliency analysis was performed in the present study. Future validation will therefore combine attention-rollout or saliency mapping with pore-size, shape, connectivity, and local stress descriptors to examine whether high-contribution regions correspond to physically critical microstructures.

Table 2. Ablation experiment variable control and configuration of IPM-FMD model

Model/Variant	CNNs Micro-topology	Swin Transformer	Multi-dimensional Attention	RAdam Optimizer	Configuration Description
IPM-FMD (Full)	✓	✓	✓	✓	Research model retaining all core mechanisms
IPM-FMD (w/o MDA)	✓	✓	×	✓	Removal of multi-dimensional feature reinforcement
IPM-FMD (w/o ST)	✓	×	✓	✓	Swin Transformer replaced by deep residual convolution
IPM-FMD (Adam)	✓	✓	✓	×	Regressor optimization degraded to standard Adam

In Table 2, the experiment strips off the multi-dimensional attention module at fixed points to test the feature purification performance, downgrades the window transformer to depth convolution to evaluate the long-range dependency capture gain, and degrades the optimizer to a standard algorithm to examine the convergence smoothness. The comprehensive regression performance and dynamic error results of each model based on this setting are shown in Figure 8.

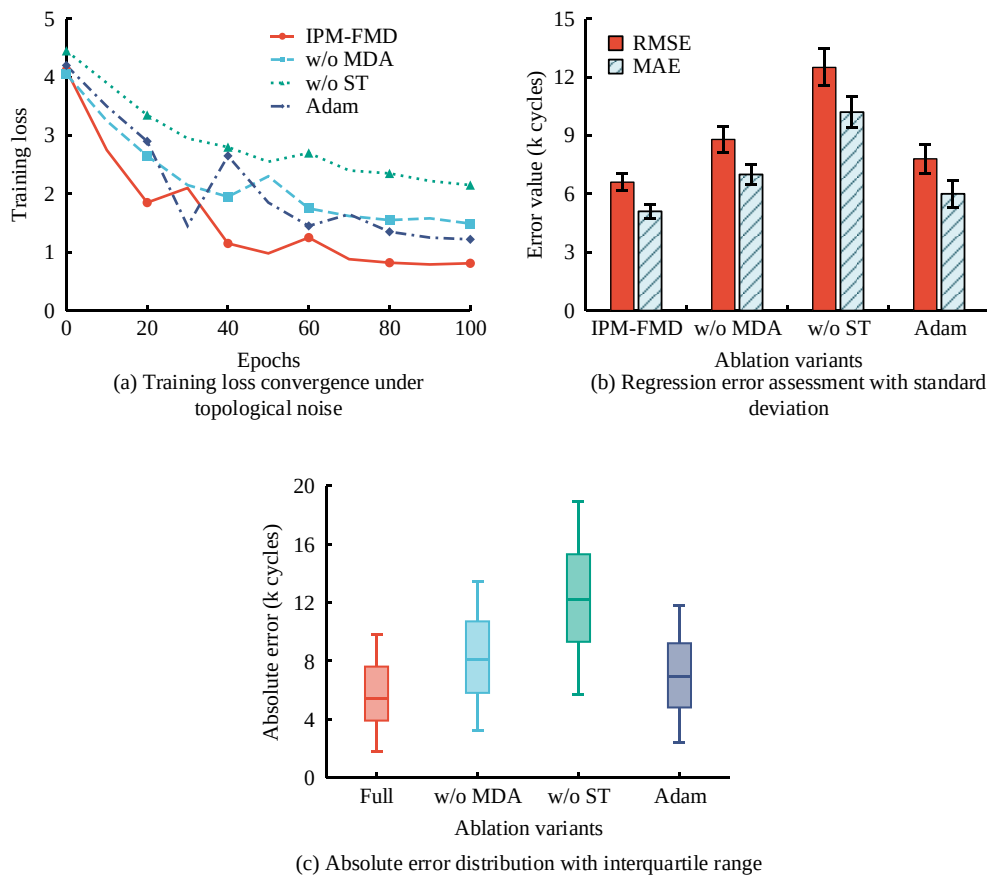


Fig. 8. Comprehensive performance comparison of ablation experiments of core components of IPM-FMD model

3.4 Analysis of Simulation Application Effects of IPM-FMD

3.4.1 Micro-Topology Directional Optimization for Fatigue-Resistant Design

Focusing on the evaluation bottleneck of shoe material foam structures in real engineering implementation, the experiment followed the Digital Rocks Portal and Zenodo open-source data sets to expand the prediction boundary to dynamic physical simulation scenarios. To bridge the mechanical gap between theoretical models and complex service conditions, the core data mapping logic was extended from single static topography analysis to multi-modal stress environment and high-throughput concurrent optimization of feature fields. To verify the topology optimization effectiveness of the predictive feedback mechanism in the reverse design of foaming formulas, experiments were conducted to quantitatively compare the anti-fatigue mechanical performance of the initial random matrix and the pore characteristics after iterative reconstruction, as shown in Figure 9.

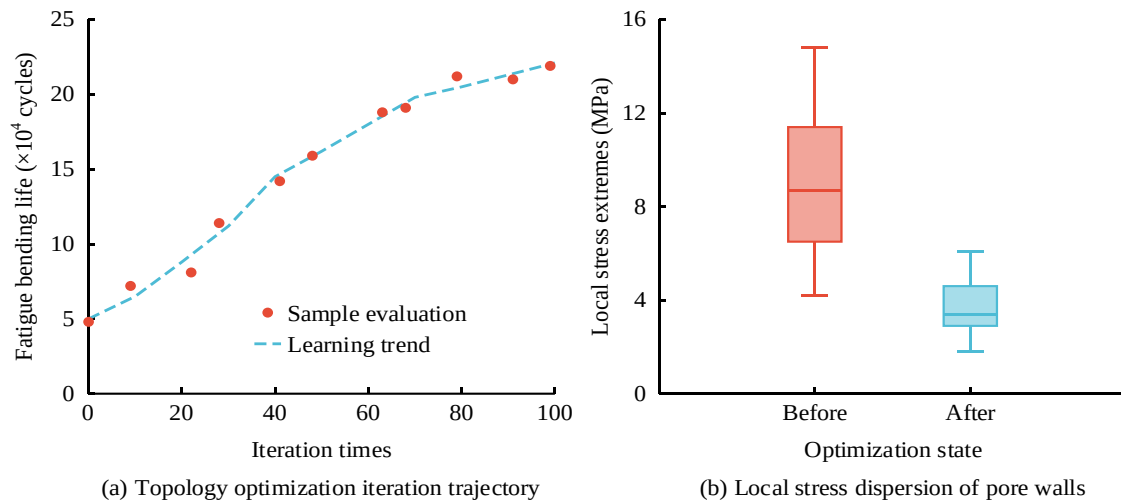


Fig. 9. Comparison of microtopology optimization trajectory and local stress dispersion based on IPM-FMD

In Figure 9(a), the topology optimization trajectory of IPM-FMD prediction feedback shows a significant nonlinear climbing trend. The initial matrix fatigue life was only 4.8×10^4 cycles. After about 80 iterations, this indicator exceeded 21.2×10^4 times and dynamically converged, verifying the efficient directional driving force of the model. As shown in Figure 9(b), before optimization, the local stress extremes of the pore wall were highly divergent, with a median of 8.7MPa and a maximum of 14.8MPa, indicating severe stress concentration. After optimization, not only did the median stress drop to 3.4MPa, but the interquartile range also narrowed significantly, with the highest extreme being controlled at 6.1MPa, which intuitively confirmed the model's powerful ability to repair microscopic defects.

3.4.2 Validation of High-Throughput Virtual Screening Performance of Massive Foaming Formulas

To verify the computational efficiency and engineering substitution potential of the IPM-FMD model in the concurrent evaluation scenario of massive recipes, experiments were carried out on six groups of representative pore gradient simulation schemes to quantitatively compare the time consumption and accuracy with traditional finite element analysis (FEA), as shown in Table 3.

In Table 3, the study quantitatively compared the screening effectiveness of the IPM-FMD model and traditional FEA on porous samples. When the porosity rises to 82.3%, the traditional FEA solution time increases to 24.6 hours, while the IPM-FMD virtual-screening time remains within 2.4 seconds, achieving an order-of-magnitude improvement in efficiency. In terms of accuracy, even in the face of extremely distorted pores, its predicted life is still highly consistent with the benchmark, and the maximum relative error is strictly controlled within 5.54%. This fully confirms that this model has extremely high engineering substitution potential in virtual concurrent screening of massive materials.

Table 3. Comparison of performance and accuracy between IPM-FMD model and traditional FEA in high-throughput simulation screening

Simulation Test Group	Equivalent Porosity (%)	Traditional FEA Time (h)	IPM-FMD Screening Time (s)	FEA Baseline Predicted Life ($\times 10^4$ cycles)	IPM-FMD Model Predicted Life ($\times 10^4$ cycles)	Absolute Relative Error (%)
Sample Group A	52.4	14.5	2.1	18.42	18.15	1.46
Sample Group B	58.7	15.2	2.2	15.75	16.02	1.71
Sample Group C	64.1	16.8	2.1	12.30	12.05	2.03
Sample Group D	71.5	18.5	2.3	8.95	9.24	3.24
Sample Group E	76.8	21.4	2.4	6.42	6.18	3.74
Sample Group F	82.3	24.6	2.4	4.15	4.38	5.54

3.4.3 Fatigue Failure Prediction and Tracking Under Complex Gait Impact Conditions

To verify the predictive robustness of the IPM-FMD model in the face of complex dynamic loads such as gait transitions, experiments quantified its dual error evolution under multi-level load transitions and tracked its long-term fatigue attenuation fit trajectory with the FEA benchmark, as shown in Figure 10.

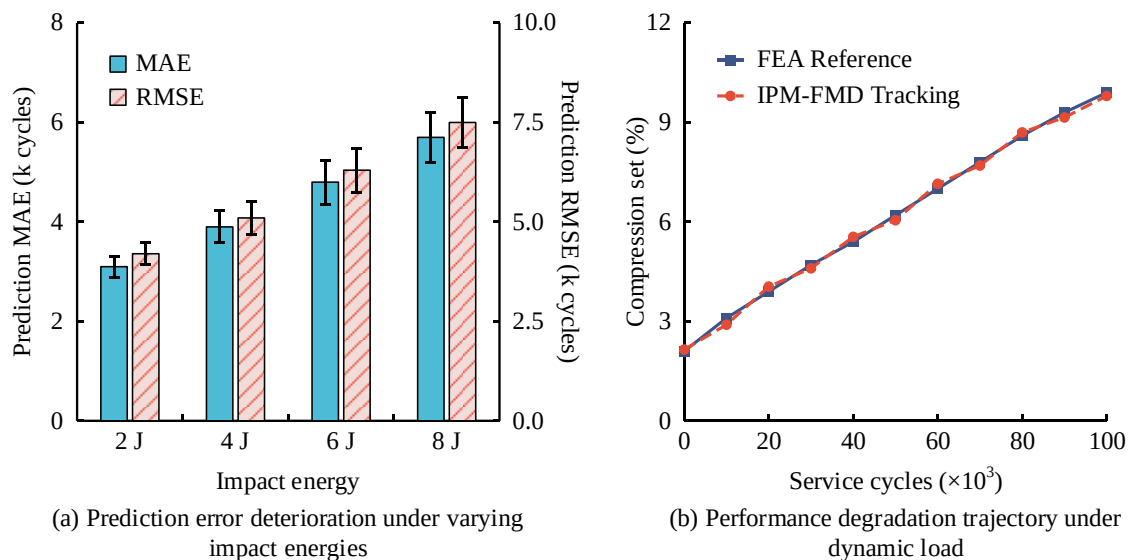


Fig. 10. Error evolution and fatigue attenuation tracking of the IPM-FMD model under variable load impact conditions

In Figure 10(a), under increasing dynamic load, the prediction error of the IPM-FMD model increases in a controllable manner. When the impact energy jumped from 2J to 8J, the maximum RMSE was still strictly controlled at 7.50 k cycles, and the maximum MAE was only 5.70 k cycles, demonstrating excellent generalization robustness in the face of complex load mutations. As shown in Figure 10(b), in one hundred thousand continuous simulation tracking, the compression deformation trajectory of the research model was highly consistent with the FEA benchmark. Even at the end of service, the predicted value (9.80%) remained close to the FEA reference value (9.90%), indicating close agreement with the numerical benchmark over the simulated loading trajectory. From an application perspective, the 32.6 ms result represents network-only batch-1 inference rather than the complete micro-CT acquisition and processing workflow. Although this computational cost supports rapid virtual material screening, it does not by itself demonstrate real-

time in-line quality control. The present evaluation also does not include real commercial footwear-foam micro-CT volumes from independent manufacturing batches. Variations in EVA or PU formulation, density, foaming and curing conditions, production batch, aging state, and loading environment may shift the morphology–durability relationship observed in the benchmark data. Accordingly, practical deployment requires material-specific calibration and external validation using independent footwear-foam batches before the model is incorporated into production screening or quality-control workflows.

4. Conclusions

To solve the problem that the mapping rules between the microscopic pore characteristics and macroscopic durability of shoe material foaming media are difficult to reveal and that traditional testing is expensive, an IPM-FMD model was constructed. This model relies on CNNs to quantify micro topology, and integrates Swin Transformer and multi-dimensional attention to accurately capture the long-range dependence of stress and purify the load-bearing skeleton, achieving high-precision nonlinear regression of macroscopic fatigue life. The results show that the median predicted RMSE is only 6.6 k cycles, and the generalization is stable under 80% extreme pore distortion. Across ten independent runs, the model achieved a mean RMSE of 6.60 ± 1.53 k cycles and a mean MAE of 4.92 ± 1.18 k cycles, while the network-only batch-1 inference latency was 32.6 ± 1.0 ms on the RTX 4090. In engineering simulation applications, model-driven micro-optimization allows the fatigue life to exceed 21.2×10^4 cycles, while the virtual-screening time remained within 2.4 s per sample, compared with up to 24.6 h for traditional finite element analysis.

Although the prediction performance is excellent, the current two-dimensional cross-section dimensionality reduction strategy is prone to losing cross-force information when dealing with complex three-dimensional connected networks. The present validation is also based on porous-media benchmark volumes and numerically generated fatigue-life references rather than independent production batches of commercial footwear foams. Differences in EVA or PU formulation, density, foaming and curing conditions, manufacturing batches, aging state, and loading environments may alter the morphology–durability relationship. Moreover, the reported 32.6 ms latency represents network-only inference rather than the complete micro-CT acquisition and preprocessing workflow. These factors limit direct extrapolation of the current results to industrial in-line quality control. In view of this limitation, follow-up research will introduce cutting-edge algorithms such as 3D point cloud reconstruction to complete the direct analytical closed loop of full 3D topology, and couple multi-modal micromechanics in-situ tracking technology to capture the microscopic stress evolution of the foamed skeleton in real time, thereby driving the intelligent prediction model to expand to more complex and realistic working conditions. Future validation will additionally incorporate real footwear-foam micro-CT data from independent manufacturing batches and examine formulation, aging, and loading variations. Attention-rollout or saliency analysis will also be introduced to relate model sensitivity to physically meaningful pore characteristics before deployment in material-screening or quality-control workflows.

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Thermo-mechanical response of fire-exposed concrete protected by intumescent coating: An equivalent-layer numerical study of coating thickness effects

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Abstract

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The thermal degradation of concrete is a result of the coupled heat penetration, temperature-dependent degradation of materials and thermally induced deformation. This paper presents an experimentally validated three-dimensional finite element study of normal strength concrete cylinders coated with different thicknesses of intumescent coatings. The temperature-displacement ABAQUS model with Concrete Damage Plasticity (CDP) and temperature-dependent material properties was validated with the experimental data of unprotected and 2 mm-coated concrete cylinders, which were exposed to 500 °C for 5400 s. The intumescent coating was represented by an equivalent-layer approach with temperature dependent thermal conductivity; its physical volumetric expansion was not explicitly modelled. A parametric study was then performed on uncoated and 1, 2 and 4mm coated cylinders at 100, 300, 500 and 700 °C. The coating thickness was found to significantly affect transient heat penetration and the subsequent mechanical response. The 4-mm coating retained 84.89% of the compressive strength at 700 °C versus 36.75% for the uncoated concrete, reducing the strength loss from 63.25% to 15.11%. Thicker coatings also reduced the heat penetration into the core, thermal induced tensile damage and displacement, proving that coating thickness is a crucial parameter to control the thermal protection and residual concrete performance.

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1. Introduction

Concrete is one of the most popular construction materials in the world due to its availability, economic efficiency, durability and capacity to withstand high compressive loads [1-3]. Its versatility, ease of casting, compatibility with reinforcement and relatively low maintenance requirements have made it a fundamental material in modern infrastructure systems [4, 5]. However, concrete, despite its good performance under normal conditions, can deteriorate significantly under extreme environmental conditions, in particular at elevated temperatures resulting from fire exposure [6, 7]. Exposure to high temperatures induces complex physical, chemical and mechanical changes in concrete. Evaporation of free and physically bound water can lead to internal pore pressure and microcracking starting at around 100°C. With increasing temperature, the concrete matrix gradually deteriorates because of the deterioration of cement hydrates and differential thermal expansion between aggregate and cement paste. This leads to an ever more pronounced decrease in compressive strength, tensile strength and elastic modulus, especially in the temperature range of approximately 300-600°C [8]. Reaching temperatures of 800°C can cause severe cracking, loss of cohesion and surface spalling, accelerating the

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deterioration and failure of structures [9]. Experimental and numerical studies have shown that temperature-dependent degradation significantly influences the structural response of concrete subjected to fire conditions [10, 11]. Numerical modelling has become an important tool for the investigation of the thermal and mechanical response of concrete at elevated temperatures. The finite element method through advanced software such as ABAQUS enables especially to simulate the temperature distribution, material degradation and structural response to thermal loading [12, 13]. The Concrete Damage Plasticity (CDP) model is one of the most used among the available constitutive models to describe the nonlinear behavior of concrete, including tensile cracking, compressive crushing, stiffness degradation and damage evolution [14, 15]. Response of concrete under elevated temperature has been simulated using coupled thermo-mechanical finite element analyses in previous studies [16-18]. However, the interplay of heat transfer, temperature-dependent material degradation and thermally induced mechanical response is still highly complex and attracts a lot of research attention. As a result, many passive fire protection systems have been developed to improve the fire resistance of concrete structures. Among these systems, intumescent coatings have attracted significant attention due to their ability to swell at high temperatures and form a thermal insulating protective layer [18, 19]. This extra layer effectively reduces heat transfer to the concrete substrate and retards the temperature increase in the member thus mitigating thermally induced deterioration [20]. The effectiveness of fire-protection coatings to preserve the post-fire performance of concrete has also been successfully validated through recent experimental studies. Li et al. observed a significant improvement in the residual compressive and splitting tensile strength of fireproof coated concrete exposed to temperatures of up to 800°C due to less heat penetration into the concrete [21]. Li et al. in a later study showed that fireproof coatings significantly improved the durability of concrete after fire by reducing water absorption and improving carbonation resistance after exposure to high temperatures [22]. More recently, Fu et al. provided a comprehensive review on the recent advances in intumescent coating technology and pointed out that coating formulation, thermal insulation efficiency and heat-transfer mitigation are still the most important factors deciding the fire-protection performance [23]. However, the authors also pointed out the need for additional studies on the coupled thermal and mechanical behavior of protected concrete members at high temperatures. In addition, Chun et al. showed that the residual mechanical performance of fire damaged concrete is mainly dependent on the degree of thermal exposure and the consequent degradation of the cementitious matrix [24]. Moreover, the comprehensive review performed by Ghiji et al. [24] showed that, although the promising thermal insulation properties of passive fire-protection systems, systematic studies on the coupled thermo-mechanical behavior of protected concrete are relatively scarce [18].

In finite element analyses, the complex expansion and progressive charring of intumescent coatings can be modelled explicitly by the evolving coating geometry and temperature dependent material behavior, or more simply using an equivalent-layer approach. In the latter approach, the thermal protection of the thermally transformed coating is represented by effective temperature-dependent thermal properties while the coating geometry is maintained. The method reduces the complexity and computational requirements of the numerical model and allows the practical evaluation of the heat transfer through the protected system. However, it does not explicitly reproduce the evolving char thickness, volumetric expansion, cracking, shrinkage nor possible local debonding of the coating during heating. Predictions obtained with an equivalent-layer representation should then be interpreted mainly in terms of the effective thermal protection transferred to the concrete rather than a direct simulation of the physical intumescence process. However, although these contributions are important, the current research has mainly focused on the thermal insulation performance or residual mechanical properties separately. Very few studies have considered the transient temperature evolution, thermally induced displacement, tensile damage evolution and residual compressive strength in a unified experimentally validated thermo-mechanical framework, particularly for thin intumescent coating systems. Therefore, the novelty of the present study is the development and experimental validation of a sequentially coupled thermo-mechanical finite element model able to quantitatively evaluate the effect of the intumescent thin coating thickness of 0, 1, 2 and 4 mm on the total thermal and mechanical behavior of normal strength concrete at elevated temperatures ranging from 100 to 700°C. The present work provides a more comprehensive evaluation of coating effectiveness than those

reported in previous investigations, by combining the thermal behavior, deformation, damage evolution and residual compressive performance in one framework.

2. Methodology and Numerical Modeling

First, a detailed sequentially coupled thermo-mechanical finite element (FE) model was developed in ABAQUS to simulate the transient thermal and post-fire mechanical behavior of concrete cylinders protected with intumescent coatings. A targeted experimental program was undertaken on five concrete cylinders to calibrate and verify the predictive accuracy of the Concrete Damage Plasticity (CDP) model. Three specimens were tested in uniaxial compression at room temperature to determine the baseline mechanical properties and calibrate the CDP parameters. The remaining two specimens (uncoated and coated) were subjected to controlled thermal exposure for numerical validation of temperature fields and residual strength. The numerical model was then cooled to ambient temperature and subjected to displacement-controlled compression to simulate the post-fire testing procedure following thermal loading. Mesh convergence study was performed to ensure the accuracy of the spatial discretization. Finally, the FE model validated by the experiments was used to perform a comprehensive parametric study on the combined effects of temperature of exposure and thickness of the intumescent coating. The overall research flowchart is summarized in Figure 1.

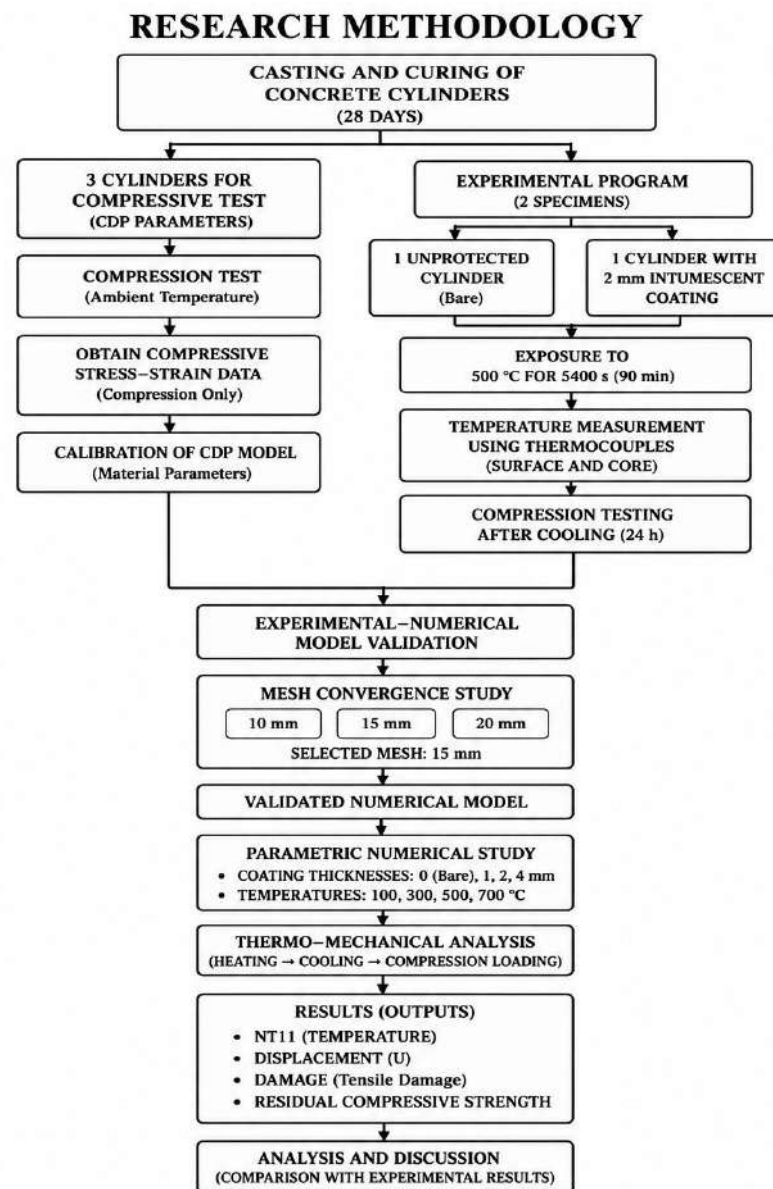


Fig. 1. Flowchart of research methodology

2.1 Finite Element Model Formulation

2.1.1 Material Model

The Concrete Damaged Plasticity (CDP) model implemented in Abaqus was used to model the concrete mechanical behavior. This constitutive model was chosen because it is able to describe the nonlinear behavior of concrete in tension (cracking) and compressive (crushing) regimes, taking into account the progressive loss of elastic stiffness due to damage. The theoretical basis of the model is based on the plastic-damage formulation proposed by Lubliner et al.[26] and its subsequent development for concrete structures by Lee and Fenves [27]. The implementation used in Abaqus provides separate tensile and compressive damage variables and allows to define stiffness degradation and recovery during load reversal [28, 29]. The corresponding damage variables, (d_t) and (d_c), vary between zero and unity, where zero indicates an undamaged material state and increasing values indicate progressive stiffness and strength degradation. The model also considers a partial recovery of stiffness during unloading and load reversal, which makes the nonlinear response of concrete more realistic. The required stress–inelastic strain relationship for the compressive response was obtained from the experimentally determined compressive stress–strain curve. The elastic strain component was subtracted from the total strain to get the inelastic strain [30]:

$$\varepsilon_c^{\sim in} = \varepsilon_c - \varepsilon_c^{el} \quad (1)$$

where:

$$\varepsilon_c^{el} = \sigma_c / E_0 \quad (2)$$

In this case $\varepsilon_c^{\sim in}$ is the inelastic compressive strain, ε_c is the total compressive strain, σ_c is the compressive stress, and E_0 is the initial elastic modulus of concrete.

The compressive damage variable was introduced only on the post-peak softening branch where the degradation of stiffness is observed after the peak compressive stress. Earlier it was assumed that the material before its softening was intact ($d_c=0$). The compressive damage variable for the post-peak response was calculated as:

$$d_c = \begin{cases} 0, & \varepsilon_c \leq \varepsilon_{c,peak} \\ 1 - \frac{\sigma_c}{f'_c}, & \varepsilon_c > \varepsilon_{c,peak} \end{cases} \quad (3)$$

where (d_c) is the compressive damage variable, (σ_c) is the post-peak compressive stress, and (f'_c) is the peak compressive stress. Thus, at the beginning of the post-peak softening, (d_c) is zero and increases gradually as the compressive stress decreases.

The cracking strain was also used to characterize the tensile response, which was calculated by subtracting the elastic strain from the total tensile strain:

$$\varepsilon_t^{\sim ck} = \varepsilon_t - \varepsilon_t^{el} \quad (4)$$

with:

$$\varepsilon_t^{el} = \sigma_t / E_0 \quad (5)$$

The tensile damage variable was also induced by the occurrence of tensile softening. The material was regarded as undamaged ($d_t=0$) prior to the peak tensile stress. The post-peak softening response was defined by the tensile damage variable as

$$d_t = \begin{cases} 0, & \varepsilon_t \leq \varepsilon_{t,peak} \\ 1 - \frac{\sigma_t}{f_t}, & \varepsilon_t > \varepsilon_{t,peak} \end{cases} \quad (6)$$

(σ_t) is the post peak tensile stress, (f_t) is the peak tensile stress and (d_t) is the tensile damage variable. Accordingly, is zero at the start of tensile softening and approaches unity with increasing loss in tensile load carrying capacity.

The concrete stress–strain behavior under compression and tension and the related damage variables were implemented in Abaqus from the experimental results and the inelastic and cracking strains calculated by Equations (1) – (6). The obtained constitutive relations were used to define the compressive and tensile behavior of concrete in the CDP framework. The main parameters used for the Concrete Damaged Plasticity model are listed in Table 1. The flow potential eccentricity, the ratio of biaxial to uniaxial compressive strength and the ratio of second stress invariant have been chosen according to the default values given in Abaqus formulation. The dilation angle and viscosity parameter were selected based on the values commonly used for normal strength concrete in the nonlinear finite element analyses. The same parameters were used in the numerical simulations to provide a unified constitutive representation of the concrete response.

Table 1. Parameters adopted for the concrete damaged plasticity model

Parameter	Value
Dilation angle, ψ	37°
Flow potential eccentricity, ϵ	0.10
f_{b0}/f_{c0}	1.16
K	0.667
Viscosity parameter, (μ)	0.0005

The CDP flow-potential and yield-surface parameters, such as dilation angle, eccentricity, f_{b0}/f_{c0} and K_c , were kept unchanged during the thermal analyses. The adopted values ($\psi = 37^\circ$, $\epsilon = 0.10$, $f_{b0}/f_{c0} = 1.16$, and $K_c = 0.667$) are in accordance with those commonly adopted in previous ABAQUS concrete simulations [31, 32]. In particular previous CDP studies have used a dilation angle of 37° together with $\epsilon = 0.10$, $f_{b0}/f_{c0} = 1.16$, and $K_c \approx 0.67$. The values of 0.10, 1.16 and 0.667 for eccentricity, f_{b0}/f_{c0} and K_c respectively are commonly used standard CDP parameters [32]. The main parts of the present model for describing the effects of elevated temperature on the concrete response were the temperature dependent mechanical properties and the constitutive stress–strain relationships. In an effort to simplify the modelling in line with established numerical practice, the CDP flow-potential and yield-surface parameters were therefore kept as temperature-independent parameters. The overall adequacy of the constitutive representation was assessed by comparing the numerical predictions with the experimental thermal and residual strength results.

The CDP yield and flow parameters were kept constant during the thermal analyses, while the temperature dependency of the concrete mechanical response was represented by the temperature-dependent stress–strain relationships and damage evolution. The importance of the temperature-dependent calibration of damage-plasticity formulations for concrete at elevated temperatures has been evidenced by experimental studies [33]. In the present study, constant CDP flow parameters were chosen to reduce the number of constitutive parameters that vary independently and to keep the formulation consistent during the parametric analysis. This assumption may affect the predicted deformation and damage response at extreme temperatures. The present study did not include a dedicated sensitivity analysis of the temperature-dependent CDP flow parameters, therefore, the mechanical predictions at the highest exposure temperatures must be interpreted with this modelling limitation in mind. The sensitivity of the predicted response to the temperature dependence of CDP parameters should be investigated in future work.

The numerical model was defined to account for the changes occurring during thermal exposure since the concrete specimens were exposed to elevated temperatures, the thermo-mechanical properties of concrete were defined in the numerical model. The thermal behavior of concrete is determined by its thermal conductivity, specific heat capacity, density and coefficient of thermal expansion, while the mechanical behavior is greatly affected by the degradation of elastic stiffness

with temperature. So, the thermal and mechanical relevant properties were taken into consideration to define the thermo-mechanical behavior of the concrete specimen. The mechanical properties of concrete and thermal expansion were temperature-dependent as specified in EN 1992-1-2 and literature. In the numerical model used here a modelling simplification was to assign constant representative values of 1.80 W/m·K for the thermal conductivity and 900 J/kg·K for the specific heat [34]. and the detailed review and experimental data presented by Kodur (2014) [3]. These references indicate that concrete undergoes significant changes in thermal and mechanical behavior with an increase in temperature. In particular, the elastic modulus decreases considerably with increasing temperature, while the thermal expansion increases generally. The thermal conductivity, specific heat capacity and density may also vary with temperature due to moisture loss and temperature-induced changes within the cementitious matrix. In the present study, the temperature dependence was mainly attributed to variations in Young's modulus and coefficient of thermal expansion, whereas thermal conductivity, specific heat capacity, density and Poisson's ratio were kept constant in the simulations. The adopted values were chosen to represent normal weight, normal strength concrete with a nominal compressive strength of about 30 MPa and to be consistent with the concrete used in the experimental program. Table 2 presents the thermo-mechanical properties of concrete used. The temperature-dependent Young's modulus was used to represent the gradual deterioration of concrete stiffness under heating, and the temperature-dependent coefficient of thermal expansion was used to represent the generation of thermal strains. Other properties were constant as defined in the numerical model. These properties were then used to simulate the transient thermal response and the subsequent thermo-mechanical behavior of the concrete specimen.

Table 2. Temperature-dependent thermal properties of concrete

Temperature (°C)	Thermal conductivity, (k) (W/m·K)	Specific heat, (cp) (J/kg·K)	Density, (ρ) (kg/m ³)	Thermal expansion coefficient, (α) (1/K)	Young's modulus, (E) (MPa)	Poisson's ratio, (ν)
20	1.80	900	2400	1.00×10^{-5}	29,791	0.18
100	1.80	900	2400	1.00×10^{-5}	28,200	0.18
200	1.80	900	2400	1.05×10^{-5}	25,200	0.18
300	1.80	900	2400	1.10×10^{-5}	20,700	0.18
400	1.80	900	2400	1.15×10^{-5}	16,300	0.18
500	1.80	900	2400	1.20×10^{-5}	11,900	0.18
600	1.80	900	2400	1.25×10^{-5}	7,600	0.18
700	1.80	900	2400	1.30×10^{-5}	3,700	0.18
800	1.80	900	2400	1.35×10^{-5}	1,500	0.18

The modelling simplification is the assumption of constant thermal conductivity and specific heat over the investigated temperature range. Although these properties are known to be temperature dependent, for the purpose of keeping a consistent definition of thermal properties within the present parametric study, representative constant values have been used. The validity of this simplification was checked by comparison with the experimental temperature-time response at 500°C for 5400 s. The good agreement between the experiments and the numerical results supports the use of the model for a comparative evaluation of the coating configurations investigated in the present study. This validation should not be interpreted as a general predictive accuracy for all temperatures or fire situations. The use of constant thermal properties can affect the absolute prediction of temperatures outside the validated condition. Therefore, future studies should fully include temperature dependent thermal conductivity and specific heat, as well as moisture effects, especially when extending the model to larger temperature ranges or structural-scale fire scenarios.

The intumescent coating was considered as a homogeneous protective material surrounding the concrete specimen. The coating layer was treated as a thermal insulation layer, whose main function was to slow down the rate of heat transfer from the external heating environment to the concrete surface and thereby delay the propagation of heat towards the specimen core. Thermal

and mechanical properties were assigned to the coating to enable its representation in the coupled thermo-mechanical finite element analysis. The thermal conductivity of intumescent coating was considered as a temperature dependent property and is presented in Table 3. The thermal conductivity adopted decreased remarkably with the increase of temperature, which was the effective reduction in heat-transfer capability of the thermal transformation of intumescent protective material. In the present model the behavior was modelled using an equivalent-layer approach; the coating geometry was kept the same and the physical volumetric expansion of the coating was not explicitly simulated. The specific heat capacity and density of the coating in the adopted mm–N–s–°C unit system were 1.47×10^9 and 1.19×10^{-9} , respectively. The elastic response of the coating was characterized by a Young's modulus of 2000 MPa and a Poisson's ratio of 0.30. The modelled coating should therefore be regarded as an equivalent thermal-protection layer, which represents the effective insulating response of the thermally transformed coating, and not as an explicit simulation of the physical intumescence process. Differences between the numerical and physical responses might be attributed to phenomena such as changing char thickness, volumetric expansion, cracking, shrinkage and local debonding, which were not explicitly modelled. The experimental database available does not allow to isolate the individual contribution of these mechanisms and a reliable independent numerical error margin for the omitted expansion process could not be estimated. Instead, the accuracy of the adopted equivalent-layer representation was directly validated using the available experimental temperature–time response.

Table 3. Thermo-mechanical properties of the intumescent coating

Temperature (°C)	Thermal conductivity, (k)	Specific heat, (cp) N.mm/(tone·°C)	Density, (ρ) tone/mm ³	Young's modulus, (E) (MPa)	Poisson's ratio, (ν)
20	0.149294	1.47×10^9	1.19×10^{-9}	2000	0.30
100	0.125649	1.47×10^9	1.19×10^{-9}	2000	0.30
300	0.066537	1.47×10^9	1.19×10^{-9}	2000	0.30
400	0.007425	1.47×10^9	1.19×10^{-9}	2000	0.30
500	0.007277	1.47×10^9	1.19×10^{-9}	2000	0.30

Note: All coating properties were characterized in the consistent mm–N–s–°C unit system used in ABAQUS model. So, the consistent mass unit is the tone. So, the coating density of 1.19×10^{-9} is equal to 1190 kg/m^3 , and the specific heat of $1.47 \times 10^9 \text{ N}\cdot\text{mm}/(\text{tonne}\cdot^\circ\text{C})$ is equal to $1470 \text{ J}/(\text{kg}\cdot^\circ\text{C})$. These values therefore correspond to the usual physical properties of the coating in the consistent unit system adopted in the numerical analysis.

2.1.2 Geometry, Mesh, Boundary Conditions, Thermal Loading, and Analysis Procedure

The numerical model consisted of a concrete cylinder of 150 mm diameter and 300 mm height. For the coated configuration, the 2 mm thick intumescent coating was geometrically modelled as three separate components, a hollow cylindrical shell enclosing the concrete cylinder and two solid circular end caps covering the upper and lower surfaces of the concrete specimen. This arrangement permitted the maintenance of a constant coating enclosure about the concrete cylinder including the lateral and end surfaces. Figure 2 presents the separate components of the concrete cylinder and the intumescent coating. Note that in the experimental program the intumescent coating was applied only on the external cylindrical surface, while the numerical model considered coating also on the upper and lower end surfaces. The numerical end caps were introduced as a geometric idealization to provide a continuous protective enclosure and a constant representation of the thermal boundary. Thus, the numerical coating configuration does not exactly reproduce the experimental coating geometry at the ends of the specimen, and this difference is acknowledged as a limitation in the interpretation of the experimental-numerical comparison.

The overall specimen configuration was comprised of the concrete cylinder and coating components concentrically placed. The computational domain was discretized with three-dimensional 8-node coupled temperature-displacement brick elements (C3D8T). To assess the impact of mesh refinement on the predicted thermal response, a mesh-sensitivity study was conducted using global element sizes of 10, 15, and 20 mm.

The temperature-time histories for the temperature histories are given in Figure 3. During the heating phase, a good consistency was found between the three discretization levels, which suggests that the predicted core-temperature response was not significantly affected by further mesh refinement in the range investigated.

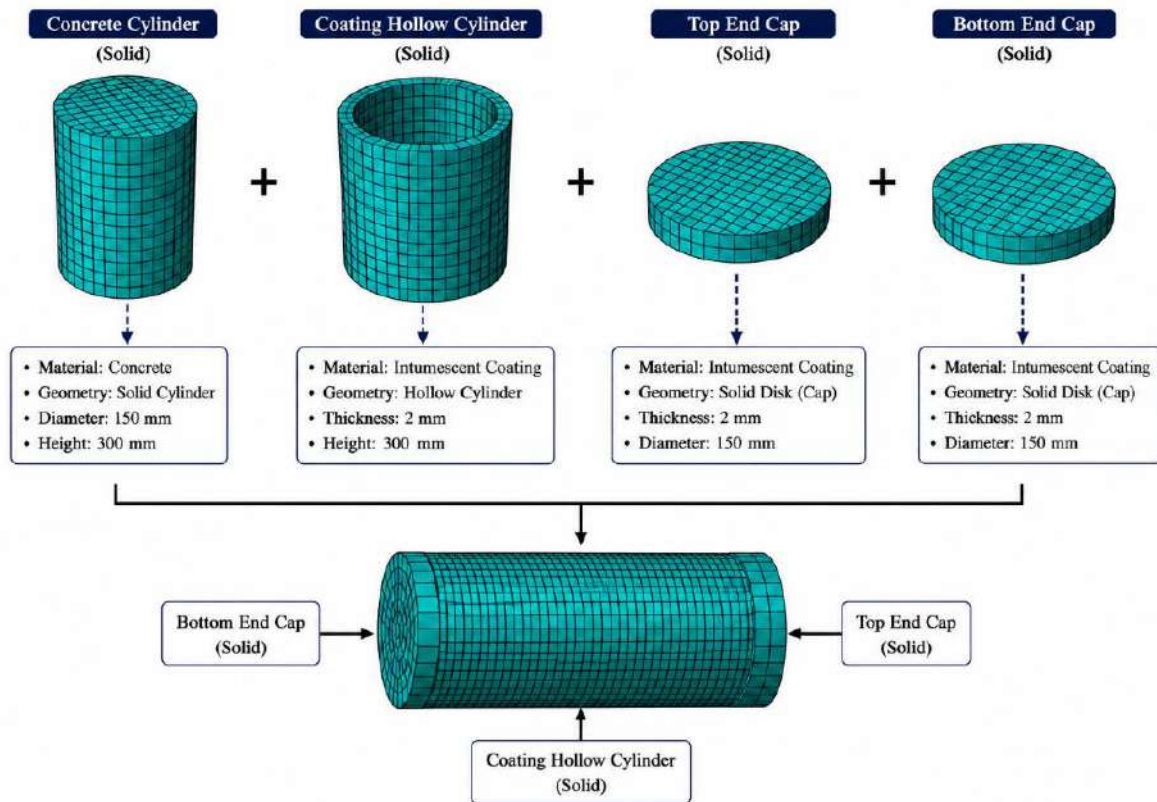


Fig. 2. Geometry and mesh configuration of the concrete cylinder and intumescent coating system

Hence, a global mesh size of 15 mm was chosen as a good compromise between numerical accuracy and computational cost. The concrete cylinder was modelled by 1,920 solid elements and 2,373 nodes with the discretization used. The 2-mm coating was discretized separately, with two elements provided through its thickness, since the coating thickness was much smaller than the global concrete mesh size. The hollow cylindrical coating had 1282 elements and 2016 nodes, and the top and bottom end caps each had 246 elements and 420 nodes. So, the whole 2 mm coating system was composed of 1774 elements and 2856 nodes. This local through-thickness refinement was used in an effort to obtain an adequate discretization of the thin protective layer while keeping the 15-mm global mesh over the larger dimensions of the specimen. The mesh-sensitivity assessment was focused on the core-temperature response as the heat penetration through the specimen governs the subsequent thermo-mechanical response. Table 4 summarizes mesh discretization details.

The bottom surface of the concrete cylinder was constrained against rigid body motion for the thermo-mechanical analyses. The axial displacement was prescribed to the upper loading surface in the subsequent compression stage. The uncoated and coated specimens were subjected to the same prescribed furnace temperature-time history during thermal exposure, from an initial ambient temperature of 20°C up to the target exposure temperature. The prescribed temperature was representative of the environment surrounding the furnace and was not imposed directly as the temperature of concrete or coating surface. The heat transfer from the furnace environment to the specimen was simulated by film convection and surface radiation. These thermal interactions were applied on the outer surface of the intumescent coating for the coated specimen and on the external concrete surface for the uncoated specimen. The heat was then transferred through the intumescent coating to the underlying concrete. Thus, the different surface temperature histories

obtained for the uncoated and coated specimens are a consequence of the thermal response of the respective configurations and not of the different imposed thermal boundary conditions. Both configurations were intentionally exposed to the same furnace temperature-time history and duration of exposure to simulate a common external heating environment and to facilitate a direct evaluation of the thermal protection offered by the intumescent coating.

Table 4. Mesh discretization details for concrete cylinder and 2 mm coating system

Component	Mesh specification	Elements	Nodes
Concrete cylinder	15-mm global mesh	1,920	2,373
Hollow cylindrical coating	2 elements through 2-mm thickness	1,282	2,016
Top coating end cap	2 elements through 2-mm thickness	246	420
Bottom coating end cap	2 elements through 2-mm thickness	246	420
Complete coating system	—	1,774	2,856

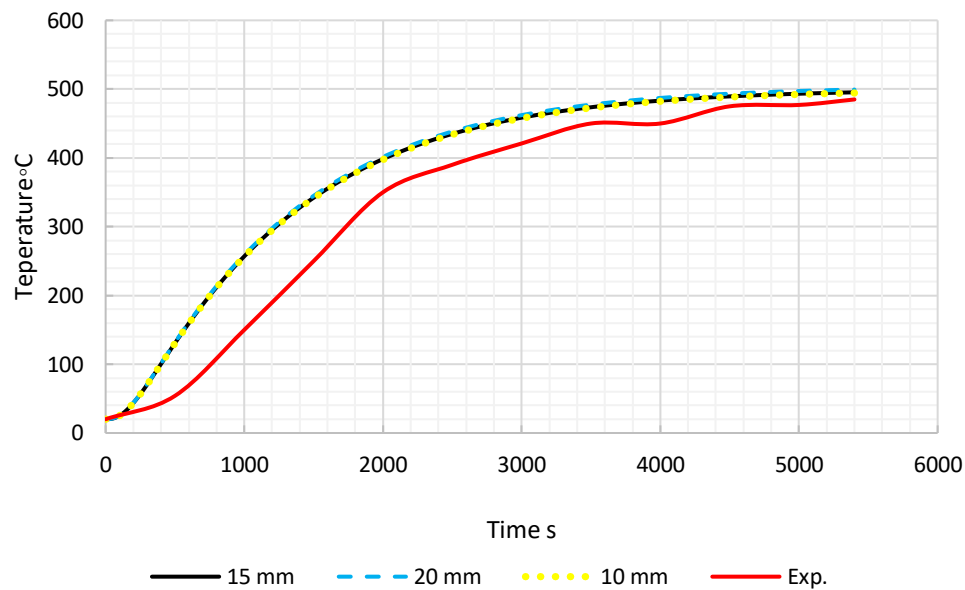


Fig. 3. Effect of mesh size on the predicted core temperature history of the coated concrete cylinder during exposure to 500°C

To reproduce the experimental heating, cooling and post-fire compression procedures, the numerical simulations were performed by a series of analysis steps. A heat transfer analysis was first performed to simulate the thermal exposure at 500°C for 5400 s (90 min) and to obtain the transient temperature distribution in the uncoated and coated concrete cylinders. For thermal validation, the predicted surface and core temperatures were compared with the experimental thermocouple data. Then, a coupled temperature-displacement sequential analysis was performed by using the validated thermal conditions and temperature-dependent material properties to simulate the thermo-mechanical response during heating including thermal expansion, stress, displacement and damage evolution. Then the specimens were numerically cooled from high temperature to 20 °C after the heating stage to simulate the experimental cooling procedure, where the specimens were cooled down in ambient air for 24 h prior to the compression testing. The thermal contraction and the mechanical and damage states associated with it were maintained at the end of the cooling stage and used as initial condition for the following residual strength simulation. Finally, a Static General compression with displacement controlled axial loading was performed to simulate the post-fire compression test. The numerical residual compressive strength was regarded as the maximum compressive stress obtained from the numerical stress-strain response using the resulting reaction force and axial displacement. The numerical results were then compared to the corresponding experimental measurements for uncoated and 2 mm intumescent coated specimens.

3. Experimental Validation Program

The experimental program was performed in order to define the mechanical behavior of normal strength concrete and to provide experimental data for the calibration and validation of the numerical model. Five concrete cylindrical specimens were cast with a diameter of 150 mm and a height of 300 mm (Figure 4b and c). The concrete was designed to achieve a nominal compressive strength of about 30 MPa after 28 days of curing. The concrete mix was made with ordinary Portland cement, clean and well graded natural river sand as fine aggregate, crushed gravel as coarse aggregate and potable water (Figure 4a). The fine aggregate had a maximum size of 4.75 mm, a specific gravity of 2.65 and a fineness modulus between 2.6 and 2.9. The coarse aggregate had a nominal maximum size of 10 mm and a specific gravity of 2.70. The physical and chemical properties of the Ordinary Portland cement used for the study are shown in Tables 5 and 6. The specimens were cured for 28 days under normal laboratory conditions after casting. After the curing period the five specimens were divided according to the intended experimental purpose. Three specimens were used to characterize the mechanical behavior of the concrete at the ambient temperature while the other two specimens were used for the thermal exposure and the experimental validation program.

Table 5. Physical properties of ordinary Portland cement (OPC)

Property	Value
Cement Type	Ordinary Portland Cement (OPC)
Standard	ASTM C150 Type I
Specific Gravity	3.15
Fineness (Blaine)	340 m ² /kg
Initial Setting Time	120 min
Final Setting Time	210 min

Table 6. Chemical properties of ordinary Portland cement (OPC)

Chemical Compound	Content (%)
CaO	63.5
SiO ₂	21.2
Al ₂ O ₃	5.41
Fe ₂ O ₃	3.30
MgO	2.13
SO ₃	2.71
Na ₂ O + K ₂ O	0.82
LOI	1.53

Three concrete cylinders (150 mm x 300 mm) were tested in uniaxial compression after 28 days of water curing as shown in Figure 4f according to ASTM C39/C39 [25]. The stress–strain relationship and the compressive strength of the concrete were obtained by continuously recording the applied load and the corresponding deformation during the tests. The average compressive strength of the three specimens was around 30 MPa, which confirms the concrete produced was in the target normal-strength grade. The experimental stress-strain response and compressive strength were then used to characterize the mechanical behavior of concrete and to define the material parameters required for the Concrete Damage Plasticity (CDP) model implemented in the software ABAQUS.

The remaining two concrete cylinders were used to investigate the thermal response and residual mechanical behavior of concrete after elevated temperature exposure. One specimen was left uncoated and used as a reference specimen, while the second specimen was coated with an intumescent coating of 2 mm thickness applied uniformly on the external cylindrical surface (Figure 4d). The intumescent coating used in this study had a density of 1.34 g/cm³ and a high-volume solids content of approximately 96%, which contributes to minimizing shrinkage during curing. The coating also exhibited a relatively high viscosity of approximately 35,000 cP, allowing

effective application on vertical surfaces with limited sagging. In addition, its volatile organic compound (VOC) content was less than 50 g/L, indicating its suitability for practical applications. The concrete surface was cleaned prior to application of the coating to remove dust, loose particles and other contaminants that could negatively influence adhesion. Surface defects (cracks and voids) were repaired to create a relatively uniform substrate. Then, an epoxy-based primer was added to improve the bond between the intumescent coating and the concrete substrate, and to reduce surface porosity. The primer was dried in ambient laboratory conditions before the application of the protective coating. Then, the intumescent coating was uniformly sprayed on the external cylindrical surface of the selected specimen to provide a nominal thickness of 2 mm. After application, the coated specimen was left to undergo the required drying and curing period under room temperature conditions to ensure sufficient adhesion and development of coating properties before thermal exposure. The uncoated and coated specimens were then exposed to a temperature of 500°C for 5400 s (90 min) under the same heating conditions (Figure 4e). This experimental setup enabled a direct comparison between the uncoated concrete (reference condition) and the concrete coated by the 2 mm intumescent coating, so that the effect of the protective layer on the heat penetration and residual mechanical behavior of the concrete could be evaluated.

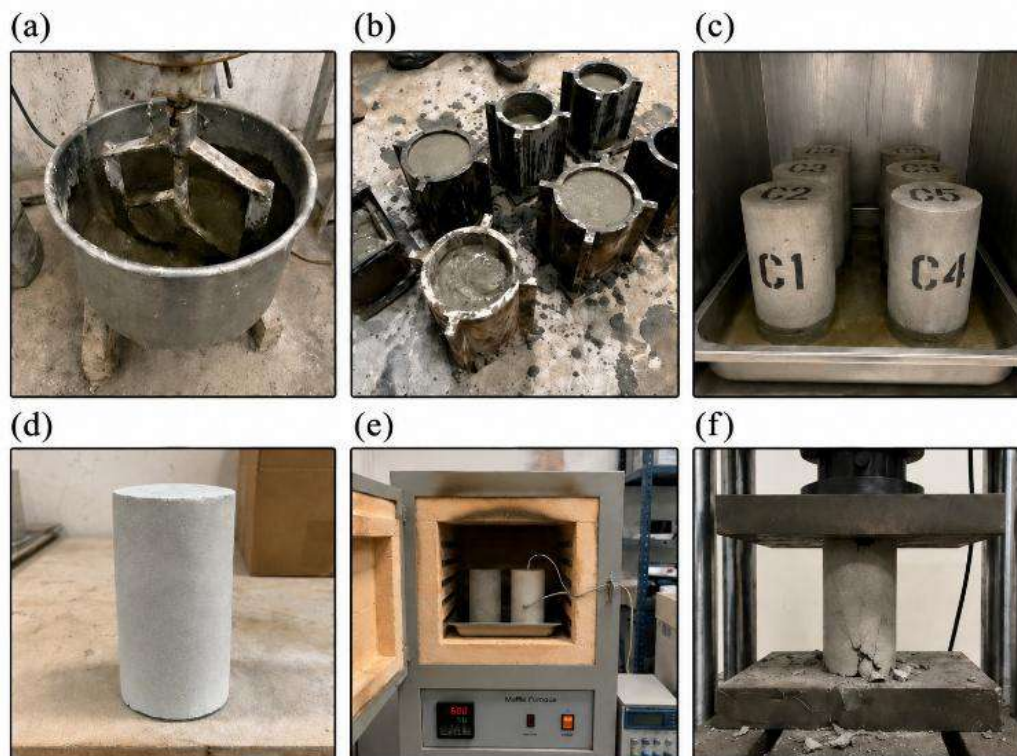


Fig. 4. (a) Concrete mixture preparation, (b) Casting of cylindrical specimens, (c) Curing of specimens, (d) Application of intumescent coating (2 mm), (e) Thermal exposure in furnace, (f) Compression testing after cooling

The temperature development in the concrete specimens during the thermal exposure was recorded by thermocouples positioned at two representative locations, i.e., in the near-surface region of the concrete and at the core of the specimen (Figure 4e). The exact depth of the near-surface thermocouple embedment was not documented during the specimen preparation, so this measurement is referred to as the near-surface temperature instead of an estimated distance from the exposed surface for the duration of this study. A core thermocouple was used to monitor the heat penetration in the interior of the concrete sample. The temperature profiles were continuously measured during the heating period for the uncoated and 2 mm-coated specimens. The measured temperature-time histories provided direct experimental information on the rate of heat penetration into the concrete and the thermal protection provided by the intumescent coating. These experimental temperature histories were then compared with the corresponding temperature predictions from the ABAQUS finite element model. The specimens were allowed to cool down to room temperature after the completion of the 90-min thermal exposure and the

residual mechanical tests were performed. Both the uncoated and 2 mm-coated specimens were then tested for axial compression. Residual compression test was conducted to assess the effect of thermal exposure on the remaining mechanical capacity of the concrete and to compare the performance of the coated and uncoated specimens. The measured mechanical response after exposure was then compared with the corresponding numerical predictions from the coupled thermo-mechanical ABAQUS analysis. The measured experimental and predicted numerical responses were compared to evaluate the reliability of the developed finite element model.

3.1 Model Validation Results and Comparison

The compressive stress-strain responses of three concrete cylinders tested at ambient temperature are shown in Figure 5. The three specimens exhibited similar stress-strain behavior with peak compressive stresses of about 30 MPa. The corresponding peak strains were in the range of 0.0015–0.0016. The stress of all the specimens decreased gradually with the increase of strain after the peak stress. The three stress-strain curves agree excellently and indicate a consistent experimental response. The stress-strain data obtained from the tests were used for the calibration of the Concrete Damage Plasticity (CDP) model in the numerical analysis.

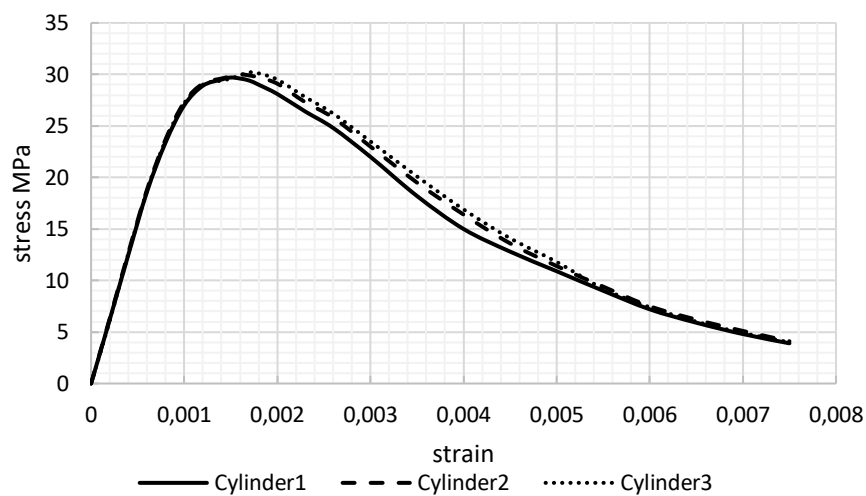


Fig. 5. Experimental compressive stress-strain curves of the three concrete cylinders at ambient temperature

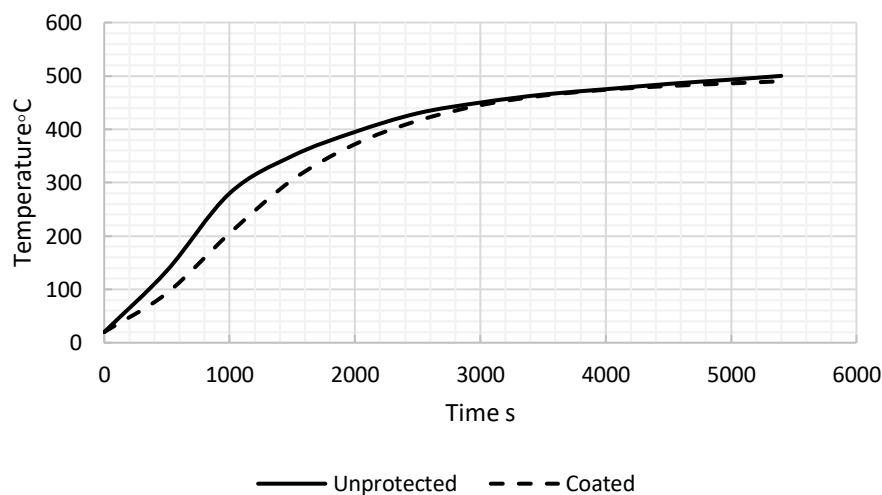


Fig. 6. Experimental surface temperature histories of the uncoated and 2 mm intumescent-coated concrete specimens during exposure to 500°C

The experimental temperature-time histories of the uncoated and 2-mm intumescent-coated concrete specimens are shown in Figures 6 and 7. Temperatures close to the exposed surface are shown in Figure 6. At 1500 s, the surface temperatures of the uncoated and coated specimens were

about 350 and 305°C, respectively, a difference of about 45°C (12.9%). The respective temperatures at 2000 s were ~395 °C and ~372 °C with a difference of ~23 °C (5.8%). The surface temperatures at 3000 s were about 450°C and 445°C, respectively. The end-of-exposure surface temperatures of the uncoated specimen and the coated specimen were about 500°C and 490°C, respectively, at the end of the 5400 s exposure period, indicating a difference of about 10°C (2.0%). In particular, small local fluctuations in the experimental temperature histories were observed during the latter part of the heating period.

Figure 7 shows the temperature histories at the geometric center of the concrete cylinders. At 1500 s, the core temperatures of the uncoated and coated samples were about 338 °C and 246 °C respectively, yielding a difference of about 92 °C (27.2%). The corresponding temperatures were approximately 393°C and 325°C at approximately 2100 s, with a difference of approximately 68°C (17.3%). The core temperatures after approximately 3000 s were approximately 430°C and 421°C, respectively, so the difference was approximately 9°C (2.1%). After the 5400 s exposure, the core temperatures of the uncoated and coated specimens were around 500°C and 485°C, respectively. The temperature difference at the end of the 5400 s exposure was 15 °C, corresponding to a reduction of 3.0%.

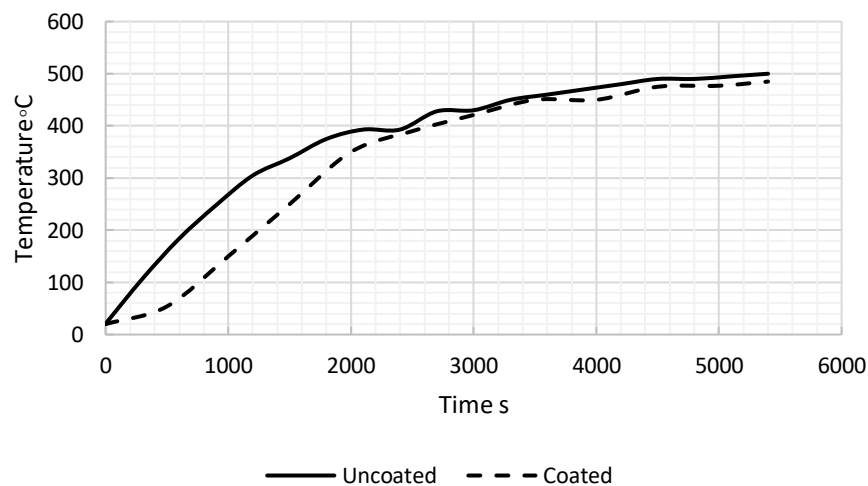


Fig. 7. Experimental core temperature histories of the uncoated and 2 mm intumescent-coated concrete specimens during exposure to 500°C

Table 7. Quantitative comparison of experimental temperatures between uncoated and coated specimens

Time (s)	Uncoated Surface (°C)	Coated Surface (°C)	Surface Reduction (°C)	Surface Reduction (%)	Uncoated Core (°C)	Coated Core (°C)	Core Reduction (°C)	Core Reduction (%)
1000	280	205	75	26.8	184	148	36	19.6
1500	350	305	45	12.9	338	246	92	27.2
2000	395	372	23	5.8	393	325	68	17.3
3000	450	445	5	1.1	430	421	9	2.1
4000	475	474	1	0.2	463	463	0	0.0
5000	493	486	7	1.4	495	478	17	3.4
5400	500	490	10	2.0	500	485	15	3

Table 7. Summary of main experimental temperature differences between uncoated and coated specimens. The maximum temperature difference was 92 °C (27.2%) at the concrete core at 1500 s. After the 5400 s exposure, the temperature difference between the uncoated and coated specimens was about 10 °C in the near-surface region and 15 °C (3.0%) in the core. The gradual decrease of the core-temperature difference with increasing exposure time can be explained by the transient nature of the heat transfer through the protective layer. In the early and middle stages of heating, the coating slowed the heat transfer into the concrete, resulting in a relatively large

temperature difference between the coated and uncoated specimens. With longer exposure the heat builds up in the coated system and continues to diffuse into the concrete core, reducing the temperature difference. Therefore, the decrease of 92 °C at 1500 s to 15 °C at 5400 s indicates that the 2 mm coating just postpones the heat penetration rather than totally prevents it during long thermal exposure times.

Figure 8 shows the compressive stress–strain behaviors of the concrete specimens before and after thermal exposure, and Table 8 presents the corresponding retained compressive strengths. The unheated concrete specimen withstood a peak compressive stress of about 29.83 MPa. The uncoated specimen exposed to 500°C for 90 min (5400 s) and then natural cooling to room temperature for 24 h had a maximum compressive stress of about 22.0 MPa, which was equivalent to the retained compressive strength of about 73.8%, and the strength loss of about 26.2%. The coated specimen attained a maximum compressive stress of about 25.85 MPa corresponding to a retained compressive strength of 86.7 % of the initial strength. Thus, the residual compressive strength of the coated specimen was about 17.5% higher than the uncoated specimen. The relative reduction of the compressive-strength loss was about 49% when compared to the uncoated specimen.

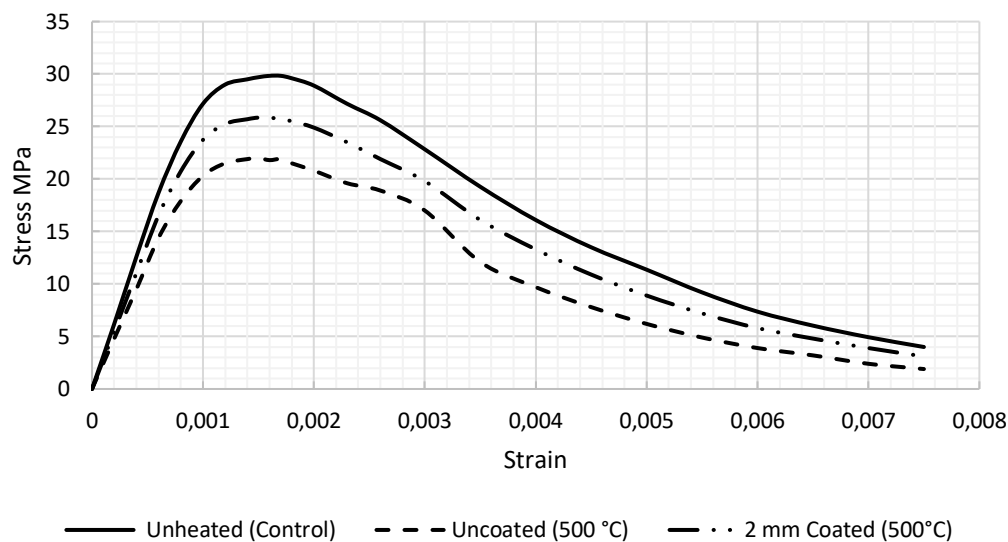


Fig. 8. Compressive stress–strain responses of concrete before and after thermal exposure at 500°C

Table 8. Summary of peak compressive stress, peak strain, and retained compressive strength

Condition	Peak compressive stress (MPa)	Peak strain	Retained strength
Before heating	29.83	0.0015–0.0016	100%
Uncoated after heating	22.00	≈ 0.0015	73.8%
Coated after heating	25.85	≈ 0.0015	86.7%

The differences in the post-peak response of the stress–strain curves are also observed between the unheated, thermally exposed uncoated and thermally exposed coated specimens. The reduction in the descending branch was greater for the uncoated specimen than for the unheated specimen. The coated specimen exhibited a stress-strain response intermediate between that of the unheated and the thermally exposed uncoated specimens. The intumescent coating was manually removed from the coated specimen after cooling, and the compressive response thereafter represented the residual response of the concrete specimen before the residual compression tests.

The thermal response of the uncoated concrete cylinder was validated by comparing the numerically predicted temperature history at the cylinder core with the corresponding experimental measure (see Figure 9). The numerical and experimental responses were in good

agreement over the 5400 s exposure time. The numerical core temperature increased from 20°C at the start of the analysis to 484°C at the end of the exposure, whereas the experimental temperature increased from 21°C to 500°C. The last numerical prediction was thus 96.8% of the experimental temperature with an absolute difference of 16 °C (3.2%). The average absolute temperature difference between numerical and experimental results was about 7.7 °C, which corresponds to the average relative deviation of about 2.2%. The maximum absolute difference was about 17°C at 2400 s. Notwithstanding this difference at the intermediate stage, the numerical temperature-time response followed the general trend of the experimental response for the whole heating period. The quantitative comparison provided an RMSE of 9.32 °C and a R^2 value of 0.9953 indicating a close agreement between the numerical and experimental core-temperature histories.

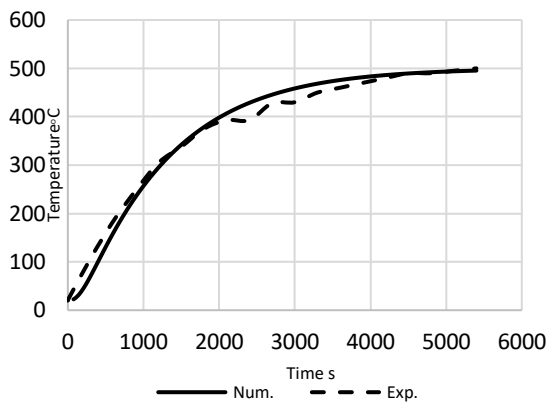


Fig. 9. Thermal validation of the uncoated concrete cylinder

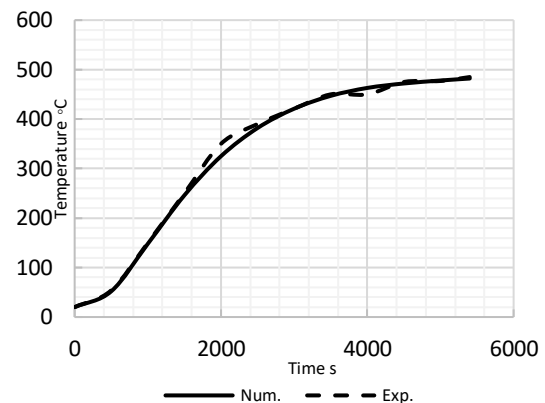


Fig. 10. Thermal validation of the 2-mm intumescent-coated concrete cylinder

The corresponding experimental measurements are also compared with the numerical thermal response of the 2-mm intumescent-coated concrete cylinder, as shown in Fig. 10. The predicted core temperature during the 5400 s exposure period increased from 20°C to 482°C and the experimental temperature increased from 20°C to 485°C. So, the difference between numerical and experimental temperatures at the end of the exposure period was 3°C (0.62%). The maximum difference was 25°C (7.14%) at 2000 s where the numerical temperature was 325°C and the experimental temperature was 350°C. At 3000 s, numerical and experimental temperatures were 421°C. The difference at the last exposure time was only 3°C. Overall the numerical and experimental temperature histories were similar during the heating period. The maximum difference was observed during the middle heating stage. The corresponding RMSE and R^2 were 8.71 °C and 0.9971, respectively, which further confirmed the good agreement between the numerical and experimental temperature histories.

4. Parametric Study

4.1 Effect of Exposure Temperature on the Thermal Response of the Uncoated Concrete Cylinder

The temperature-time response of the uncoated concrete cylinder exposed externally to 100, 300, 500, and 700 °C is shown in Figure 11. The internal thermal response shows a distinct and systematic increase with increasing exposure temperature. In all cases, the temperature increases slowly from the initial temperature of 20 °C, but the magnitude and rate of heating increases significantly with the imposed exposure temperature. The slowest thermal response is observed for the 100 °C exposure, with the core temperature rising steadily and reaching about 99 °C at the end of the 5400 s exposure period. The 300 °C case results in a final core temperature of about 297 °C, whereas a 500 °C exposure results in a much higher final core temperature of about 495 °C. The 700 °C exposure causes the most severe thermal response with the core temperature reaching a maximum of approximately 693 °C at the end of the 5400 s analysis. The differences between the four curves are most obvious at the initial heating stage. The steepest temperature increase is observed for the 700 °C case, followed by the 500, 300 and 100 °C cases, respectively. This shows that the rise of the external exposure temperature leads to an increase of the thermal driving force

between the heated boundary and the interior of the concrete and, thus, to an acceleration of the heat transfer into the specimen. For example, the 500 °C curve reaches ~100 °C after ~410 s and ~300 °C after ~1325 s, while the 300 °C exposure reaches ~200 °C after ~2300 s. The response at 700 °C is even faster, reaching over 300 °C in about 850 s and 500 °C in about 1800 s. The differences indicate that the temperature of the imposed exposure controls the rate at which thermal energy penetrates through the concrete and the final thermal state of the cylinder. Regardless of the different heating rates, all four curves show a nonlinear, asymptotic response with a relatively fast temperature increase in the initial heating phase, followed by a gradual decrease in the heating rate. This behavior is consistent with transient heat conduction in concrete, where the temperature gradient is initially high but gradually decreases as the internal temperature approaches the imposed thermal environment.

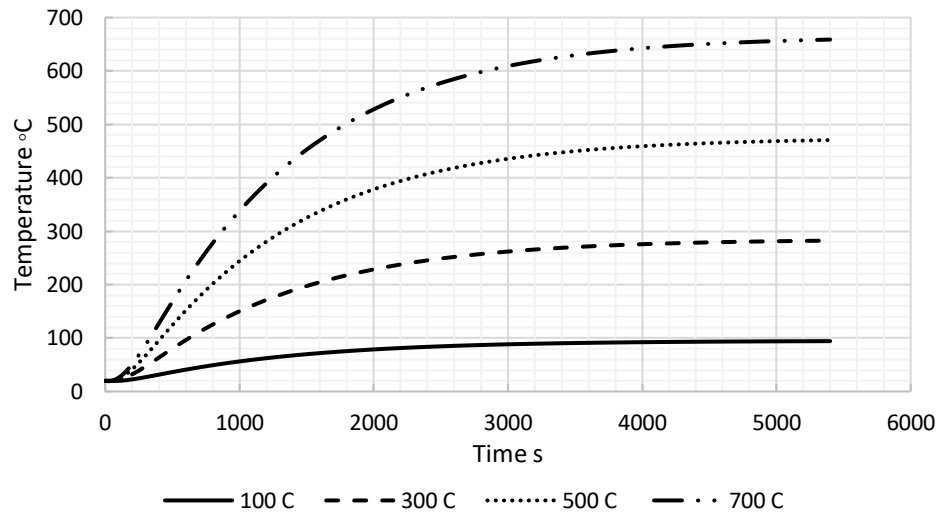


Fig. 11. Effect of exposure temperature on the thermo-mechanical response of uncoated concrete cylinder at core

The reduction of the slope of each curve indicates the gradual reduction of the thermal driving force and the increasing thermal equilibration within the cylinder. The results also showed that the uncoated concrete cylinder is very sensitive to the severity of thermal exposure. The final internal temperature, compared to the initial 100 °C case, increases by a factor of ~7 with increasing exposure temperature from 100 to 700 °C. This temperature dependent response is critical from a thermo-mechanical point of view since the higher thermal fields associated with severe exposure are expected to produce larger thermal expansion and thermal gradients and consequently more significant thermally induced stresses and deformation. Furthermore, the temperature-dependent degradation of mechanical properties of concrete, such as the loss of elastic stiffness and strength at elevated temperatures, is likely to increase the mechanical effects of thermal exposure. Therefore, the results presented in Figure 11 represent a reference case in order to evaluate the capability of the intumescent protective coating to reduce the heat penetration and the resulting thermo-mechanical response, being the uncoated cylinder the reference case.

4.2 Effect of Intumescent Coating Thickness

The effect of intumescent coating thickness on the thermal response of the concrete cylinder was examined by applying coating thickness of 1, 2 and 4 mm under exposure temperature of 100, 300, 500 and 700°C. The results show that the temperature development in the concrete core was controlled by the effect of the external exposure temperature and the thickness of the protective coating. Higher final core temperatures were generally achieved with higher exposure temperatures, but the effectiveness of thermal protection was strongly dependent on coating thickness. For the 1-mm coating, the final core temperatures after 5400 s were 88.04 °C, 276.76 °C, 471.89 °C, and 665.85 °C for exposure temperatures of 100 °C, 300 °C, 500 °C, and 700 °C, respectively. The ultimate core temperatures were reduced to 70.00, 264.12, 455.61 and 640.32°C when the coating thickness was increased to 2 mm. The reductions compared to the 1-mm coating

were 20.5%, 4.6%, 3.5%, and 3.8%, respectively. The 2-mm coating provided a measurable improvement in thermal protection, although the reduction in core temperature became relatively limited at the higher exposure temperatures. The 4-mm coating gave extra protection against heat. The final core temperatures at the end of the 5400 s exposure period were approximately 71.5, 275, 460 and 648 °C for exposure temperatures of 100, 300, 500 and 700 °C, respectively. These values exhibit a physically consistent increase in core temperature with increasing exposure temperature.

Table 9. Final concrete core temperatures at 5400 s and reductions relative to the uncoated condition

Exposure temperature (°C)	Uncoated core temp. (°C)	1 mm core temp. (°C)	Reduction (%)	2 mm core temp. (°C)	Reduction (%)	4 mm core temp. (°C)	Reduction (%)
100	99.21	95.11	4.1	74.70	24.7	71.50	27.9
300	297.29	285.00	4.1	281.50	5.3	275.00	7.5
500	495.38	475.00	4.1	482.00	2.7	460.00	7.1
700	693.47	664.00	4.2	686.00	1.1	648.00	6.6

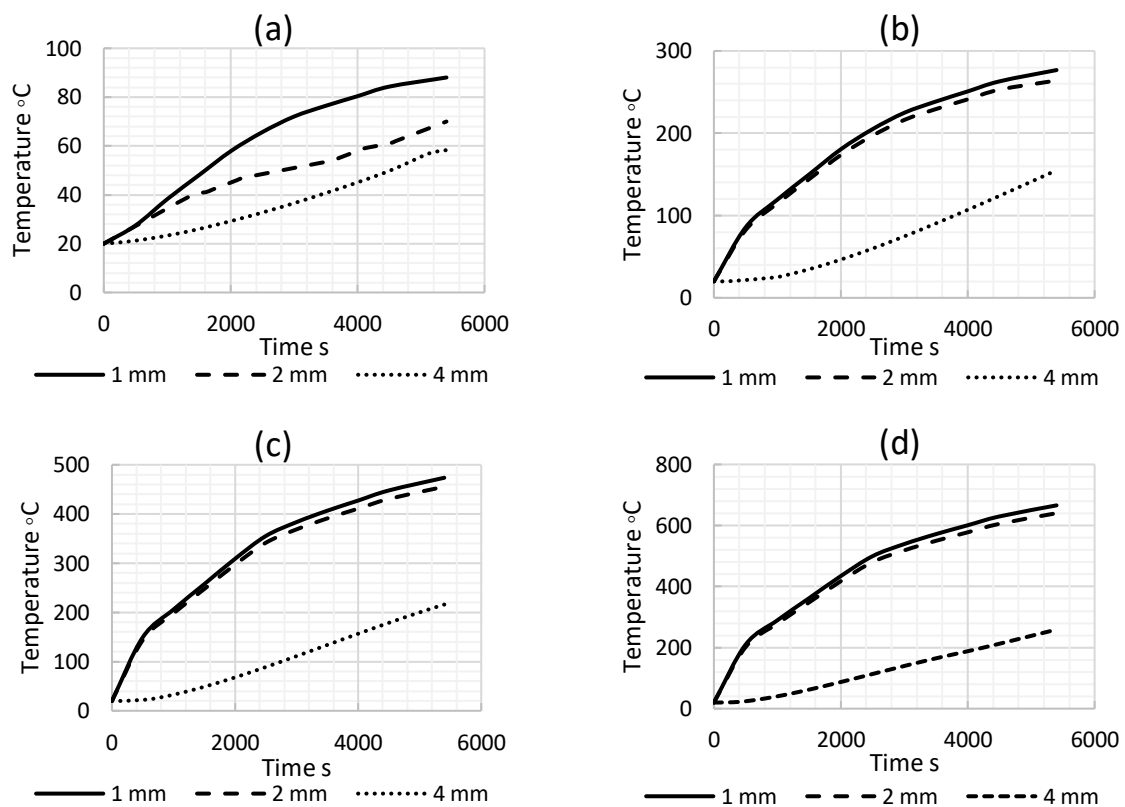


Fig. 12. Effect of intumescent coating thickness (a) Exposure temperature = 100°C, (b) Exposure temperature = 300°C, (c) Exposure temperature = 500°C, and (d) Exposure temperature = 700°C

The final core temperature decreased by about 18.8%, 0.6%, 2.5% and 2.7% at 100, 300, 500 and 700 °C respectively, when the coating thickness was increased to 4 mm compared to the 1-mm coating. The corresponding decreases were approximately 4.3%, 2.3%, 4.6% and 5.5% relative to the 2 mm coating. The results thus indicate that an increased coating thickness provides additional thermal protection, but the level of improvement is dependent on the exposure temperature and is not proportional to the increase of coating thickness. The results confirm that the overall thermal performance of the intumescent coating is governed by the interaction between exposure temperature and the thickness of the protective layer. The 4-mm coating provided the best thermal

protection, particularly important at high thermal exposure. The ability to reduce the final concrete core temperature is about 61.1% compared to the 1-mm coating at 700°C, which shows the improved ability to delay the penetration of heat and to reduce the thermal severity experienced by concrete.

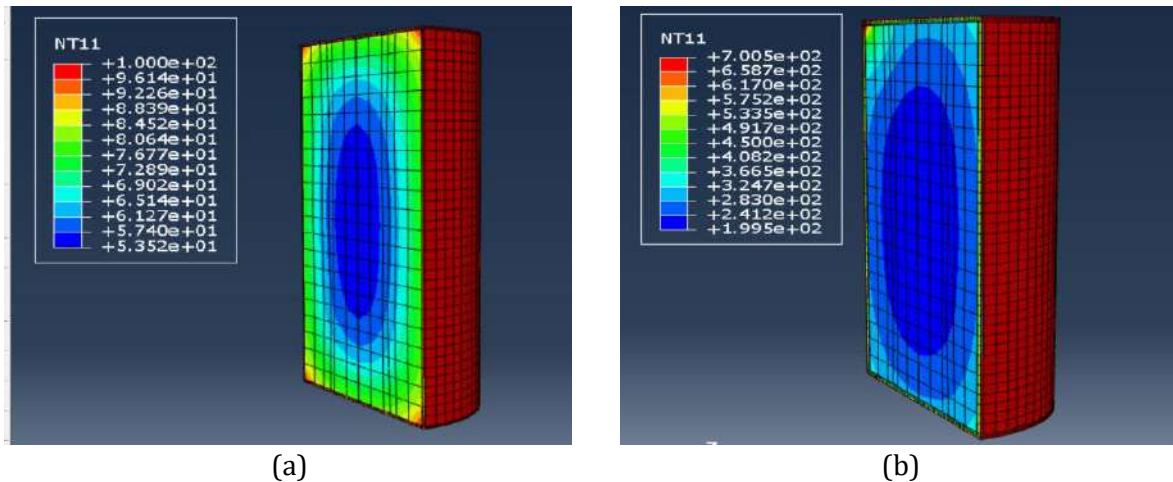


Fig. 13. Comparison of temperature contours under the least severe thermal condition (a: 1-mm intumescent coating exposed to 100°C) and the most severe thermal condition (b: 4-mm intumescent coating exposed to 700°C), illustrating the combined influence of exposure temperature and coating thickness on heat penetration

Therefore, the 4 mm intumescent coating was identified as the most effective thermal barrier and the most promising thickness to reduce the thermal degradation of the concrete cylinder under elevated temperature conditions. For a direct quantitative comparison of the investigated coating thicknesses, the final concrete core temperatures at 5400 s and the corresponding percentage reductions relative to the uncoated condition are summarized in Table 9. Figure 12 summarizes the effect of intumescent coating thickness. The lowest final core temperature was obtained with 4 mm coating at all investigated exposure temperatures as shown in Table 9. However, the extent of the thermal benefit was affected by the exposure temperature, confirming that the effect of the coating thickness was not strictly proportional in all the explored thermal conditions.

The temperature contours shown in the Figure 13 provide a visual representation of the combined effects of the exposure temperature and intumescent coating thickness on the heat penetration into the concrete cylinder. The comparison of 1-mm vs 4-mm coatings indicates the significant enhancement of thermal protection with the increase of coating thickness. The thinner coating allows for much greater heat penetration to the concrete core in severe exposure conditions, while the thicker 4 mm coating maintains a substantially cooler internal region. The contour distribution also confirms the presence of a steep temperature gradient passing through the coated system indicating that the intumescent layer effectively delays heat propagation. These visual observations are consistent with the temperature-time results and further support the superior thermal protection of the thicker coating.

4.3 Effect of Intumescent Coating on Mechanical Response

4.3.1 Displacement Response

The thermo-mechanical displacement behavior of the coated concrete cylinder was studied by means of the displacement histories obtained at the exposed surface and at the cylinder core for the three investigated intumescent coating thicknesses. For all coating configurations, displacement values were extracted from two identical reference locations within the concrete cylinder: a node at the geometric center of the concrete cylinder (core location) and a node at the center of the upper concrete surface (surface location). Therefore, the surface displacement was acquired from the concrete surface, not the outer surface of the intumescent coating. The displacement histories showed a general overall trend for all analyzed cases: an initial transient contraction and a gradual transition to positive displacement as the thermally induced expansion

took over. The first negative displacement is not an indication of actual shrinkage of concrete on heating. Rather it is a transient thermo-mechanical response associated with the rapid and non-uniform development of temperature field at the onset of thermal exposure. The exposed surface is heated and tends to expand faster than the relatively cooler concrete core. The temperature difference across the section temporarily restrains the free thermal expansion and produces a redistribution of thermal stresses and deformation. This transient interaction may cause a small initial displacement in the direction of contraction. With the heat progressively penetrating toward the core and the temperature field developing in the whole specimen, the thermally induced expansion dominates and the displacement changes to the positive direction. The exposure temperature and coating thickness therefore mainly affect the magnitude and speed of displacement development and not the general qualitative evolution from initial transient response to later thermal expansion. Also, the displacements histories showed a strong dependence on the thermal exposure conditions. As the exposure temperature increased, the temperature evolution inside the concrete was faster and higher, and the thermally induced deformation was increasing accordingly. This trend can be observed in the final displacement values in Table 10. Surface and core displacements generally increased with increasing exposure temperature. The surface displacement of the 1mm coating increased from 0.1213mm at 100°C to 0.8518mm at 700°C, while the core displacement increased from 0.0557mm to 0.4125mm. The 2- and 4-mm coatings showed the same temperature dependent response. This observation further supports the conclusion that the magnitude of the final displacement was primarily governed by the extent of thermal exposure. It was also obvious the difference between the exposed surface and the core of the cylinder. The core displacement was lower than the corresponding surface displacement for all coating thicknesses and exposure temperatures. This behavior is explained by the non-uniform distribution of the temperature all over the cylinder and the delayed thermal response of the core. The exposed surface responds more directly to the imposed thermal boundary condition, while the core experiences a delayed and reduced temperature increase as heat diffuses inward. Hence, the difference between the surface and core displacement is representative of the developed thermal gradient through the concrete section. The raw displacement histories showed that the difference developed progressively during the analysis, not constantly confirming the transient nature of the coupled thermal and mechanical response. The effect of coating thickness was particularly notable in the severe exposure condition at 700°C given in Figure 14. But all three configurations showed the same general evolution from contraction to expansion, but their displacement magnitudes differed greatly. The maximum final surface displacement of 2-mm coating was 1.2174 mm while the maximum final surface displacement of the 1- and 4-mm coatings were 0.8518 and 0.8245 mm, respectively. Hence, the final surface displacement for the 4-mm coating was reduced by about 32.3% compared to the 2-mm coating and by 3.2% compared to the 1-mm coating. For the 1-, 2-, and 4-mm coatings, the corresponding core displacements were 0.4125, 0.7371, and 0.3803 mm, demonstrating that the 4-mm configuration reduced the final core displacement by approximately 48.4% compared to the 2-mm coating and 7.8% compared to the 1-mm coating. The relatively larger displacement of the 2 mm configuration with respect to the 1 and 4mm configurations should not be interpreted as degradation of the thermal protection provided by the coating. The numerical output was checked, and it was confirmed that this non-monotonic response was not caused by the location of the displacement extraction, as the same concrete surface and core reference locations were used for all three coating thicknesses. Rather, the displacement response is governed by the combined interaction of the transient temperature field, thermal expansion and contraction, temperature gradients and the temperature dependent mechanical behavior of concrete. In particular, the 4-mm coating offered much higher thermal insulation, which highly reduced the heat penetration in the concrete and hence limited the final thermally induced deformation. This interpretation is consistent with the temperature results which demonstrated that the internal thermal field was affected by the coating thickness, but the extent of thermal benefit was not strictly proportional to the coating thickness under all exposure conditions. The results of the displacement measurements thus confirm that the protective coating modifies the mechanical response mainly through its control on transient heat transfer and the resulting thermal field. The displacement histories in general show a consistent thermo-mechanical response for all the configurations investigated while the deformation magnitude was controlled

by the combined effects of exposure temperature and coating thickness. The results indicated that the 4 mm coating was the most effective in reducing the final core displacement under severe thermal exposure, which was confirmed with its superior thermal insulation performance. The displacement response thus confirms the high coupling between heat penetration and thermally induced deformation, while the subsequent tensile-damage contours provide a complementary assessment of the spatial consequences of this thermo-mechanical response.

Table 10. Final displacement response at the concrete surface and core after 5400 s

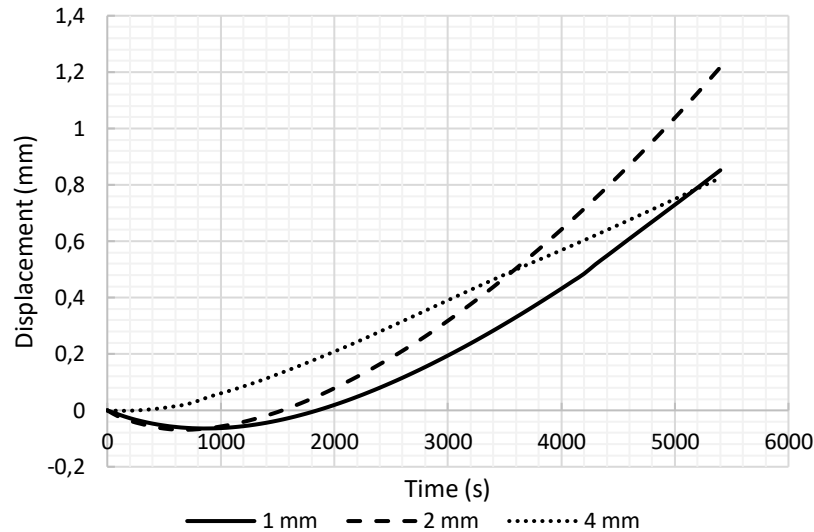


Fig. 14. Displacement histories at surface of concrete cylinders with different intumescent coating thicknesses at 700°C

4.3.2 Tensile Damage Response

The contours of tensile damage, which qualitatively represent the evolution of damage in the concrete cylinders under different thermal exposure conditions and intumescent coating thicknesses, are shown in Figure 15. The contours were chosen to represent the main trends found in the entire analyzed set of configurations, and not to reproduce each individual case. In general, the distribution and intensity of tensile damage increased with increasing temperature of exposure, especially for the uncoated and thinner coated cylinders. The uncoated cylinder with the lower exposure temperature showed only limited localized damage, whereas the severe thermal exposure caused a substantially greater extent and intensity of damage. The results also show the protective effect of the intumescent coating on the evolution of the thermally induced tensile damage. Increasing the coating thickness generally reduced the thermal penetration into the concrete and therefore limited the extent of damage development for a given exposure temperature. This effect was more evident under the more severe thermal exposure, where the thicker protective layers resulted in lower temperatures within the concrete and reduced the thermal severity experienced by the material. The damage contours also show that the damage was not uniformly spread across the cylinder but rather grew preferentially in areas subjected to higher thermal gradients and non-uniform thermally induced deformation. The damage pattern observed is consistent with the thermal results shown earlier, where the temperature within the concrete was considerably reduced as the intumescent coating thickness increased. In the concrete damage-plasticity model, the tensile damage is related to the tensile deformation and the cracking induced by temperature. Therefore, the decrease in temperature penetration by the protective coating led to the corresponding decrease in the development and propagation of tensile damage. Thus, the 4-mm coating demonstrated the best overall damage mitigation of the tested configurations, especially at the highest thermal exposure conditions. The spatially varying temperature field and the corresponding non-uniform thermally induced deformation led to the localization and non-uniformity of tensile damage contours in concrete cylinder. This behavior is in agreement with the thermomechanical response of concrete at elevated temperatures where temperature-dependent

degradation of mechanical properties, thermal gradients and localized cracking mechanisms contribute to spatially heterogeneous damage development. [35, 36]. Thus, the damage distribution observed is expected to be heterogeneous in the cylinder and not to develop homogeneously in the whole concrete domain.

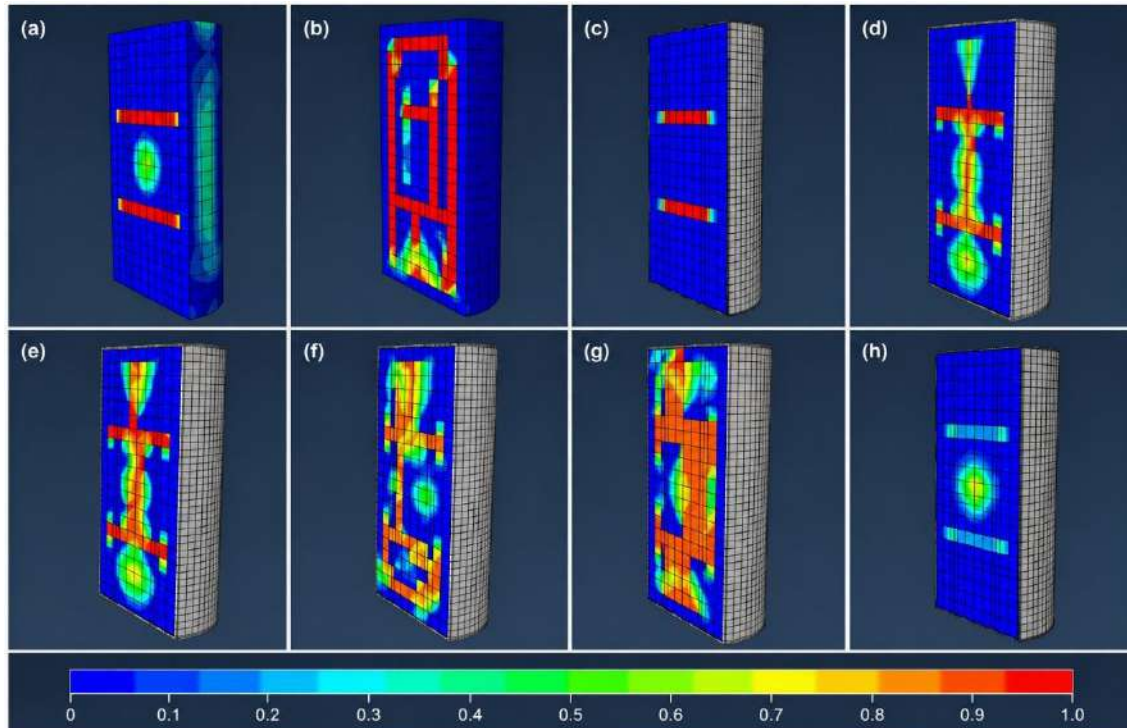


Fig. 15. Representative tensile damage contours illustrating the effects of thermal exposure temperature and intumescent coating thickness. (a) uncoated cylinder at 100 ° C, (b) uncoated cylinder at 700 ° C, (c) 1 mm coated cylinder at 100 ° C, (d) 1 mm coated cylinder at 500 ° C, (e) 1 mm coated cylinder at 700 ° C, (f) 2 mm coated cylinder at 500 ° C, (g) 2 mm coated cylinder at 700 ° C, (h) 4 mm coated cylinder at 500 ° C

This interpretation is in line with previous studies on fire-protected concrete systems that demonstrated that intumescent coatings alter the internal temperature distribution by reducing the heat ingress and consequently the thermal exposure of the concrete . [37, 38]. Therefore, the reduction in thermal severity with increasing coating thickness can help to alleviate the thermally induced tensile damage and maintain the integrity of the concrete core, as also reported in experimental studies of fire-protected concrete exposed to elevated temperatures.[21]. Therefore, the tensile-damage contours for the present study were interpreted qualitatively, based on the overall spatial distribution and severity of damage, rather than based on isolated element values or as mesh-independent predictions of individual crack paths. Some contours exhibit grid-aligned localization, which may be partly due to the sensitivity of strain-softening damage formulations to the finite element discretization. The present analysis does not include a dedicated mesh-objectivity study of the localized damage patterns and an objectivity regularization or damage-stabilization procedure. Thus, the predicted damage bands should not be taken literally in terms of their local width and fine geometry. The contours are primarily used for comparing the gross extent and position of damage for the thermal conditions studied. Future studies should investigate mesh-objective regularizations and damage-stabilization approaches when detailed prediction of localized cracking is desired.

4.3.3 Residual Mechanical Performance

The residual compressive performance of the concrete cylinders was evaluated by the peak compressive stress value after thermal exposure. The retained compressive strength and the corresponding strength loss were calculated with respect to the reference compressive strength of the unheated concrete, and the results are summarized in Table 11. The findings pointed to a clear temperature-dependent decrease in the mechanical capacity of the concrete while the effectiveness

of the intumescent coating was improved with the increase of coating thickness and the intensity of exposure. The mechanical degradation was low for all investigated configurations at the exposure temperature of 100 °C. The uncoated cylinder lost 1.83% of the original compressive strength (98.17% of the original compressive strength). The retained strength slightly increased to 98.43%, 99.0% and 99.98% for the 1-, 2- and 4-mm coatings, respectively. The small differences between the configurations show that the concrete mechanical properties were affected only to a small extent at the low temperature, and therefore the influence of the coating thickness was limited at this temperature. The loss in compression performance was more severe at 300 °C. The uncoated cylinder retained 88.5% of its original strength or had a strength loss of 11.5%. By increasing the coating thickness to 1 and 2 mm, the retained strengths were 89.32% and 90.10%, respectively, while the 4-mm coating increased the retained strength to 94.1% and the strength loss was reduced to 5.9%. The gradual improvement with the increase of coating thickness shows that the thicker protective layer gradually limited the severity of thermal deterioration of the concrete. At 500°C the effect of the coating was much more pronounced. The original compressive strength of the uncoated cylinder was maintained at 75% (a 25% reduction in strength). The retained strength increased to 77, 88 and 93% for 1, 2 and 4mm coatings, respectively. Thus, the 4-mm coating reduced the strength loss from 25% to 7%, which is about 72% reduction in the strength loss, compared to the uncoated configuration. This important improvement shows the higher efficiency of the thicker coating to preserve the mechanical capacity of the concrete with the increase of the thermal attack. The most pronounced difference between studied configurations was observed at 700 °C. The uncoated cylinder lost 63.25% of its compressive strength, retaining only 36.75% of the original. The 1 and 2mm coatings showed only slight improvements with the retained strengths of 41.23% and 43.0% respectively. In contrast, the 4-mm coating retained 84.89% of the initial compressive strength, and the loss of strength was only 15.11%. Thus, the 4-mm coating reduced the strength loss by about 76.1% compared with the uncoated configuration and preserved about 2.31 times the residual compressive capacity of the uncoated cylinder. The small difference in retained compressive strength between the 1-mm and 2-mm coatings at 700°C, despite the better thermal protection offered by the thicker coating, suggests that the relationship between thermal protection and residual mechanical performance is not necessarily linear. The residual compressive strength relates to the combined effect of the internal temperature field, the thermal gradients and the degradation accumulated in the concrete rather than the coating thickness or a single temperature measure. Both the 1-mm and 2-mm configurations were exposed to the severe 700°C exposure, which is within a high temperature regime for significant concrete degradation, so the added thermal protection of increasing the coating thickness from 1 mm to 2 mm provided only a limited improvement in retained strength. However, the 4-mm coating was thermally stable enough to significantly reduce the thermal severity on the concrete, resulting in the substantially higher retained strength of 84.89 %. Coating expansion, interface degradation and potential debonding were not explicitly modelled in the current equivalent-layer formulation and therefore these mechanisms were not used to directly explain the predicted residual strength response. The results showed that the coating thickness effect was highly dependent on the intensity of thermal exposure. At low temperatures, concrete retained most of its original mechanical capacity and the differences between coating thicknesses were limited. The protective effect of the intumescent coating increased with increasing exposure temperature. This is especially true at 700°C where the 4mm coating resulted in a much higher retained strength than either the 1mm or 2mm coatings. The significant improvement at this exposure temperature is attributed to the ability of the thicker protective layer to reduce the thermal intensity experienced by the concrete and, thereby, suppress the degradation of its mechanical capacity. The results on residual strength also provide a quantitative description of the mechanical effects of the thermo-mechanical response discussed above. The results showed that the severity of the thermal exposure led to greater deterioration of the mechanical capacity of concrete, while the thicker intumescent coating significantly mitigated this deterioration. The observed relation between exposure temperature, coating thickness and residual strength is physically consistent with the temperature-dependent deterioration mechanisms reported for concrete subject to elevated temperatures and with the protective role of thermal barrier systems reported in previous studies [13, 22, 24]. The results of these studies support the interpretation that reducing the heat

penetration can decrease the severity of the thermal damage and improve the residual mechanical performance of the concrete. The results demonstrate that the intumescent coating was effective in maintaining the residual compressive performance of concrete cylinders, and its benefit was more significant under severe thermal exposure. The 4 mm coating resulted in the highest retained compressive strength and the lowest strength loss of all investigated configurations. It showed a significant performance at 700°C, reducing the strength loss from 63.25% for the uncoated cylinder to only 15.11%. Therefore, the remaining mechanical results quantitatively show that the increase of the intumescent coating thickness can significantly reduce the degradation of concrete compressive capacity and preserve a larger portion of its original mechanical performance after severe thermal exposure.

Table 11. Summary of the residual compressive performance of the coated concrete cylinders

Exposure Temperature (°C)	Coating Thickness (mm)	Peak Stress (MPa)	Retained Strength (%)	Strength Loss (%)
100	0	29.2873	98.17	1.83
100	1	29.3649	98.43	1.57
100	2	29.535	99	1
100	4	29.83	99.98	0.011
300	0	26.4025	88.5	11.5
300	1	26.6471	89.32	10.68
300	2	26.88	90.10	9.9
300	4	28.0731	94.1	5.9
500	0	22.375	75	25
500	1	22.9716	77	23
500	2	26.2533	88	12
500	4	27.745	93	7
700	0	10.9638	36.75	63.25
700	1	12.3002	41.23	58.77
700	2	12.8283	43	57
700	4	25.3255	84.89	15.11

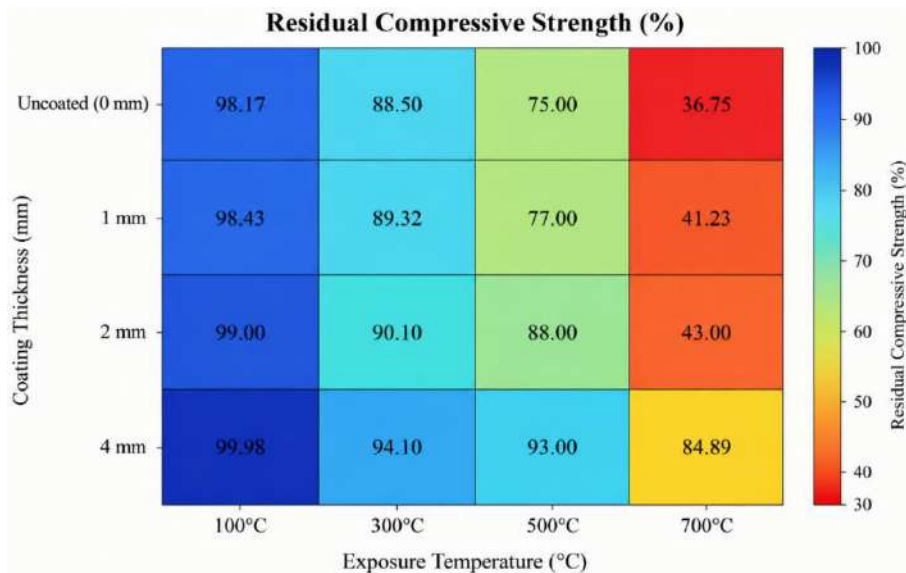


Fig. 16. Heatmap of retained compressive strength as a function of exposure temperature and intumescent coating thickness

As shown in Figure 16, the retained compressive strength generally decreased with the increasing exposure temperature while increasing the thickness of the intumescent coating improved the residual mechanical performance of the concrete. The effect of coating thickness was most evident at the highest exposure temperature, where the 4-mm coating retained significantly higher residual strength compared to the uncoated and thinner-coated cases.

5. Conclusions

The thermo-mechanical response of a concrete structure exposed to fire was studied in the present work using an experimentally validated sequentially coupled finite element framework. The results suggest that the protective effect of the coating is controlled not only by its thickness but also by the interaction between the exposure temperature, the transient heat transfer, thermal gradients and the consequent mechanical response of the concrete. The numerical model has been shown to agree well with the experimentally observed thermal response and has provided a suitable framework to examine the influence of coating thickness when considering the investigated thermal conditions.

The intumescent coating mainly acted to delay the penetration of heat into the concrete and change the evolution of the internal temperature field. In general, increasing the coating thickness improved the thermal protection. The lowest final core temperatures of the coating thickness investigation were obtained for the 4 mm configuration. However, for all exposure conditions the improvement was not strictly proportional to coating thickness. Thus, coating performance should not be assessed solely on the basis of thickness, but also on the time and intensity of thermal exposure and the transient heat-transfer characteristics of the protected system.

The thermo-mechanical results also indicated that the deformation of concrete was closely related to the fluctuating temperature distribution and thermal gradients. The non-monotonic displacement response observed for some coating configurations indicates that a lower temperature at an individual location does not directly translate to a proportionally lower displacement. Instead, the deformation response is due to a combination of spatial temperature gradients, thermal expansion and contraction, and temperature-dependent mechanical behaviour. Similarly, the predicted tensile damage patterns show that controlling heat penetration can decrease the amount of thermally induced damage and hence improve the residual mechanical performance after high temperature exposure.

These results support the application of increased intumescent coating thickness as an effective approach to enhancing the thermal protection of concrete exposed to high temperatures. The best overall combination of thermal protection reduced thermally induced deformation and improved residual response was obtained with the 4 mm coating in the investigated range. However, the results also suggest that the selection of protective coating thickness should be based on the anticipated thermal exposure rather than assuming a linear improvement with increasing thickness. Therefore, the validated modelling framework may be used to facilitate a comparative assessment and preliminary optimization of protective coating configurations under the studied conditions.

5.1 Limitations and Scope of Applicability

The current numerical results are to be understood by considering the studied specimens and modelling assumptions. The model was developed and validated using relatively small unrestrained concrete cylinders under controlled thermal exposure. Therefore, the predicted thermal and thermo-mechanical responses should not be directly extrapolated to full-scale structural members where sustained mechanical loading, restraint conditions, member geometry, reinforcement and non-uniform fire exposure may significantly impact the response. Also, transient creep and other load-temperature interaction effects were not explicitly considered in the present model. The three-dimensional geometry used in this study is representative of the cylindrical specimens tested and does not reflect the more complex heat transfer paths and restraint conditions that may exist in beams, columns, slabs, or structural connections. The results are therefore primarily intended for comparative assessment of the investigated coating thicknesses under the validated conditions rather than for direct structural fire design predictions. Future research should be directed toward the extension of the proposed framework to loaded and restrained reinforced-concrete members, inclusion of transient creep and additional temperature-dependent material behavior, and validation of the predictions against larger-scale fire tests.

6. Recommendations

Based on the results of the present study, the following recommendations are presented:

- The intumescent coating thickness should be considered as an important design parameter for the development of thermal protection systems for concrete subjected to extreme elevated temperatures.
- Thicker protective layers shall be prioritized for the case of severe thermal exposure conditions where the preservation of the concrete core temperature and residual compressive capacity is required.
- The performance of fire-protected concrete should not be judged solely on the basis of the reduction in surface or core temperature but rather considering the coupled thermal and mechanical response.
- Numerical predictions should be supported by experimental thermal and mechanical data in the evaluation of new coating materials or coating thicknesses.

7. Future Work

Future work should aim to validate the predicted thermo-mechanical response experimentally for a wider range of intumescent coating thicknesses and formulations. Further research should also consider other concrete strength classes and aggregate types as well as the influence of coating thermal properties and expansion behavior on heat transfer and damage development. In addition, the effect of elevated temperature in combination with sustained or transient mechanical loading should be studied to extend the present material-level analysis towards more realistic structural conditions. Finally, optimization-based studies could be performed to find coating thicknesses which provide a good balance between thermal protection, residual mechanical performance, material consumption and practical application requirements.

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A hybrid real-time safety monitoring framework for power battery packs using autoencoders and adaptive extended Kalman filter-based HARA

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Abstract

Power battery packs are a critical component of electric vehicles (EVs), and their safety and stability directly dictate the reliability and commercial viability of modern electrified transportation. Traditional battery monitoring systems often rely on static threshold-based models, which struggle to provide adaptive and accurate safety diagnostics under highly dynamic operating conditions. This paper integrates real-time sensor measurements with a systematic Hazard Analysis and Risk Assessment (HARA) framework to propose a real-time monitoring and analysis model for power battery packs. The system employs an Adaptive Extended Kalman Filter (AEKF) to adjust system parameters using sensor signals derived from battery performance and monitors various fault types based on HARA results. An autoencoder network enhances the performance of state estimation and monitoring. Experiments conducted on OBD, CALCE, and self-constructed datasets validate the model's performance in terms of fault detection consistency, isolation accuracy, and State of Charge prediction. The experimental results show that the fault detection consistency of the monitoring system based on sensor signals and HARA reaches $94.28\% \pm 0.76\%$, the insulation resistance estimation error is $1.492 \pm 0.067 \Omega/V$, and the SOC prediction accuracy is $98.46\% \pm 0.42\%$, which is better than the other three models.

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1. Background

With the accelerated growth of the electric vehicle industry, the application of artificial intelligence algorithms has become very common to ensure smooth system running [1]. Power battery packs are an indispensable part of electric vehicles, and the working efficiency of battery packs will have a direct effect on the running conditions and range of the vehicles [2]. When batteries undergo abusive conditions such as overcharging or internal short circuits, severe thermal runaway (or heat accumulation) may occur, potentially leading to catastrophic safety hazards such as fires or explosions [3]. Consequently, real-time monitoring of battery packs has emerged as a paramount and highly active research focus globally. Real-time monitoring systems are primarily based on direct detection by sensor signals and state estimation algorithms. These methods require high accuracy for battery models, and key parameters of the battery model, such as ohmic resistance, polarization parameters, and open circuit voltage, will fluctuate with temperature, aging, and changes in operating conditions, resulting in parameter uncertainty. There is a model mismatch between the simplified model and the real battery. Voltage, current, and temperature sensor noise and sampling errors can introduce disturbances. The non-linear characteristics of the battery itself also increase the difficulty of online identification. Therefore, the key values of the model are prone to insufficient accuracy, leading to state estimation errors and increased computational costs [4]. By contrast, the Hazard Analysis and Risk Assessment (HARA) approach provides a structured

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approach to systematically assess safety hazards. This reduces subjective bias in safety assessments and provides the opportunity, allowing the optimization of computational resource allocation based on the assessed risk levels. This will balance the related costs and safety more efficiently [5]. Thus, this study establishes an integrated real-time monitoring and analysis model for power battery packs by coupling sensor measurements with the HARA framework. This model is capable of overcoming the prediction limitations and processing delays of conventional monitoring schemes. Moreover, this model helps to improve battery reliability and risk prevention capabilities. The innovation and contributions of the work emerge at two ends. First, it introduces a novel hybrid safety monitoring framework that seamlessly integrates data-driven diagnostics with proactive risk evaluation. Secondly, it provides a technical solution for real-time monitoring of power battery packs under experimental conditions.

2. Related Studies

Since HARA can enhance the effectiveness and reliability throughout the lifecycle of detection targets, numerous scholars both domestically and internationally have joined the research. To better monitor the states during the power battery charging process, Kim et al. put forward a strategy based on an ensemble split learning algorithm. This strategy enhanced the accuracy of battery life prediction using the ensemble split algorithm, observed the temperature and current of batteries, and conserved power. Experimental results show that the strategy achieves an accuracy of 93.40% on the test set [6]. To solve the situation where the monitoring capability of batteries reduces each year, Saba et al. put forward a measurement strategy that utilized the digital twin and deep deterministic policy gradient method. This strategy monitored the life of batteries by recording digital twin values such as the charging state, and batteries' safety and reliability were improved by training the model based on the deep deterministic policy gradient method. The monitoring method considered battery safety and reliability [7]. For the battery monitoring issue, the researchers from Zhao and Bai put forward a model that utilized the whale optimization algorithm and long short-term memory algorithm. This model optimized the battery fault diagnosis by using the whale optimization algorithm, and the safety management capability of batteries was enhanced. Experiment results showed that the monitoring accuracy of this model was 95.3% [8]. Bai et al. put forward an electrochemical-magnetic-field-thermal coupling three-dimensional model for monitoring the operating state of batteries. This model fulfilled the battery state monitoring task by identifying the internal short-circuiting condition and crack defects, as well as the distribution of the magnetic field surrounding the battery [9]. The researchers from Wang et al. put forward a lithium battery type monitoring system with the aim of monitoring the power batteries. This system constructed monitoring programs through the laboratory virtual instrument engineering platform and conducted systematic testing to complete power battery pack monitoring. Results showed that the voltage measurement error of this monitoring system was constrained within 5% [10].

In summary, HARA related research provides a structured risk assessment process, but its analysis is mostly based on typical scenarios in the development stage, with relatively fixed risk parameters and insufficient coupling with real-time sensor signals and state estimation results. Research on battery monitoring has made progress in life prediction, digital twin, fault diagnosis, and condition monitoring. However, some methods rely on high-precision battery models, which are affected by parameter uncertainty, model mismatch, and sensor noise. Some methods have limited support for unknown anomaly detection and risk grading decisions, and there is also a trade-off between real-time performance and computational overhead. Therefore, this study combined sensor signals and the HARA method to propose a real-time monitoring model applied to power battery packs, achieving the goal of efficiently identifying hazard types and accurately monitoring battery operating states. It is expected that this model can effectively enhance real-time monitoring performance and provide reliable technical decisions for the stable operation of power battery packs.

3. A Battery Monitoring and Analysis System Combining Sensor Signals And HARA

3.1 Design of A Battery Safety Analysis Framework Integrating Sensor Signals and HARA

During the monitoring of power battery packs, issues such as inaccurate functional state estimation and inconsistent monitoring results easily occur. Traditional real-time monitoring methods obtain data with limited dimensions, which restricts the amount of available information and makes precise real-time monitoring difficult to achieve [11-12]. In contrast, HARA systematically analyzes potential issues in power battery systems and performs risk assessment. Based on risk exposure, severity, and controllability, it defines reasonable safety levels and provides reliable safety mechanisms [13]. In addition, battery signals collected by sensors show higher objectivity and directness. Therefore, this study applies HARA combined with sensor signals to perform real-time monitoring and analysis of power battery packs. The approach achieves high-precision and high-efficiency battery state monitoring. The application process of battery monitoring using HARA combined with sensor signals is shown in Figure 1.

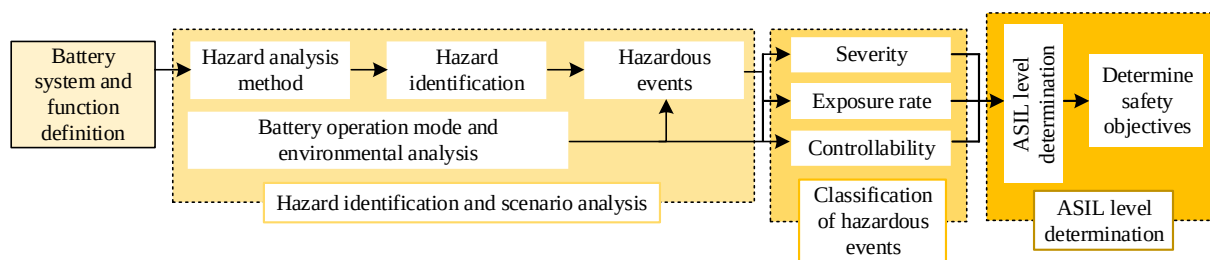


Fig. 1. Process of battery status monitoring based on the HARA method and sensor signals

As shown in Figure 1, the monitoring and analysis process of power battery packs based on HARA mainly includes hazard identification and scenario analysis, hazard event classification, and determination of the Automotive Safety Integrity Level (ASIL). During hazard identification and scenario analysis, HARA identifies potential hazards and adverse outcomes of power battery packs based on system definitions and hazard analysis methods. It then determines specific operating scenarios. HARA subsequently performs risk assessment and defines safety objectives for each hazard based on the assessment results. It also provides functional safety concepts to complete state monitoring. HARA hazard analysis and safety objectives are shown in Table 1.

In Table 1, severity, exposure/occurrence and controllability are classified according to S0-S3, E0-E4 and C0-C3 respectively. The detection conditions are composed of voltage, current, temperature, SOC, AE reconstruction error, AEKF innovation/residual and other signals. The trigger threshold is used to convert the risk level into executable monitoring logic. The threshold is set based on 12.8 v/60 ah lithium iron phosphate group and 10 hz/100 Hz sampling. The control action is performed by the battery management system (BMS). ASIL is determined according to ISO 26262 risk matrix. The standard HARA is oriented to vehicle functional safety. It usually conducts static analysis according to the system definition and typical scenarios in the development stage, and the risk parameters and ASIL levels are relatively fixed after being determined. The battery system has the characteristics of electrochemical nonlinearity, aging, multi-sensor coupling and rapid evolution of thermal runaway. In this paper, the hazard types are extended to battery specific faults such as overcharge, over discharge, internal short circuit, thermal runaway spread and insulation fault. The detection conditions introduce voltage, current, temperature, SOC, AE reconstruction error, AEKF innovation and residual. The exposure/occurrence probability and controllability are reevaluated with the real-time working conditions. The safety objectives and trigger thresholds directly correspond to the BMS control action. The hazard types, safety objectives and ASIL levels determined by HARA in the development phase are used as static baselines and are updated in rolling combination with sensor signals and state estimation results during operation. AEKF adjusts SOC, SOH and internal resistance estimates in real time according to voltage, current and temperature, re-evaluates the exposure/occurrence probability and controllability with the change of working conditions, and adjusts the trigger threshold and control action accordingly [14-15].

Table 1. HARA hazard analysis and safety objectives of power battery pack

Hazard Type	Operating Scenario	Detection Conditions	Severity S	Exposure E	Controllability C	ASIL	Safety Objective	Trigger Threshold	Control Action
Overcharge	Fast charging or slow charging, SOC close to full charge	Cell voltage, charging current, SOC, voltage consistency, temperature rise	S3	E3	C2	C	Prevent thermal runaway caused by cell overcharge	Cell voltage ≥ 3.65 V for 2 s, or the voltage difference between the highest and lowest cell ≥ 100 mV and temperature rise ≥ 1 °C/min	Reduce the charging current and cut off the charging relay, start cooling and alarming, and prohibit recharging
Over-discharge	Driving or static state, low SOC	Cell voltage, discharge current, SOC	S2	E3	C2	B	Prevent deep discharge and copper dissolution	Cell voltage ≤ 2.50 V for 5 s, or SOC $\leq 5\%$ and discharge current $> 0.1C$	Limit the discharge power, cut off the load when necessary, and prompt charging
Overtemperature	High-rate discharge, fast charging or high-temperature environment	Temperature, temperature rise rate, temperature difference, current	S3	E3	C2	C	Prevent thermal runaway	Temperature ≥ 60 °C, or temperature rise ≥ 1.5 °C/min, or maximum temperature difference ≥ 8 °C	Reduce power or cut off output, start cooling and trigger an alarm
Internal short circuit	Charging, driving or static state	Sudden voltage drops, temperature rise, abnormal SOC, AE reconstruction error, internal resistance estimation	S3	E3	C3	D	Identify and isolate internal short circuits at an early stage	Voltage drop ≥ 200 mV/s, or temperature rise ≥ 2 °C/min, or AE reconstruction error $> 3\sigma$	Cut off the high-voltage relay, prohibit charging, and start thermal management and alarming
Connection fault / Increased contact resistance	Vibration, aging and high-current working conditions	Connection point temperature, internal resistance estimation, voltage difference	S2	E3	C2	B	Prevent local overheating and electric arcs	The estimated internal resistance increases by $\geq 30\%$ compared with the baseline, or the connection point temperature ≥ 70 °C for 3 s, or the voltage difference ≥ 100 mV	Reduce power, cut off the faulty branch and give maintenance prompts
Voltage sensor offset / Stuck / Reduced accuracy	All working conditions	Voltage residual, adjacent cell consistency, AEKF innovation	S2	E3	C2	B	Avoid misjudgment caused by wrong voltage input	Voltage residual > 50 mV for 10 s, or variance < 1 mV when stuck, or accuracy deviation $> 2\%$	Switch to redundant or model-estimated voltage, degrade monitoring, limit fast charging

Current sensor noise / Rate deviation	Charge and discharge, dynamic working conditions	Current residual, SOC estimation deviation, power integration	S2	E2	C2	A	Ensure reliable SOC/SOP estimation	Standard deviation of current noise>0.05C, or rate deviation>5% for 10 s	Enable filtering or reconstructed current, reduce power and calibrate the sensor and trigger an alarm
Temperature sensor reading drift	All working conditions	Temperature residual, thermal model prediction deviation, temperature difference	S2	E2	C2	A	Avoid thermal management failure caused by temperature misjudgment	Temperature residual>5 °C for 30 s, or drift>2 °C	Adopt redundant or model temperature, reduce power and trigger an alarm
Thermal runaway spread	After collision, internal short circuit or overcharge	Temperature, temperature rise rate, smoke or air pressure, voltage	S3	E3	C3	D	Prevent occupant injury and thermal runaway spread within the pack	Temperature ≥80 °C, or temperature rise ≥5 °C/min, or sudden change in smoke or air pressure	Cut off high voltage, carry out emergency ventilation and fire extinguishing, unlock doors and trigger an alarm
Insulation fault	Water immersion, aging, collision	Insulation resistance, leakage current	S3	E2	C3	C	Prevent high-voltage electric shock	Insulation resistance <100 Ω/V, or leakage current>10 mA	Cut off high voltage, prohibit power-on and trigger an alarm

The HARA baseline monitoring strategy is used under normal conditions, and the sampling frequency is increased and the corresponding safety measures are triggered under dynamic or abnormal conditions. Therefore, this study combines AEKF with HARA to enhance real-time performance and adaptability. Therefore, this study combines AEKF with HARA to enhance real-time performance and adaptability. The battery model employs a second-order RC equivalent circuit. Let $x = [\text{SOC}, U_1, U_2, R_0]^T$ be the state vector, $u = [I, T]^T$ the input vector, and $Y = V_t$ the output vector. the expressions for the state and output equations are given by Equation (1).

$$\begin{cases} \text{SOC}_{k+1} = \text{SOC}_k - \frac{\eta \Delta t I_k}{Q_k} + W_{\text{SOC},k} \\ U_{1,k+1} = e^{-\Delta t/(R_1 C_1)} U_{1,k} + R_1 (1 - e^{-\Delta t/(R_1 C_1)}) I_k + W_{1,k} \\ U_{2,k+1} = e^{-\Delta t/(R_2 C_2)} U_{2,k} + R_2 (1 - e^{-\Delta t/(R_2 C_2)}) I_k + W_{2,k} \\ R_{0,k+1} = R_{0,k} + W_{R,k} \\ Y_k = \text{OCV}(\text{SOC}_k) - I_k R_{0,k} - U_{1,k} - U_{2,k} + V_n \end{cases} \quad (1)$$

In Equation (1), SOC represents the state of charge. U_1 and U_2 represent the polarization voltages of the two RC branches, respectively. R_0 represents the ohmic internal resistance. I represents the discharge current. T represents the battery temperature. V_t represents the terminal voltage. η represents the coulombic efficiency. Δt represents the sampling time. R_1, C_1 and R_2, C_2 represent the polarization resistance and polarization capacitance of the two RC branches, respectively. $\text{OCV}(\cdot)$ represents the open-circuit voltage function. $W_{\text{SOC},k}$, $W_{1,k}$, $W_{2,k}$, and $W_{R,k}$ represent process noise. and V_n represents measurement noise. The parameters are obtained through a combination of offline identification and online adjustment. During the offline phase, hybrid pulse power

characterization tests are conducted on the battery pack. pulse currents are applied at various states of charge (SOC) while terminal voltage responses are recorded, and the Recursive Least Squares (RLS) method is used to fit the initial values of ohmic resistance, polarization resistance, and polarization capacitance. The relationship between open-circuit voltage and SOC is determined via low-current charge/discharge and rest tests, utilizing polynomial fitting. In the online phase, the AEKF adaptively adjusts the process noise covariance and measurement noise covariance based on the innovation sequence, thereby maintaining parameter validity amidst aging and changing operating conditions. The battery pack has a rated capacity of 60 Ah and a nominal voltage of 12.8 V, utilizing a lithium iron phosphate chemistry. The parameters are set as follows: ohmic internal resistance $R_0 = 80 \text{ m}\Omega$. polarization resistance $R_1 = 0.5 \text{ m}\Omega$. polarization capacitance $C_1 = 2000 \text{ F}$. polarization resistance $R_2 = 0.3 \text{ m}\Omega$. polarization capacitance $C_2 = 5000 \text{ F}$. and coulombic efficiency $\eta = 0.98$. The AEKF linearizes nonlinear data by calculating the Jacobian matrix, which consists of the partial derivatives of the state and output equations with respect to the state variables. AEKF linearizes nonlinear data by calculating the Jacobian matrix. The state estimation process of AEKF is shown in Equation (1).

$$\hat{x}_{n|n-1} = \phi_{n|n-1} \hat{x}_{n-1|n-1} \quad (1)$$

In Equation (1), $\hat{x}_{n-1|n-1}$ represents the estimated value at the $n - 1$ -th step. $\phi_{n|n-1}$ represents the system transition matrix at the n -th step. $\hat{x}_{n|n-1}$ represents the prior estimate at the n -th step. The process of calculating the error covariance matrix in AEKF is shown in Equation (2).

$$P_{n|n-1} = \phi_{n|n-1} P_{n-1|n-1} \phi_{n|n-1}^T + Q \quad (2)$$

In Equation (2), $P_{n|n-1}$ represents the error covariance matrix of the prior estimate. Q represents the covariance matrix of system noise, $Q = \text{diag}(1 \times 10^{-8}, 1 \times 10^{-6}, 1 \times 10^{-6}, 1 \times 10^{-10})$. The calculation of the Kalman gain is shown in Equation (3).

$$K_{n|n-1} = P_{n|n-1} C^T (C P_{n|n-1} C^T + R)^{-1} \quad (3)$$

In Equation (3), $K_{n|n-1}$ represents the Kalman gain. R represents the covariance matrix of measurement error, with a value of $1 \times 10^{-6} \text{ V}^2$, corresponding to a standard deviation of 1 mV for the terminal voltage measurement. C represents the measurement matrix.

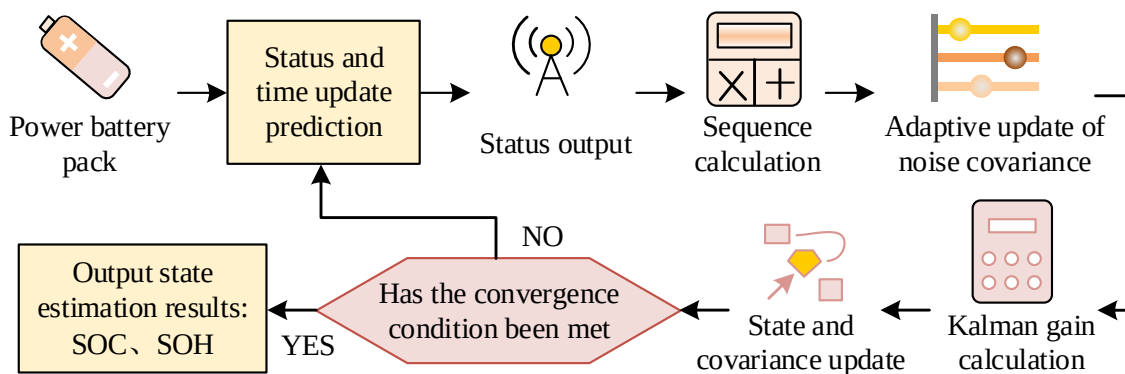


Fig. 2. Process of battery state estimation and monitoring based on the AEKF algorithm

The covariance matrix estimation for the next time step is shown in Equation (4).

$$\begin{cases} P_{n|n} = (I - K_{n|n-1} C) P_{n|n-1} \\ x_n = \phi_{n|n-1} x_{n-1|n-1} + W_{n-1} \\ Y_n = C x_n + V_n \end{cases} \quad (4)$$

In Equation (4), $P_{n|n}$ represents the covariance estimation at the next time step. W_{n-1} represents system noise. V_n represents measurement noise. The battery state estimation and monitoring process based on this algorithm framework is shown in Figure 2.

As shown in Figure 2, AEKF updates the initial state estimate and covariance matrix of the battery pack to obtain the prior state estimate and prior error covariance. It then calculates the measured voltage difference using the innovation sequence. After updating and computing the covariance matrix, the algorithm determines the weight allocation between predicted values and measured values. The state and covariance measurements are then corrected to output the posterior state estimate. The algorithm performs a prediction and correction step at each sampling instant, with sampling intervals set to 10 Hz (0.1 s) or 100 Hz (0.01 s) depending on the experiment. The convergence criteria for the iteration are that the Euclidean distance between consecutive state estimates is less than 1×10^{-4} , or the rate of change of the innovation covariance is less than 1%. If the convergence criteria are met or the maximum of 50 iterations is reached, the real-time state estimate is output; otherwise, the process returns to the prediction stage to continue iterating. The monitoring process concludes when the total experiment duration reaches 5000 s or the battery pack ceases operation. This study combines HARA with AEKF to form the AEKF-HARA algorithm. The algorithm achieves high safety objective compliance and strong technical traceability. It also addresses the influence of redundant signals. The process of adjusting battery pack adaptability and observation noise is shown in Equation (5).

$$N_k = \hat{y}_k y_k^T - H_k P_{k|k-1} H_k^T \quad (5)$$

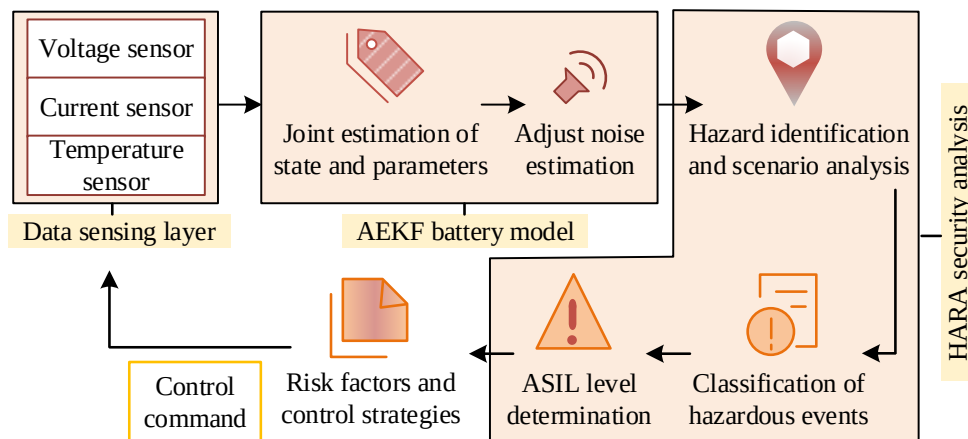


Fig. 3. Process of battery pack monitoring and analysis using the AEKF-HARA algorithm

In Equation (5), N_k represents the innovation covariance. H_k^T represents the observation matrix. The AEKF utilizes sliding statistics of the innovation to update the process and measurement noise covariance matrices online. The noise covariances are increased when innovation fluctuations rise, thereby enhancing dynamic adaptability, and are reduced when the innovation is stable to maintain estimation consistency. A forgetting factor is introduced to accelerate the update process when the rate of change of the innovation exceeds a specific threshold. The updated noise covariances are then employed for prediction and gain calculation in the subsequent time step, enabling adaptation to aging effects and changes in operating conditions. The monitoring and analysis process of power battery packs using AEKF-HARA combined with sensor signals is shown in Figure 3.

As shown in Figure 3, AEKF-HARA collects physical battery signals through sensors and inputs them into AEKF for parameter adjustment. AEKF predicts battery state estimation and error covariance based on sensor signals. It adjusts noise covariance in real time, maintains battery performance, and outputs the state of charge. The state of charge enters the HARA framework definition. HARA performs fault diagnosis of the battery pack through rule-based and residual analysis. It analyzes hazard types and events, evaluates hazard risks, and determines the ASIL level. Finally, AEKF-HARA triggers corresponding management decisions based on risk assessment results. It executes safety measures defined in HARA and outputs control commands that affect sensors. This process completes real-time monitoring and analysis of the power battery pack. The correction process of estimation values in AEKF-HARA is shown in Equation (6).

$$\hat{x}_{n|n} = \hat{x}_{n|n-1} + K_{n|n-1}(Y_n - C\hat{x}_{n|n-1}) \quad (6)$$

In Equation (6), $\hat{x}_{n|n}$ represents the posterior estimate. Y_n represents the measured value.

3.2 Establishment of A Real-Time Monitoring Model with the Introduction of the AE Algorithm

The HARA-AEKF algorithm predicts state estimation and trend evolution through AEKF, while HARA performs reliable and protective safety actions. The two components work together to complete real-time monitoring of power battery packs. However, this algorithm still shows limitations in data utilization and detection of unknown faults. Autoencoder (AE) extracts historical data and automatically learns operating patterns under normal conditions. It explores deeper and unknown faults as well as nonlinear relationships. This process improves the utilization of sensor signals and enhances real-time and automatic monitoring performance [16-17]. Therefore, this study introduces AE to optimize the AEKF-HARA algorithm and develops the AE-AEKF-HARA real-time monitoring model for power battery packs, referred to as the AAH model. The model aims to achieve accurate state tracking and provide real-time and reliable hazard mitigation solutions. The real-time monitoring process based on AE is shown in Figure 4.

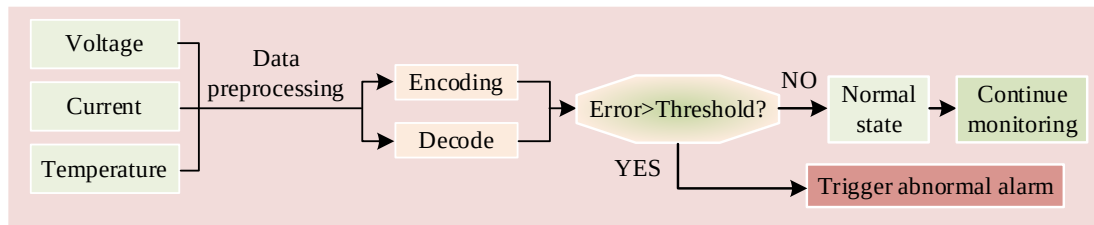


Fig. 4. Process of real-time battery pack monitoring based on the AE algorithm

In Figure 4, AE collects historical normal operation data of power battery pack, including voltage, current and temperature. The data were normalized by Z-score, and the missing values were supplemented by linear interpolation. The network input dimension is 3, the encoder and decoder are symmetrical, the number of neurons in the hidden layer is 16, 8, 4, 8 and 16, the activation function is relu, and the output layer is sigmoid. Adam optimizer was used for training, with a learning rate of 0.001, a batch size of 64, and 200 iterations. The training set and validation set were divided by 8:2. The abnormal threshold is determined based on the mean and standard deviation of the reconstruction error of the normal verification set. The threshold $\tau = \mu + 3\sigma$ is taken and verified on the fault verification set to ensure that the false positive rate and false negative rate meet the requirements. After determining the reconstruction error of each sample, the algorithm performs real-time anomaly detection. If the reconstruction error exceeds the threshold, the sample shows abnormal patterns and triggers an anomaly alarm. If the reconstruction error is less than or equal to the threshold, the battery sample remains in a normal state and the algorithm continues monitoring the next anomaly point. This process completes battery pack state monitoring. The encoder output of AE is shown in Equation (7).

$$h = g(x) = \sigma(Wx + b) \quad (7)$$

In Equation (7), g represents the encoder. h represents the hidden variable output of the encoder. σ represents the activation function of the encoder. b and W represent the bias vector and weight matrix of the encoder. The decoder output is shown in Equation (8).

$$\hat{x} = f(g(x)) = \sigma'(W'h + b') \quad (8)$$

In Equation (8), $\hat{x}$ represents the decoder output. f represents the decoder. h represents the hidden variable. σ' represents the activation function of the decoder. W' and b' represent the weight matrix and bias vector of the decoder. The loss function of reconstruction error in AE is shown in Equation (9).

$$L_{MSE}(\theta) = \frac{1}{n} \sum_{i=1}^n \|x_i - \hat{x}_i\|^2 \quad (9)$$

In Equation (9), θ represents the parameters used in the encoder and decoder. The training process minimizes reconstruction error through parameter θ . n represents the number of samples. x_i represents the original data. $\hat{x}_i$ represents the reconstructed data. The state monitoring and analysis process of power battery packs based on the AAH model with AE is shown in Figure 5.

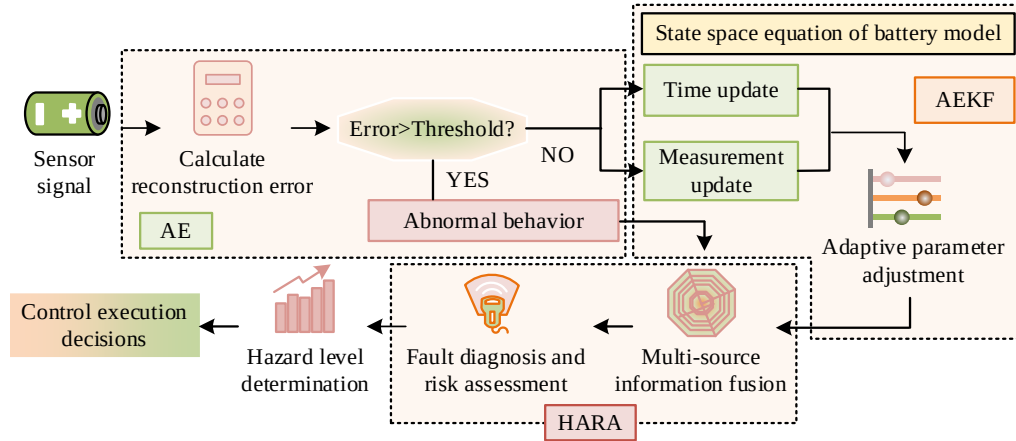


Fig. 5. Process of battery pack status monitoring and analysis based on the AAH model

As shown in Figure 5, the AAH model preprocesses physical signals collected from power battery packs by sensors and inputs them into the AE module. The encoder compresses sensor signals into low-dimensional features, and the decoder reconstructs the features into the original dimensions. AE performs primary anomaly detection. If the reconstruction error exceeds the threshold, the data pattern is abnormal and triggers a primary anomaly flag. If the error remains within the threshold, the AE output data enter the AEKF module. In AEKF, the algorithm updates time and measurement step and combines the battery state model to adjust parameters such as real-time noise. This process improves the accuracy of state of charge and state of health detection. Then, the model takes the self-encoder reconstruction error, anomaly flag, innovation and normalized residual of extended Kalman filter, state of charge, health state and internal resistance change as HARA inputs. The severity is determined in combination with hazard type, voltage difference, temperature rise and state of charge. The degree of occurrence or exposure shall be determined in combination with abnormal signs, residual overrun duration and occurrence frequency. Controllability is determined in combination with response time, redundant sensors and execution capability. ASIL level is determined according to ISO 26262 risk matrix, and corresponding safety objectives and trigger thresholds are selected from table 3-1. The control decision is implemented according to the level. Level D cuts off the high voltage and starts the emergency thermal management. Level C reduces the power and alarms. Level B limits the charge and discharge and encrypts the sampling. Level A record, prompts and calibrates. If the self-encoder anomaly flag triggers or the normalized residual continues to exceed the limit, the unknown anomaly or sensor inconsistency is input into HARA as an additional hazard, triggering degradation monitoring, redundancy switching or model estimation replacement. The domain reconstruction error calculation process of the AAH model is shown in Equation (10).

$$r(x) = \|x - \hat{x}\|^2 - \text{mean}_{z \in N_k(x)} (\|z - \hat{z}\|^2) \quad (10)$$

In Equation (10), $r(x)$ represents the neighborhood reconstruction error. z represents neighborhood samples. $N_k(x)$ represents the set of k -domain samples used to train observation values. The neighborhood size is set to 5, and the distance metric uses Euclidean distance. The domain sample set is composed of 80% randomly selected from normal training data, with a total of 10000 normal training samples. Therefore, the NN contains 8000 samples, and the remaining 2000 samples are used as the validation set. For each sample, first search for the 5 nearest neighboring samples from the domain sample set, calculate the mean reconstruction error of these neighboring samples, and then subtract the mean from the reconstruction error of the sample itself

to obtain the domain reconstruction error. This value is used to measure the degree of deviation of the sample reconstruction error from its neighborhood average level, with larger deviations being more likely to be abnormal. The process of detecting the variation rate of observation noise in the AAH model is shown in Equation (11).

$$\gamma = \left| \frac{\sum_{i=k-N}^k N_i - \sum_{i=k-2N}^{k-N} N_i}{\sum_{i=k-2N}^k N_i} \right| \quad (11)$$

In Equation (11), γ represents the estimated variation rate of observation noise and reflects the degree of observation noise variation. $2N$ represents the width of the time window. k represents the current time step. The process of adjusting the forgetting factor based on the variation rate of observation noise is shown in Equation (12).

$$b = b_{\min} \max_{\max} \quad (12)$$

In Equation (12), b represents the forgetting factor at the current time step. $b_{\max}$ represents the maximum value of the forgetting factor. $b_{\min}$ is the minimum value of the forgetting factor.

4. Performance of The Battery Monitoring Model Combining Sensor Signals And HARA

4.1 Verification of Monitoring and Analysis Performance Of AEKF-HARA

To verify the real-time detection and analysis performance of the AEKF-HARA algorithm, this study conducted comparative experiments with three real-time monitoring algorithms: Bidirectional Long Short-Term Memory Network (BiLSTM), Adaptive Neuro-Fuzzy Inference System (ANFIS), and CNN-SVM, which combined Convolutional Neural Network (CNN) with Support Vector Machine (SVM). The bidirectional long short-term memory network can capture the temporal dependencies of battery signals. The adaptive neural fuzzy inference system combines the adaptive capabilities of fuzzy inference and neural networks. CNN-SVM uses convolutional layers to extract local features and is classified by support vector machines. These three types of methods represent typical applications of deep learning, fuzzy reasoning, and hybrid models in battery monitoring and fault diagnosis. To ensure fairness in comparison, all baseline methods were trained and tested under the same dataset partitioning, input features, and evaluation conditions. The experimental environment was set up as follows: the operating system was Windows 11, the CPU was Intel Core i9-14900K, the GPU was NVIDIA RTX 4090 24GB, the memory was 64 GB, the deep learning framework was PyTorch 2.0.1, and the programming language was Python 3.9. BiLSTM is set to two layers, with 64 and 32 hidden units respectively, dropout of 0.2, optimizer Adam, learning rate of 0.001, batch size of 64, maximum iteration of 200 rounds, using early stopping strategy, and patience value of 20. ANFIS adopts Gaussian membership functions, with 3 membership functions for each input variable. The training algorithm is mixed learning, with a maximum of 100 iterations. In CNN-SVM, CNN consists of two convolutional layers with kernel sizes of 3 and 3, and numbers of 16 and 32, respectively. The pooling layer uses max pooling, and the fully connected layer outputs 64 dimensional features. SVM uses radial basis kernel functions, and the penalty parameters C and gamma are determined on the validation set through grid search. All baseline methods use the same training, validation, and testing sets as AEKF-HARA, with consistent input features and evaluation metrics, and repeat the experiment 5 times to take the mean. The hyperparameters are determined on the validation set through grid search or early stopping strategy to ensure optimal performance for each baseline. The datasets used in the experiment are the Oxford Battery Degradation Dataset (OBD) and the University of Maryland CALCE Battery Research Group Dataset (CALCE). The OBD dataset uses Oxford Battery Degradation Dataset 1, which includes 8 Kokam SLPB 533459H4 batteries with a sampling interval of 1 second. The features include voltage, current, temperature, and SOC, and the operating conditions are constant current charging and discharging and dynamic driving cycles. The CALCE dataset uses CS2 and CX2 series, selects 12 batteries with a sampling interval of 1 second, and features voltage, current, temperature, and capacity. The operating conditions are constant current charge discharge and DST. Both datasets are divided in chronological order, with the top 70% used as the training set, 15% as the validation set, and 15%

as the testing set to avoid time series data leakage. The training set is used for model parameter fitting, the validation set is used for hyperparameter selection and anomaly threshold determination, and the test set is only used for final performance evaluation. All experiments were repeated 5 times, and the results were expressed as mean $\pm$ standard deviation. One way analysis of variance was used for inter group comparison, with a significance level of 0.05. This study conducted experiments with the AEKF-HARA algorithm and the three comparison algorithms (BiLSTM, ANFIS, and CNN-SVM) on the OBD dataset, using two metrics to measure performance: Root Mean Square Error (RMSE) of state of charge estimation accuracy and voltage boundary prediction accuracy. The root mean square error of the state of charge estimation accuracy is obtained by taking the square root of the difference between the predicted value and the true value, expressed as a percentage. The smaller the value, the smaller the estimation error. The accuracy of voltage boundary prediction is the proportion of samples whose predicted voltage falls within the allowable boundary of the true voltage. The allowable boundary is taken as 2% of the true voltage. This indicator is dimensionless, and the closer it is to 1, the more reliable the predicted voltage is. The results are shown in Figure 6.

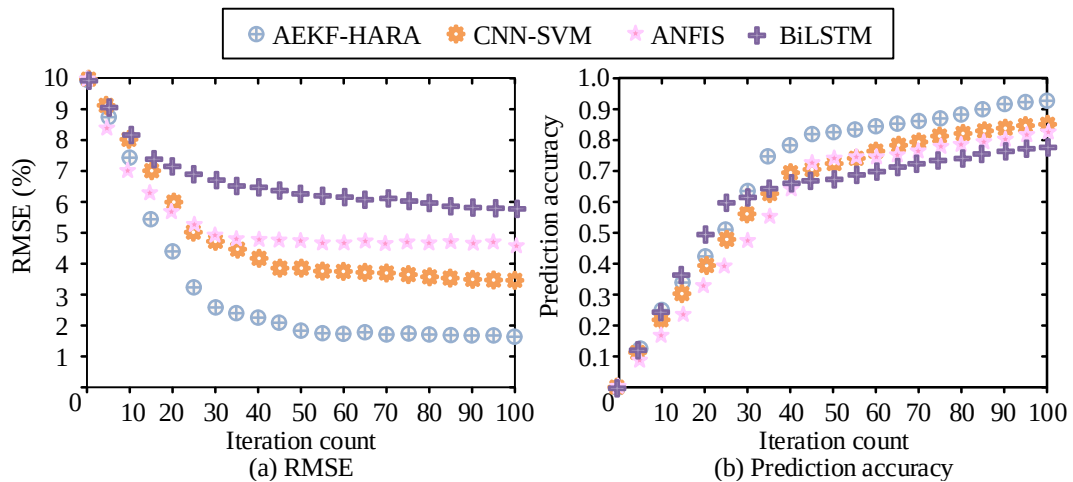


Fig. 6. Comparison of algorithm performance indicators on the OBD dataset

As shown in Figure 6(a), the root mean square error of the state of charge estimation accuracy of AEKF-HARA algorithm is $1.712\% \pm 0.085\%$, while that of BiLSTM algorithm is $5.804\% \pm 0.216\%$. The difference between the two is 4.092 percentage points, and the difference is statistically significant ($p < 0.05$). From the results in Figure 6(b), the accuracy of voltage boundary prediction using the AEKF-HARA algorithm reached a value of 0.934, which showed an improvement of 0.123 over the BiLSTM algorithm. The earlier results showed that the AEKF-HARA algorithm was capable of achieving higher accuracy in the estimation of the state of charge and voltage boundary prediction of the battery. By using the benefits provided by AEKF's adaptive adjustment of parameters, the analysis of the algorithm was improved. To further verify the superiority of the real-time monitoring performance of the AEKF-HARA algorithm, this study conducted experiments with it and the three comparison algorithms on the CALCE dataset, using two metrics for evaluation: fault identification delay time and identification accuracy. The fault recognition delay is the average delay of all fault types, including overcharge, over discharge, overtemperature, internal short circuit, connection fault, voltage sensor fault, current sensor fault, temperature sensor fault, thermal runaway spread, and insulation fault. The results are shown in Figure 7.

As shown in Figure 7 (a), the delay time for fault recognition using the AEKF-HARA algorithm is 29.87 ± 1.24 ms, and the recognition accuracy is $92.41\% \pm 1.05\%$. From Figures 7 (b), (c), and (d), it can be seen that the CNN-SVM algorithm achieved good results in various indicator tests. However, compared with the AEKF-HARA algorithm, its delay time increased by 10.72 ms and the recognition accuracy decreased by 5.22 percentage points, with statistical significance ($p < 0.05$). From the above results, it could be concluded that the proposed AEKF-HARA algorithm could react promptly to the identification of dangerous battery types and efficiently capture fault features. The timely and reliable data support offered by the proposed algorithm could play an important role in maintaining battery operation stability. This was due to the efficient combination of the AEKF

algorithm and HARA, enabling fast identification and feature extraction, increasing the efficiency of identification. To further analyze the real-time performance of each method, Table 2 compares the inference time, model size, and memory usage of each component of AEKF-HARA with the comparison model.

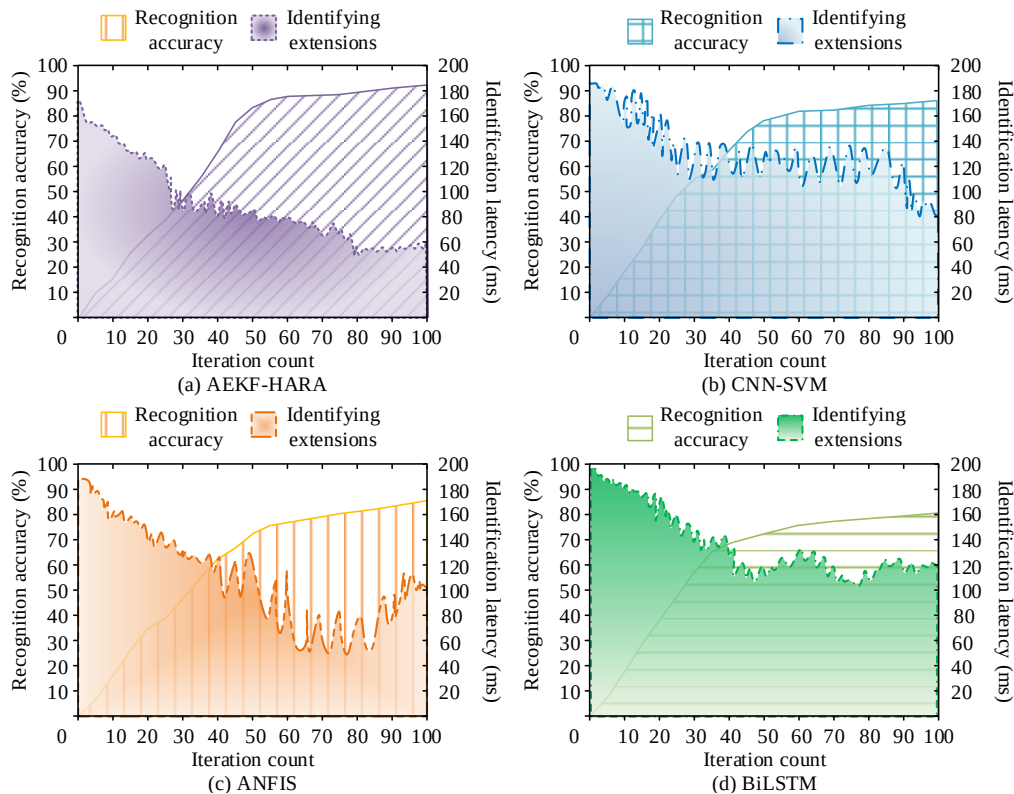


Fig. 7. Comparison of real-time monitoring performance on the CALCE dataset

Table 2. Comparison of real-time performance of different models

Model	Inference Time (ms)	Model Size (MB)	Memory Usage (MB)
AEKF-HARA	29.87±1.24	0.65	120
CNN-SVM	40.59±1.86	1.80	210
ANFIS	46.32±2.05	1.20	180
BiLSTM	52.14±2.31	2.50	260
Model based on Whale Optimization Algorithm and Long Short-Term Memory Algorithm	38.65±1.72	2.10	240

According to Table 2, the inference time of AEKF-HARA is 29.87±1.24 ms, which is lower than other models. The inference time of CNN-SVM and models based on whale optimization algorithm and long short-term memory algorithm are 40.59±1.86 ms and 38.65±1.72 ms, respectively. The inference time of ANFIS and BiLSTM both exceed 45 ms. In terms of model size and memory usage, AEKF-HARA is 0.65 MB and about 120 MB, respectively, which are lower than other models. The above results indicate that AEKF-HARA has lower inference latency and resource consumption while maintaining high recognition accuracy, making it suitable for embedded deployment. To verify that the proposed algorithm possessed more advantages in real-time monitoring performance, the experiments are carried out with the proposed algorithm and the other three algorithms on the OBD dataset and CALCE dataset. The experiments assessed the algorithms based

upon three metrics: monitoring precision, recall, and F1 score. At the same time, whale optimization algorithm and long short-term memory algorithm [8] were introduced. The experimental results are shown in Table 3.

Table 3. Comparison of algorithm performance indicators

Dataset	Algorithm	Precision (%)	Recall (%)	F1 Score (%)
OBD	AEKF-HARA	96.62%±0.58%	95.18%±0.62%	95.89%±0.55%
	CNN-SVM	91.35%±0.82%	90.47%±0.86%	90.90%±0.79%
	ANFIS	88.24%±0.91%	86.91%±0.95%	87.57%±0.88%
	BiLSTM	87.71%±0.96%	86.05%±1.02%	86.87%±0.94%
	The method proposed by Zhao and Bai	95.02%±0.67%	94.76%±0.70%	94.89%±0.65%
CALCE	AEKF-HARA	96.33%±0.61%	95.12%±0.64%	95.73%±0.58%
	CNN-SVM	90.58%±0.85%	89.96%±0.88%	90.27%±0.82%
	ANFIS	89.61%±0.89%	88.44%±0.93%	89.02%±0.91%
	BiLSTM	86.86%±0.98%	85.27%±1.05%	86.06%±0.96%
	The method proposed by Zhao and Bai	94.85%±0.69%	94.58%±0.72%	94.71%±0.67%

As shown in Table 3, the AEKF-HARA algorithm has higher accuracy, recall, and F1 score than other comparison methods on the OBD and CALCE datasets. On the OBD dataset, its F1 value is 95.86%±0.55%, which is 4.95, 8.29, 9.02, and 0.97 percentage points higher than CNN-SVM, ANFIS, BiLSTM, and models based on whale optimization algorithm and long short-term memory algorithm, respectively. On the CALCE dataset, its F1 value is 95.73%±0.58%, which is 5.58, 9.51, 8.70, and 1.02 percentage points higher than the four methods, respectively. The above differences are statistically significant ($p < 0.05$). The above data results indicated that the AEKF-HARA algorithm demonstrated excellent monitoring and analysis performance on both datasets, possessing accurate feature capture capability. This was because the HARA method in this algorithm provided accurate hazard type and risk assessment standards, and combined with the adaptive filtering capability of the AEKF algorithm, it adjusted and optimized battery system parameters in real time, monitoring battery states more effectively.

4.2 Performance Analysis of The AAH Battery Monitoring Model

After verifying the superiority of the real-time monitoring performance of the AEKF-HARA algorithm, this study conducted comparative experiments with the AAH model and three monitoring models: CNN-SVM, ANFIS, and BiLSTM. The experimental environment parameter settings were the same as those used in the experiments verifying the real-time monitoring performance of the AEKF-HARA algorithm. The experimental datasets used were the OBD dataset, CALCE dataset, and a self-made dataset. The content details of the self-made dataset are shown in Table 4.

As shown in Table 2, the self-built dataset is based on a second-order RC equivalent circuit model, with a rated capacity of 60 Ah and a nominal voltage of 12.8 V for the battery pack, using a lithium iron phosphate system. The circuit parameters are Ohmic internal resistance of 0.8 m Ω , polarization resistance R1 of 0.5 m Ω , R2 of 0.3 m Ω , polarization capacitance C1 of 2000 F, C2 of 5000 F, and Coulomb efficiency of 0.98. Fault injection includes voltage sensor offset of ± 50 mV, ± 100 mV, stuck holding constant, accuracy decrease noise standard deviation of 5 mV, current sensor noise standard deviation of 0.05C, rate deviation of $\pm 5\%$, temperature sensor reading drift of ± 2 $^{\circ}\text{C}$, and connection fault internal resistance increase of 30% and 50%. The duration of fault injection is randomly distributed between 10 s and 60 s, and each fault type is tested 10 times. The truth value is generated by the injection time and fault type label, and normal samples and fault samples are mixed in chronological order.

Table 4. Details of the self-made dataset

Type	Parameter	Specific settings
Battery data and model	Cell model	Second order RC equivalent circuit model
	Battery pack configuration	Rated capacity: 60 Ah Nominal voltage: 12.8 V (3.2V/cell)
	Experimental data source	Standard driving cycle (NEDC/WLTC) generates voltage, current, and temperature data
Fault injection setting	Fault Type	Generation of voltage, current, and temperature data based on standard driving cycles (NEDC/WLTC)
		Voltage sensor: offsets of ± 50 mV and ± 100 mV, signal sticking, and accuracy degradation (noise standard deviation of 5 mV)
		Current sensor: noise injection (standard deviation of 0.05C) and rate deviation of $\pm 5\%$ Temperature sensor: reading drift of $\pm 2^\circ\text{C}$ Connection faults: 30% and 50% increases in connection point internal resistance
Experimental setup	Injection timing	The time distribution was randomized throughout the experiment, lasting from 10 s to 60 s.
	Sampling frequency	10 Hz & 100 Hz
	Experimental time	5000s
	Ttest scenario	Environmental temperature: 25 $^\circ\text{C}$
		Charge discharge rate: 1C

The data is divided in chronological order, with the first 70% being the training set, 15% being the validation set, and 15% being the testing set. The experiment was repeated 5 times, and the results were expressed as mean standard deviation. One way analysis of variance was used to compare the differences between the models. The fault injection in the self-built dataset is divided into sensor level faults and physical battery faults. Sensor level faults include voltage sensor offset, jamming, and decreased accuracy, current sensor noise and rate deviation, and temperature sensor reading drift. Physical battery failures include increased internal resistance at connection points, which extends to internal short circuits and overcharging. The AAH model simultaneously detects these two types of objects, AEKF determines sensor signal consistency through information and residuals, AE identifies unknown anomalies through reconstruction errors, and HARA determines ASIL levels based on hazard type, severity, exposure/occurrence, and controllability. Sensor anomalies are mapped as voltage, current, or temperature sensor faults according to Table 2, triggering redundant switching or model estimation substitution. When voltage dip, temperature rise, and AE reconstruction error occur simultaneously, they are mapped as physical battery hazards such as internal short circuits, triggering the disconnection of high voltage and emergency thermal management. In this way, sensor level faults and physical battery hazards can be distinguished in monitoring and risk assessment, and corresponding safety decisions can be formed. This study conducted experiments with it and the three comparison models on the CALCE dataset, using two metrics to evaluate the monitoring performance of the models: Coefficient of Determination (R^2) and fault detection consistency. The coefficient of determination R^2 is calculated according to the standard formula, and the closer it is to 1, the better the fit. The consistency of fault detection is the proportion of samples whose fault types output by the model are consistent with the actual fault types, expressed as a percentage of the total sample size. The larger the proportion, the more reliable the monitoring results will be. The experimental results are shown in Figure 8.

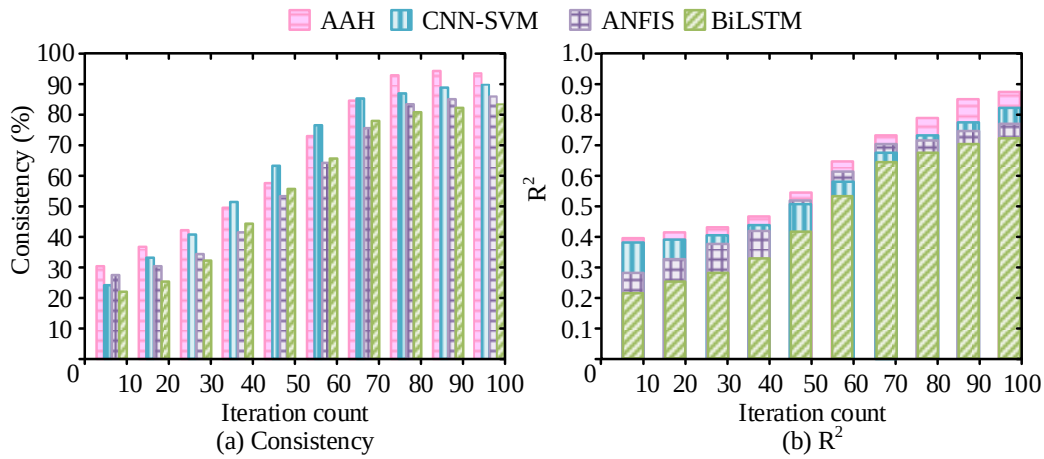


Fig. 8. Comparison of model performance indicators on the CALCE dataset

As shown in Figure 8(a), the AAH model achieved a result of $94.28\% \pm 0.76\%$ in the fault detection consistency test, outperforming the other three comparative models and showing an absolute improvement of 4.27 percentage points over the CNN-SVM model. The difference is statistically significant ($p < 0.05$). In Figure 8(b), the R^2 value of the AAH model reached 0.872 ± 0.015 , improving by 0.048 and 0.101 compared to the CNN-SVM model and ANFIS model, respectively, and the difference was statistically significant ($p < 0.05$). The above results indicated that the AAH model had high consistency, proving that this model could accurately identify battery fault features, effectively controlling the occurrence of missed or false alarms, and could deeply capture the intrinsic features of the battery system, thereby enhancing the real-time monitoring capability of the model.

In order to further evaluate the real-time monitoring performance of the AAH model, this study tested the AAH model and the three comparison models mentioned above on the OBD dataset, and evaluated them through insulation resistance estimation error (MAE). The estimation error of insulation resistance is obtained by dividing the absolute value of the difference between the estimated isolation resistance after fault isolation and the true isolation resistance by the nominal voltage, and then taking the average of all samples. The unit is Ω/V . The smaller the value, the higher the isolation accuracy. The test results are shown in Figure 9.

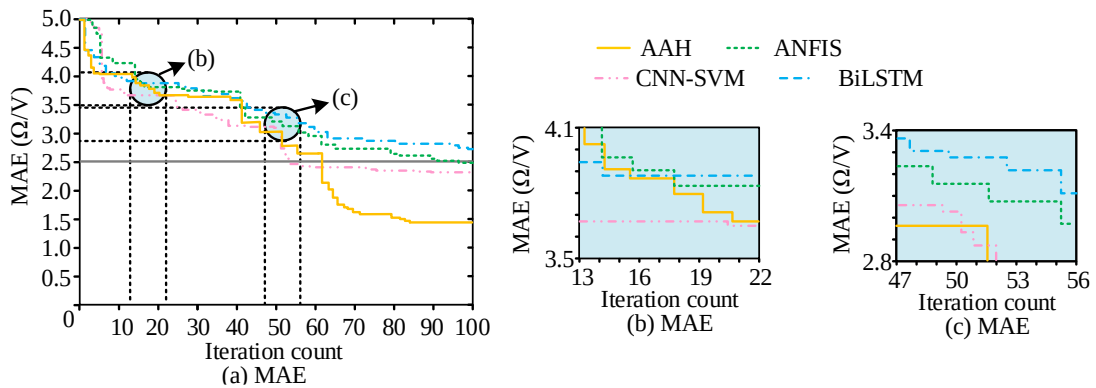


Fig. 9. Comparison results of insulation resistance estimation errors

As shown in Figure 9(a), After 100 iterations, the AAH model achieved the best performance with an estimated insulation resistance error (MAE) of $1.492 \Omega/V$. The weakest performer was the BiLSTM model, with an MAE value of $2.716 \pm 0.113 \Omega/V$, an increase of $1.224 \Omega/V$ compared to the AAH model, and the difference is statistically significant ($p < 0.05$). In Figures 9(b) and 9(c), the error reduction trend of the AAH model in the early stage was at a moderate level, but as the number of iterations increased, the error reduction speed of the AAH model accelerated and gradually stabilized at the lowest level. The above results demonstrated that the AAH model could perform effective error control and accurately locate fault positions, possessing excellent error convergence capability and the ability to capture nonlinear data features. This benefited from the AEKF structure, which enhanced the model's ability to process complex data and the adaptability of

battery performance, reducing the possibility of errors. To more comprehensively evaluate the real-time monitoring performance of the AAH model, this study tested it and the three comparison models on the self-made dataset, measuring through the comparison of the gap between predicted State of Charge (SOC) values and actual SOC values. The accuracy of state of charge prediction is the proportion of samples whose predicted SOC falls within the range of plus or minus 2% of the true SOC to the total number of samples, expressed as a percentage. The larger the percentage, the more accurate the prediction. The test results are shown in Figure 10.

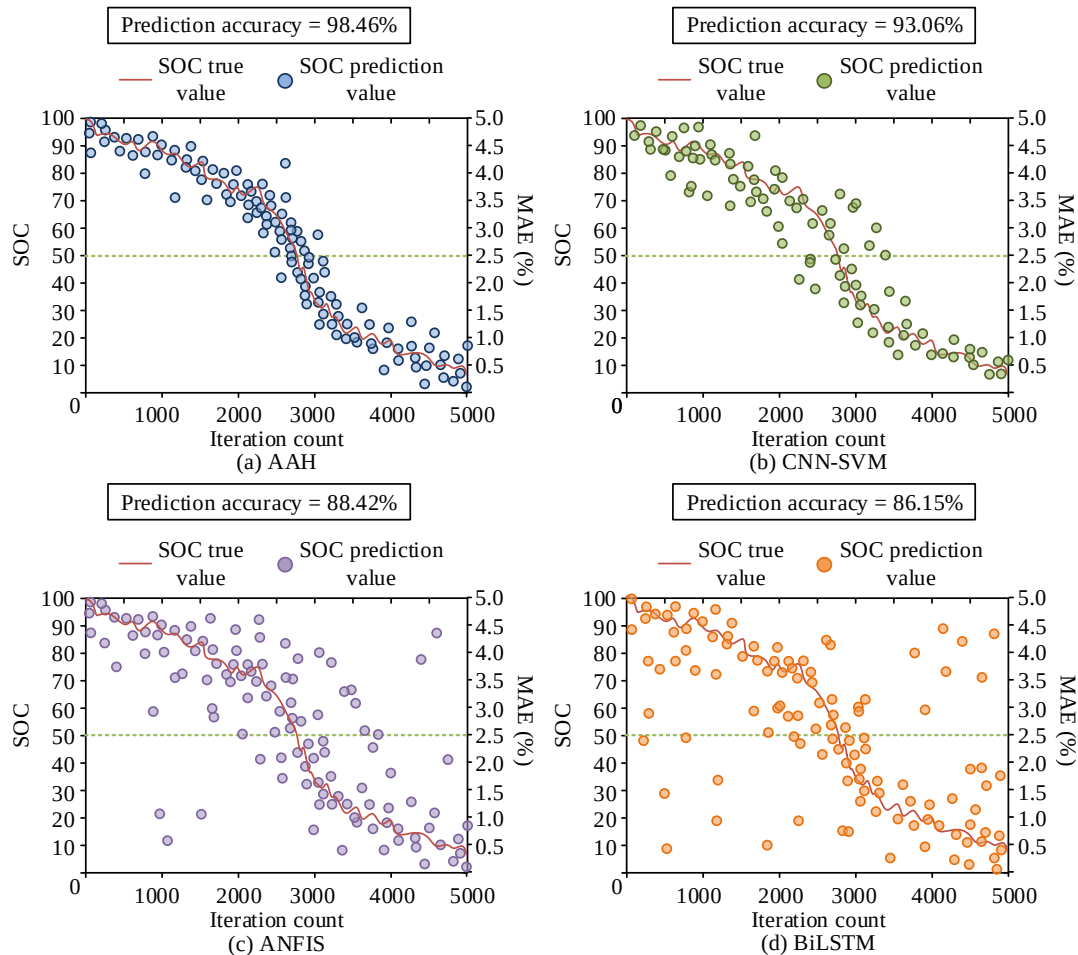


Fig. 10. Comparison of SOC prediction results

As shown in Figure 10(a), the average State of Charge (SOC) estimation accuracy of the AAH model reached $98.46\% \pm 0.42\%$ (representing a tracking error of less than 1.54%). In Figures (b), (c), and (d), the BiLSTM model had the lowest prediction result, reaching $86.15\% \pm 0.95\%$, and some prediction points were relatively scattered. Compared to the AAH model, the prediction ratio decreased by 12.31%, and the difference was statistically significant ($p < 0.05$). The above results indicated that the prediction points of the AAH model were closely distributed near the actual values, demonstrating that this model had highly accurate battery state estimation and measurement stability. This is because the HARA method in the AAH model can monitor the battery operating status in real time and reflect battery status information. While the AEKF structure and AE algorithm achieved detection and analysis of complex battery dynamics through adaptive and weight allocation mechanisms, dynamically adjusting parameters and improving the overall real-time monitoring performance of the model. To analyze the contribution of each module in the AAH model, this study conducted ablation experiments; the results are shown in Table 5.

As shown in Table 5, it can be seen that the fault detection consistency of AEKF when used alone is $85.34\% \pm 0.91\%$, the isolated MAE is $2.318 \pm 0.095 \Omega/V$, and the SOC prediction is $91.27\% \pm 0.74\%$, indicating its limited state estimation and noise adaptation capabilities. When used alone, the fault detection consistency of HARA is $82.16\% \pm 1.02\%$, and the isolated MAE is $2.506 \pm 0.108 \Omega/V$. The fault detection consistency of AEKF-HARA has increased to $90.57\% \pm 0.83\%$, the isolated MAE has

decreased to $1.896 \pm 0.082 \text{ } \Omega/\text{V}$, and the SOC prediction has reached $94.13\% \pm 0.61\%$, indicating that HARA's risk classification can improve monitoring decisions. The fault detection consistency of AE-AEKF is $88.92\% \pm 0.88\%$, the isolated MAE is $2.105 \pm 0.089 \text{ } \Omega/\text{V}$, and the SOC prediction is $93.58\% \pm 0.66\%$, indicating that AE enhances the ability of anomaly detection, but lacks the safety objectives and risk classification of HARA. The fault detection consistency of AAH reached $94.28\% \pm 0.76\%$, R^2 was 0.872 ± 0.015 , isolated MAE was $1.492 \pm 0.067 \text{ } \Omega/\text{V}$, and SOC prediction was $98.46\% \pm 0.42\%$, all of which were superior to other ablation models, and the differences were statistically significant ($p < 0.05$). The above results indicate that AEKF, HARA, and AE respectively undertake state estimation, risk decision-making, and anomaly detection functions. The combination of the three can effectively improve the accuracy and stability of real-time monitoring of battery packs. To evaluate the stability of the model under different operating conditions, voltage noise was taken as 1, 2, and 5 mV, current noise was taken as 0.01C, 0.05C, and 0.1C, temperatures were taken as -10, 25, and 45 °C, loads were taken as constant current, DST, and dynamic driving cycles, and aging states were taken as SOH of 100%, 90%, and 80%. The monitoring results under different noise levels, temperatures, load curves, and aging states are shown in Table 6.

Table 5. Results of ablation experiments on the AAH model

Model	Fault detection consistency (%)	R^2	Isolation MAE (Ω/V)	SOC prediction (%)
AEKF	$85.34\% \pm 0.91\%$	0.781 ± 0.018	2.318 ± 0.095	$91.27\% \pm 0.74\%$
HARA	$82.16\% \pm 1.02\%$	/	2.506 ± 0.108	/
AEKF-HARA	$90.57\% \pm 0.83\%$	0.834 ± 0.016	1.896 ± 0.082	$94.13\% \pm 0.61\%$
AE-AEKF	$88.92\% \pm 0.88\%$	0.812 ± 0.017	2.105 ± 0.089	$93.58\% \pm 0.66\%$
AAH	$94.28\% \pm 0.76\%$	0.872 ± 0.015	1.492 ± 0.067	$98.46\% \pm 0.42\%$

Table 6. Robustness results under different conditions

Condition	F1 score (%)	SOC RMSE (%)	Isolation MAE (Ω/V)
Benchmark, 25 °C, 1 mV, 0.01C, constant current, SOH 100%	$95.86\% \pm 0.55\%$	$1.712\% \pm 0.085\%$	1.492 ± 0.067
Voltage noise of 5 mV, current noise of 0.1C	$93.12\% \pm 0.85\%$	$2.45\% \pm 0.12\%$	1.82 ± 0.08
Temperature -10 °C	$92.35\% \pm 0.92\%$	$2.68\% \pm 0.14\%$	1.91 ± 0.09
Temperature 45 °C	$93.05\% \pm 0.88\%$	$2.31\% \pm 0.11\%$	1.78 ± 0.08
DST working condition	$94.20\% \pm 0.72\%$	$1.95\% \pm 0.10\%$	1.58 ± 0.07
Dynamic driving cycle	$93.88\% \pm 0.79\%$	$2.05\% \pm 0.10\%$	1.65 ± 0.07
SOH 90%	$93.56\% \pm 0.81\%$	$2.18\% \pm 0.11\%$	1.72 ± 0.08
SOH 80%	$92.74\% \pm 0.90\%$	$2.52\% \pm 0.13\%$	1.86 ± 0.09

From Table 6, it can be seen that under the conditions of voltage noise of 5 mV and current noise of 0.1C, the F1 score decreased to $93.12\% \pm 0.85\%$, a decrease of 2.74 percentage points compared to the baseline conditions, and the difference was statistically significant ($p < 0.05$). The SOC RMSE increased to $2.45\% \pm 0.12\%$, and the isolated MAE increased to $1.82 \pm 0.08 \text{ } \Omega/\text{V}$, with statistically significant differences compared to the baseline ($p < 0.05$). At -10 °C and 45 °C, the F1 scores were $92.35\% \pm 0.92\%$ and $93.05\% \pm 0.88\%$, respectively, with statistically significant differences compared to the baseline ($p < 0.05$).

The SOC RMSE were $2.68\% \pm 0.14\%$ and $2.31\% \pm 0.11\%$, respectively, and the isolated MAE were 1.91 ± 0.09 and $1.78 \pm 0.08 \text{ } \Omega/\text{V}$. Under DST and dynamic driving cycles, the F1 score remained above 93.88%, with statistical significance ($p < 0.05$) compared to the baseline, but the decrease

was relatively small. When SOH decreased from 100% to 80%, F1 score decreased from 95.86%±0.55% to 92.74%±0.90%, SOC RMSE increased from 1.712%±0.085% to 2.52%±0.13%, and isolated MAE increased from 1.492±0.067 to 1.86±0.09 Ω/V , with statistically significant differences ($p<0.05$). Overall, the model shows good robustness under extreme conditions, with F1 scores still above 92%, SOC RMSE below 2.7%, and isolated MAE below 2.0 Ω/V . To analyze the impact of hyperparameter selection on performance, F1 scores and SOC predictions were calculated for different network structures, anomaly thresholds, forgetting factors, and neighborhood sizes. The results are shown in Table 7.

Table 7. Sensitivity analysis of hyperparameters

Parameter	value	F1 score (%)	SOC prediction (%)
AE hidden layer neurons	8-4-2-4-8	92.15%±0.85%	96.12%±0.55%
	16-8-4-8-16	94.28%±0.76%	98.46%±0.42%
	32-16-8-16-32	94.35%±0.74%	98.51%±0.41%
Abnormal threshold	$\mu+2\sigma$	93.10%±0.82%	97.85%±0.48%
	$\mu+3\sigma$	94.28%±0.76%	98.46%±0.42%
	$\mu+4\sigma$	93.56%±0.80%	98.12%±0.45%
Lower limit of forgetting factor	0.90	93.42%±0.81%	97.95%±0.47%
	0.95	94.28%±0.76%	98.46%±0.42%
	0.98	94.05%±0.78%	98.30%±0.44%
Neighborhood size	3	93.65%±0.80%	98.05%±0.46%
	5	94.28%±0.76%	98.46%±0.42%
	7	94.12%±0.77%	98.35%±0.43%

5. Summary and Outlook

As the backbone of electric vehicles (EVs), the reliable and optimal function of power battery packs remains a significant task in optimizing automotive powertrain efficiency. However, conventional battery monitoring strategies often suffer from the deterioration of efficiency related to environmental factors and the lack of real-time capability. To address the aforementioned issue, the researchers suggested a new strategy of a sensor signal and HARA algorithm-combined battery monitoring and analysis model. This strategy not only employed sensor signal data with AEKF, aiming to improve the optimization of battery state identification and optimization, as well as the application of HARA to identify abnormal type classification. Subsequent to that, the strategy utilized the AE approach aiming to globally optimize the real-time capability of monitoring. Experimental findings confirmed that the RMSE of the state-of-charge estimation via the AEKF-HARA strategy was 1.712%±0.085%, the tracking accuracy was 0.934±0.012 (R^2), the monitoring precision exceeded 96%, the fault identification delay time was limited to 29.87±1.24 ms, and the identification accuracy reached 92.41%±1.05%, which is an absolute improvement of 5.22 percentage points compared to the CNN-SVM algorithm strategy. Meanwhile, the consistency of the AAH model strategy was measured to be 94.28%±0.76%, the insulation MAE is 1.492±0.067 Ω/V , and the predicted SOC accuracy was measured as 98.46%±0.42%, presenting a huge superiority over the other three models. This experiment successfully confirmed that the proposed AAH model strategy exhibited extremely high precision capability of monitoring and high-speed responses to the system monitoring. Although this experimental strategy has shown advantages over other methods in terms of monitoring efficiency and accuracy, its generalization ability in different battery chemistry systems and complex real-world operating conditions still needs to be improved. Therefore, future work will focus on integrating physics-informed machine learning (PINN) and transfer learning algorithms into the AAH framework to accurately predict long-term capacity degradation and state-of-health (SOH) trajectories under diverse operating histories and environmental temperatures.

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Comparison of shear strength testing methods and bearing capacity estimation for problematic soils in Mosul city, Northern Iraq

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Abstract

The study aims to investigate the strength characteristics, as represented by shear parameters, of various soil types that pose numerous engineering problems in and around Mosul, Iraq. Five soils, including low-plasticity clay, high-plasticity clay, silty organic soil, fill organic materials (10.2%), and gypseous soil (63.4%), were selected for the study, having serious geotechnical problems like swelling, collapse, high organic content, and high gypsum content. The testing program includes direct shear, residual shear, unconfined compression, oedometer, chemical tests, and the index properties of the study soils. The results generally showed that direct shear tests yield strength parameters that lead to overvaluation of bearing capacity. In contrast, unconfined compression tests yield more conservative results, and residual shear tests yield the smallest values due to the test conditions. The results indicated that shear properties can vary significantly across test methods, highlighting the need for careful interpretation of bearing capacity values. Taking safety considerations into account, the bearing capacity may range from 50 to 150 kN/m². Also, for the design of shallow foundations on Mosul soils, any available field or laboratory strength test for soil should be used in combination with tests that identify geotechnical issues in the soil, relying primarily on the test that yields the lowest value—or at least a reasonable value with engineering judgment—for the strength parameters.

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1. Introduction

The city of Mosul in Iraq is constantly expanding, and its construction rests on a variety of soil types. Since shear properties are important in determining soil bearing capacity, which is essential in foundation design, it is vital to highlight these soils and their shear values. Recognizing that shear strength parameters can vary with testing methods helps engineers make more informed decisions. The shear strength parameter is a key factor in evaluating soil bearing capacity and stability [1]. Many factors influence the shear strength, including the roughness of the soil particles' surfaces, the soil's mechanical properties, and the applied load. The shear strength at the interface, soil cohesion, angle of internal friction, and residual strength increase with increasing interface roughness. In contrast, the shear strength tends to remain constant with increasing shear displacement [2]. Shear resistance criteria are important for slope stability analyses to avoid landslide failure [3].

A fundamental understanding of shear behavior at the interface with foundation elements is important in the design of many geotechnical structures. The parameters of soil and surface characteristics that may control shear load-transfer mechanisms and shear strength at the interface between surfaces with structured roughness and cohesionless (sandy) soils require

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further study [4]. The shear strength of expansive soils is very important for engineering stability, not only because it shows typical variability with changes in moisture retention but also because it exhibits significant attenuation due to swelling-shrinkage behavior, fissuring, and over-consolidation [5]. [6] Investigate the impact of a humid, hot environment on the residual strength characteristics of soil, and study how changes in soil microstructure correlate with strength reduction in cycle tests. The soil's saturated shear strength continued to decrease because the interaggregate connections remained broken. In contrast, the deterioration of the intergranular connection in the aggregate was relatively slow, and the angle of internal friction in the soil showed a slight decrease, even remaining constant at a later stage. Several comparative studies were conducted to highlight differences between shear coefficients obtained from direct and residual shear tests, noting their effects on the geotechnical properties of clay soils [7–9].

Some previous studies have focused on the geotechnical properties of clay soils, both low- and high-plasticity, found in certain parts of Iraq, with an emphasis on soil resistance and shear strength [10–17]. The geotechnical properties of low-plastic-clay soil from Mosul, with added sand, have been studied by [18]. The researchers focus on resistance and shear properties, as measured by direct shear testing, unconfined compressive strength, and residual shear strength. The study results showed that increasing the amount of coarse, non-cohesive material, such as sand, increased shear strength to varying degrees depending on the test type, and led to a notable increase in bearing capacity of soil [18]. Other research has focused on the geotechnical properties and engineering problems associated with alluvial or clayey silt soils, which are abundant on riverbanks and in areas with dense vegetation, such as forests. These soils present several engineering challenges, including high organic and salt content and high susceptibility to collapse [14,19,20]. Gypsum formations cover vast areas of Iraq and are mixed with clay, silt, and sandy soils in varying concentrations. The presence of gypsum affects the engineering properties of soils, especially when they are exposed to water, which can cause gypsum to dissolve, thereby impacting soil strength, compressibility, and stability. Therefore, this topic has received considerable attention from researchers [21–26].

This study aims to determine the strength and resistance properties of soils found in northern Iraq, specifically in Mosul and its surrounding areas, which are experiencing significant urban expansion. The study also aims to compare the bearing capacity of the soils under investigation and to review the geotechnical complications associated with the soils, providing appropriate recommendations for foundation types and designs for various structures.

2. Materials of The Study

Five soil types were selected that cover almost the entire city of Mosul (divided by the Tigris River into two parts: the right and left banks), the surrounding areas, and many populated areas and cities in Nineveh Governorate. The samples were collected between 1.0 and 3.0 m below the ground level, which corresponded to the depths of most buildings' foundations in the city. Table 1 shows some geotechnical properties of soils used in the study. Figure 1 shows soil particles and the locations of the materials used in the study.

Low-plasticity clay soils are found in many residential areas of Mosul and its outskirts, and are among the most common soil types in northern Iraq [27,28]. High-plasticity clay soils are prevalent in parts of northern Mosul [27,28]. Furthermore, clay-silty soils with high gypsum content are found south of Mosul, extending to the southern borders of Nineveh Governorate [27,28]. Silty soils, with high organic content, are widespread along the banks of the Tigris River and in the forests of Mosul. Very high-organic-content soil deposits are concentrated in the center of Mosul, including the city's historic and archaeological areas and its markets. Many soil investigation reports for building, and several studies, show that the soil fill materials with high organic content are soils that have accumulated as a result of previous construction work; although they are not naturally deposited soils, they can reach depths of more than 10 meters in some locations [27,29].

Table 1. Geotechnical properties of soils used in the study

Soil type		Low plasticity clay	High plasticity clay	Low plasticity Silt	Organic soil	Gypsum soil
Sampling depth (m)		1.0	3.0	1.5	2.0	2.5
Soil condition at the site	Water content (%)	15	23.2	11.2	19.3	20.7
	Dry density kN/m ³	15.5	16.4	13.6	14.1	16.1
Atterberg limit	Liquid limit (w/c %)	46	87	27	35	30
	Plastic limit (w/c %)	26	38	22	N.P.	21
	Plasticity index (w/c %)	20	49	7	N.P.	9
Grain size analysis	Sand (%)	7	13	20	13	45
	Silt (%)	51	33	70	77	33
	Clay (%)	42	54	10	10	22
U.S.C.S	Group symbol	CL	CH	ML	OL	CL-ML
Compaction	Max. unit weight, (γ, kN/m ³)	15.59	13.65	17.68	14.59	16.66
	Optimum water content (w/c %)	20.5	18	12.6	23.2	10.0



Fig. 1. Picture of soil particles used in the study

3. Methodology

The soils collected for the study's tests were oven-dried at 100°C except for gypseous and organic soils, which were dried at 60°C, cleaned, and sifted through the specified sieve for each test

requiring disturbed samples. The tests require that undisturbed samples (for direct shear and oedometer tests) be prepared from soil lumps or a Shelby tube, with an average of more than three specimens (per test). Furthermore, immediately after extraction from the field, the samples were preserved along with their water content to ensure they were subjected to as little disturbance as possible. The samples for the unconfined compression and residual shear test were prepared from disturbed samples, depending on the compaction characteristics parameters. The specimens were prepared under suitable laboratory conditions and tested using equipment in the geotechnical laboratory at the College of Engineering, Mosul University, as shown in Figure 2, and tested in an average of three samples with an acceptable standard deviation according to the specifications listed in Table 2.



Fig. 2. Equipment used in the study, left: direct shear, middle: unconfined compression, right: residual shear.

Table 2. Tests and specifications of soils

Test category	Test name	Standard specifications
Type of samples	Disturbed samples (DS)	[30]
	Undisturbed Samples (US)	[31]
Soil Classification	Atterberg Limits (LL and PL)	[32]
	Grain Size Analysis	[33]
	U.S.C. S	[34]
Moisture-density relation	Compaction test	[35]
Oedometer tests	Consolidation test	[36]
	Swelling test	[37]
	Collapse test	[38]
Strength tests	Unconfined compression	[39]
	Direct shear test	[40]
	Residual Shear Strength	[41]
Chemical tests	Sulphate content (SO ₃ %)	[42]
	Total soluble salts (T.S.S %)	
	Gypsum content (CaSO ₄ .2H ₂ O)	
	Organic matter content (%)	

4. Calculation of Bearing Capacity

The bearing capacity of the study soils was estimated using the general Meyerhof gross bearing capacity equation [43], based on strength parameters obtained from the direct shear and residual shear tests.

$$q_{ult.} = c N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (1)$$

Where:

($q_{all}=q_{ult}/F.O.S$)

q : $\gamma \times D_f$ (effective stress at the level of the bottom of the foundation); D_f : depth of foundation (supposed to be 1.5 m); c' : effective cohesion

γ : unit weight of soil

B : width of foundation (the foundation is supposed to be a square with $B = 1.5$ m)

N_c, N_q, N_γ bearing capacity factors, $F_{cs}, F_{qs}, F_{\gamma s}$: $F_{cd}, F_{qd}, F_{\gamma d}$: depth factors, $F_{ci}, F_{qi}, F_{\gamma i}$: load inclination factors[44–47].

The bearing capacity from the unconfined compression test was estimated using the Buisman bearing capacity equation for a foundation on saturated clay. (43)

$$q_{ult.square} = \underbrace{5.14 \left(1 + \frac{0.5 \frac{B}{H} - 0.707}{5.14} \right)}_{N_c (square)} c_u + q \quad (for H < 0.707B) \quad (2)$$

Where:

H : the depth of the soil layer under the base of the foundation, and taken as ($1.0m < 0.707B$) to give minimum values of N_c and q_u/fN_c and $q_{ult.}$, c_u : the undrained shear strength (Stress, $q_u/2$).

The assumption of saturated clay (zero internal friction angle, $\phi = 0$) was intentionally adopted for comparative purposes and to simulate a worst-case scenario. This assumption is justified by the field conditions in Mosul city, where the groundwater table is remarkably shallow and close to the ground surface in many areas, creating potentially saturated, undrained conditions.

The purpose of specifying the safety factor, width, and depth of the foundation is to facilitate comparison with the parameters used in the load-bearing capacity equation. The safety factor (3) is appropriate for the soil used in this study, as it is cohesive fine soil according to the aforementioned classification. As for the selection of the foundation width and depth (1.5 meters), these dimensions are used for shallow foundations commonly employed in standard buildings in the city of Mosul and its outskirts.

5. Results and Discussion

The index and geotechnical properties, especially the shear characteristics, of five problematic soils were determined. The soils are widely distributed across northern Iraq, particularly in and around Mosul city.

5.1 Geotechnical Parameters of Problematic Soils

Table 3 presents the parameters and the primary geotechnical issues associated with each soil used in this study, compared with the reference soil (low-plasticity soil). The problems of these soils have direct or indirect effects on their strength and bearing capacity. The obtained parameters of the soils used in the study show a clear geotechnical problem that exceeds the allowable limits. Geotechnical problems like swelling, collapse, high organic content, and high gypsum content often control the design of the foundation and lead to conservative values of allowable bearing capacity of soil, as shown in the bearing capacity results.

5.2 Shear Strength Characteristics of Soils

Table 4 shows the results of the shear strength characteristics of soils used in the study, represented by effective cohesion (c') and effective internal friction angle (ϕ'), obtained from different tests (unconfined compression, direct shear, and residual shear test). From Table 4 and Figure 3, the single most prominent observation to emerge from the data comparison was that the unconfined compression values give a misleading understanding of the strength of the five soils under study. Whereas the least problematic soil (CL) exhibits intermediate-to-weak shear strength parameters, the most problematic soils (OL and CL-ML) yield the highest strength values. The results must be interpreted with the test conditions in mind.

Table 3. Main geotechnical problems of the soils used in the study

	Soil type	Low plasticity clay (CL)	High plasticity clay (CH)	Low plasticity Silt (ML)	Organic soil (OL)	Gypsum soil (CL-ML)
Oedometer tests	Compression index (Cc)	0.15	0.18	0.23	0.31	0.15
	Pre-consolidated pressure (Pc, kN/m ²)	120	130	130	220	210
	Swell pressure (kN/m ²)	75	350	Nil	Nil	Nil
	Swell percentage (%)	1.6	22	Nil	Nil	Nil
	Collapse Index *(%)	Nil	Nil	2.2	2.5	0.9
Chemical tests	Sulphate content (SO ₃ %)	0.03	0.34	0.25	1.1	0.47
	Total soluble salts (T.S.S %)	1.2	5.4	3.2	8.5	2.45
	Gypsum content (CaSO ₄ .2H ₂ O)	5.5	5.0	1.3	18.4	63.4
	Organic matter content (%)	1.2	1.5	1.1	10.2	5.5
Main geotechnical problem	low plasticity clay (CL)	Reference soil				
	High plasticity clay (CH)	High shrinkage and swelling (plasticity index, P.I.> 35, very high swelling potential) [48]				
	Low plasticity Silt (ML)	Moderate collapse and high compressibility (collapse index, C.I.>2 and Compression index, Cc> 0.2) [38, 49]				
	Organic soil (OL)	Medium organic soil (organic content >6.0) [50] Moderate collapse and high compressibility (collapse index, C.I.>2 and Compression index, Cc> 0.2)				
	Gypsum soil (CL-ML)	gypseous soil (Gypsum content% > 50 consider Gypcrete**) [51]with high gypsum-soluble materials				

*Collapse Index was calculated at a stress level of 200 KN/m²

*Gypcrete is a hardened, natural soil layer rich in secondary gypsum (calcium sulfate), forming in arid/semi-arid regions where sulfate-rich water evaporates, leaving gypsum to cement soil particles like sand and clay into a crust or mass. It is a significant geological feature in dry climates, often found near the surface, that acts as an evaporite deposit, poses construction challenges due to potential instability and settlement, and serves as a raw material for plaster.

The heterogeneous composition of organic soil, which may include stones, gravel, or debris from previous buildings, can give a false indication of unconfined compressive strength. In addition, the low moisture content of the gypseous soil sample is a notable limitation during the unconfined compressive strength test, yielding high values that do not accurately reflect the soil's condition under higher moisture levels, which can lead to significant soil settlement or collapse due to gypsum dissolution. With the exception of organic soils (OL), significant differences, especially in silty soil (ML), can be observed in the internal friction angle values obtained from direct shear tests and those from residual shear strength tests, due to the difference in the soil type and the test condition. The fact that there is a predetermined failure plane in the direct shear test, one of the test's limitations, may result in relatively high values of effective internal friction angle (ϕ') in low-shear-strength soils, such as silty soils. In contrast, the residual shear strength test does not have a predetermined failure plane, which explains the significant variation in the internal friction angle

values obtained from the two tests. It is widely recognized that organic soils (OL) exhibit unpredictable or uncharacteristic results in various engineering tests. As the organic matter content increases, the soil displays increasingly erratic behavior during these evaluations. Consequently, these soils must be handled with extreme caution in engineering projects. As shear strain increases toward the residual state, any initial structure or apparent cohesion from organic fibers is destroyed, causing effective cohesion to approach zero. However, the residual friction angle is governed primarily by the ultimate arrangement, orientation, and intrinsic interparticle friction of the residual mineral and decomposed organic matrix. Because the fundamental soil fabric and particle surface characteristics remain unchanged after large displacement alignment, the residual friction angle reaches a steady-state value that is almost entirely independent of the original structure or initial cohesive bonds.

Table 4. Shear parameters of soils used in the study

Test type	Parameter	Soil type				
		CL	CH	ML	OL	CL-ML
Unconfined compression	Stress (q_u) (kN/m ²)	73.4	107	99.1	135.1	222.6
	Optimum water content (w/c%)	16.5	22	10.1	19.0	9.5
Direct shear	Cohesion (c) (kN/m ²)	14	35	4	11	40
	Eff. internal friction angle (ϕ')	26	27	28	19	34
Residual Shear Strength	Eff. cohesion (c') (kN/m ²)	0	10	5	1	5
	Eff. internal friction angle (ϕ')	9	6.3	10.2	18.8	12.1

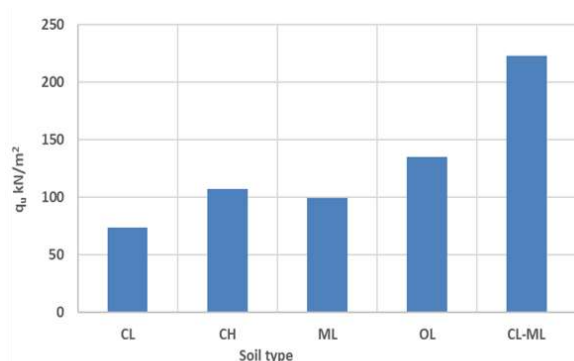


Fig. 3. The unconfined compression strength of the soils of the study

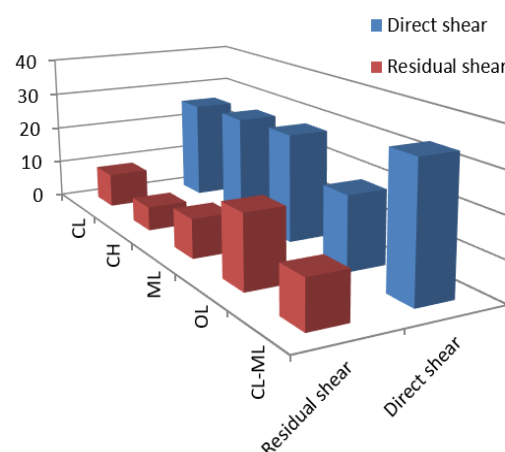


Fig. 4. The effective internal friction angle values of the study soils from the direct shear test

Table 4 and Figures 4 and 5 show the results of the direct shear test and the strength parameters (c' and ϕ') of soils used in the study. Once again, although direct shear testing is supposed to yield more reasonable values than unconfined compression testing, (CH and CL-ML) soils produced the highest shear modulus values. While the testing conditions account for the worst-case scenario of fully saturated soil with a consolidated drained test (slow test with a rate of 0.007mm/min), determining the test's failure level may lead to overestimates of the shear parameter values.

The results of the strength parameters (c' and ϕ') obtained from residual shear strength are presented in Table 4 and Figures 4 and 5. The results showed a logical and expected decrease in the internal friction angle of soils due to the test's nature, which yields a residual shear value after failure. Furthermore, a significant decrease in cohesion values is observed, approaching almost zero. This result is expected in this test because the expected residual shear value depends on the frictional force between the particles only. In the residual shear test, the device allows for continuous, unlimited displacement in one direction. This continuous rotation forces the platy clay particles to rearrange in a way so that they are perfectly parallel to the shear plane. This mechanism makes the resistance caused solely by sliding friction between the smooth surfaces of the particles and complete destruction of bonds and cohesive strength.

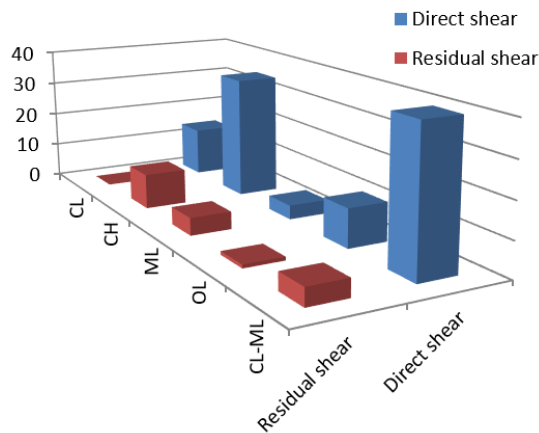


Fig. 5. The cohesion values of the study soils from the direct shear test

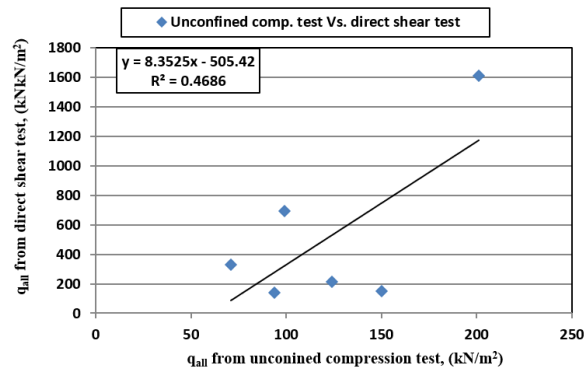


Fig. 6. The relation of allowable bearing capacity values of the study soils obtained from the different shear tests, with the coefficient of determination (R^2) between the types of tests

5.3 Bearing Capacity Estimation

Soil bearing capacity is the primary objective when conducting shear resistance tests and determining soil shear strength coefficients. Table 5 presents the bearing capacity of the study soils, calculated according to the assumptions outlined in the study's methodology. The experimental data yielded bearing-capacity results that are controversial, and there is no consensus on which values are suitable for foundation design. Furthermore, there is no clear correlation between the results of the soil bearing capacity calculated from any two shear strength tests in the study. As shown in Figure 6, there is no logical or statistically reliable relationship between the laboratory tests of the direct shear test and the unconfined compression test. The correlation coefficient (R^2) among the test method or the calculation method for load-bearing capacity values is very low for all relations in Figure 6. The differences in the method and nature of the test, the preparation of the specimen, and the method used to calculate the bearing capacity from each test provide a possible explanation for the differences in the calculated bearing capacity results.

Generally, the direct shear test yields overestimated allowable bearing capacity values due to test conditions, and it is recommended that these values not be used directly without first assessing the main geotechnical problems of the soils. The unconfined compression test yielded more conservative allowable bearing capacity values. The equation used to calculate the bearing capacity from unconfined compressive strength is for saturated soils (angle of internal friction = 0), so it yields lower values (worst-case conditions). This option is suitable for soils with engineering issues, such as gypsum, organic, and silty soils, due to their high potential for saturation. Meanwhile, calculating bearing capacity from direct shear testing uses effective shear modulus values that are somewhat higher, resulting in larger values. Nevertheless, it showed some high values for the CL-ML soil. The high bearing capacity of gypsum soil stems from its high cohesion, which is due to the soil being in a dry state; the soil is strong as a result of the cementitious nature of the gypsum binder. The residual shear test yields the lowest allowable bearing capacity, with some CL soil values underestimated and considered safe design estimates for post-failure scenarios. Figure 7 presents a comparison of the bearing capacities of the soils used in the study, and the dotted rectangle indicates the approximate reasonable range of bearing capacity (50 to 150 kN/m²) for a similar situation. The bearing capacity results were used in different locations of structural design in Mosul City and the surrounding area, as shown in Table 6. The soils recorded in Table 6 were the same soils used in this study, and the shear strength parameters were obtained from different types of laboratory and field tests. Although the shear strength values obtained from laboratory tests (particularly the unconfined compressive strength test and the direct shear test) and various field tests are similar to or even higher than those obtained from the study, the bearing capacity values took into account the prominent geotechnical issues for each soil type. The bearing

capacity used for these locations has approximately the same range suggested by this study, as shown in Figure 8. Generally, the use of bearing capacity values requires suitable experience and sufficient knowledge of the soil type and the associated geotechnical problems. Problems like significant soil settlement, collapse due to gypsum dissolution, and wetting and drying cycles should be considered before bearing capacity estimation.

Table 5. Bearing capacity of soils used in the study (kN/m²)

Soil type	Test type					
	Unconfined compression		Direct shear		Residual Shear	
	qult.	qall.	qult.	qall.	qult.	qall.
CL	213	71	993	331	51	17
CH	298	99	2083	694	149	50
ML	372	124	638	213	121	40
OL	283	94	426	142	167	56
CL-ML	602	201	4830	1610	162	54

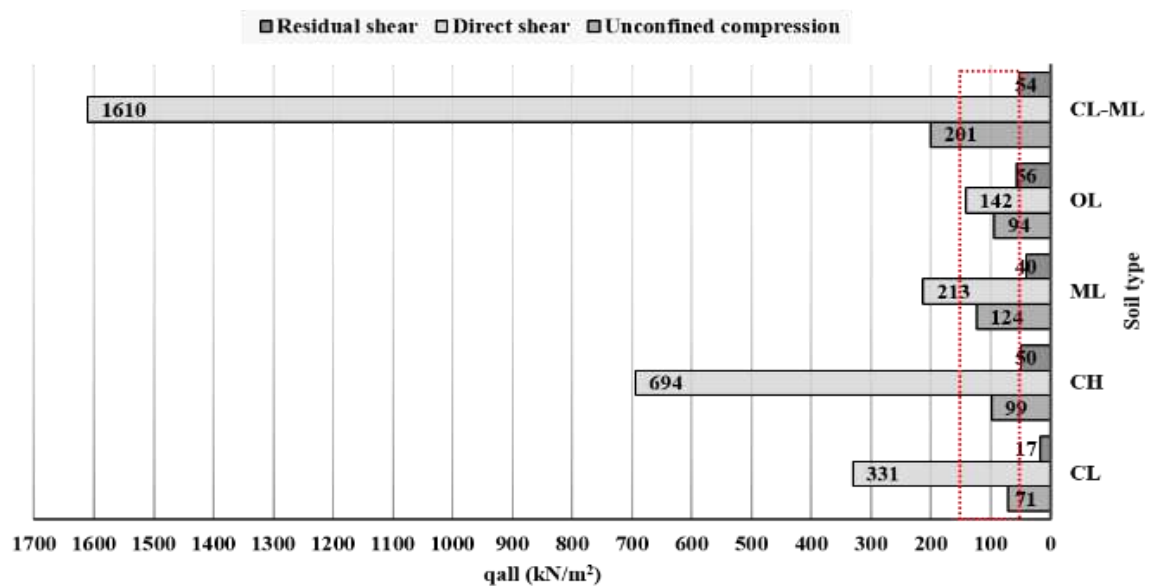


Fig. 7. Comparison of bearing capacity of soils used in the study (kN/m²)

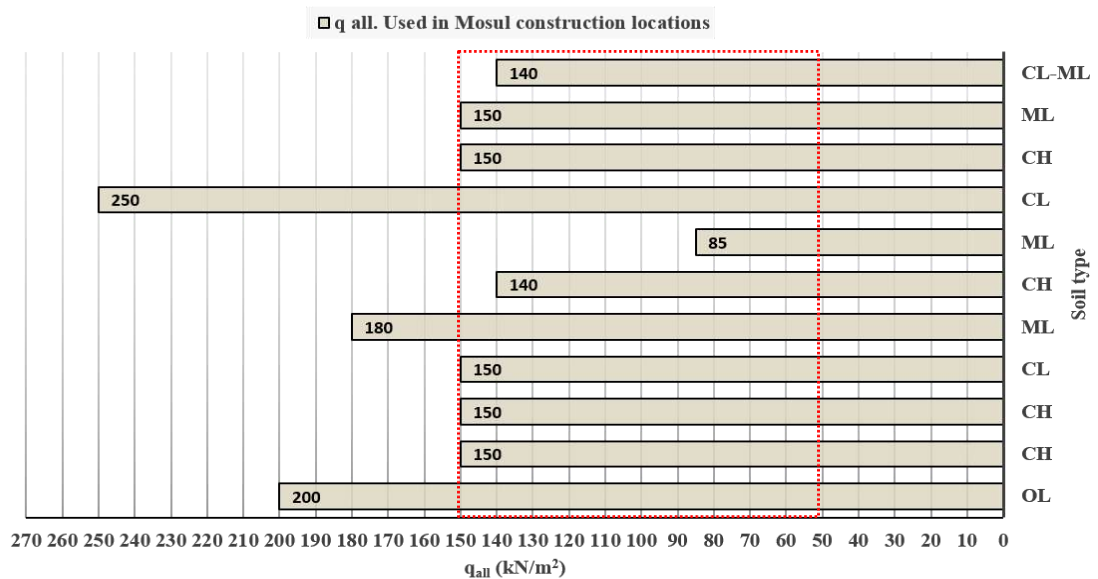


Fig. 8. Bearing capacity of soils used in different locations of construction in Mosul and around (kN/m²)

Table 6. Shear parameters and bearing capacity of some locations in Mosul city

Site	Depth of sample (m)	Type of soil	Laboratory strength tests		
			Shear characteristics from the direct shear		Shear characteristics from the unconfined compression
			C cohesion (kN/m ²)	φ Angle of friction (degrees)	Stress (qu) (kN/m ²)
School In the old city of Mosul	2	OL	24	26	----
Buildings of Nineveh Drug Company (Mosul, left banks)	1.5	CH	75	25.6	----
	2.5	CH	100	28	---
Cars Park in (Mosul, right banks)	4-10	CL	---	---	135-165
Building at the Mosul airport (Mosul, right bank)	0.1-4.0	ML	4	28	
Building in (Mosul, right banks)	2-10	CH			369-482
Building in (Mosul, right banks)	1.0	ML			
Hospital in Mosul, left banks)	6.0	CL	20	28	
Private University in Mosul, left banks)	5-15	CH	18-25	21-25	180-240
Health dispensary in Mosul, left bank	6.0	ML			60
Secondary electricity station west of Mosul city	1.5	CL-ML	25	27	

Site	Field strength tests			Allowable bearing capacity used for the site (kN/m ²)
	Results of Standard Penetration	Results of Dynamic Cone Penetration	Results of plate load	
	SPT (N) (Blows/30cm)	Number of Blows (N)/10cm	Allowable bearing capacity (kN/m ²)	
School In the old city of Mosul	---			200
Buildings of Nineveh Drug Company (Mosul, left banks)	---			150

Cars Park in (Mosul, right banks)	44-59			150
Building at the Mosul airport (Mosul, right bank)		2-12		180
Building in (Mosul, right banks)				140
Building in (Mosul, right banks)			85	85
Hospital in Mosul, left bank)				250
Private University in Mosul, left bank.				150
Health dispensary in Mosul, left bank.				150
Secondary electricity station west of Mosul city				140

6. Conclusions

Five soil types from the city of Mosul and its surrounding areas were selected to identify their geotechnical problems, study their strength properties, and estimate their bearing capacity. Based on the results obtained, the following conclusions can be drawn:

- The geotechnical problems in the soils of Mosul, in northern Iraq—such as settlement, collapse, swelling, and high organic and gypsum content—have a significant impact on the assessment of the bearing capacity of those soils. These problems are a notable limitation to indicating a high bearing capacity obtained from the bearing capacity equation.
- When estimating or approximating the allowable bearing capacity of soil in the city of Mosul, it is important to take into account that the strength coefficients depend directly on the type of test and the test conditions, which significantly affect the bearing capacity values.
- For the design of shallow foundations on Mosul soils, swelling tests are essential for high plasticity soil (CH); collapse and consolidation tests with organic content are required for silty soil (ML) and fill materials (OL); and consolidation tests with gypsum content are important for Gypsum soil (CL-ML).
- For the design of shallow foundations on Mosul soils, any available field or laboratory strength test for soil should be used in conjunction with tests that identify geotechnical issues in the soil, relying primarily on the test that yields the lowest value—or at least a reasonable value—for the strength parameters used to calculate bearing capacity.
- Test conditions had a significant impact on the strength parameters used in bearing capacity calculations, as evidenced by the variation in results, with the Direct shear tests yielding strength parameters that lead to overestimation of bearing capacity. In contrast, unconfined compression tests yield smaller values, and residual shear tests yield the smallest values due to the test conditions.
- For ordinary construction equivalent to a two- to five-story building resting on the soils used in this study at a depth of 2 to 3 meters, a bearing capacity of 50 to 150 kN/m² is suitable, provided other geotechnical issues are taken into account.

Further studies are needed on soils with engineering problems in and around the city of Mosul. Selecting the appropriate test method to obtain strength parameters that provide the best estimate of the bearing capacity required for structures subject to high soil pressures requires further study. In future studies, it is recommended to compare the bearing capacity values of soils obtained from field tests, such as plate load tests and standard penetration tests, with those from laboratory tests, taking into account the test conditions, as well as to study the effect of the gypsum content in the soil and the degree of saturation on the bearing capacity of problematic soils from Mosul city.

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Comparative evaluation of acid, mechanical, and thermal pre-treatment of recycled coarse aggregate in concrete

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Abstract

This study investigates the mechanical and selected durability-related performance of concrete incorporating recycled coarse aggregate (RCA) subjected to three pre-treatment methods, namely acid treatment (RCAA), mechanical treatment (RCAM), and thermal treatment (RCAT), with untreated RCA (RCAU) and conventional concrete (CC) used as reference groups. RCA replacement levels of 25, 50, 75 and 100% were considered. Workability, compressive strength, splitting tensile strength and flexural strength were evaluated at curing ages up to 365 days. Water absorption and sorptivity were evaluated for the conventional and untreated-RCA concrete series. The results show decreasing workability and strength with increasing RCA replacement, mainly because of higher porosity, adhered mortar and weaker interfacial transition zones. At 28 days, the 100% untreated-RCA concrete showed a compressive-strength reduction of about 17.1% relative to the control, while the 25% replacement level showed smaller reductions. Among the pre-treatment series, acid treatment gave the highest reported compressive-strength retention, followed by mechanical treatment, whereas thermal treatment gave lower values than the corresponding untreated-RCA series in the reported dataset. The durability-related results available in this study show that increasing untreated RCA replacement increased water absorption and sorptivity, indicating greater susceptibility to water ingress. The findings support the need for controlled RCA replacement and appropriate pre-treatment, while the durability benefit of the pre-treatment methods requires further experimental verification.

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1. Introduction

Rapid urbanization and large-scale infrastructure development have increased demand for construction materials, particularly natural aggregates (NA), which constitute a major volume fraction of concrete [1,2]. Intensive extraction of NA contributes to resource depletion and environmental impacts, while the generation of construction and demolition waste (CDW) creates additional disposal challenges. RCA produced from CDW has therefore emerged as a potential alternative to NA and as an important component of circular construction practices [3,4]. However, its structural application remains constrained by the presence of adhered old mortar (OM),

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microcracking and associated increases in porosity and water absorption [5–10]. Mechanical, chemical and thermal pre-treatment methods have been investigated to reduce adhered mortar and improve RCA quality [11–13]. Mechanical abrasion can remove part of the adhered mortar with comparatively low chemical waste [14–19], while acid treatment can dissolve calcium-rich hydration products but requires careful control of concentration, exposure and washing [20–24]. Thermal treatment can exploit differential thermal expansion to loosen adhered mortar, although excessive heating may damage aggregate and requires additional energy [25–29]. Hybrid treatments have also been reported [30–33]. Because the present study uses Portland Pozzolana Cement (PPC), the binder should also be considered when interpreting RCA performance. The pozzolanic constituents of PPC can participate in secondary reaction products and may contribute to matrix densification and interfacial development over extended curing periods. Previous studies on recycled aggregate concrete incorporating supplementary cementitious materials have reported changes in strength and durability associated with such binder systems [12,16]. PPC was therefore selected as the binder in the present investigation to examine RCA concrete under a commonly available blended-cement system rather than to imply equivalence with OPC.

The OM layer enclosing RCA particles changes dramatically in its properties. It increases porosity [6], water absorption and angularity and decreases specific gravity and strength [7]. Adhered mortar results in a weak interfacial transition zone (ITZ) between aggregate and fresh cement paste, which is a key factor for load transmission and overall concrete integrity [8]. In general, RCA absorbs 3–10% water which is substantially greater than 0.5–2% for NA leading to poorer density and reduction of 10–40% in compressive strength of concrete made using RCA known as recycled aggregate concrete (RAC) [9,10]. Therefore, pre-treatment techniques should be employed to enhance the RCA quality and to assure the structural reliability and durability of RAC. Various pre-treatment techniques have been proposed in an attempt to decrease adherent mortar and enhance aggregate surface characteristics including mechanical abrasion, chemical acid soaking and thermal exposure [11–13].

The physical technique of detaching the OM layer from the RCA particles by mechanical abrasion, ball milling or rotating friction is known as the mechanical treatment [14,15]. This method can remove 30–50% of the adhering mortar thereby decreasing porosity and increasing the strength and density of the aggregate [16]. However, severe wear may cause surface cracking and abnormalities in shape that are detrimental to the workability of concrete [17]. It has been shown that mechanical treatment optimization can boost the RCA crushing strength by 10–15% and enhance the RAC compressive strength by 10–20% [18]. The key advantages are the simplicity of operation, cost-effectiveness and little chemical waste, which makes it applicable for industrial large-scale applications [19].

Another potential option is the acid treatment that employs acidic solutions such as hydrochloric acid (HCl), sulfuric acid (H₂SO₄) or acetic acid (CH₃COOH) to dissolve calcium-rich hydration products in the OM [20,21]. The chemical reaction of acid with calcium hydroxide (Ca(OH)₂) or calcium silicate hydrate (C–S–H) phases, deteriorates the link between mortar and aggregate leading to its removal [22]. Studies have indicated that soaking RCA in 0.1–0.5 M HCl for 24 h can decrease adherent mortar by 20–30% and water absorption by 25–40%, leading to better aggregate quality [23]. However, the effectiveness of acid treatment is highly dependent on the acid content, the exposure time and the washing conditions. The aggregate surface can be damaged by overexposure and leftover acid can remain and impact the durability of concrete [24]. Therefore, the treatment parameters must be optimized to achieve a good compromise between mortar removal and aggregate integrity.

Thermal treatment is the application of heat at high temperatures (normally 250–600 °C) to RCA to create distinct thermal stress between natural aggregate and attached mortar to separate them [25,26]. It has been demonstrated that heating at ~300–400 °C, followed by mechanical abrasion or water quenching, can be successful in removing 25–45% of the mortar without harming the aggregate structure [27]. This procedure minimizes water absorption by up to 35% and increases the compressive strength of RAC by 10–25% compared to untreated RCA [28]. However, heating beyond 600 °C causes degradation of the mineral structure of the aggregate and micro-cracking

and damage due to expansion. Thermal treatment is energy expensive yet environmentally sound when used in conjunction with waste heat recovery systems in precast or cement industries [29].

Comparative investigations have indicated that these pre-treatment approaches have varying strengths and limitations. Mechanical treatment is inexpensive and scalable but less effective in mortar removal than acid or thermal treatment [30]. Acid treatment has more efficiency of mortar removal but increases environmental concern for handling of acids and disposal of effluents. Thermal treatment improves aggregate quality substantially but requires higher energy input [31]. To circumvent these limitations, hybrid treatments like mechanical–acid or thermal–mechanical combinations have been developed. For example, mild heating at 300 °C combined with mechanical abrasion led to a reduction in adhering mortar by up to 50% and an increase in the compressive strength of RAC by up to 25% [32,33].

The mechanical performance of RAC made with treated RCA is influenced by the quality of the aggregate, the mix design, the water-to-cement ratio and the curing regime. Studies have shown that strength recovery of 90–100% of conventional concrete can be obtained with optimum RCA treatment and replacement levels up to 50% [34]. Furthermore, treated RCA also improves durability parameters such as water absorption, chloride ion penetration, and shrinking [35]. SEM and XRD microstructural studies confirm a denser ITZ and a lower microcrack density, confirming the usefulness of RCA treatment in improving the overall performance [36].

Most previous studies have evaluated individual RCA treatment methods under different source concretes, replacement ratios and test protocols. The present work contributes a controlled comparison of untreated, acid-treated, mechanically treated and thermally treated RCA under a common M30 concrete framework, together with curing ages extending from 7 to 365 days. The extended curing period allows the evolution of strength to be followed beyond the conventional 28-day reference age. The experimental dataset is examined through direct descriptive comparisons and trend analysis to relate RCA replacement and treatment condition to concrete performance. These features provide a performance-oriented basis for interpreting the applicability of treated RCA, while recognizing the limitations of the available replicate-level dataset.

2. Research Objectives and Scope

The study aims to evaluate the influence of RCA replacement on concrete performance and to compare untreated RCA with acid-, mechanically-, and thermally treated RCA under common experimental conditions. The specific objectives are to: (i) evaluate workability and strength development at 7, 14, 28, 56, 90, 180, 270 and 365 days; (ii) compare the influence of RCA treatment on compressive, splitting tensile and flexural strengths; (iii) examine water absorption and sorptivity for the conventional and untreated-RCA series; and (iv) identify replacement levels and treatment conditions associated with improved strength retention. The study does not include direct SEM/XRD characterization or comparative durability testing of all treated-RCA concrete series; these are identified as areas for future research.

3. Materials and Methods

3.1 Cement

PPC conforming to IS 1489 was used as the binding material. The reported specific gravity of the cement was 3.15, as given in Table 1. This value is retained as the reported measured value in the supplied experimental dataset; no retrospective substitution with a typical PPC value has been made. The cement was selected to represent a blended-cement binder commonly used in practice. The possible contribution of pozzolanic reaction to continued matrix densification is considered when interpreting the long-term strength development. Table 2 represents the chemical properties of PPC.

3.2 Fine Aggregate

The fine aggregate used in this study was river sand conforming to IS 383, which is a naturally occurring material sourced from riverbeds. The sand was described as clean and free from organic impurities. The material was oven-dried prior to the reported physical-property measurements.

The fine aggregate passing the 4.75 mm sieve was used, with its measured properties provided in Table 3.

Table 1. Physical Properties of PPC

S.No.	Description	Result
1	Fineness (m^2/kg)	337
2	Standard consistency (%)	37
3	Setting time (minutes)	
	a. Initial	186
	b. Final	335
4	Soundness, mm	3
5	Specific gravity	3.15
6	Compressive strength, MPa	
	3 days	26.95
	7 days	35.70
	28 days	55.75

Table 2. Chemical properties of PPC*

S.No.	Description	Result
1	Insoluble residue (%)	29.85
2	Loss on ignition (%)	2.05
3	Sulphuric anhydride (%)	2.38
4	Magnesia (%)	1.24
5	Total chlorides (%)	0.021

*Source: Ultra Tech cements Ltd.

Table 3. Properties of fine aggregate

S.No.	Description	Result
1	Specific gravity	2.65
2	Bulk relative density, kg/m^3	1635
3	Bulking of sand	26%
4	Fineness modulus	2.64
5	Zone	II
6	Water absorption, %	1.35

3.3 Coarse Aggregate

Crushed granite coarse aggregate of 20 mm and 10 mm nominal sizes was used. The aggregates were angular, clean and free from deleterious materials, and conformed to IS 383 requirements. Physical properties including specific gravity and water absorption were determined using the relevant aggregate test procedures; coarse-aggregate specific gravity and water absorption were considered with reference to IS 2386 (Part III). The measured properties are presented in Table 4.

Table 4. Properties of coarse aggregate

S.No.	Description	Result
1	Specific gravity	2.65
2	Fineness modulus	2.43
3	Bulk density (kg/m^3)	1670
4	Crushing strength (%)	34
5	Impact value (%)	24
6	Water absorption (%)	0.50
7	Abrasion value (%)	30

3.4 Recycled Coarse Aggregate

3.4.1 Source of Construction and Demolition Waste

The RCA was produced exclusively from CDW, obtained from reinforced concrete structural elements of residential/commercial buildings in Chennai. No brick masonry, mortar debris, tiles, or mixed construction waste was included. The parent concrete was estimated to be of M25 grade, based on available construction records and field assessment, with an approximate service age of 25–30 years.

3.4.2 Crushing Procedure

The demolished concrete was initially broken into manageable sizes and subjected to mechanical crushing using a laboratory-scale jaw crusher for primary size reduction. Impurities such as steel, wood and plastic were manually removed. The crushing and subsequent processing were intended to produce RCA fractions suitable for concrete production. The RCA was then divided into untreated, acid-treated, mechanically treated and thermally treated groups for the experimental program described below.

3.4.3 Screening and Processing

The crushed material was sieved using conventional sieves, and the required size fractions were collected as RCA. Oversized particles were discarded or crushed again. The sieved RCA was air-dried and stored in closed containers to minimize moisture variation before testing and concrete production. The RCA used in the concrete consisted of 60% of the fraction passing 25 mm and retained on 20 mm, and 40% passing 12.5 mm and retained on 10 mm. The material was subsequently processed as untreated, acid-treated, mechanically treated and thermally treated RCA according to the experimental groups.

3.4.4 Physical Properties of RCA

The RCA exhibited lower density and higher water absorption than the natural coarse aggregate, consistent with the presence of adhered old mortar and micro-porosity introduced during crushing. The physical properties of the natural coarse aggregate and RCA were determined using relevant procedures of IS 2386. The measured physical properties of RCA are summarized in Table 5. These properties reflect the parent-concrete quality and the adopted crushing and screening process.

Table 5: Properties of recycled coarse aggregate

S.No.	Description	Result
1	Specific gravity	2.38
2	Fineness modulus	6.75
3	Bulk density (kg/m ³)	1380
4	Crushing strength (%)	26
5	Impact value (%)	21
6	Water absorption (%)	4.50
7	Abrasion value (%)	20

3.4.5 Treated Recycled Coarse Aggregate

The treated RCA groups were produced using the specified acid, mechanical and thermal pre-treatment procedures. Their measured physical properties are summarized in Table 6. The treatment processes did not improve every measured aggregate property uniformly; therefore, the treatment effects are interpreted parameter by parameter rather than assuming that all treatments enhance all properties.

Table 6. Properties of treated recycled coarse aggregate

S.No	Description	Acid	Mechanical	Thermal
1	Specific gravity	2.59	2.55	2.40
2	Aggregate crushing value (%)	7.20	7.10	6.80
3	Aggregate impact value (%)	29	26	22
4	Impact value (kj/m ²)	18	29	25
5	Abrasion value (%)	2.10	3.10	4.08
6	Abrasion	25	30	33

3.5 Water and Superplasticizer

Potable water suitable for drinking was used for mixing and curing. The superplasticizer SikaViscoCrete-SC 001 was used in the study, with dosage adjusted according to the workability requirement. The available manuscript records provide the product name, specific gravity and pH, but do not provide the manufacturer's dry-extract/solid-content value or the manufacturer's recommended dosage range. These values have therefore not been inferred or fabricated. The characteristics of superplasticizer are represented in Table 7.

Table 7. Technical data of superplasticizer

S.No.	Characteristics	Value
1	Color	Dark brown liquid
2	Specific gravity	1.08
3	Air entrainment	Maximum 1%
4	pH	7 to 8

3.6 Preparation and Curing of Specimen

The concrete mixtures were prepared using a laboratory pan mixer. The dry constituents were first blended to achieve homogeneous distribution, followed by the addition of water and superplasticizer. The fresh concrete was placed into moulds in layers and compacted using a table vibrator. Specimens were demoulded after 24 h and immersed in water at ambient temperature until the designated testing age. Exact mixing and vibration durations and the number of filling layers were not recorded in the supplied manuscript and are therefore identified as reproducibility limitations rather than supplemented with assumed values.

3.7 Test Methods

- Slump tests were performed to evaluate workability in accordance with IS 1199.
- Compressive strength tests were performed on 150 mm cube specimens using a 2000 kN compression testing machine at curing ages of 7, 14, 28, 56, 90, 180, 270 and 365 days. The reported test procedure was based on the applicable Indian Standard for concrete compressive strength (IS 516).
- Cylindrical specimens were cast to determine splitting tensile strength at the same curing ages. The applicable test procedure was based on IS 5816. The exact cylinder dimensions and loading rate were not reported in the supplied manuscript and have not been inferred.
- Prism specimens under third-point loading were evaluated for flexural strength at the reported curing ages. The applicable procedure was based on IS 516. The exact prism dimensions and loading rate were not reported in the supplied manuscript and have not been inferred.
- Durability-related performance was assessed through water absorption and sorptivity tests for the conventional and untreated-RCA concrete series. Water absorption was evaluated with reference to ASTM C642 and sorptivity with reference to ASTM C1585. The supplied manuscript does not contain corresponding concrete-level durability results for the acid-, mechanically- and thermally treated RCA series; therefore, no unsupported treated-series durability values have been added.

3.8 Statistical Analysis

Because the supplied dataset does not contain independent replicate-level observations for every treatment $\times$ replacement $\times$ curing-age combination, pooled standard deviations, coefficients of variation and inferential ANOVA results.

4. Concrete Mix Proportions

The concrete mixtures were designed for M30 grade in accordance with IS 10262:2019. The nominal water-to-cement ratio was kept at 0.45. Natural coarse aggregate was replaced by RCA at 0, 25, 50, 75 and 100% by mass in the experimental mixtures, while cement, water and fine aggregate contents were kept constant. The constituent ratio was 1: 1.48: 3.05, with a nominal w/c ratio of 0.45. Because the aggregates had different specific gravities, the reported mixtures are explicitly treated as mass-replacement mixtures; the supplied experimental record does not permit retrospective re-casting or reconstruction of the actual batches using an absolute-volume correction. This limitation is now stated to avoid implying volumetric equivalence. The detailed mix proportions in cubic meter of concrete are given in Table 8.

Table 8. Mix proportions of concrete with and without RCA (mass-based RCA replacement)

Mix ID	RCA Replacement (%)	Cement (kg/m ³)	Water (kg/m ³)	w/c Ratio	Fine Aggregate (kg/m ³)	Coarse Aggregate (kg/m ³)	Recycled Coarse Aggregate (kg/m ³)	Superplasticizer (% of cement)
M0	0	385	173	0.45	570	1174	0	0
M25	25	385	173	0.45	570	881	294	0.6
M50	50	385	173	0.45	570	587	587	0.8
M75	75	385	173	0.45	570	294	881	0.9
M100	100	385	173	0.45	570	0	1174	1

5. Concrete Properties

5.1 Fresh Concrete Properties

As illustrated in Figure 1, the reported slump value decreased from 100 mm for the control mix to 95, 81, 68 and 50 mm at 25, 50, 75 and 100% untreated RCA replacement, respectively. RCA has higher water absorption and a rougher, more angular surface than the natural aggregate. These characteristics increase particle-to-particle friction and interlocking and reduce the free water available for flow, thereby decreasing workability. The supplied dataset does not contain slump values for the acid-, mechanically- or thermally treated concrete series; consequently, Figure 1 and the discussion are limited to the conventional and untreated-RCA series.

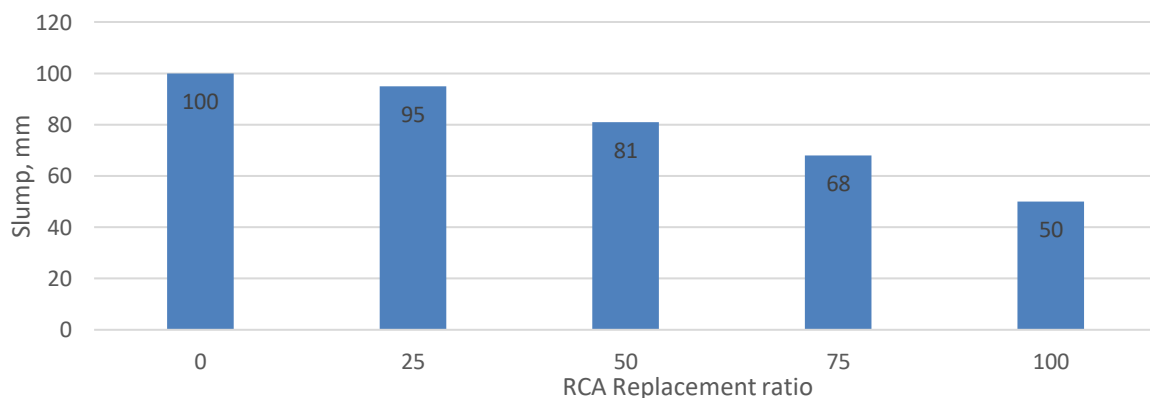


Fig. 1. Slump versus untreated RCA replacement

5.2 Hardened Concrete Properties

5.2.1 Compressive Strength

The compressive-strength values in Table 9 show progressive strength development from 7 to 365 days for the conventional and RCA concrete series. The control mix reached 34.50 MPa at 28 days and 43.00 MPa at 365 days. The results indicate that increasing RCA replacement generally reduced compressive strength, while the magnitude of reduction depended on treatment and replacement level. At 100% replacement, the untreated-RCA mixture reached 28.60 MPa at 28 days, compared with 34.50 MPa for the control, corresponding to a reduction of about 17.1%. The reduction is associated with the higher porosity, adhered mortar and weaker interfacial transition zones of untreated RCA. Among the treated-RCA mixtures, the acid-treated series generally retained the highest compressive strength in the reported dataset. At 25% replacement, RCAA25% reached 34.00 MPa at 28 days, compared with 33.20 MPa for RCAU25%. The results indicate that acid treatment improved strength retention relative to untreated RCA under the conditions investigated.

Mechanical treatment also improved strength relative to the corresponding untreated RCA in the reported dataset; for example, RCAM25% reached 33.40 MPa at 28 days. In contrast, the thermally treated series was consistently lower than the corresponding untreated series in the reported strength; for example, RCAT25% and RCAT100% reached 32.80 and 28.00 MPa at 28 days, compared with 33.20 and 28.60 MPa for RCAU25% and RCAU100%, respectively. Accordingly, thermal treatment should not be described as superior to untreated RCA in this study. The 25% replacement level generally provided the smallest strength reduction across the reported treatment series, whereas higher replacement levels produced larger reductions. The long-term data show continued strength development up to 365 days; however, this does not eliminate the initial strength penalty associated with higher RCA replacement. Overall, the results indicate that appropriate RCA pre-treatment can improve strength retention compared with untreated RCA, with acid treatment showing the highest reported retention in the present dataset.

Table 9. Compressive strength of various treated and untreated RCA

S.No	Replacement Ratio of CA	Compressive Strength, N/mm ²							
		7 Days	14 Days	28 Days	56 Days	90 Days	180 Days	270 Days	365 Days
1	CC	22.50	27.80	34.50	36.80	38.20	40.50	41.80	43.00
2	RCAU25%	21.80	26.90	33.20	35.10	36.50	38.20	39.50	40.60
3	RCAU50%	20.60	25.40	31.80	33.60	35.00	36.50	37.80	38.90
4	RCAU75%	19.20	23.90	30.20	31.90	33.30	34.80	36.00	37.20
5	RCAU100%	18.00	22.50	28.60	30.10	31.50	33.00	34.20	35.50
6	RCAA25%	22.10	27.30	34.00	36.20	37.60	39.80	41.00	42.20
7	RCAA50%	21.50	26.60	33.00	35.00	36.50	38.50	39.80	41.00
8	RCAA75%	20.80	25.80	32.00	33.80	35.20	36.80	38.00	39.20
9	RCAA100%	19.60	24.20	30.50	32.20	33.60	35.20	36.50	37.80
10	RCAM25%	21.60	26.70	33.40	35.40	36.80	38.60	39.90	41.10
11	RCAM50%	21.00	25.90	32.30	34.10	35.60	37.20	38.50	39.70
12	RCAM75%	20.00	24.50	31.00	32.70	34.10	35.60	36.90	38.00
13	RCAM100%	18.80	23.20	29.50	31.10	32.50	34.00	35.30	36.50
14	RCAT25%	21.20	26.20	32.80	34.60	36.00	37.50	38.80	40.00
15	RCAT50%	20.20	24.80	31.20	32.90	34.30	35.80	37.00	38.20
16	RCAT75%	19.00	23.40	29.80	31.40	32.80	34.20	35.50	36.70
17	RCAT100%	17.80	22.00	28.00	29.60	31.00	32.40	33.60	34.80

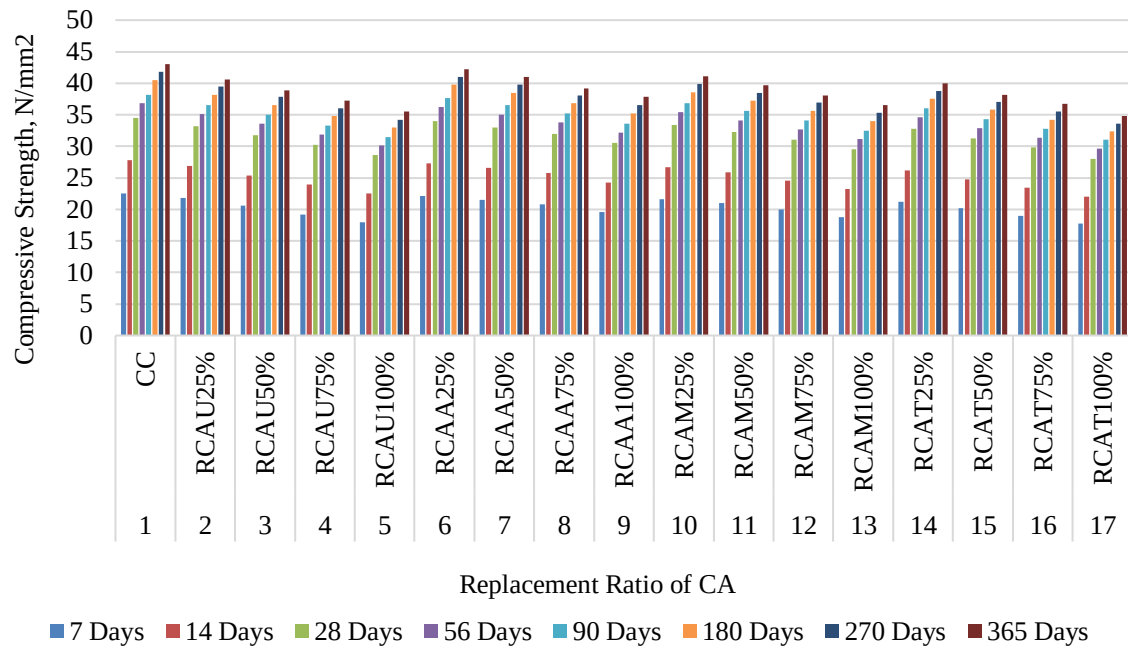


Fig. 2. Compressive strength versus curing age and RCA replacement

The statistical metrics given in Table 10 provide a better insight of the consistency, dependability and overall performance of RCA than CC. The average compressive strength of the CC was 35.64 MPa, with a standard deviation of 7.22 MPa (coefficient of variation (COV) = 20.26%). This implies a considerable degree of variability often associated with conventional concrete mixtures.

The use of recycled coarse materials causes a little decrease in the mean strength for all categories. RCAU has the lowest mean strength of 32.34 MPa, which corresponds to 9.25% drop compared to CC. This reduction is due to the deficiencies of the untreated recycled aggregates, such as high porosity, microcracks, and the presence of adhering mortar, which degrade the interfacial bonding in the concrete matrix. Better performance was observed for processed RCA mixes in comparison. The mean strength of RCAA is 33.94 MPa with only 4.77% reduction, which indicates that chemical treatment is an efficient method to improve the aggregate quality by removal of adhering mortar and improvement of surface texture. Similarly, the RCAM exhibits a moderate reduction of 6.20% which indicates that mechanical processing enhances aggregate integrity but is somewhat less effective than chemical treatment. RCAT shows a decrease of 8.30% showing it is less efficient in enhancing the overall performance.

The standard deviation values for all combinations are very close, from 6.95 to 7.22 MPa, indicating that the variation in strength is constant across all categories. This is also evidenced by COV values in the range of 20.26 to 21.58%. The minor rise of COV of RCA mixes suggests a somewhat higher variability, which can be justified by the heterogeneous character of recycled aggregates. However, the deviation is still within acceptable engineering limitations. This shows that the use of RCA does not affect the homogeneity of the concrete appreciably.

Table 10. Mean strength, standard deviation and standard reduction and COV

RCA (%)	Mean Strength (MPa)	Standard Deviation (MPa)	Standard Reduction (%)	COV (%)
CC	35.64	7.22	-	20.26
RCAU	32.34	6.98	9.25	21.58
RCAA	33.94	7.05	4.77	20.78
RCAM	33.43	7.01	6.20	20.97
RCAT	32.68	6.95	8.30	21.27

5.2.2 Split Tensile Strength

The split tensile strength data exhibited a steady increase with curing age across all mixtures, analogous to the behavior of compressive strength. The control mix attained a 28-day tensile strength of 3.40 MPa, whereas RAC demonstrated a decrease contingent upon the replacement level and treatment procedure. Untreated RCA mixes showed the lowest performance, with a reduction of approximately 17–20% at 100% replacement. Among the treatment methods, RCAA exhibited the best performance, with only an 11.8% reduction compared to the control mix at 28 days. Mechanical treatment showed moderate improvement, whereas thermal treatment resulted in comparatively lower strength due to microstructural damage caused by heating. The reduction in tensile strength is primarily attributed to the ITZ and increased porosity associated with RCA. However, treatment methods effectively enhanced bonding characteristics and reduced strength loss. Overall, up to 50% replacement of natural coarse aggregate with treated RCA can be recommended without significant compromise in tensile performance. The results are presented in Table 11.

Table 11. Split tensile strength of various treated and untreated RCA

S.No	Replacement Ratio	7 Days	14 Days	28 Days	56 Days	90 Days	180 Days	270 Days	365 Days
1	CC	2.10	2.60	3.40	3.60	3.75	3.95	4.05	4.15
2	RCAU25%	2.00	2.50	3.25	3.45	3.60	3.80	3.90	4.00
3	RCAU50%	1.90	2.35	3.10	3.30	3.45	3.60	3.70	3.80
4	RCAU75%	1.75	2.20	2.95	3.10	3.25	3.40	3.50	3.60
5	RCAU100%	1.60	2.00	2.80	2.95	3.10	3.25	3.35	3.45
6	RCAA25%	2.05	2.55	3.35	3.55	3.70	3.90	4.00	4.10
7	RCAA50%	2.00	2.50	3.25	3.45	3.60	3.80	3.90	4.00
8	RCAA75%	1.90	2.40	3.15	3.30	3.45	3.60	3.70	3.80
9	RCAA100%	1.75	2.20	3.00	3.15	3.30	3.45	3.55	3.65
10	RCAM25%	2.00	2.50	3.30	3.50	3.65	3.85	3.95	4.05
11	RCAM50%	1.95	2.45	3.20	3.35	3.50	3.70	3.80	3.90
12	RCAM75%	1.85	2.30	3.05	3.20	3.35	3.50	3.60	3.70
13	RCAM100%	1.70	2.15	2.90	3.05	3.20	3.35	3.45	3.55
14	RCAT25%	1.95	2.40	3.20	3.35	3.50	3.70	3.80	3.90
15	RCAT50%	1.85	2.30	3.05	3.20	3.35	3.50	3.60	3.70
16	RCAT75%	1.75	2.15	2.90	3.05	3.20	3.35	3.45	3.55
17	RCAT100%	1.60	2.00	2.75	2.90	3.05	3.20	3.30	3.40

5.2.3 Flexural Strength

The flexural strength results exhibited a consistent increase with curing age for all concrete mixes as shown in Table 12. The control mix exhibited a 28-day flexural strength of 4.80 MPa, whereas the flexural strength of recycled aggregate concrete gradually diminished with increased replacement. RCAA exhibited optimal performance, achieving a 28-day strength of 4.75 MPa at a 25% replacement rate, equivalent to the control mix. Mechanical treatment marginally enhanced strength, however thermal treatment diminished strength due to microstructural degradation after heating. The ITZ and diminished crack resistance in recycled aggregates decrease flexural strength. Treatment protocols significantly enhanced bonding, minimizing strength degradation. Processed RCA can substitute up to 50% of natural aggregates without compromising flexural performance.

Table 12. Flexural strength of various treated and untreated RCA

S.No	Replacement Ratio	7 Days	14 Days	28 Days	56 Days	90 Days	180 Days	270 Days	365 Days
1	CC	3.20	3.80	4.80	5.10	5.30	5.60	5.80	6.00
2	RCAU25%	3.10	3.70	4.60	4.90	5.10	5.30	5.50	5.70
3	RCAU50%	2.95	3.50	4.40	4.70	4.90	5.10	5.30	5.50
4	RCAU75%	2.80	3.30	4.20	4.40	4.60	4.80	5.00	5.20
5	RCAU100%	2.60	3.10	4.00	4.20	4.40	4.60	4.80	5.00
6	RCAA25%	3.15	3.75	4.75	5.00	5.20	5.50	5.70	5.90
7	RCAA50%	3.05	3.65	4.60	4.85	5.05	5.30	5.50	5.70
8	RCAA75%	2.95	3.50	4.45	4.70	4.90	5.10	5.30	5.50
9	RCAA100%	2.80	3.30	4.20	4.45	4.65	4.90	5.10	5.30
10	RCAM25%	3.10	3.70	4.65	4.90	5.10	5.35	5.55	5.75
11	RCAM50%	3.00	3.60	4.50	4.75	4.95	5.20	5.40	5.60
12	RCAM75%	2.85	3.40	4.30	4.55	4.75	5.00	5.20	5.40
13	RCAM100%	2.70	3.20	4.10	4.35	4.55	4.80	5.00	5.20
14	RCAT25%	3.05	3.60	4.55	4.80	5.00	5.25	5.45	5.65
15	RCAT50%	2.90	3.45	4.35	4.60	4.80	5.00	5.20	5.40
16	RCAT75%	2.75	3.25	4.15	4.40	4.60	4.80	5.00	5.20
17	RCAT100%	2.60	3.05	3.95	4.20	4.40	4.60	4.80	5.00

5.3. Durability Properties

5.3.1 Water Absorption

The water absorption consistently rises with the increasing amount of RCA. The control concrete exhibited a minimum absorption of 1.5%, while the concrete containing 100% RCA had the highest absorption at 4.5%, representing an estimated 200% increase compared to the control concrete, as detailed in Table 13 and Figure 3.

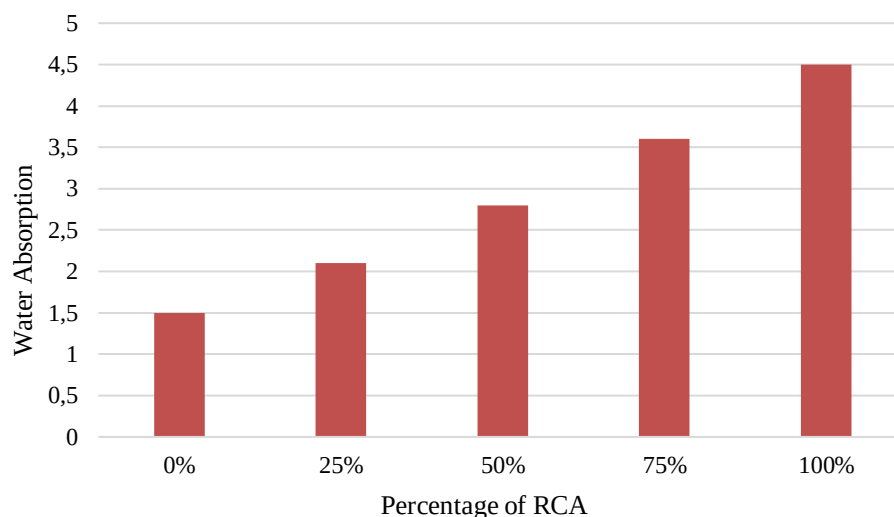


Fig. 3. Water absorption versus untreated RCA replacement

The incremental rise in water absorption with the increase in RCA concentration is mainly ascribed to the porous nature of RCA and the presence of adherent OM on their surfaces. These features develop more capillary holes in the concrete matrix resulting in higher water absorption than

typical concrete manufactured with natural sand. The results of water absorption clearly show that the inclusion of RCA considerably enhances the porosity of concrete. The control mix had a water absorption of 1.50%, whereas those with 25, 50, 75, and 100% RCA had values of 2.10, 2.80, 3.60, and 4.50%, respectively. This means the concrete water absorption is increased by 40%, 86.70%, 140% and 200%, respectively, in comparison to the traditional concrete. The increase in water absorption is mainly due to the existence of adherent mortar and microcracks in recycled aggregates resulting in high permeability. The results show that greater RCA replacement levels negatively affect durability-related attributes and consequently suitable treatment methods are needed to reduce the water absorption in RAC.

Table 13. Water Absorption (%)

RCA (%)	0%	25%	50%	75%	100%
Absorption	1.50	2.10	2.80	3.60	4.50
Increase in Absorption (%)	—	40.00	86.70	140.00	200.00

5.3.2 Sorptivity

The increase in water absorption with untreated RCA replacement is attributed to the porous nature of RCA, adhered old mortar and microcracks. These features provide additional pathways for water ingress. The term 'higher' or 'increased water absorption' is used rather than 'better water absorption'.

Table 14. Sorptivity Test

RCA (%)	0%	25%	50%	75%	100%
Sorptivity (mm/ $\sqrt{\text{min}}$)	0.12	0.16	0.21	0.27	0.34
Increase in Sorptivity (%)	—	33.30	75.00	125.00	183.30

The sorptivity results are for the conventional and untreated-RCA concrete series. Sorptivity increased from 0.12 mm/ $\sqrt{\text{min}}$ for the control mix to 0.34 mm/ $\sqrt{\text{min}}$ at 100% untreated RCA, corresponding to an increase of approximately 183.3%. The increase indicates greater capillary suction and potential susceptibility to water ingress with increasing untreated RCA content. Comparative sorptivity performance of treated-RCA concretes remains to be established in future experimental work. The results are presented in Table 14 and Figure 4.

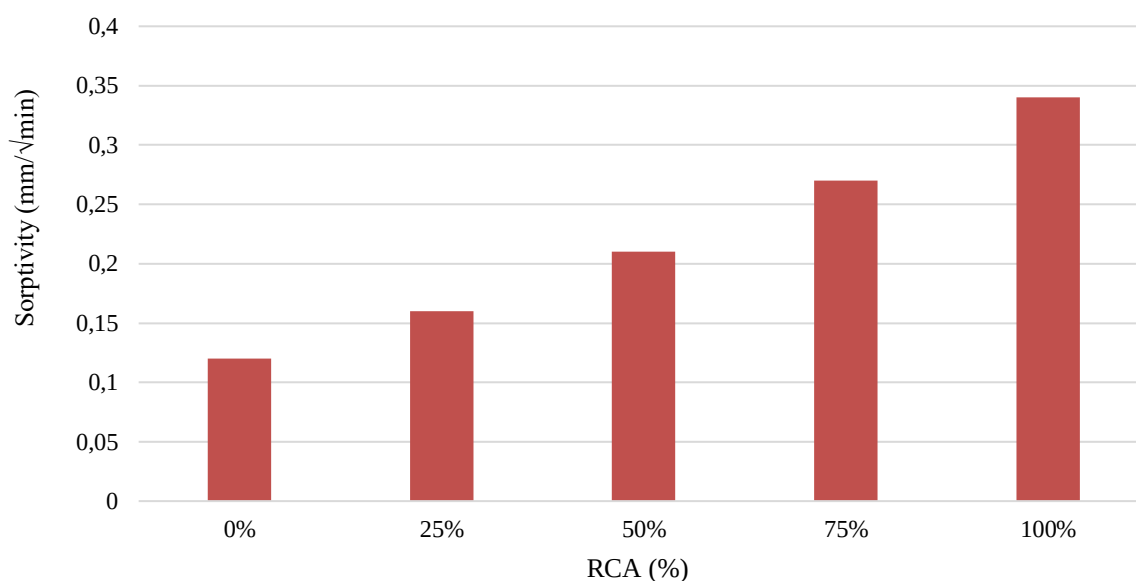


Fig. 4. Sorptivity versus untreated RCA replacement

6. Conclusions

This study compared untreated, acid-treated, mechanically treated and thermally treated RCA in M30 concrete over curing ages up to 365 days. The reported results show that increasing RCA replacement generally reduces workability and mechanical strength, while treatment effects depend on the treatment method and replacement level. Acid treatment gave the highest reported strength retention among the investigated treatments, whereas thermal treatment produced lower values than the corresponding untreated series. The available durability data are limited to conventional and untreated-RCA concrete; therefore, conclusions regarding durability benefits of the treatment methods are intentionally limited.

The comparative results indicate that acid treatment improved strength retention by reducing the influence of adhered mortar under the tested conditions. Mechanical treatment also provided strength retention benefits relative to untreated RCA. Thermal treatment did not improve the reported compressive, splitting tensile or flexural strengths relative to the corresponding untreated series and therefore requires further optimization before its effectiveness can be established.

- 1. RCA can be used as a partial substitute for natural coarse aggregate, but the suitable replacement level depends on the required performance and treatment condition.
- 2. At 28 days, compressive strength decreased from 34.50 MPa for the control to 28.60 MPa for RCAU100%, while lower replacement levels showed smaller reductions.
- 3. Acid-treated RCA showed the highest reported compressive-strength retention among the three pre-treatment methods. Mechanical treatment showed intermediate retention, while thermal treatment produced lower values than the corresponding untreated RCA series in the reported dataset.
- 4. Splitting tensile strength decreased with increasing untreated RCA replacement; at 28 days, the control and RCAU100% values were 3.40 and 2.80 MPa, respectively. Acid-treated RCA showed higher retention than untreated RCA at comparable replacement levels.
- 5. Flexural strength also decreased with increasing RCA replacement. At 28 days, the control and RCAU100% values were 4.80 and 4.00 MPa, respectively, while RCAA25% reached 4.75 MPa.
- 6. Water absorption increased from 1.50% for the control to 4.50% for RCAU100%, representing a 200% increase. This result indicates increased water-ingress susceptibility with increasing untreated RCA replacement.
- 7. Sorptivity increased from 0.12 to 0.34 mm/ $\sqrt{\text{min}}$ between the control and RCAU100%, representing an increase of approximately 183.3%.
- 8. RCA recycling can contribute to resource conservation and CDW diversion. Practical application should be based on the required performance, quality of the RCA source, treatment condition and replacement level.

7. Future Research

Future research should focus on comparative durability testing of concrete incorporating acid-, mechanically- and thermally treated RCA, microstructural characterization using SEM/XRD, optimization of thermal and mechanical treatment parameters, detailed life-cycle and energy assessment, and field-scale validation.

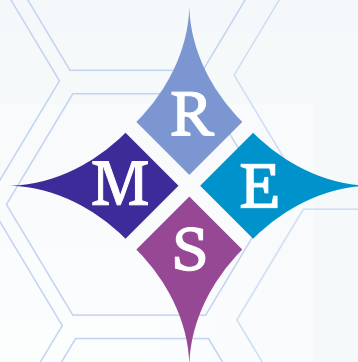
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