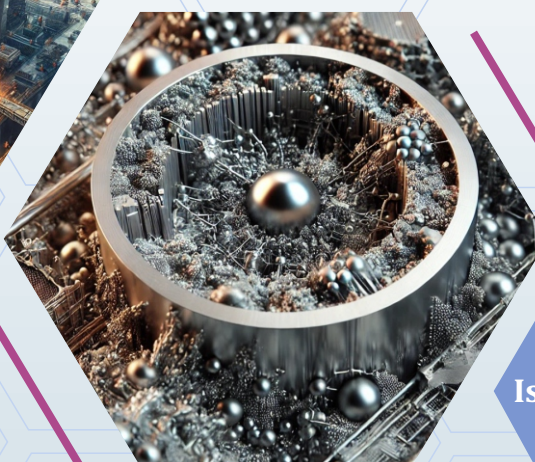




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Behavior of self-connected composite columns under eccentric load

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Abstract

This study presented a detailed structural evaluation of the behavior of composite columns under the effect of eccentric loads. The study focused on the impact of circular and square openings in the steel section. The study was carried out on 10 composite column specimens with different opening sizes and at three positions in the flanges and the web. The effect of these openings on the load-carrying capacity of the columns, as well as the slip behavior between the concrete and the steel section and their interaction, was studied. The study also aims to compare the effects of openings with those of shear studs and to explore the extent to which openings can replace shear studs to improve the structural efficiency of composite columns and simplify the construction process. The results of the study showed that the use of openings, especially those in the flanges, reduces slip and friction between the steel section and the concrete, thereby increasing the efficiency of force transfer between the concrete and the steel section and enabling them to act as a single element. Square openings reduced slip by 51.5%, while circular openings reduced it by 43.4%. Accordingly, this study represents an essential contribution to improving the design of composite columns, enhancing structural performance, and leveraging the mechanical properties of the section itself during self-connection with concrete to ensure load transfer, as well as the possibility of replacing shear studs with openings in future designs.

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1. Introduction

Composite structural systems have been widely used in structural frameworks and bridges for several decades to minimize material use. The benefits of using composite elements include reducing construction time, decreasing the overall weight of the structure, and minimizing vertical floor space. This construction system may help provide cost-saving services to the community. The composite column with shear connectors is a well-known integrated structural element of composite structures, offering economic feasibility and excellent structural performance. Shear connectors are used to connect two structural elements at an interfacial stratum to prevent the upward separation of a slab from a steel girder and to provide smooth transmission of longitudinal shear forces [1]. Load-slip between steel and concrete and shear capacity are essential characteristics for the design of steel-concrete composite structures [2,3]. The shear connector is the component that ensures shear transfer between the steel profile and the reinforced concrete slab, enabling composite action in composite beams. Several different types of connectors have been studied, proposed, and used in the past. [4,5]. Numerous studies have investigated various types of shear connectors, including headed studs, channels, perforated plates, and innovative geometric designs. These investigations focus on parameters such as on-site installation, load-carrying capacity, failure modes (e.g., concrete crushing, steel yielding, or stud shearing), and

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allowable interface slip [6,7,8]. These studies vary according to design codes and the intended structural function, using different types of shear connectors compared with the most common type, namely the headed-stud shear connector [9-11]. Despite the many advantages of hybrid systems, more attention is needed to improve the bond between steel and concrete, aiming to maximize efficiency and reduce material consumption. From reviewing most of the studies, it was observed that they mainly focused on the use of connectors and on developing some of them to ensure bonding between the concrete and the steel section in composite structural members. Few studies, however, relied on openings in the steel section for bonding. This method was previously used in composite beams and was referred to as Self-connected. A self-connected, partly enclosed composite beam with a perforated steel section and concrete shear connectors was compared with conventional composite beams [12]. This type of bonding has not been previously used in composite columns, and the current study is the first to address its use.

2. Materials

The cement used in this study is Iraqi cement produced in Nineveh Governorate at the Badush Cement Plant, in accordance with Iraqi standard specifications [13], and it meets the requirements for ordinary Portland cement. Table 1 shows the chemical properties of the cement, and Table 2 outlines the physical properties of the cement used. All these tests were conducted in the laboratories of the University of Mosul. River gravel with a maximum particle size of 12.5 mm was selected for casting. Specific gravity 2.51 and density 1508 kg/m³.

Table 1. Chemical analysis of cement and its compounds

NO	Oxides of cement components	Percentage(%)	Specification limits (%) [13]
1	Al ₂ O ₃	5.4	3.0-8.0
2	SiO ₂	23.4	17.0-25.0
3	Fe ₂ O ₃	3.25	0.5-6.0
4	CaO	61.8	60.0-67.0
5	SO ₃	2.25	Does not exceed 2.8%
6	MgO	3.5	Does not exceed 5.0%
Main compounds of cement			
7	C ₃ S	38.53	31.03-41.05
8	C ₂ S	33.45	28.61-37.90
9	C ₃ A	11.99	11.96-12.30
10	C ₄ AF	7.92	7.72-8.02

A sieve analysis was carried out on the gravel to verify its conformity with the American standard specifications [14], as detailed in (Fig. 1). Sand sourced from the Knihesh area was utilized for casting molds. The sieve analysis confirmed that the sand meets the American standard specifications [14]. Specific gravity 2.41, the fineness modulus is 2.97, and the density is 1758 kg/m³. The clay content was measured at 2%, which is below the maximum permissible limit of 3% for clay materials, as indicated in (Fig. 2).

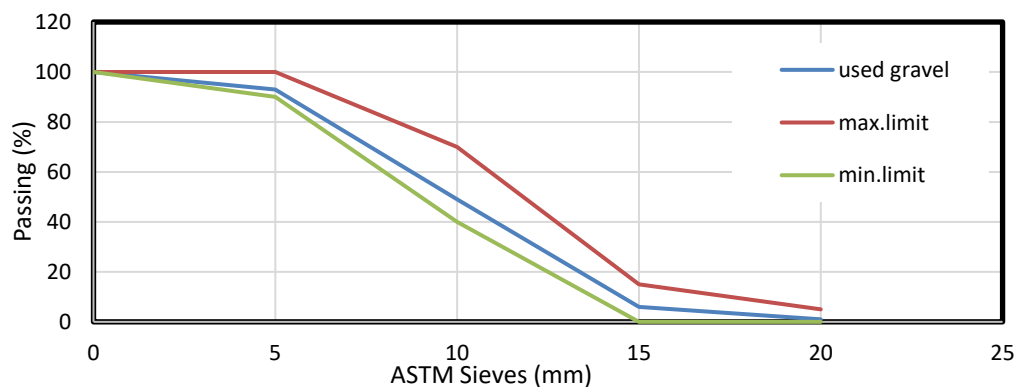


Fig. 1. Sieve analysis of coarse aggregate

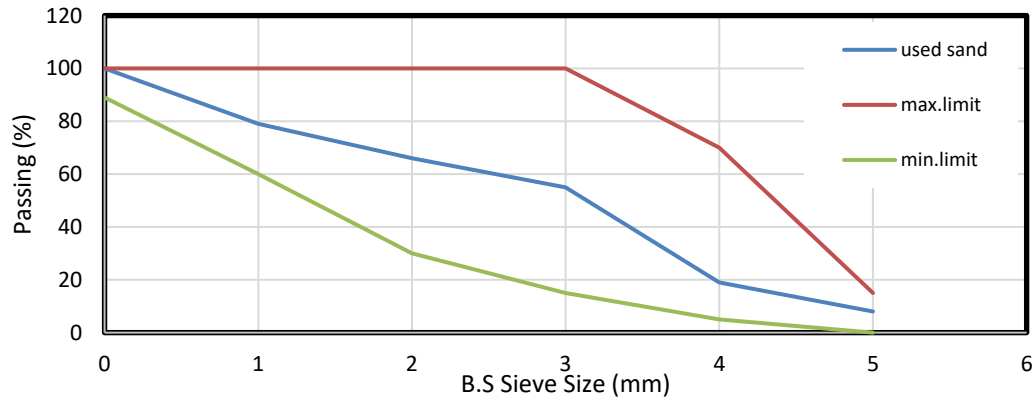


Fig. 2. Sieve analysis of fine aggregate

The water used in the concrete mix and for curing the concrete is potable. The composite concrete columns were reinforced with eight mm-diameter longitudinal bars and six mm-diameter hoops. Tensile tests on the reinforcement steel were conducted using a universal testing machine with a capacity of 600 kN; the test results are presented in Table 3. All the bars used comply with the ASTM Standard [15]. An H-Steel section was used in the composite concrete columns, with dimensions of 80 mm for the flange (6 mm thick) and 64 mm for the web (4 mm thick) (H80*80*8*6), and a length of 2800 mm. The tensile properties of the flange and web were verified through a coupon test [16]. The results indicated that the section complied with the ASTM specification [17], as presented in Table 4.

Table 2. Physical properties of the used cement

NO	Properties	Results	Specification limits (%) [13]
1	Fineness (percentage retained on sieve No. 170)	3%	Does not exceed 10%
Setting Time			
1	Initial (minutes)	120	Not less than 45 minutes
2	Final (minutes)	225	Does not exceed 600 minutes
Compressive Strength (MPa)			
1	3 Days	19.7	Not less than (16 MPa)
2	7 Days	24.8	Not less than (24 MPa)
Tensile Strength (MPa)			
1	3 Days	1.77	Not less than (1.6 MPa)
2	7 Days	2.45	Not less than (2.4 MPa)

Table 3. Steel bar properties

Bar db	fy (MPa)	fu (MPa)	Elongation %	Standard Limits [15]%
6	378	458	16.4	≥ 9%
8	433	541	15.8	≥ 9%

Table 4. H steel section properties

H Steel section	Plate Thickness mm	fy (MPa)	Standard Limits for fy [17]	fu (MPa)	Standard Limits for fu [17]	Elongation %	Standard Limits Elongation% [17]
flange	8	278	>250	427	400-550	21	>20
web	6	261	>250	409	400-550	24	>20

3. Concrete Mix Design

The concrete mix was designed according to the American standard [18]. Several trial mixes were conducted to achieve a compressive strength of 30 MPa, as shown in Tables 5 and 6. Coarse aggregate with a maximum size of 12.5 mm was selected to ensure uniformity of the mix and ease of handling. The water-to-cement ratio was set at 0.48 to balance strength and durability requirements. The slump value was adjusted to range between 75-100 mm, ensuring an adequate level of workability and ease of pouring. The final concrete mix demonstrated high workability.

Table 5. Properties Of the Concrete Mix Used.

Materials	Quantity per 1m ³
Water/cement ratio	0.48
Water (Kg/m ³)	216
Cement (Kg/m ³)	450
Fine aggregate (Kg/m ³)	714
Coarse aggregate (Kg/m ³)	912

Table 6. Compressive strength test of cubes

Sample	Compressive Strength (MPa)
1	31.7
2	32.6
3	32.1
4	33.9
5	32.3
6	31.8
Average	32.4

4. Experimental Program

Ten composite columns 140x140x2800mm were fabricated using plywood forms. The H steel sections were placed and leveled in the molds with the 4 bars with 8mm diameter longitudinal reinforcement and stirrups of 6mm diameter each 150mm spacing, as shown in (Fig.3). Concrete with a compressive strength of 30 MPa was cast and used for the molds by employing a vibrator during casting. Six standard concrete cubes (150x150x150 mm) were used to test compressive strength, and six standard cylinders (150x300 mm) were used to test tensile strength. The concrete was cured by spraying potable water for 28 days, then transported to the laboratory for testing. Four strain gauges of type (FLAB-6-11-3LJC-F) of 6 mm length were used on each column, glued to the top and bottom flanges of the steel section. Another two gauges (PFL-30-11-3LJC-F with a length of 30 mm) were fixed on the concrete at the center of the column, as shown in (Fig.4). Two types of openings were used: a circular opening with two diameters of 15mm and 30mm, and square openings of 13.35 mm and 26.6 mm. The openings were placed in symmetrical positions at the top and bottom flanges and web, with a distance in the longitudinal direction of 150mm on centers, as shown in (Fig.5). Specimens made with 12.5mm diameter and 17.5mm length stud shear connectors were installed and welded at locations similar to the openings' locations, as shown in (Fig.6) and Table (7) shows the details of the specimens. The LVDT (Linear Variable Differential Transformer) with an accuracy of 0.001 was mounted to measure the slip between the concrete and steel sections. The eccentric loads were applied to the specimen through a thick steel plate (25mm) using a hydraulic jack having a capacity of 1000 kN with a load cell connected with all other measuring instruments (LVDT and Strain gauge) to the data logger used to record readings during the test as shown in (Fig.7). All test data were exported from the data logger to the computer in Excel format. The work conducted in the Civil engineering department's material laboratory, at university of Mosul, which has a certificate from the C.O.S.Q.C.I.Q. (Iraqi Central Organization for Standardization and Quality Control) and Iraqi Accreditation System. The accuracy of the measurements and testing ensured by a qualified technician, double check all measurements and using a calibrated equipment.

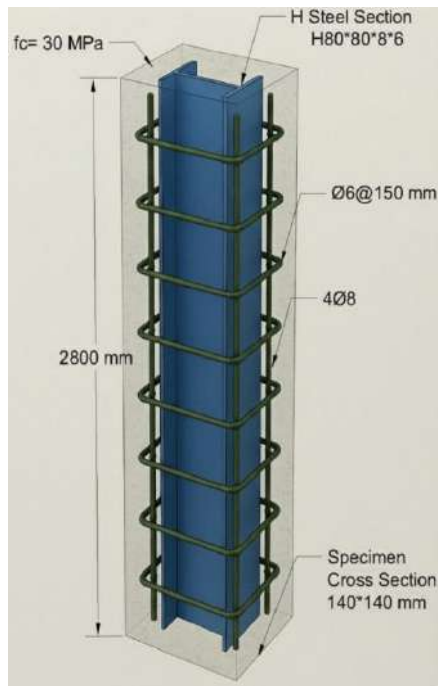


Fig. 3. Composite columns reinforcement

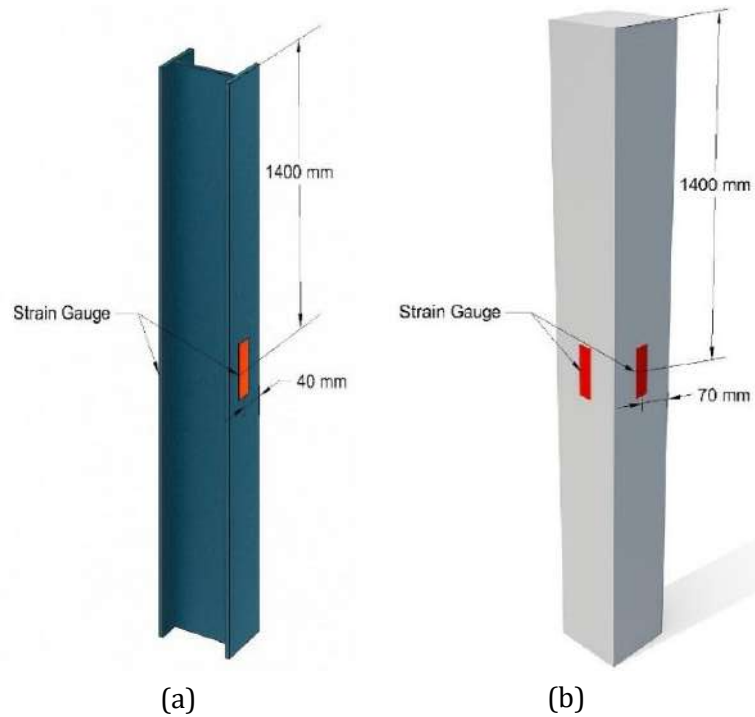


Fig. 4. Strain gauge locations

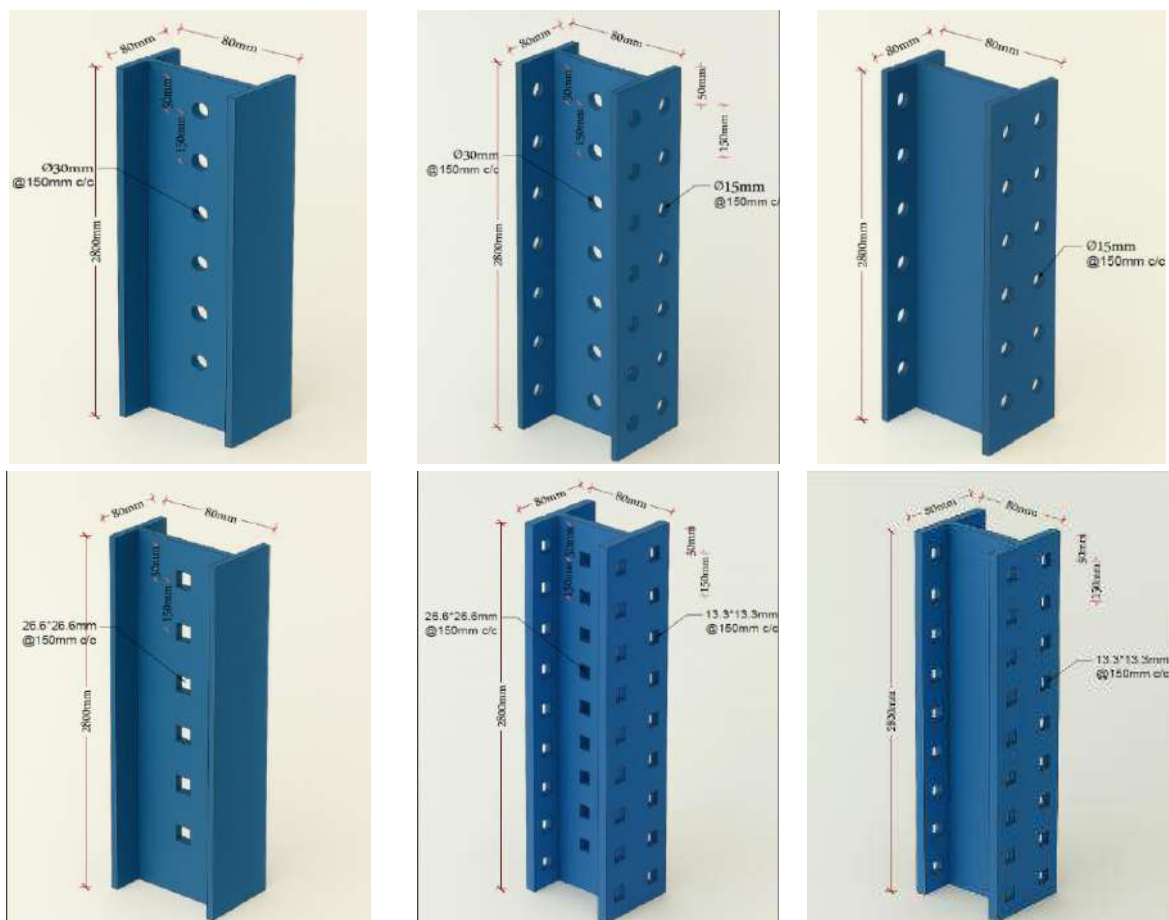


Fig. 5. Shape, details, and locations of the openings

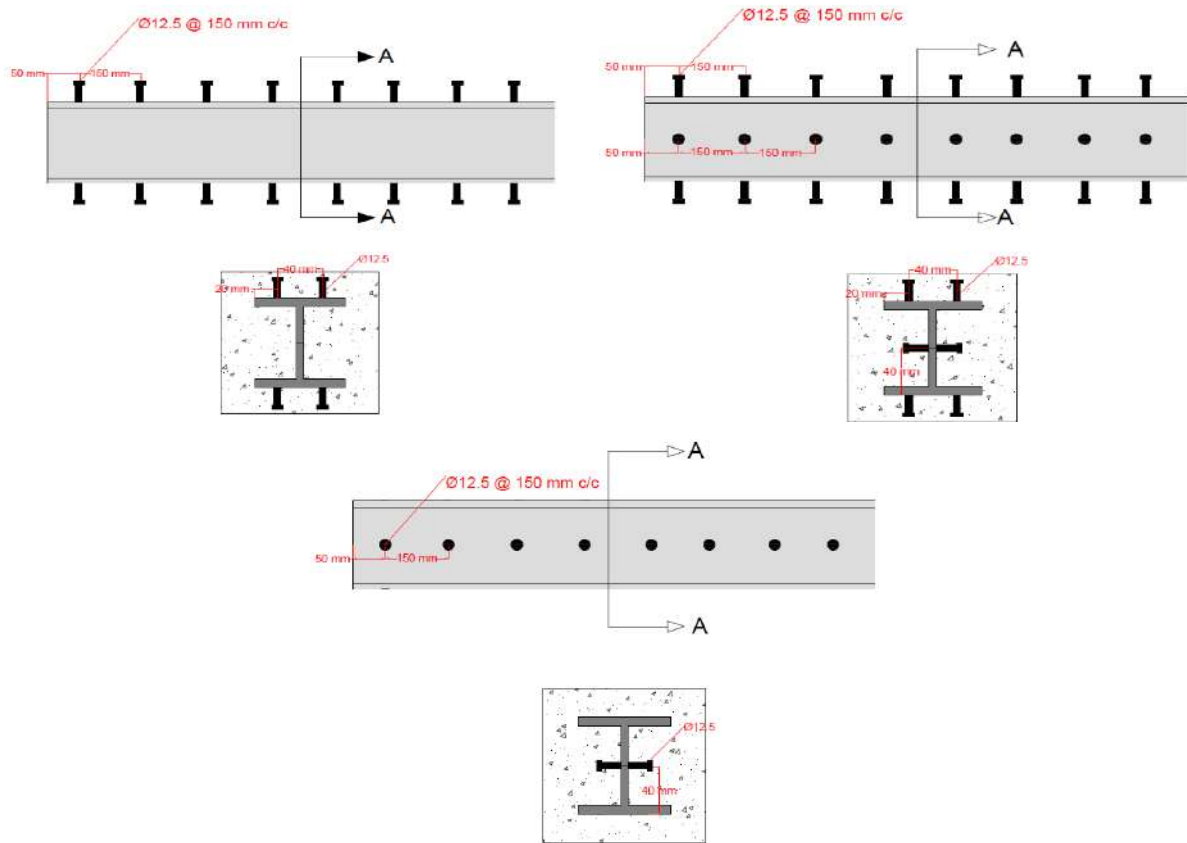


Fig. 6. Shape, details, and locations of the headed studs

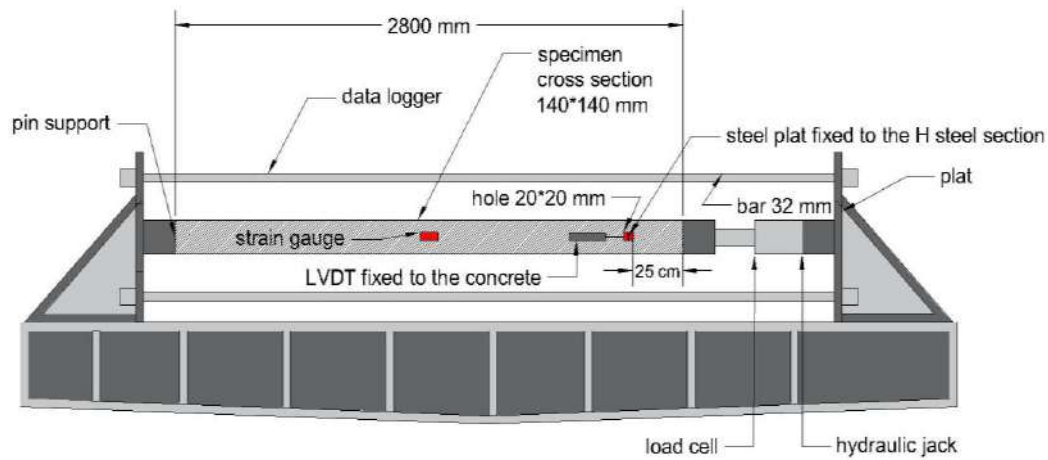


Fig. 7. Illustrates the preparation of the specimen for testing and the details of the main testing apparatus

Table 7. Specimen details

No.	Specimen	Hole Shape	Number of Holes	Cross-Section Area mm ²	Location of Parameter	Dimension of opening mm
1	CO2	no hole	No holes	1664	Non	nill
2	OFS2	square openings	72	1238.4	Flange	13.3*13.3
3	OWS2	square openings	18	1504.4	Web	26.6*26.6
4	OFWS2	square openings	90	1078.8	Flange and web	13.3*13.3 26.6*26.6
5	OFC2	circular openings	72	1184	Flange	Ø15
6	OWC2	circular openings	18	1484	Web	Ø30
7	OFWC2	circular openings	90	1064	Flange and web	Ø15 Ø30

No.	Specimen	Connector	Number of Connector	Cross-Section Area mm ²	Location of Parameter	Dimension of Connector mm
1	SF2	Headed shear studs	72	1664	Flange	Ø12.5
2	SW2	Headed shear studs	36	1664	Web	Ø12.5
3	SFW2	Headed shear studs	90	1664	Flange and web	Ø12.5

5. Mechanism for Applying Loads to The Specimen

The eccentric load, applied from one direction at a distance of $e=35$ mm (from the center of the applied load on the specimen, corresponding to a ratio of $(e/h = 0.25)$) This ratio is commonly used in experimental studies as it represents a moderate eccentric loading condition, not too small not to affect the behavior, and not too large to cause an uncontrollable failure. This type of loading was designed to simulate realistic structural loading conditions, in which loads are often applied outside the section's centroid, thereby generating additional moments that influence the column's structural behavior. The specimens were supported during testing using a pin support, as illustrated in Fig.8, which details the loads and the mechanism of their application on the specimens.

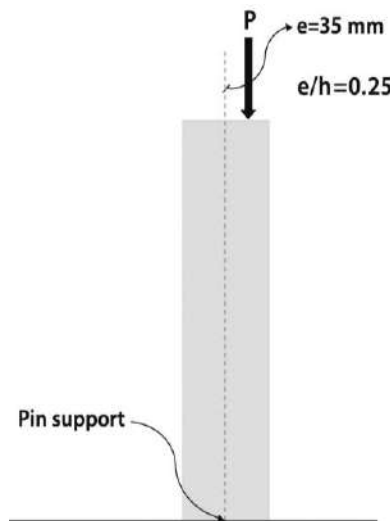


Fig. 8. Mechanism for applying loads

6. Test Results

6.1 Composite Columns Containing Circular Openings

This study evaluated the behavior and performance of three different models of composite columns featuring openings in the steel section subjected to eccentric loads (OFC2, OWC2, OFWC2) are shown in (Fig.9) (Fig.10). The analysis was directed towards several factors including the effect of the openings on the load-bearing capacity of the column as compared to hosts without openings. Also, the load-slip response between the steel and concrete parts was studied, and possible failure mechanisms were explored. During the load application process on the model (OFC2), cracks appeared in the concrete at the mid-height of the column. As the applied loads increased, the cracks continued to propagate from the middle towards both sides of the column. Additionally, the column exhibited buckling with cracks forming in the first third of the column in the direction of the applied load. These cracks continued to develop until final failure at the column head, where the concrete crushed in a layered manner, with one layer stacking over another. For model (OWC2), cracks initially appeared at mid-height of the column during load application and continued to increase as

the column buckled, leading to final failure at the column center. In the last model (OFWC2), cracks appeared near the load application area and continued to propagate in a layered manner until final failure, when a fracture occurred in the steel section, resulting in the flange separating from the web. Minimum slip levels in the column with circular openings are attributed to several interlinked factors, compared to the reference column.

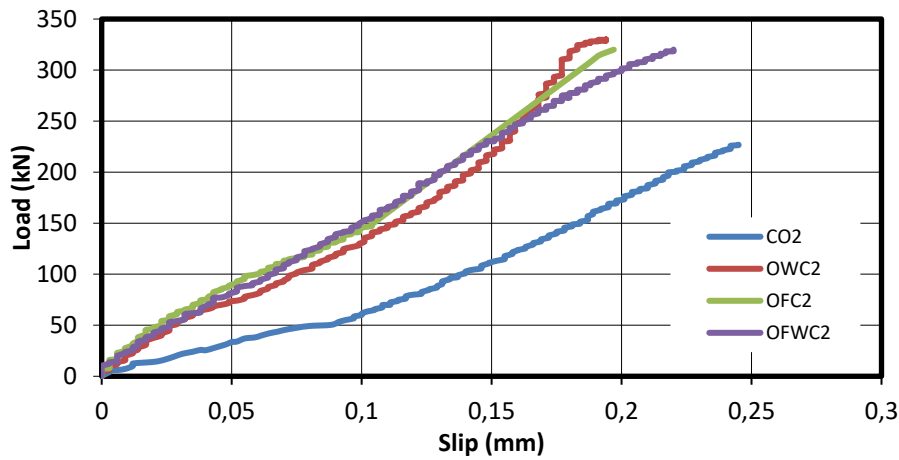


Fig. 9. Load-Slip for Specimen CO2, OFC2, OWC2 and OFWC2



(a)



(b)



(c)

Fig. 10. Failure mode of the specimen. (a) OFC2, (b) OWC2, and (c) OFWC2

Firstly, the openings assist in redistributing stresses in the Steel section, reducing the concentration of forces at specific points and thus minimizing slip. Furthermore, the steel's exposure area is reduced, leading to reduced direct frictional impact a significant cause of slip in the reference column. Besides this, the slots enhance the column's versatility, allowing it to better adjust to applied loads with no or minor slip. They also reduce stress concentration at the surface of the steel section, thereby decreasing the forces that cause slip. In addition, the openings change the failure mechanism. The column with openings depends more on composite action than on frictional resistance, unlike the reference column. The bearing stresses at the contact area of concrete inside the circular opening will increase the connections and enhance axial load resistance but the rotation depends only on the friction between the steel and concrete at the contact face. Meanwhile, the use of square opening will cause stress concentration at corners, which enhance the rotation resistance rather than the bearing stress. [18] Finally, the openings also increase the column's stiffness and allow it to accept loads more easily, with fewer slips than the reference column.

6.2 Composite Columns with Square Openings

To investigate the influence of eccentric loads on the behavior and performance of composite columns with square openings (OFS2, OWS2, OFWS2) in their steel sections, tests were carried out on three models of composite columns. These openings were used to assess their impact on the columns' load-carrying capacity, compared with results from models without openings. The interaction between the steel and concrete components was analyzed regarding load-slip behavior to identify potential modes of failure for each model. During the testing process of model (OFS2), cracks began to appear in the lower third of the column, accompanied by buckling. The concrete continued to fracture and buckle until reaching ultimate failure. As for model (OWS2), during loading, cracks formed in the upper third of the column, along with buckling towards the back and upward. These cracks continued to propagate along with the buckling until ultimate failure. In the last model (OFWS2), cracks initially appeared in the middle of the column, accompanied by upward buckling. The buckling continued to increase, along with the expansion of cracks, until the column reached ultimate failure.

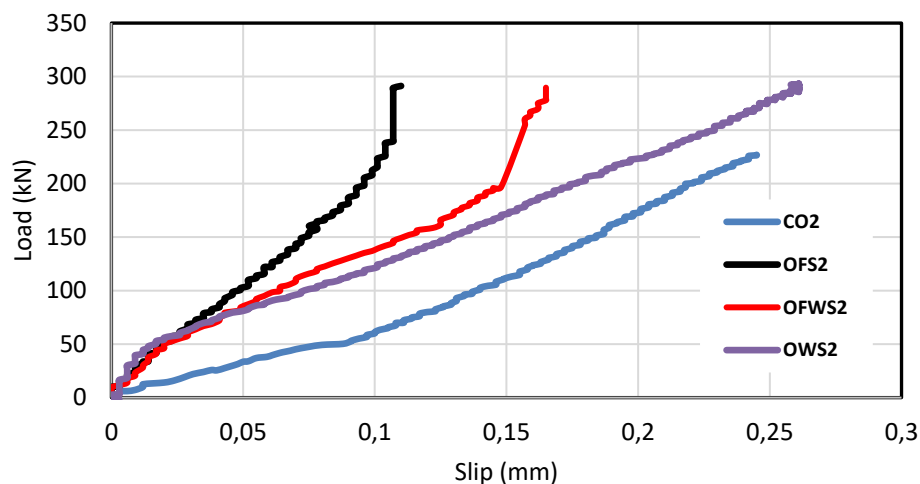


Fig. 11. Load-slip for specimen CO2, OFS2, OWS2 and OFWS2

The load-slip curves obtained from the tests are shown in (Fig.11) (Fig.12), illustrating the response of the specimen under the effect of eccentric load. The presence of openings leads to greater stress concentrations in the regions surrounding the openings; therefore, the columns typically fail at discrete locations around the opening(s) rather than, for example, gradually failing throughout the whole column along its length or width. Regarding the position of openings, flange openings significantly influence the slip response more than web openings. The flange is a critical transfer element for axial loads between steel and concrete. Thus, a few openings in the flange decrease the effective contact area and spread the stresses to other places, thereby minimizing slip.



(a)



(b)



(c)

Fig. 12. Failure mode of the specimen. (a) OFS2. (b) OWS2. (c) OFWS2

6.3 Composite Columns with Shear Studs

The study was carried out on composite columns with shear studs in the steel section (SF2, SW2, SFW2) under eccentric loading. The test was conducted on three composite column models. The bond behavior between the steel section and the concrete was studied. The load–slip curve was drawn separately for each model, providing a deeper understanding of the effect of vertical shear studs on the cohesion and bond between the steel section and the concrete, as well as the ultimate load capacity under eccentric loads. During testing of the model (SF2), the concrete began to fracture near the top of the column. It continued to break until ultimate failure, when the flange separated from the web at the load application area. The same failure pattern was observed in model (SW2). As for the last model (SFW2), the concrete also began to fracture at the top of the column and continued to deteriorate until reaching ultimate failure. The load–slip curves were plotted, as shown in (Fig.13), and the Failure mode of the specimens in (Fig.14), illustrating the response of the specimen under the effect of eccentric load. Shear studs improve the connection between steel and concrete, reducing slip. However, the effectiveness of the studs depends on their location. Since the flange primarily transfers loads between steel and concrete, shear studs placed in the flange are more effective in reducing slip. On the other hand, studs placed on the web are less effective at transferring forces than those in the flange, limiting their impact.

Regarding load-bearing capacity, the reference column has a higher capacity and exhibits more ductile failure. Stress is distributed more uniformly when loads are applied, resulting in better load-bearing capacity for the column than for columns with shear studs. Columns with shear studs show slightly lower load-bearing capacity and more brittle failure. This means that failure is not uniform throughout the column; rather, it is more severe in areas of localized stress around the studs.

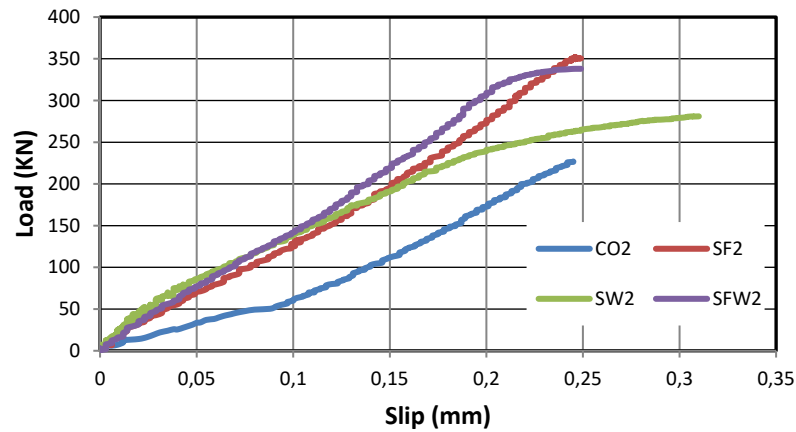


Fig. 13. Load-slip for Specimen CO2, SF2, SW2 and SFW2



(a)



(b)



(c)

Fig. 14. Failure Mode of The Specimen (a)SF2. (b)SW2. (c)SFW2

7. Effect of The Shape and Location of Openings

Six composite column models with various openings were tested to investigate the effects of openings under eccentric loads. There were three columns with circular openings (OFC2, OWC2, OFWC2) and three with square openings (OFS2, OWS2, OFWS2). The openings were placed at different locations in each model, and two opening shapes were used to study the effects of location and shape on slip behavior and load-bearing capacity. The models were tested under eccentric loads, and load-slip curves were drawn for each. The curves provided a deep understanding of the models' performance under the influence of eccentric loads. The curves also showed the effects of the shape and location of the openings on the mechanical properties of the composite columns. They allowed comparison of the structural performance of composite columns with openings. The

load-slip curves indicate that the slip between the steel and concrete sections of a composite square column is lower than that of a composite column with a circular opening. There are several reasons for this effect; the first is that the square shape has sharp corners, which act as resistance factors, preventing the limited concrete in the steel section from transferring, reducing its tendency to rotate. These corners help to develop the mechanical bond between concrete and steel much better than the circular openings do.

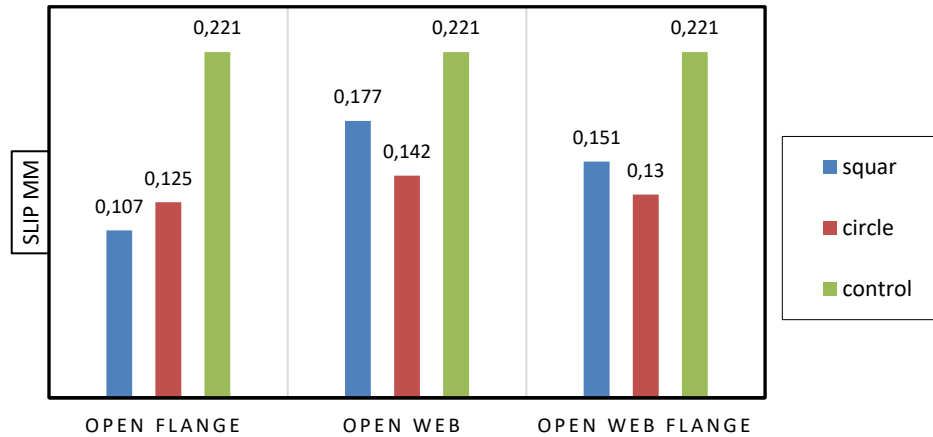


Fig. 15. Slip values at a load of 200 kN for columns with circular and square openings

Table 8. Slip at load 200 (kN)

Code	Slip (mm)	%Slip Decrease
CO2	0.221	reference
OFS2	0.107	51.5
OFC2	0.125	43.4
OWS2	0.177	20
OWC2	0.142	35.7
OFWS2	0.151	31.6
OFWC2	0.130	41

Shown in Figure 15 and Table 8 is the effect of the shape of the openings on reducing slip between the concrete and steel portions of the column under a load of 200 kN. Square edges allow for greater contact between materials because they have a larger internal surface area than circular edges. This increased contact helps improve friction resistance between the concrete and steel, lessening slip under axial loads. The additional friction from the larger contact area enhances the column's cohesion and load-bearing capacity. As a result, it can be concluded that the use of square openings improves bonding between concrete and steel.

Table 9. Summary results

Code	Load at yield (kN)	Slip at yield Load (mm)	Ultimate Load (kN)	Slip at ultimate Load (mm)	Load Increasing%	Ductility index
CO2	140	0.175	227	0.245	reference	1.3
OFC2	170	0.11	320	0.197	40.96	1.79
OFS2	150	0.09	300	0.168	32.15	1.87
OWC2	156	0.116	328	0.194	44.5	1.67
OWS2	180	0.157	285	0.261	25.55	1.66
OFWC2	173	0.118	320.1	0.22	40.97	1.86
OFWS2	140	0.133	287	0.23	26.43	1.73

The minimum slip gained from using opening in flanges can be attributed to the increasing in rotation resistance mainly depends on the coupling of tension and compression forces at both tension and compression zones. Moreover, the presence of openings increased the ductility index and the ultimate load capacity of composite columns with openings, as shown in Table 9. This improvement can be attributed to the fact that the presence of openings enhanced the columns' ability for plastic deformation and increased their capacity to absorb the applied loads.

8. Comparison Between Composite Columns with Shear Studs and Columns with Openings in the Steel Section Under Axial Load

The objective of this study is to investigate eccentric loaded behavior of composite columns with shear studs, without shear studs but possessing openings in the steel section, and to explore whether it is possible to remove shear studs and use openings instead. They included three with square openings, three with a circular opening, and three with shear studs. Load-slip curves were used as a diagnostic tool to examine each model's response to variations in cohesion and slip behavior under load. Results indicate that shear studs in those columns develop high levels of cohesion and minimize slip, whereas openings can affect the force transfer mechanisms and stress distributions. It was shown that the potential of using cohesion as an effective alternative in cases where columns with openings achieve safe and efficient designs that satisfy structural performance requirements and even exhibit behavior similar to that of columns with shear studs, which comply with the results obtained by Al-Darzi, 2007 [18]. The results also showed a close similarity in the behavior of columns with shear studs and those with openings, as the openings in the steel section significantly improve slip resistance between the steel section and the surrounding concrete., as shown in Table 10.

Table 10. Slip decreases at load 200 (kN)

Code	Slip (mm)	%Slip Decrease
CO2	0.221	reference
SF2	0.151	31.67
OFS1	0.107	51.5
OFC1	0.125	43.4
SW2	0.157	28.95
OWS1	0.177	20
OWC1	0.142	35.7
SFW2	0.139	37.1
OFWS1	0.151	31.6
OFWC1	0.130	41

Table 11. Summary of the results for all specimens tested

Code	Load at yield (kN)	Slip at yield Load (mm)	Ultimate Load (kN)	Slip at ultimate Load (mm)	Load Increasing%	Ductility index	Standard Deviation
CO2	140	0.175	227	0.245	reference	1.3	0.06725
OFC2	170	0.11	320	0.197	40.96	1.79	0.04078
OFS2	150	0.09	300	0.168	32.15	1.87	0.03761
SF2	150	0.119	350	0.247	54.18	2.08	0.07366
OWC2	156	0.116	328	0.194	44.5	1.67	0.06003
OWS2	180	0.157	285	0.261	25.55	1.66	0.08739
SW2	191	0.151	281	0.31	23.78	2.05	0.09717
OFWC2	173	0.118	320.1	0.22	40.97	1.86	0.06772
OFWS2	140	0.133	287	0.23	26.43	1.73	0.05500
SFW2	176	0.126	338	0.249	48.9	1.99	0.06886

Moreover, the presence of shear studs leads to an increase in the ductility index of the composite columns, in addition to an increase in the ultimate load capacity of the columns. Table 11 summarizes the results for all specimens tested. The standard deviation shown in Table 11 explained that the results deviate with no more than 9%.

9. Conclusions

In this study, the test results for self-connected composite columns were obtained. The effect of openings in the steel section on the slip behavior and ultimate load-bearing capacity of the composite columns was investigated through load–slip curves. The influence of the shape and location of the openings on the column behavior was also examined. Furthermore, these columns were compared with columns with vertical shear studs to assess the feasibility of replacing conventional shear studs with openings. Based on the experimental results, the main conclusions obtained within the limits of the present work were as follows:

- The use of headed stud connectors will increase the column ultimate load resistance with a ductile behavior having less slip compare with the control specimen.
- The composite column with opening gives larger resistance compare with control specimen and reduce the slip between the steel and concrete. The results showed that the composite columns containing openings exhibited an increase in ultimate load-bearing capacity. The increase in columns with square openings in the flange reached 32.15%, while the rise in columns with circular openings reached 40.96%.
- The composite column with opening gives lesser resistance compare with headed stud specimen, but reduce the slip between the steel and concrete.
- The shape of opening affects the behavior of the composite column. The composite columns with square openings in the flange give a larger slip reduction than those with circular openings. The slip reduction for columns with square openings was 51.5%, while it was 43.4% for columns with circular openings. It can be stated that the use of openings leads to better bonding between the concrete and the steel, thereby enhancing composite interaction within the column and making openings a more effective means of improving structural performance.
- The results show that the shape and location of the openings have a significant effect on slip in composite columns, as the results indicated that openings located in the flange area play an essential role in reducing slip. The web area plays a minor role in opposing slip compared to the flange, which governs the steel and concrete's load transfer and bond mechanism. These openings can create a mechanical bond between the steel section and the concrete by interlocking the concrete with the steel section.
- The composite columns containing openings exhibited a higher value of the ductility index, indicating that the presence of openings enhanced the columns' ability for plastic deformation and increased their capacity to absorb the applied loads.

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References

- [1] Eghbali NB, Andamnejad P. Structural performance of rigid shear connectors in concrete encased steel composite columns. *Structures*. 2023;54:348-368. <https://doi.org/10.1016/j.istruc.2023.05.040>
- [2] Haroon M, Lee HD, Shin KJ, Woo JH, Lee JS. Performance of angle shear connectors for U-shaped steel concrete infilled composite beams. *Heliyon*. 2024;10(3):e25119. <https://doi.org/10.1016/j.heliyon.2024.e25119>
- [3] Wang W, Mi J, Zhu B, Chen Y, Xu J, Jia Y. Study on bond-slip mechanism and interface shear transfer of corrugated web PEC columns with studs. *Constr Build Mater*. 2024;413:134899. <https://doi.org/10.1016/j.conbuildmat.2024.134899>

- [4] Soalih HA, Demir S. Current practice and recent developments of shear connectors for timber-concrete composite applications: a state-of-the-art review. *J Struct Eng Appl Mech.* 2023;6(5):422-440. <https://doi.org/10.31462/jseam.2023.05422440>
- [5] Zhou F, Liang H, Jiang J, Zhou Z, Zou Y. Shear behavior of prefabricated steel-concrete connectors in prefabricated steel-concrete composite beams. *Sci Rep.* 2025;15:10993. <https://doi.org/10.1038/s41598-024-82933-3>
- [6] Wang X, He Q, An Z, Liu G, Wen X, Wang Y, et al. Experimental study of perfobond rib shear connector under lateral force. *Appl Sci.* 2021;11(19):9088. <https://doi.org/10.3390/app11199088>
- [7] Zheng Z, Zou Y, Chou Y, Qin F, Chen F, Di J, et al. Shear behaviour and calculation methods of bearing-shear connectors for prefabricated steel-concrete composite beams. *Materials (Basel).* 2023;16(13):4616. <https://doi.org/10.3390/ma16134616>
- [8] Pardeshi RT, Patil YD. Review of various shear connectors in composite structures. *Adv Steel Constr.* 2021;17(4):394-402.
- [9] Hadi BA, Saleh SM. Effect of stud connector diameter on the behavior of steel-lightweight self-compacting concrete composite beams with partial shear connection. *Academia Open.* 2024;9(2). <https://doi.org/10.21070/acopen.9.2024.8385>
- [10] Lai Z, Weng X, Yang X, Zhao H. Shear behavior and design of headed studs embedded in steel-UHPC composite structures. *Structures.* 2024;59:105788. <https://doi.org/10.1016/j.istruc.2023.105788>
- [11] Du W, Hu Z, Zhou Z. Analytical study of stud shear connector behavior in steel-UHPC composite structures. *Buildings.* 2024;14(12):3807. <https://doi.org/10.3390/buildings14123807>
- [12] Maarooof AAA, Abdullah JA, Kasim SY. Performance of steel perforated and partially-encased composite self-connected beams. *J Kejuruteraan.* 2022;34(4):703-717. [https://doi.org/10.17576/jkukm-2022-34\(4\)-18](https://doi.org/10.17576/jkukm-2022-34(4)-18)
- [13] Ministry of Planning. Iraqi standard specification I.Q.S. No. 5M: Portland cement. Baghdad (Iraq): Iraqi Central Organization for Standardization and Quality Control; 2016. Arabic.
- [14] ASTM International. Standard specification for concrete aggregates. ASTM C33/C33M-18. West Conshohocken (PA): ASTM International; 2018.
- [15] ASTM International. Standard specification for deformed and plain carbon-steel bars for concrete reinforcement. ASTM A615/A615M-16. West Conshohocken (PA): ASTM International; 2016.
- [16] ASTM International. Standard test methods for tension testing of metallic materials. ASTM E8/E8M-22. West Conshohocken (PA): ASTM International; 2022.
- [17] ASTM International. Standard specification for carbon structural steel. ASTM A36/A36M-19. West Conshohocken (PA): ASTM International; 2019.
- [18] Al-Darzi SYK. Effects of concrete nonlinear modeling on the analysis of push-out test by finite element method. *J Appl Sci.* 2007;7(5):743-747. <https://doi.org/10.3923/jas.2007.743.747>

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Thermo-mechanical behavior of a symmetric single-walled carbon nanotube/polymer reinforced beam laminate

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Abstract

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The symmetrical laminates reinforced with carbon nanotubes (CNTs) offer exceptional performance, due to their high specific stiffness, low density, and excellent fracture toughness. Through the understanding of their thermo-mechanical behavior under various stresses and temperature is essential for their design. Moreover, if the distribution and orientation of CNTs are adjusted and their agglomeration is avoided, their integration into a polymer matrix allows the macroscopic application of their exceptional characteristics in different fields. This work provides a micromechanical modeling approach to predict the effective thermo-elastic behavior of nanocomposites. In order to determine these effective parameters, the Mori-Tanaka homogenization approach is used. With a volume fraction equal or less than ten percent which, modify significantly thermo-elastic properties of the composite and also, a simplified model where CNTs are modeled as hollow cylindrical shells. These effective properties are combined via MATLAB for ensuring a structural analysis based on the classical laminate theory and first-order shear deformation theory to examine the mechanical behavior and the classical laminate theory to evaluate the thermal behavior of symmetric laminated nanocomposite beams. The model focuses particularly on the thickness distribution of normal as in-plane and out-of-plane shear stresses that vary over thickness and stresses, deformations due to the temperature variation. The results show the need to capture shear deformation effects and indicate the validity of proposed model for analyzing stress continuity and identifying critical locations in nanocomposite laminates and so how each ply reacts to a temperature variation despite an overall geometry without curvature.

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1. Introduction

Carbon nanotubes (CNTs) have been drawing continuing interest since their development in the early 1990s [1]. This is explained by their remarkable mechanical, electrical and thermal properties. Recent publications [2,3] have already established these features, making them interesting choices for composite reinforcement [4]. A little amount of CNTs can improve the overall thermo-elastic characteristics of the composite [5], which prompted the use of numerous frequent methodologies to study the thermo-elastic behavior of fiber-reinforced composites. In this side, Seidel and Lagoudas achieved it for carbon nanotube-reinforced composites (CNTRC) [6]. X. Li et al. obtained an excellent agreement between their simulation outcomes and experimental data when using self-consistent methods, as they combined it with Halpin-Tsai theory in their simulation of nanocomposites single-walled carbon nanotubes (SWCNTs)/Epoxy [7]. Z. He et al. obtained almost 20% of reduction for the coefficient of thermal expansion and 40%. Increasing the

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thermal conductivity by just adding 2% of weight of CNTs [8]. Due to multiple variables that complicate the experimental field and the fact that the rule of mixtures (ROM) cannot be taking into account, some important parameters such as the agglomeration effect of CNTs [9] and since that it's swift, operates one as analytical approximation and doesn't ask more of computational resources. Several writers [10, 11] choose the use of the Mori-Tanaka (MT) [12], which is regarded as one of the best analytical approaches for calculating the effective constants of the composite. At large scale, classical laminate theory (CLT) was employed to simulate multilayer structures, using the effective characteristics determined previously [13, 14]. Heshmati and Yas [15] also had combined the MT approach with a finite element method to analyze CNTs' agglomeration and dispersion effects on FG Euler–Bernoulli beams. Authors of [16] captured the CNTs agglomeration effects on Young's modulus, then utilized the non-linear von Kármán strain–displacement approximation to obtain the nonlinear strain of the continuous system via Euler-Bernoulli beam theory. At [17] Huang, X.-H et al. proposed an analytical model based on CLT to predict thermal expansion coefficients. By means of the Timoshenko beam model in [18], both the critical buckling resistance and post-buckling path of the multilayered beams generated from GPLR nanocomposites were examined. Dynamic modelling of the shafts was mainly carried out utilizing equivalent modulus beam theory (EMBT), where its formulation has been established based on the Timoshenko beam theory, as well as layer wise beam theory (LBT) formulations [19]. Also, an FGM model was proposed with taking into account, volume fraction to study thermal bending with FSDT [20]. On the other hand, nonlinear dynamic and energetic assessments of multi-layer plates having an auxetic core were conducted by employing the nonlinear expansion of the kinematic equations of Reddy's higher-order shear deformation theory (HSDT) in [21]. The main objective for developing these advanced components is their application in structures, such as beams [22-24], plates [25, 26], shells [27] or in thermal environments [26, 27]. These constructions are subjected to different applied loads such as deflection [28], buckling [28-31], bending [29, 30] and stress [32-34]. Moreover, the calculation of stresses due to temperature variation which cause deformations on a microscopic scale, leads to residual stresses which can be significant for different phenomena [35]. In this work, a micromechanical modeling approach is adopted to calculate the effective elastic constants of SWCNTs-reinforced nanocomposite laminate. This approach uses MATLAB toward an accurate simple model where carbon nanotubes are modeled as hollow cylindrical shells [36]. In addition, the effective wall thickness of SWCNTs is limited to the value of $t = 0.142 \text{ nm}$, following the recommendation reported in [37]. The effective elastic properties are then used in first-order shear deformation theory (FSDT) to calculate the in-plane normal and shear stress, with out-of-plane shear stress distributions of the laminate, which is an essential step in evaluating the strength and stability of structures. By incorporating the effects of transverse shear deformation, the model enables more accurate stress predictions in medium-thickness laminates, where CLT may be insufficient. Also, the study of thermal constraints will not allow to harm the composite structures because of the difference between the thermal expansion coefficients of the matrix and the reinforcement. This multiscale framework thus provides a consistent and comprehensive representation of the thermo-mechanical behavior of CNTRC laminates, linking microscale interactions and macroscale structural response.

2. Elementary Equations

2.1. Mechanical Moments and Bending Stresses

The equations of a symmetric laminated beam are provided by [38] in order to determine the stress behavior of a laminated beam composed of carbon nanotube-based nanocomposite.

$$\begin{bmatrix} M_x \\ M_y \\ M_{xy} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{16} \\ D_{12} & D_{22} & D_{26} \\ D_{16} & D_{26} & D_{66} \end{bmatrix} \begin{bmatrix} k_x \\ k_y \\ k_{xy} \end{bmatrix} \quad (1)$$

k_x , k_y and k_{xy} correspond to the curvatures, the flexural stiffnesses are $D_{ij=1,2,6}$ and the flexural moments are M_x , M_y , and M_{xy} while Δ is the matrix $[D_{ij}]$ determinant and D_{11}^* is an element of the inverse matrix.

$$D_{11}^* = \frac{1}{\Delta} (D_{22}D_{66} - D_{26}^2) \quad (2)$$

$$\Delta = D_{11}D_{22}D_{66} + 2D_{12}D_{16}D_{26} - D_{11}D_{26}^2 - D_{22}D_{16}^2 - D_{66}D_{12}^2$$

The bending occurs along the x axis here, where the bending and shear moments resolve to zero respectively

$$M_y = 0 \text{ and } M_{xy} = 0 \quad (3)$$

Then the deflection will only be a function of x

$$k_x = -\frac{d^2w_0}{dx^2} = D_{11}^*M_x \quad (4)$$

Since the $\frac{L}{b}$ ratio is high enough, the equation (4) can be expressed as follows:

$$\frac{d\phi_x}{dx} = \frac{d^2w_0}{dx^2} = -\frac{M}{E_x I} \quad (5)$$

Where M is the bending moment, I is the quadratic moment of the beam's cross-section with respect to the (x, y) plane, E_x is the flexural modulus of the beam and are written as:

$$E_x = \frac{12}{h^3 D_{11}^*}, \quad I = I_{xy} = \frac{bh^3}{12}, \quad M = bM_x \quad (6)$$

The stresses components within the k^{th} layer can be expressed as

$$\sigma_{xx}^k = z a_{xx}^k \frac{M}{I} \text{ where } a_{xx}^k = \frac{h^3}{12} \sum_{j=1,2,6} Q_{1j}^k D_{1j}^* \quad (7)$$

$$\sigma_{yy}^k = z a_{yy}^k \frac{M}{I} \text{ where } a_{yy}^k = \frac{h^3}{12} \sum_{j=1,2,6} Q_{2j}^k D_{1j}^* \quad (8)$$

$$\sigma_{xy}^k = z a_{xy}^k \frac{M}{I} \text{ where } a_{xy}^k = \frac{h^3}{12} \sum_{j=1,2,6} Q_{6j}^k D_{1j}^* \quad (9)$$

While $L = 10mm$, $b = 1mm$ and $h = 1mm$ denote the length, width and the height of the laminated beam respectively and Q_{ij} represents the rigidity coefficients. The transverse shear stress is calculated by

$$\sigma_{xz}^k = -a_{xx}^k \tau_0 \left[4 \left(\frac{z}{h} \right)^2 + d_k \right] \quad (10)$$

Such that, d_k is a constant which can be found as a first step by making σ_{xz}^k zero both the high and the below surfaces of the beam and guaranteeing that the lamina interfaces respect the continuity used. The maximum shear τ_0 is obtained at $z = 0$ and provided by

$$\tau_0 = \frac{3 \left(\frac{dM}{dx} \right)}{2bh} \quad (11)$$

Assuming a load P who occurs on a simply supported beam at its midpoint, the maximum bending stress in the k^{th} layer of the symmetric laminated beam can be calculated by

$$\sigma_{xx}^k = -2 a_{xx}^k \sigma_0 \frac{z}{h} \quad (12)$$

The maximum tensile stress σ_0 which is found at $\left(z = -\frac{h}{2} \right)$ and the maximum shear stress which is given at $z = 0$ are respectively expressed to normalize the stress results

$$\sigma_0 = \frac{3PL}{2bh^2}, \tau_0 = -\frac{3P}{4bh} \quad (13)$$

The elastic constant values are provided from [39] by

$$\begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{23} \\ \sigma_{13} \\ \sigma_{12} \end{bmatrix} = \begin{bmatrix} n & l & l & 0 & 0 & 0 \\ l & k+m & k-m & 0 & 0 & 0 \\ l & k-m & k+m & 0 & 0 & 0 \\ 0 & 0 & 0 & 2m & 0 & 0 \\ 0 & 0 & 0 & 0 & 2p & 0 \\ 0 & 0 & 0 & 0 & 0 & 2p \end{bmatrix} \begin{bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{23} \\ \varepsilon_{13} \\ \varepsilon_{12} \end{bmatrix} \quad (14)$$

According to [36], the Hill's elastic moduli are the constants k, l, m, n and p respectively represent, the transverse plane-strain bulk modulus, the transverse cross modulus, the transverse shear modulus, the axial tensile modulus and the axial shear modulus, which are calculated using the MT method as follows

$$k = \frac{E_m \{E_m c_m + 2k_r(1 + v_m)[1 + c_r(1 - 2v_m)]\}}{2(1 + v_m)[E_m(1 + c_r - 2v_m) + 2c_m k_r(1 - v_m - 2v_m^2)]} \quad (15)$$

$$l = \frac{E_m \{c_m v_m [E_m + 2k_r(1 + v_m)] + 2c_r l_r(1 - v_m^2)\}}{(1 + v_m)[2c_m k_r(1 - v_m - 2v_m^2) + E_m(1 + c_r - 2v_m)]} \quad (16)$$

$$m = \frac{E_m [E_m c_m + 2m_r(1 + v_m)(3 + c_r - 4v_m)]}{2(1 + v_m)\{E_m [c_m + 4c_r(1 - v_m)] + 2c_m m_r(3 - v_m - 4v_m^2)\}} \quad (17)$$

$$n = \frac{E_m^2 c_m (1 + c_r - c_m v_m) + 2c_m c_r (k_r n_r - l_r^2)(1 + v_m)^2 (1 - 2v_m)}{(1 + v_m)\{2c_m k_r(1 - v_m - 2v_m^2) + E_m(1 + c_r - 2v_m)\}} + \frac{E_m [2c_m^2 k_r(1 - v_m) + c_r n_r(1 - 2v_m + c_r) + 4c_m c_r l_r v_m]}{2c_m k_r(1 - v_m - 2v_m^2) + E_m(1 + c_r - 2v_m)} \quad (18)$$

$$p = \frac{E_m [E_m c_m + 2(1 + c_r)p_r(1 + v_m)]}{2(1 + v_m)[E_m(1 + c_r) + 2c_m p_r(1 + v_m)]} \quad (19)$$

$$E_L = n - \frac{l^2}{k}, E_T = \frac{4m(kn - l^2)}{kn - l^2 + mn} \quad (20)$$

$$G_{LT} = 2p \text{ and } v_{LT} = \frac{l}{2k} \quad (21)$$

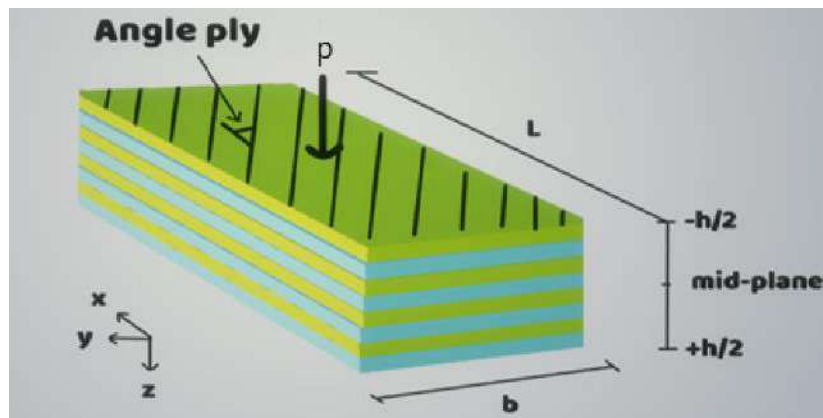


Fig. 1. Geometry of nanocomposite laminate beam in orthogonal axis

Due to the assumed small thickness of the plies, plane stress conditions are assumed in each ply, because of the assumed plane stress conditions, only four elasticity parameters E_L, E_T, G_{LT}, v_{LT} , which are related to the local axes of orthotropy, are necessary to define the elasticity matrix of the material [40]. Fig.1 displays more details of the laminate beam in an orthogonal axis.

2.2. Thermal Effects

The symmetric laminates are a good fit for the idea of determining its coefficients of thermal expansion since the coupling stiffness matrix $[B] = 0$ and no bending happens under thermal stresses. Because thermal stresses in this kind of laminate produce zero resultant forces, the n-ply laminate is introduced by [41] as follows

$$[N^T] = \begin{bmatrix} N_x^T \\ N_y^T \\ N_{xy}^T \end{bmatrix} = \Delta T \sum_{k=1}^n \begin{bmatrix} \overline{Q_{11}} & \overline{Q_{12}} & \overline{Q_{16}} \\ \overline{Q_{12}} & \overline{Q_{22}} & \overline{Q_{26}} \\ \overline{Q_{16}} & \overline{Q_{26}} & \overline{Q_{66}} \end{bmatrix}_k \begin{bmatrix} \alpha_x \\ \alpha_y \\ \alpha_{xy} \end{bmatrix}_k (h_k - h_{k-1}) \quad (22)$$

While $[A_{ij}]$ is the extensional matrix, $[N^T]$ is the resultants thermal force and moments (thermal moments $[M^T] = 0$ under uniform temperature variation at symmetric laminates) which is given by

$$\begin{bmatrix} N_x^T \\ N_y^T \\ N_{xy}^T \end{bmatrix} = \begin{bmatrix} A_{11} & A_{12} & A_{16} \\ A_{12} & A_{22} & A_{26} \\ A_{16} & A_{26} & A_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{bmatrix} \quad (23)$$

Using the thermoelastic extremum principle, the laminate thermal expansion coefficients are developed by

$$\alpha_1 = \frac{1}{E_L} (\alpha_{cnt} E_{cnt} c_r + \alpha_m E_m c_m), \alpha_2 = (1 + \nu_{cnt}) \alpha_{cnt} c_r + (1 + \nu_m) \alpha_m c_m - \alpha_1 \nu_{LT} \quad (24)$$

The transformation matrix $[T]$ must be added in order to determine the thermal expansion coefficients of each ply

$$\begin{bmatrix} \alpha_x \\ \alpha_y \\ \frac{\alpha_{xy}}{2} \end{bmatrix} = [T]^{-1} \begin{bmatrix} \alpha_1 \\ \alpha_2 \\ 0 \end{bmatrix} \quad (25)$$

The global strains in each laminate ply can be computed, the real strains in the laminate are represented by these global strains. However, mechanical strains are composed of the difference between the real strains and the free expansion strains. The k^{th} ply's mechanical strains are determined by

$$\begin{bmatrix} \varepsilon_x^M \\ \varepsilon_y^M \\ \gamma_{xy}^M \end{bmatrix} = \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} - \begin{bmatrix} \varepsilon_x^T \\ \varepsilon_y^T \\ \gamma_{xy}^T \end{bmatrix} \quad (26)$$

The symbol M denote mechanical, T denote thermal while ΔT denote difference temperature, where the thermal and global strains are introduced by

$$\begin{bmatrix} \varepsilon_x^T \\ \varepsilon_y^T \\ \gamma_{xy}^T \end{bmatrix}_k = \Delta T \begin{bmatrix} \alpha_x \\ \alpha_y \\ \alpha_{xy} \end{bmatrix}_k \quad (27)$$

The thermal stresses in any ply are then given by

$$\begin{bmatrix} \sigma_x^T \\ \sigma_y^T \\ \tau_{xy}^T \end{bmatrix}_k = - \begin{bmatrix} \overline{Q_{11}} & \overline{Q_{12}} & \overline{Q_{16}} \\ \overline{Q_{12}} & \overline{Q_{22}} & \overline{Q_{26}} \\ \overline{Q_{16}} & \overline{Q_{26}} & \overline{Q_{66}} \end{bmatrix}_k \begin{bmatrix} \varepsilon_x^T \\ \varepsilon_y^T \\ \gamma_{xy}^T \end{bmatrix}_k = -\overline{Q}_k \cdot \alpha_k^T \cdot \Delta T \quad (28)$$

By applying the transformed reduced stiffness matrix to the mechanical strain vector, the mechanical stress components in the k^{th} ply of the laminate can be calculated as follows:

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix}_k = \begin{bmatrix} \overline{Q_{11}} & \overline{Q_{12}} & \overline{Q_{16}} \\ \overline{Q_{12}} & \overline{Q_{22}} & \overline{Q_{26}} \\ \overline{Q_{16}} & \overline{Q_{26}} & \overline{Q_{66}} \end{bmatrix}_k \begin{bmatrix} \varepsilon_x^M \\ \varepsilon_y^M \\ \gamma_{xy}^M \end{bmatrix}_k \quad (29)$$

3. Validation

As shown at fig.2, the present work has a relative deviation of 1.84% at the normalized normal stress and satisfy the boundary conditions at the normalized out-of-plane shear stress as [32]. A polystyrene matrix is the involved polymer matrix with a Young's modulus of $E_m = 1.9 \text{ GPa}$ and Poisson's ratio as $\nu^m = 0.3$, the diameters of the nanotubes are taken as $d = 0.2 \text{ nm}$, the wall thickness as $t = 0.142 \text{ nm}$, then, $k_r = 30 \text{ GPa}$, $l_r = 10 \text{ GPa}$, $m_r = p_r = 1 \text{ GPa}$, $n_r = 450 \text{ GPa}$ as Hill's elastic constants for a SWCNTs (10,10) with a volume fraction of $c_r = 10\%$.

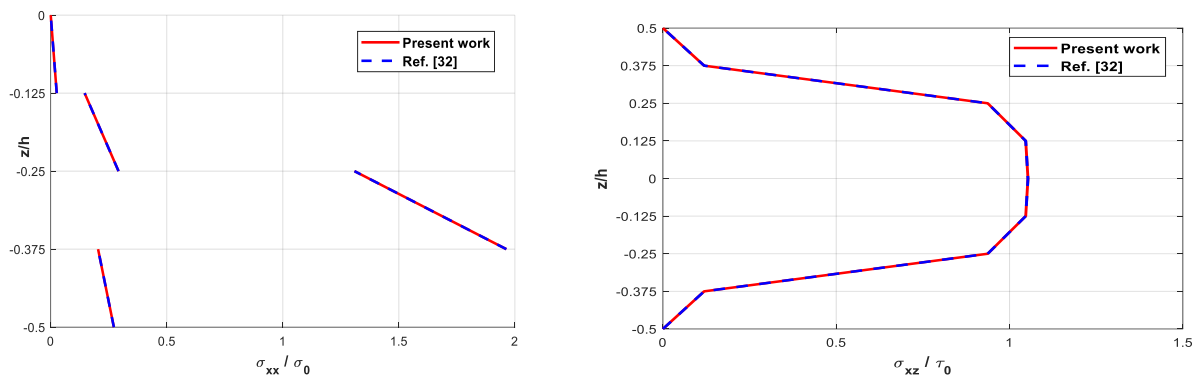


Fig. 2. A comparison of dimensionless normal and shear stresses alongside the normalized thickness of the laminate for a stacking sequence of $[45^\circ/0^\circ/-45^\circ/90^\circ]_s$ for present work and [32]

4. Results and Discussion

It is important to note that, in practice, CNTs concentrations do not usually exceed 5~10% [42]. The reason is related to the fact that obtaining uniform filler dispersions at high concentrations is an intricate task due to the tendency of CNTs to agglomerate in bundles. For the CNTRC with various stacking sequences, the numerical calculations use the engineering constants that were taken from [43] to table 1. using the micromechanical equations that were mentioned above.

The variation of the normalized in-plane stresses (longitudinal, transverse and shear) by the maximum tensile stress with respect to $\frac{z}{h}$ is introduced in Fig. 3. A symmetric laminate with a stacking sequence of $[45^\circ/0^\circ/-45^\circ/90^\circ]_s$ is investigated. Where the composite contains the same polymer matrix and is reinforced with various materials, each with a volume fraction of $c_r = 10\%$. When all materials have the same geometry and are undergoing to the same external load P , a lower stress value doesn't necessarily mean that the ply is more resistant.

Table 1. Hill's elastic constants of constituent phases

	Graphene	Graphite	CNT
$k_r \text{ [GPa]}$	850	620	30
$l_r \text{ [GPa]}$	6.8	15	10
$n_r \text{ [GPa]}$	102000	36.5	450
$m_r \text{ [GPa]}$	369	440	1
$p_r \text{ [GPa]}$	102000	4	1

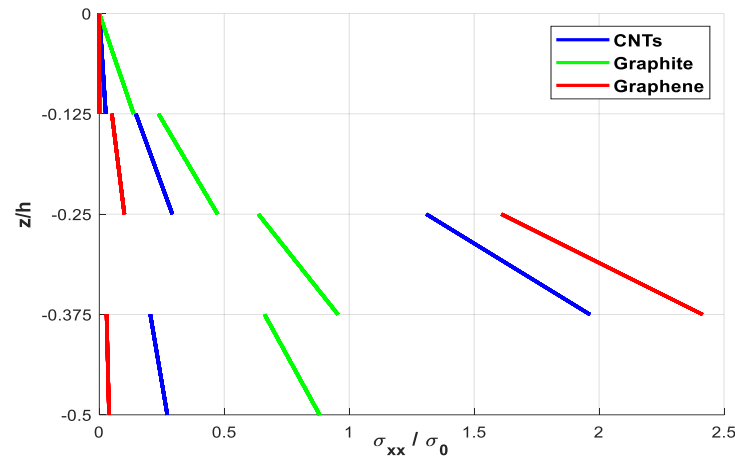


Fig. 3. Distribution of $[45^\circ/0^\circ/-45^\circ/90^\circ]_s$ for dimensionless normal stress across normalized thickness

In pure bending, CLT allows linearity in each layer of the laminate. In fig.3, the 0° layer, thanks to its high bending strength e.g., approximately 8 times greater than that of the 45° -fold for CNTs, generates a lot of stress and can withstand a greater number of the bending force, making it the most stressed layer in tension/compression. The upper surface $z > 0$ is subjected to maximum compression, and the lower surface $z < 0$ to maximum tension. It is in this layer that the failure criteria will most often be verified first, as poor interlaminar behavior may be suspected. The behavior of the reinforcements, through the normalized stress $\frac{\sigma_{xx}}{\sigma_0}$, varies significantly depending on the orientation of the layers in the laminate. In $\pm 45^\circ$ orientations, the highest stress value in these folds is less than 1, which reflects a partial contribution of the reinforcements to longitudinal bending via coupling mechanisms.

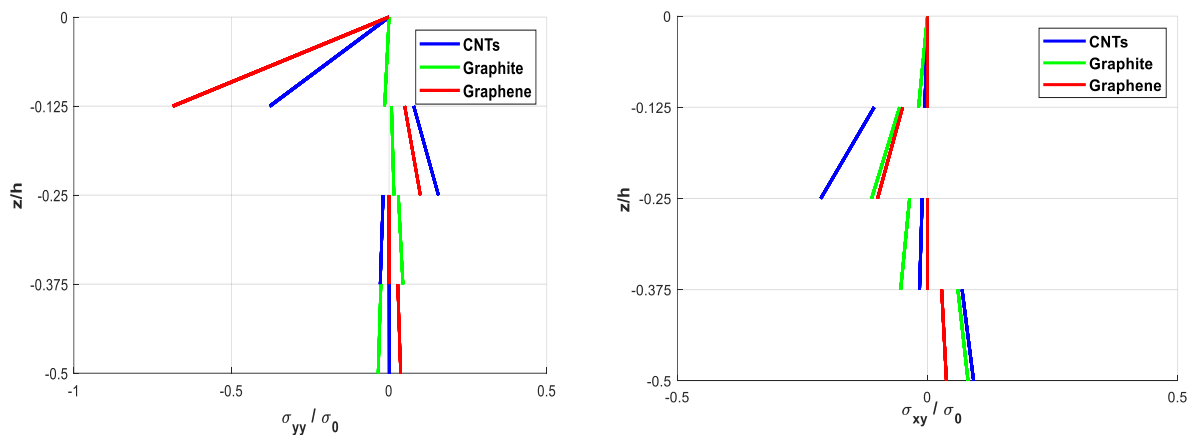


Fig. 4. Distribution of $[45^\circ/0^\circ/-45^\circ/90^\circ]_s$ for: a) dimensionless transverse stress across normalized thickness, b) dimensionless in-plane shear stress across normalized thickness

In contrast, in the 0° oriented layer, where the reinforcements are aligned with the bending direction, the stress peaks around 1, 2 and 2.5 for graphite, CNTs and graphene, respectively, indicating maximum reinforcement mobilization in longitudinal tension or compression. In the 90° layer, the stress drops back in value, the highest stress value of the three reinforcements does not even reach 0.25 in the bending direction. Since the main reinforcement is aligned in the x-plane for the 0° oriented layers and in the diagonal for the 45° layers. Then, there will be no bending in the y-plane. The main deformation is in ε_{xx} , so the σ_{yy} stresses will be closer to zero. As seen in fig. 4a, we can see also that these orientations do not mobilize the reinforcement in the y-axis. CNTs and graphene, being highly anisotropic, generate compressive transverse stresses in 90° folds, where the Poisson effect becomes dominant. The folds oriented at $\pm 45^\circ$ in fig. 4b, essentially generate the

normalized shear stress $\frac{\sigma_{xy}}{\sigma_0}$, in accordance with the symmetry of the stack and with opposite contributions in sign. The folds at 0° and 90° confirm the predominant role of the oblique layers in the transfer of σ_{xy} and induce stress values between 0.02 and -0.08.

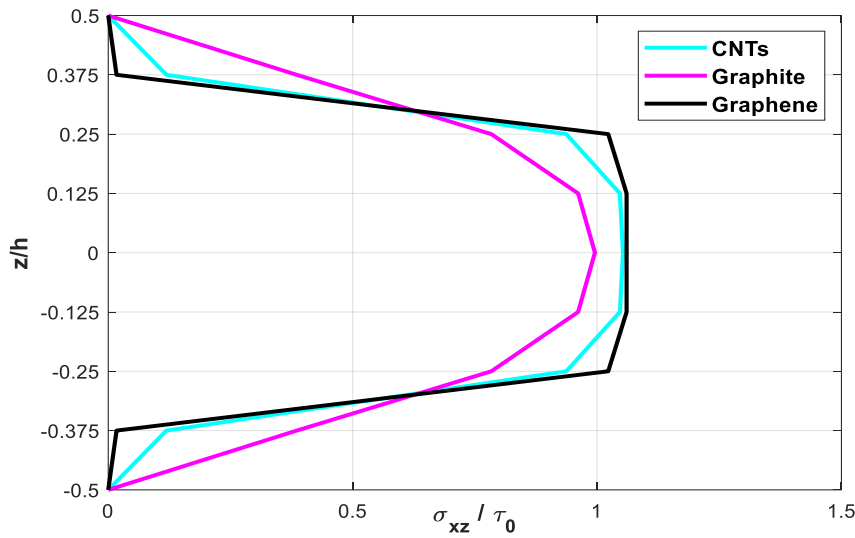


Fig. 5. Distribution of $[45^\circ/0^\circ/-45^\circ/90^\circ]_s$ for dimensionless out-of-plane shear stress across normalized thickness for various reinforcement

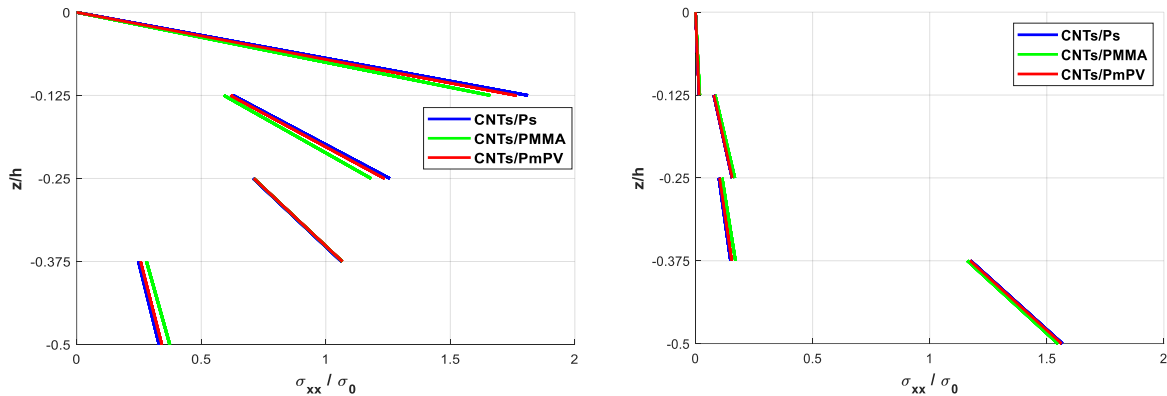


Fig. 6. Distribution of dimensionless normal stress across normalized thickness for a) a stacking sequence of $[90^\circ/45^\circ/-45^\circ/0^\circ]_s$, b) a stacking sequence of $[0^\circ/45^\circ/-45^\circ/90^\circ]_s$

The distribution of the out-of-plane shear stress normalized by the maximum shear stress with respect to $\frac{z}{h}$ is displayed in fig. 5 with the same properties as the laminate, which is displayed above in fig.3 and fig. 4. The coefficient $a_{xx}(k)$ acts as a local stiffness weight for each layer in Fig. 5, which shows a parabolic distribution of σ_{xz} in the thickness and vanishes at the edges. Graphene results show values ranging from 0 to 0.17 at the 45° bend and from 1.0229 to 1.0605 at the -45° bend, this behavior is expected since it is extremely stiff in in-plane tension, but weak in out-of-plane shear, especially in inclined orientations. CNTs behave better than graphene with out-of-plane transverse shear in $\pm 45^\circ$ bends and have the same behavior at 0° and 90° bends, they both have no contribution, while graphite is somewhat stable in all sequences. At the mid-plane, all reinforcements reach their maximum out-of-plane shear stresses.

At fig. 6 and fig. 7, a SWCNTs (10,10) volume fraction of $c_r = 10\%$ is mixed with different matrices on different stacking sequences: the polystyrene matrix, which is introduced above and two other matrices, which are polymeric, isotropic and possess identical Poisson's ratios $\nu^m = 0.34$, while the Young's modulus of the Poly(methyl methacrylate) (PMMA) matrix is $E_m = 2.5 \text{ GPa}$ and $E_m = 2.1 \text{ GPa}$ for the Poly(meta-phenylene vinylene-co-2,5-dioctoxy para-phenylene) (PmPV) matrix

[44]. The normal stresses σ_{xx} in each stacking sequence in fig. 6, are highly dependent on the ply orientation. Since the 0° layers are generally the ones which contribute the most to bending because the reinforcement there is aligned with the main load direction, these layers consequently experience higher normal stresses and significantly improve axial stiffness when they are located on the outside of the laminate, as in the $[0^\circ/45^\circ/-45^\circ/90^\circ]_s$ sequence. On the other hand, the centered position of the 0° layers in the $[90^\circ/45^\circ/-45^\circ/0^\circ]_s$ sequence limits their direct contribution to bending and favors a transfer of normal stresses to the inner layers.

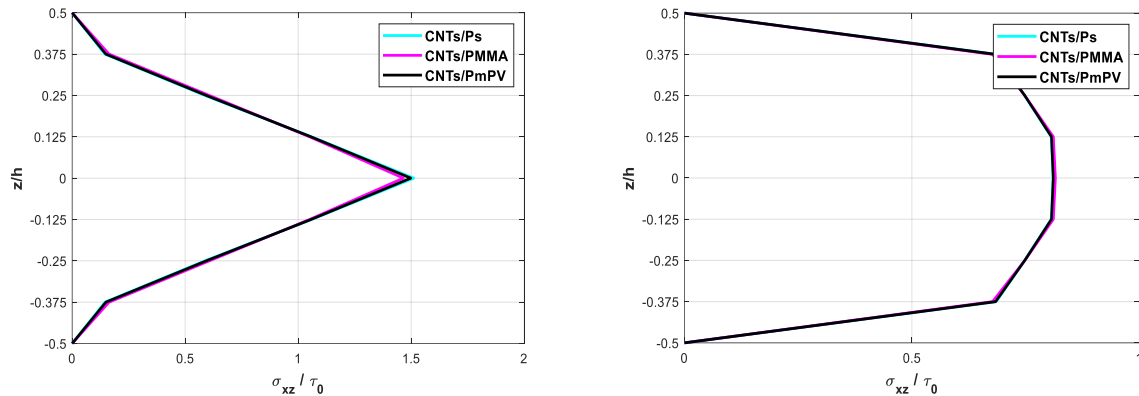


Fig. 7. Distribution of dimensionless out-of-plane shear stress across normalized thickness for :
a) a stacking sequence of $[90^\circ/45^\circ/-45^\circ/0^\circ]_s$, b) a stacking sequence of $[0^\circ/45^\circ/-45^\circ/90^\circ]_s$

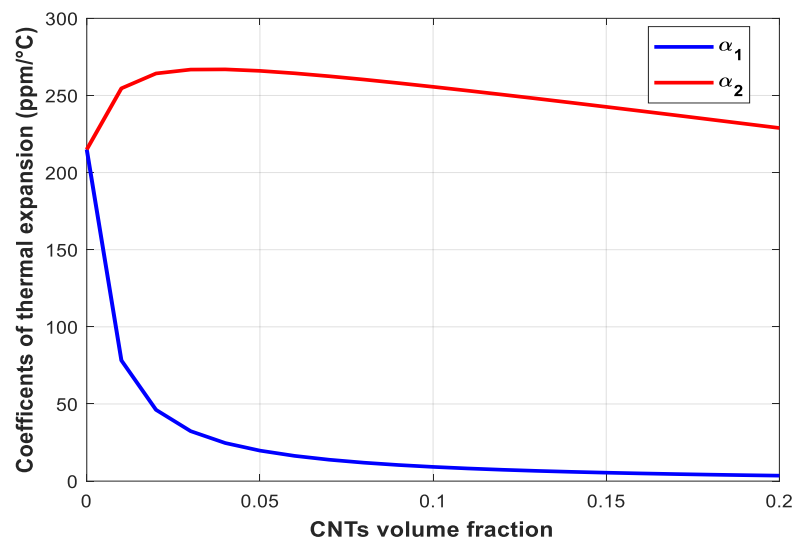


Fig. 8. Effect of CNTs volume fraction on longitudinal and transverse CTE

In Fig. 7, the matrix is highly stressed in 90° shear since the fibers do not support well the longitudinal loads. The results clearly indicate that the layers oriented at $\pm 45^\circ$, which, support the highest shear stresses, are less stressed in the stacking sequence of $[0^\circ/45^\circ/-45^\circ/90^\circ]_s$. Despite the difference in stiffness of the three matrices, PS being the softest, PMMA being the stiffest and PmPV being intermediate, there is almost no big difference in response to stresses. The reinforcement dominates the mechanical behavior and the matrix has little influence at out-of-plane shear stress.

The variation in length per unit of variation in temperature is known as the coefficient of thermal expansion (CTE). For a laminate with a stacking sequence of $[0^\circ/90^\circ]_s$, three coefficients of thermal expansion are defined: α_x in the x-direction, α_y in the y-direction, α_{xy} in the x-y-plane direction. Assuming that $\alpha_{cnt} = -0.9 \times 10^{-6} (K^{-1})$ at room temperature is the CTE for the reinforcing phase, SWCNTs (10, 10) [45]. While $\alpha_m = 215 \frac{ppm}{^\circ C}$ at $29.9^\circ C$ is the CTE of the PMMA matrix [46], its glass transition temperature T_g , which separates two regimes of the polymer behavior, justifies the

relation; $T < T_g$, the CTE of the matrix can be taken as $\alpha_m = 215 \frac{\text{ppm}}{^\circ\text{C}}$. Because α_m remains constant from 10°C to 80°C , the variation of temperature is taken as $\Delta T = 70^\circ\text{C}$. The longitudinal, transverse CTE, Young's modulus and Poisson's ratio of the laminate are respectively calculated as $\alpha_1 = 9.22 \times 10^{-6} \text{K}^{-1}$, $\alpha_2 = 2.556 \times 10^{-4} \text{K}^{-1}$, $E_L = 46.634 \text{ GPa}$ and $\nu_{LT} = 0.3883$, where Young's modulus and Poisson's ratio of CNTs are taken from [47] as $E_{cnt} = 600 \text{ GPa}$ and $\nu_{cnt} = 0.19$ while CNTs volume fraction is taken as $V_{cnt} = 10\%$. The transformed coefficients of thermal expansion are

$$\begin{bmatrix} \alpha_x \\ \alpha_y \\ \alpha_{xy} \end{bmatrix}_{0^\circ} = \begin{bmatrix} 9.22 \times 10^{-6} \\ 2.556 \times 10^{-4} \\ 0 \end{bmatrix} \text{K}^{-1}, \begin{bmatrix} \alpha_x \\ \alpha_y \\ \alpha_{xy} \end{bmatrix}_{90^\circ} = \begin{bmatrix} 2.556 \times 10^{-4} \\ 9.22 \times 10^{-6} \\ 0 \end{bmatrix} \text{K}^{-1} \quad (30)$$

The variation of both longitudinal and transverse CTE of the laminate as a function of CNTs volume fraction is illustrated in fig. 8. The longitudinal direction in the lamina, which is aligned with the orientation of the CNTs, shows a significant reduction in α_1 despite the matrix being highly thermally expansive. This behavior results from the negative CTE of the CNTs and their high axial stiffness, which allows them to have a significant impact in the effective response through a stiffness-weighted averaging. In contrast, α_2 is only slightly reduced, since the matrix governs the transverse properties due to the low volume fraction and limited transverse stiffness of the CNTs.

Table 2. Distribution of strains in a $[0^\circ / \overline{90^\circ}]_s$ laminate with respect to z under a temperature change of 70°C

Ply no.	ε_x	ε_x^T	ε_x^M	ε_y	ε_y^T	ε_y^M	$\gamma_{xy}, \gamma_{xy}^T, \gamma_{xy}^M$
1 (0°)		6.454×10^{-4}	0.0013		0.0179	-0.0142	0
2 (90°)	0.001946	0.0179	-0.016	0.003691	6.454×10^{-4}	0.003	0
3 (0°)		6.454×10^{-4}	0.0013		0.0179	-0.0142	0

Considering the thermal expansion coefficients and mechanical stiffness of each layer, the strains of each ply for a symmetric laminate with a stacking sequence of $[0^\circ / \overline{90^\circ}]_s$ are evaluated in tab. 2. The composite composed of SWCNTs (10,10) as a reinforcement and PMMA as a matrix. Since there are no mechanical loads, there is only a constant variation of temperature in all the laminate. There will be no thermal bending moments $[M]^T = 0$ and no curvatures $[k] = 0$. Then, through the thickness, the mid-plane strain remains constant and the same strain occurs in every lamina. Since, the CTEs are different in the fiber and transverse directions. Thermal stresses vary depending on the ply orientation, which allows for the calculation of mechanical deformations.

Table 3. Distribution of stresses in a $[0^\circ / \overline{90^\circ}]_s$ laminate with respect to z under a temperature change of 70°C

Ply no.	$\sigma_x [\text{MPa}]$	$\sigma_x^T [\text{MPa}]$	$\sigma_y [\text{MPa}]$	$\sigma_y^T [\text{MPa}]$	$\sigma_{xy}, \sigma_{xy}^T$
1 (0°)	45.8058	-49.728	-38.1546	-50.545	0
2 (90°)	-41.33	-50.545	123.85	-49.728	0
3 (0°)	45.8058	-49.728	-38.1546	-50.545	0

These thermal strains, denoted ε^T , are computed from the temperature change and the effective CTE of each layer and generate internal stresses even in the absence of external mechanical loads. The analysis of tab. 3. shows that even with a uniform-imposed deformation ε^0 , the mechanical stresses vary according to ε^T , which depends on the bend angle. The 0° and 90° bends are the most constrained bends, because the greater the difference between ε^0 and ε^T , the higher the mechanical stresses will be. The negative sign for the stresses indicates that there is compressive stress. Fig.9

shows the linear distribution of normal thermal stress due to a temperature variation of 70°; it is constant in the 0° bends and changes by approximately 1.64% in the 90° bend.

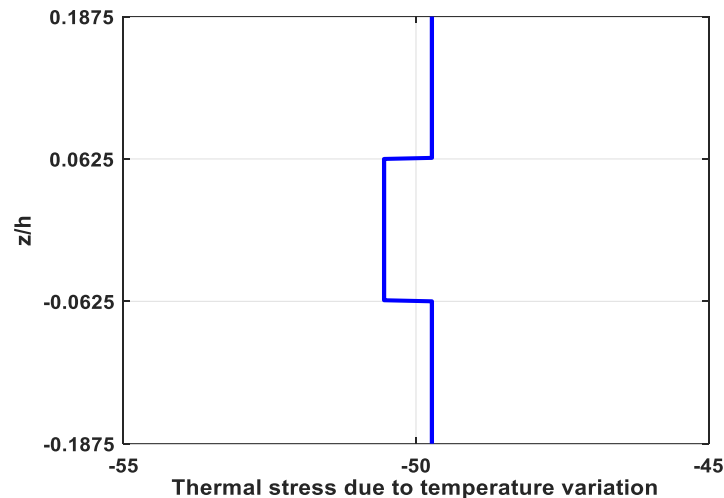


Fig. 9. Thermal stress σ_x^T distribution across laminate thickness for a stacking sequence of $[0^\circ / \overline{90}^\circ]_s$

5. Conclusions

To evaluate the elastic response of CNT-reinforced nanocomposite beams, a multiscale analytical model was developed using effective properties derived from the Mori-Tanaka homogenization method. With the help of MATLAB, the structural behavior of the beams was analyzed at the macroscopic scale, although its reinforcement occurs at the nanoscale, reflecting its applicability to real-scale engineering systems in many fields such as defense, aeronautics, automotive and civil engineering. Using a parabolic distribution across the normalized thickness, the out-of-plane transverse stress was calculated by FSDT theory, which takes into account the transverse shear effects neglected by CLT. The obtained results illustrate the impact of the reinforcement volume fraction and stacking sequences on the mechanical performance of the reinforced laminate beam. A moderate quantity of CNTs can improve the mechanical, thermal response of the structure. On the other hand, the arrangement of the layers strongly affects the stress distribution, while placing the 0° layers on the outside allows for better axial load support and consistent bending behavior. Also, the matrix plays a more important role than the fibers in sequences where the reinforcements are not aligned with the primary load direction. The results sheds light on the anisotropic response of reinforcements, where they contribute strongly to the normal stress in 0° bends, moderately at $\pm 45^\circ$ and almost without any contribution at 90°. The analysis of thermal strains and global stresses due to temperature variation in a symmetric laminate involves understanding how each ply reacts to difference of temperature despite an overall geometry without curvature. Moreover, increasing the volume fraction of CNTs in a polymer matrix will lead to a marked reduction in the longitudinal thermal expansion coefficient α_1 , making the composite more thermally stable in the principal direction. While results show the poor thermal behavior at the transverse direction, there is not an important change in the transversal thermal expansion coefficient α_2 . Finally, The MATLAB implementation created, will serve as a flexible framework for upcoming investigations on non-symmetric laminates, higher-order shear deformation theories, and various composite geometries.

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References

- [1] Iijima S. Helical microtubes of graphitic carbon. *Nature*. 1991;354:56-58. <https://doi.org/10.1038/354056a0>
- [2] Ince Yardimci A, et al. SEM images of carbon nanotubes (CNTs): effect of voltage and spot size on image resolutions. *Research & Design*. 2024;1(1):43-51. <https://doi.org/10.17515/rede2024-004en1116rs>
- [3] Sahai RS, et al. Study on performance of multiwall carbon nanotubes and functionalized multiwall carbon nanotubes/ poly aryl ether ketone polymer composite gears. *Res Eng Struct Mater*. 2025;11(1):273-285. <http://dx.doi.org/10.17515/resm2024.217na0324rs>
- [4] Pak SY, et al. Micro-Macroscopic coupled modeling for the prediction of synergistic improvement on the thermal conductivity of boron nitride and multi-walled carbon nanotube reinforced composites. *Compos Part A*. 2021;148:106474. <https://doi.org/10.1016/j.compositesa.2021.106474>
- [5] Mesfin KK, Ananda BA, Tilak R. Three-phase modelling and characterization of elastic behavior of MWCNT reinforced GFRP composites: A combined numerical and experimental study. *Mater Today Proc*. 2020;26:944-949. <https://doi.org/10.1016/j.matpr.2020.01.152>
- [6] Seidel G, Lagoudas D. Micromechanical analysis of the effective elastic properties of carbon nanotube reinforced composites. *Mech Mater*. 2006;38(8-10):884-907. <https://doi.org/10.1016/j.mechmat.2005.06.029>
- [7] Li X, et al. Reinforcing mechanisms of single-walled carbon nanotube-reinforced polymer composites. *J Nanosci Nanotechnol*. 2007;7(7):2309-2317. <https://doi.org/10.1166/jnn.2007.410>
- [8] He Z, et al. Insight into the mechanical properties and thermal expansion behavior of epoxy nanocomposites reinforced with multi-walled carbon nanotube solvent-free nanofluids. *Polym Compos*. 2024;45(5):4401-4411. <https://doi.org/10.1002/pc.28069>
- [9] Heshmati M, Yas MH. Free vibration analysis of functionally graded CNT-reinforced nanocomposite beam using Eshelby-Mori-Tanaka approach. *J Mech Sci Technol*. 2013;27(11):3403-3408. <https://doi.org/10.1007/s12206-013-0862-8>
- [10] Skocek J, Zeman J, Sejnoha M. Effective properties of textile composite: application of the Mori-Tanaka method. *Model Simul Mater Sci Eng*. 2008;16(8):085002-15. <https://doi.org/10.1088/0965-0393/16/8/085002>
- [11] Fedotov A. A numerical-analytical model of elasto-plastic deformation of matrix composites. *Acta Mech*. 2024;235:3633-3642. <https://doi.org/10.1007/s00707-024-03918-x>
- [12] Mori T, Tanaka K. Average stress in matrix and average elastic energy of materials with misfitting inclusions. *Acta Metall*. 1973;21:571-574. [https://doi.org/10.1016/0001-6160\(73\)90064-3](https://doi.org/10.1016/0001-6160(73)90064-3)
- [13] Ghorbanpour Arani A, Maghamikia S, Mohammadimehr M, Arefmanesh A. Buckling analysis of laminated composite rectangular plates reinforced by SWCNTs using analytical and finite element methods. *J Mech Sci Technol*. 2011;25(3):809-820. <https://doi.org/10.1007/s12206-011-0127-3>
- [14] Tahoun V, Naei MH. Using Eshelby-Mori-Tanaka scheme for 3D free vibration analysis of sandwich curved panels with functionally graded nanocomposite face sheets and finite length. *Polym Compos*. 2016. <https://doi.org/10.1002/pc.23929>
- [15] Heshmati M, Yas MH. Free vibration analysis of functionally graded CNT-reinforced nanocomposite beam using Eshelby-Mori-Tanaka approach. *J Mech Sci Technol*. 2013;27:3403-3408. <https://doi.org/10.1007/s12206-013-0862-8>
- [16] Dabbagh A, Ebrahimi F. Postbuckling analysis of meta-nanocomposite beams by considering the CNT's agglomeration. *Eur Phys J Plus*. 2021;136:1168. <https://doi.org/10.1140/epjp/s13360-021-02160-x>
- [17] Huang XH, et al. A comparative analysis of thermo mechanical behavior of CNT reinforced composite plates: Capturing the effects of thermal shrinkage. *Case Stud Therm Eng*. 2022;38. <https://doi.org/10.1016/j.csite.2022.102347>
- [18] Yang J, Wu H, Kitipornchai S. Buckling and postbuckling of functionally graded multilayer graphene platelet-reinforced composite beams. *Compos Struct*. 2017;161:111-118. <https://doi.org/10.1016/j.compstruct.2016.11.048>
- [19] Udatha P, Sekhar AS, Velmurugan R. Eigen value analysis of composite hollow shafts using modified EMBT formulation considering the shear deformation along the thickness direction. *DefTechnol*. 2023;28:1-12. <https://doi.org/10.1016/j.dt.2022.12.020>
- [20] Li J, et al. Thermomechanical bending of functionally graded carbon nanotubes reinforced composite plate by mechless method. *Polym Compos*. 2024;45(14):13063-13075. <https://doi.org/10.1002/pc.28686>
- [21] Zhu X, Zhang J, Zhang W, Chen J. Vibration frequencies and energies of an auxetic honeycomb sandwich plate. *Mech Adv Mater Struct*. 2019;26(23):1951-1957. <https://doi.org/10.1080/15376494.2018.1455933>

- [22] Vo-Duy T, Ho-Huu V, Nguyen-Thoi V. Free vibration analysis of laminated FG-CNT reinforced composite beams using finite element method. *Front Struct Civ Eng*. 2019;13:324-336. <https://doi.org/10.1007/s11709-018-0466-6>
- [23] Shafiei H, Setoodeh AR. An analytical study on the nonlinear forced vibration of functionally graded carbon nanotube-reinforced composite beams on nonlinear viscoelastic foundation. *Arch Mech*. 2020;72:81-107.
- [24] Civalek Ö, Akbas S, Akgöz B, Dastjerdi S. Forced vibration analysis of composite beams reinforced by carbon nanotubes. *Nanomaterials*. 2021;11:571. <https://doi.org/10.3390/nano11030571>
- [25] Wu CP, Chang SK. Stability of carbon nanotube-reinforced composite plates with surface-bonded piezoelectric layers and under bi-axial compression. *Compos Struct*. 2014;111:587-601. <https://doi.org/10.1016/j.compstruct.2014.01.040>
- [26] Ebrahimi F, Nouraei M, Dabbagh A, Rabczuk T. Thermal buckling analysis of embedded grapheneoxide powder-reinforced nanocomposite plates. *Adv Nano Res*. 2019;7(5):293-310.
- [27] Shen H. Torsional postbuckling of nanotube-reinforced composite cylindrical shells in thermal environments. *Compos Struct*. 2014;116:477-488. <https://doi.org/10.1016/j.compstruct.2014.05.039>
- [28] Akbas SD. Post-buckling analysis of a fiber reinforced composite beam with crack. *Eng Fract Mech*. 2019;212:70-80. <https://doi.org/10.1016/j.engfracmech.2019.03.007>
- [29] Chitour M, et al. A Novel high order theory for static bending of functionally graded (FG) beams subjected to various mechanical loads. *Res Eng Struct Mater*. 2024;10(4):1523-1539. <https://doi.org/10.17515/resm2024.141me0104rs>
- [30] Polit O, Anant C, Anirudh B, Ganapathi M. Functionally graded graphene reinforced porous nanocomposite curved beams: Bending and elastic stability using a higher order model with thickness stretch effect. *Compos B Eng*. 2019;166:310-327. <https://doi.org/10.1016/j.compositesb.2018.11.074>
- [31] Amani MA, Ebrahimi F, Dabbagh A, Rastgoo A, Nasiri MM. A machine learning-based model for the estimation of the temperature-dependent moduli of graphene oxide reinforced nanocomposites and its application in a thermally affected buckling analysis. *Eng Comput*. 2021;37(3):2245-2255. <https://doi.org/10.1007/s00366-020-00945-9>
- [32] Wuite J, Adali S. Deflection and stress behaviour of nanocomposite reinforced beams using a multiscale analysis. *Compos Struct*. 2005;71:388-396. <https://doi.org/10.1016/j.compstruct.2005.09.011>
- [33] Akmal J, et al. Development and validation of an accurate analytical solution for hoop stress distribution in cylindrical functionally graded materials. *Res Eng Struct Mater*. 2025;11(5):2521-2535. <https://doi.org/10.17515/resm2025-777an0324rs>
- [34] Patton A, Faroughi S, Reali A. Efficient equilibrium-based stress recovery for isogeometric laminated Euler-bernoulli curved beams. *Compos Struct*. 2024;118374. <https://doi.org/10.1016/j.compstruct.2024.118374>
- [35] Shokrieh MM. Residual stresses in composite materials. Woodhead Publishing Series in Composites Science and Engineering; 2014. No. 48. <https://doi.org/10.1533/9780857098597.1.173>
- [36] Khaldi A, Tekili S, Khadri Y. Vibration of nanocomposite beams reinforced by SWCNTs under the action of harmonic load. *Res Eng Struct Mater*. 2025. <http://dx.doi.org/10.17515/resm2025-1013ma0706rs>
- [37] Vodenitcharova T, Zhang LC. Effective wall thickness of a single-walled carbon nanotube. *Phys Rev B*. 2003;68:165401. <https://doi.org/10.1103/PhysRevB.68.165401>
- [38] Berthelot JM. Composite materials, mechanical behaviour and structural analysis. New York: Springer-Verlag; 1999. <https://doi.org/10.1007/978-1-4612-0527-2>
- [39] Popov VN, Doren VE, Balkanski M. Elastic properties of crystals of single-walled carbon nanotubes. *Solid State Commun*. 2000;114:359-399. [https://doi.org/10.1016/S0038-1098\(00\)00070-3](https://doi.org/10.1016/S0038-1098(00)00070-3)
- [40] Altenbach H, Öchsner A. Encyclopedia of Continuum Mechanics. Springer Nature; 2019. <https://doi.org/10.1007/978-3-662-53605-6>
- [41] Estrada H, Lee L. Mechanics of composite materials. In: The international handbook of FRP composites in civil engineering. 2013. p. 51-78.
- [42] Coleman JN, Khan U, Blau WJ, Gun'ko YK. Small but strong: A review of the mechanical properties of carbon nanotube-polymer composites. *Carbon*. 2006;44:1624-1652. <https://doi.org/10.1016/j.carbon.2006.02.038>
- [43] G-Macias E, R-Tembleque L, Saez A. Bending and free vibration analysis of functionally graded graphene vs. carbon nanotube reinforced composite plates. *Compos Struct*. 2017.
- [44] Garcia-Macias E, Guzmán CF, Saavedra Flores EI, Castro-Triguero R. Multiscale modeling of the elastic moduli of CNT-reinforced polymers and fitting of efficiency parameters for the use of the extended rule-of-mixtures. *Compos Part B Eng*. 2019;114-131. <https://doi.org/10.1016/j.compositesb.2018.09.057>
- [45] Shirasu K, Yamamoto G, Hashida T. Application of aligned carbon nanotube-reinforced polymer composite to electrothermal actuator. Carbon nanotubes- current progress of their polymer composites; 2016. <https://doi.org/10.5772/62509>

- [46] Zhi CY, Bando Y, Wang WL, Tang CC, Kuwahara H, Golberg D. Mechanical and thermal properties of polymethyl Methacrylate-BN nanotube composites. J Nanomater. 2008. <https://doi.org/10.1155/2008/642036>
- [47] Kamarian S, Shakeri M, Yas M, et al. Free vibration of functionally graded nanocomposite sandwich beams resting on Pasternak foundation by considering the agglomeration effect of CNTs. J Sandw Struct Mater. 2015;17:632-665. <https://doi.org/10.1177/1099636215590280>

Profitability-aware machine learning for forecasting fabrication durations in PEB projects

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In Pre-Engineered Building (PEB) production, forecasting production timelines accurately is crucial, especially as tasks like cutting, welding, and painting vary depending on structural complexity and job size. Many conventional scheduling methods fall short when dealing with the dynamic, multi-stage nature of fabrication workflows. This study introduces a profitability-aware machine learning-based approach that predicts both overall and stage level fabrication durations using Random Forest regression. The model is trained on part-level production data, with features including project tonnage, number of parts, and statistical descriptors of task durations. To improve learning performance, projects are grouped into six distinct complexity classes based on fabrication characteristics. A profitability-oriented evaluation is also proposed, which labels prediction outcomes as Profitable, Acceptable, or Excess depending on how closely they align with business targets. The model is tested on real data from 34 completed PEB projects, showing clear improvements over conventional estimation methods. Results demonstrate that stage-level predictions outperform start-date-based forecasts, ensuring profitability-aligned outcomes even in high-complexity projects. The proposed framework helps bridge the gap between technical forecasting accuracy and real-world production goals, offering a practical solution for smarter planning in steel fabrication environments.

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1. Introduction

The construction industry continues to adopt industrialized production systems such as Pre-Engineered Buildings (PEBs) to meet rising demands for speed, efficiency, and cost control. Unlike conventional structures, PEBs rely heavily on steel-based components fabricated off-site. These components pass through sequential, tightly coupled stages including cutting, welding, drilling, and surface treatment. While this modularization accelerates installation, it introduces new challenges. Stage-wise fabrication durations are difficult to predict due to component variability, production concurrency, and task interdependencies.

Accurate forecasting of fabrication timelines is vital to ensure just-in-time project delivery, optimize shop-floor operations, and maintain profitability. Traditional estimation methods are often based on historical averages or parametric rules. These methods lack the granularity and adaptability failing to capture the dynamic fabrication environments. In response, researchers have increasingly turned to machine learning to address this complexity [1]. For instance, A demand forecasting model for steel manufacturing was introduced using ML techniques [2]. Stage-wise

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prediction in fabrication was shown to improve accuracy by isolating each task [3]. More recently, an ML-driven approach for material quantity estimation in structural fabrication was proposed [4]. Both highlighted the potential of data-driven models to outperform rule-based systems under uncertainty [3,4]. Beyond material and demand forecasting, several studies have explored ML applications in activity-level duration prediction. Ensemble-based methods such as Random Forest have been applied to capture non-linear dependencies in production workflows [5,6,7].

Despite these advances, limitations remain. First, they treat fabrication as a monolithic process, forecasting total durations without capturing variability across individual stages. Second, they often fail to incorporate structural and project-level complexity factors which can significantly impact process performance. Recent efforts to overcome these limitations include the use of classification-aware preprocessing [8,9], feature engineering using statistical descriptors [10], and profitability-aligned evaluation frameworks [11,12]. These approaches shape the methodology of the present study.

This research proposes a machine learning based predictive framework that forecasts both overall and stage-specific fabrication durations in PEB. Using Random Forest regression and statistical features such as mean, median, and mode from part-level fabrication records, the model is trained on data from 34 real-world projects. Projects are stratified into six mutually exclusive classes (P1C1–P2C3) based on their structural complexity and production scale, improving model specificity and interpretability. Forecast outputs are further evaluated using a profitability-aware classification system that categorizes predictions into “Profitable,” “Acceptable,” or “Excess” based on deviation thresholds tied to business performance. The key contributions of this study are as follows:

- Profitability-aware forecasting framework: We propose a machine learning framework tailored to Pre-Engineered Building (PEB) fabrication, which integrates profitability considerations into predictive modeling — an aspect rarely addressed in prior work.
- Activity-level prediction strategy: Unlike conventional project-level or start-date-based forecasting, this study introduces and validates an activity-specific modeling approach that reduces error propagation and enhances stability across fabrication stages.
- Empirical validation across complexity tiers: Using a dataset of 34 projects stratified into low, medium, and high complexity classes, the framework is empirically tested, demonstrating robust accuracy (low RMSE, narrow confidence intervals) and superior profitability classification compared to baseline models.
- Decision-making support in industrial environments: The framework provides actionable insights for PEB managers by linking predictive accuracy to profitability outcomes, enabling informed scheduling, resource allocation, and financial planning.

Together, these contributions advance both the academic understanding of machine learning in construction manufacturing and the practical capabilities of PEB firms to achieve profitability-aware project control.

2. Literature Review

2.1. Machine Learning Applications in Construction and Fabrication Forecasting

Machine learning (ML) has emerged as a transformative approach in construction and industrial fabrication, offering data-driven alternatives to conventional scheduling and forecasting techniques [13]. Early studies in this domain developed ML-based productivity models for construction crews, demonstrating adaptability under site variability [14]. Computational intelligence was also applied to inventory management, where improvements in material flow were achieved within construction supply chains [15].

In steel and modular fabrication, demand forecasting has become a crucial use case for ML. A tailored model was designed to forecast demand in steel production environments [2]. Regression-based learning was later applied to estimate material quantities in structural fabrication tasks [4]. Neural network approaches were also validated for raw material inventory control in steel plants, confirming their usefulness for shop-floor planning [16].

Recent studies in materials and construction domains further underscore the versatility of machine learning in predictive tasks. Advanced neural architectures and optimization techniques have been successfully employed beyond scheduling and fabrication. For instance, an SRS-optimized long short-term memory (LSTME) model achieved high accuracy in predicting the mechanical properties of friction stir processed Al-6061–Alumina composites, with R^2 values exceeding 0.90 across multiple material performance indicators [13]. Similarly, an artificial neural network (ANN)-based soft-computing model was developed to forecast the compressive strength of eco-friendly recycled aggregate concrete, demonstrating strong agreement between experimental and predicted outcomes [1]. These contributions highlight the broader applicability of ML in handling complex, nonlinear behaviors across engineering domains, thereby reinforcing the rationale for its adoption in forecasting multi-stage fabrication durations in PEB projects.

2.2. Activity-Level Forecasting and Multi-Stage Modeling

Traditional duration prediction models often struggle to adapt to the variability present in complex fabrication workflows. To address this, recent studies have proposed breaking projects into discrete stages or tasks. A predictive model was developed that independently forecasted fabrication flow by stage, improving overall forecast fidelity [3]. ML techniques were also applied to estimate production costs at the activity level, showing that disaggregation helps isolate uncertainty [17].

Data-driven timeline forecasting in modular housing was explored through ML applications that estimated process-specific durations across prefabricated units [18]. Multi-objective ML models for scheduling optimization were also introduced, reinforcing the effectiveness of activity-aware modeling [19]. A comprehensive review of hybrid predictive scheduling models in industrial construction further emphasized the value of integrating multiple algorithms across project stages [20]. Ensemble-based models for activity-level forecasting have shown superior accuracy in dynamic workflows. Support vector machines were applied for scheduling in engineering-procurement-construction (EPC) projects [21].

2.3 Feature Engineering and Model Architecture in Predictive Scheduling

Feature engineering plays a pivotal role in the success of ML-based forecasting models. Classification-aware preprocessing was incorporated to improve generalization in scheduling models [8]. Hybrid models that combined statistical features such as mean, median, and mode with learning algorithms were shown to enhance prediction accuracy [22]. The integration of IoT-based sensor data for real-time forecasting in fabrication environments was also explored, marking a shift toward dynamic model updating [23].

The comparison of ML architectures has been a subject of interest in the construction domain. Decision trees, support vector machines, and Random Forest models were evaluated for forecasting applications. Random Forest, among ensemble models, has proven effective in activity-level forecasting by capturing non-linear interactions and handling heterogeneous features, making it suitable for fabrication duration prediction [24,6,7]. Statistical descriptor-based feature sets, similar to those employed in the present study, were also tested and found to improve duration prediction accuracy [10].

2.4 Complexity-Aware Stratification and Classification-Driven Modeling

Given the heterogeneity in project types and part geometries in fabrication workflows, classification-aware approaches have gained traction. Component-type clustering was applied to isolate variance in project complexity, which improved learning performance [9]. A task-classification model was implemented that adjusted its prediction pipeline based on structural and geometric load [22]. These strategies directly inform the use of class-based segmentation in your work (e.g., P1C1 to P2C3).

Profitability-aware forecasting framework further advanced this concept by classifying predictions into zones based on expected value-added versus deviation risk [11]. This aligns with your study's implementation of three forecast-based profitability classes Profitable, Acceptable, and Excess which convert RMSE accuracy into financial insight.

2.5 Profitability-Driven Forecasting

Several studies have emphasized that forecasting precision must translate into operational or financial gains. Forecast deviation was linked to profitability erosion, with recommendations that error tolerance thresholds be embedded into scheduling systems [25]. Cost-sensitive ML models were developed to penalize over- or underestimation differently, introducing a concept later integrated into classification-based evaluation frameworks [12]. Random Forest models were also tested in high-variability scheduling environments, demonstrating that profitability-aware learning produced more practical predictions [26].

2.6 Practical ML Integration

Industrial implementation studies demonstrated the practical integration of ML in fabrication environments. Real-time dashboards were developed for monitoring fabrication progress [27]. ML was also embedded into production feedback loops to support adaptive decision-making [28]. Cost and lead-time improvements in a PEB production facility were further documented after the adoption of stage-level ML predictions, affirming the direct applicability of such methods in real-world steel fabrication environments [29].

3. Data Acquisition and Preparation

3.1 Data Source and Collection

The foundation of this study was based on real-time fabrication data collected from a Pre-Engineered Building (PEB) production facility. The structured data pipeline was designed to preserve the fidelity of production workflows while ensuring compatibility with machine learning (ML) algorithms. The primary data source was a centralized Google Sheets interface referred to as the PRED Sheet (Production Raw Entry Database). This live document was managed by the factory manager and is continuously updated to reflect real-time progress. It contains both static and dynamic fields. Static entries, such as job code (JC), member part mark, quantity (Q), and weight in Kgs (TW), are recorded once upon project initiation. Dynamic entries, including date-wise progress for each activity stage, are updated daily for every part mark involved in the manufacturing process.

Table 1. PRED Sheet – Production Raw Entry Database

JC	Part	Q	TW	C	S	BS	SW		FP
.	.	.	.	.	.	.	.		.
.	.	.	.	.	.	.	.		.
.	.	.	.	.	.	.	.		.
18	Rafter/Portal	1	255.41	17/2/22	17/2/22	20/2/22	22/2/22		10/5/22
18	Rafter/Portal	1	219.37	17/2/22	17/2/22	20/2/22	21/2/22		10/5/22
18	Rafter/Portal	1	309.63	20/2/22	20/2/22	21/2/22	22/2/22		10/5/22
18	Rafter/Portal	1	309.63	20/2/22	20/2/22	21/2/22	23/2/22		10/5/22
19	Plates	1	0.3	24/1/22	NA	NA	NA		27/1/22
19	Plates	1	0.3	24/1/22	NA	NA	NA		27/1/22
19	Plates	1	0.3	24/1/22	NA	NA	NA		27/1/22
19	Plates	1	0.3	24/1/22	NA	NA	NA		27/1/22
.	.	.	.	.	.	.	.		.
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.	.	.	.	.	.	.	.		.
.	.	.	.	.	.	.	.		.

Each fabricated component undergoes ten sequential production stages: Cutting (C), Splicing (S), Built-up Section (BS), SAW Welding (SW), Fit-up (FW), Welding (W), Drilling (D), Cleaning & Grinding (CG), Primer Application (P), and Finish Painting (FP), followed by Dispatch. Table 1 presents a sample excerpt from the PRED Sheet.

3.2 Feature Engineering

To support the modeling pipeline, the PRED Sheet was supplemented with calculated fields using spreadsheet automation. These derived metrics fall into two categories: (i) Start Date-based durations, which record the number of working days taken to complete each task from the official project start date (SD), and (ii) individual activity durations (AD), which isolate the duration of each production stage from its own start to end date. Table 2 shows a representative sample of these calculated fields.

Table 2. Automated activity duration metrics from PRED sheet

JC	Part	C		S		BS		SW		FU		W			FP	
		SD	AD	SD	AD	SD	AD	SD	AD	SD	AD	SD	AD		SD	AD
.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
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.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
18	Rafter/Portal	1	1	1	1	4	4	6	3	9	4	10	2		83	5
18	Rafter/Portal	1	1	1	1	4	4	6	3	10	5	10	1		83	5
18	Rafter/Portal	1	1	1	1	4	4	5	2	14	10	16	3		83	5
18	Rafter/Portal	1	1	1	1	4	4	5	2	14	10	16	3		83	5
18	Rafter/Portal	1	1	1	1	2	2	3	2	7	5	13	7		80	5
19	Plates	1	1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		4	1
19	Plates	1	1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		4	1
19	Plates	1	1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		4	1
19	Plates	1	1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		4	1
19	Plates	1	1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		4	1
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These engineered features, along with the raw material attributes, were preprocessed for machine learning input. Missing values in activity durations occurred primarily when certain fabrication stages were not applicable (e.g., splicing absent for plate-type members). Categorical variables such as part marks were encoded using label encoding, and missing values were imputed using the median duration for each respective activity within the same project.

3.3 Project Complexity Stratification

Statistical summaries were generated automatically from the PRED Sheet to reveal trends in fabrication timelines. These summaries were pivotal for understanding the variability and central tendency of production activities across projects. Specifically, for each project and each fabrication activity, descriptive statistics including minimum, maximum, mean, median, and mode were calculated. This statistical analysis enabled the identification of outliers, bottlenecks, and typical process durations—insights that are critical for enhancing model accuracy and interpretability.

Two key types of aggregations were performed: the first computed statistics relative to the Start Date (SD) to assess cumulative project timelines, and the second isolated durations for individual activities, independent of their relationship to project commencement. The former allows understanding of the overall project flow, while the latter isolates efficiency metrics for each task.

Tables 3 and 4 provide these aggregated statistics. Table 3 summarizes the project-level fabrication durations from SD, offering insight into end-to-end timeline distributions. Table 4 breaks down the statistical behavior of each fabrication activity across projects, illuminating activity-specific variabilities.

Table 3. Project-Wise Summary of Fabrication Durations (From Start Date)

Code	Total Fabrication Parts	Cutting					Splicing					
		Min	Max	Mean	Median	Mode	Min	Max	Mean	Median	Mode	

.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	
.	.	.	.	.	.	.	.	.	.	.	.	.
19	50	1	1	1	1	1	0	0	0	0	0	
21	18	1	1	1	1	1	2	2	2	2	2	
22	399	1	1	1	1	1	1	2	1	1	1	
24	550	1	1	1	1	1	1	2	1	1	1	
27	24	1	1	1	1	1	1	1	1	1	1	
29	288	1	1	1	1	1	1	2	1	1	1	
30	339	1	1	1	1	1	1	1	1	1	1	
31	901	1	1	1	1	1	1	2	1	1	1	
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.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	.

Table 4. Project-Wise Activity Duration Summary (Independent of SD)

Code	Total Fabrication Parts	Cutting					Splicing					
		Min	Max	Mean	Median	Mode	Min	Max	Mean	Median	Mode	
.	.	.	.	.	.	.	.	.	.	.	.	
.	.	.	.	.	.	.	.	.	.	.	.	.
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.	.	.	.	.	.	.	.	.	.	.	.	.
19	50	1	1	1	1	1	0	0	0	0	0	
21	18	1	1	1	1	1	2	2	2	2	2	
22	399	1	1	1	1	1	1	2	1	1	1	
24	550	1	1	1	1	1	1	2	1	1	1	
27	24	1	1	1	1	1	1	1	1	1	1	
29	288	1	1	1	1	1	1	2	1	1	1	
30	339	1	1	1	1	1	1	1	1	1	1	
31	901	1	1	1	1	1	1	2	1	1	1	
.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	
.	.	.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.	.	.

This level of statistical analysis formed the basis for selecting input features and tuning model performance in subsequent stages. Moreover, it highlights how specific activities vary in duration, guiding the identification of process stages with the greatest predictive uncertainty.

3.4 Classification Based Preprocessing of Data

To ensure that the machine learning model could learn from structurally homogeneous and operationally consistent data, a robust classification scheme was implemented to stratify the dataset based on fabrication characteristics. This multi-step preprocessing and grouping strategy focused on both the type of structural elements fabricated and the scale of production, thereby improving model specificity and interpretability.

The dataset initially included 71 PEB projects. Incomplete, subcontracted, or conventionally fabricated projects were excluded, leaving 34 with verified fabrication records. A further filter was applied based on the presence of critical load bearing components specifically columns and rafters which are essential to PEB structure. Projects containing both components were designated as type P1, while those containing either component were categorized as type P2. This binary classification ensured the inclusion of structurally significant and fabrication-relevant projects.

The filtered projects were then grouped according to fabrication quantity thresholds measured in metric tons (MT). Projects were categorized into three scales: small (0–30 MT), medium (30–80 MT), and large (>80 MT). Only medium and large-scale projects were retained to mitigate skewness

and reduce noise from under-representative records, resulting in a final dataset of 10 P1 and 12 P2 projects.

To further delineate complexity, projects were classified into three component-based categories: C1, C2, and C3. These categories represent increasing structural and fabrication complexity, with C1 comprising primary members such as columns and rafters, C2 incorporating additional load-distribution elements like joist and portal beams, and C3 encompassing specialized components such as crane beams and jack beams. Each project was assessed based on the proportion of its total fabricated components that belonged to each combination. Minimum thresholds were imposed to ensure dominant representation: 75% for C1, 80% for C2, and 85% for C3. These values were determined in consultation with domain experts from the fabrication facility to align with operational practice, where a component group is considered representative only when the majority of fabricated parts belong to it. Projects meeting these thresholds were respectively categorized into P1C1–P2C1, P1C2–P2C2, and P1C3–P2C3 classification groups.

This dual-layered classification based on fabrication type and component complexity yielded six mutually exclusive and structurally coherent project groups. Each group reflects a distinct fabrication profile, allowing for stratified training and evaluation of the predictive model. Notably, among the 29 active fabrication components observed across all projects, 15 were common to all classification tiers, while 11 appeared in at least two groups, and three were exclusive to the C3 category. This distribution reinforces the validity of the classification framework and its alignment with real-world fabrication heterogeneity.

After preprocessing and removal of incomplete records, the final modelling dataset comprised 34 completed PEB projects. These instances were distributed across the three complexity classes as follows: P1C1–P2C1: [20] records, P1C2–P2C2: [25], P1C3–P2C3: [28]. This breakdown ensured that each class contained a sufficient number of observations for training and evaluation, while preserving the intended separation between project types and complexity tiers.

The final result of this preprocessing is a dataset that captures the diverse operational realities of PEB fabrication while maintaining sufficient uniformity within each group to enable accurate and interpretable machine learning predictions.

4. Methodology

This section outlined the predictive modeling approach adopted for forecasting fabrication durations in PEB projects using real-time production data. The methodology incorporated ensemble machine learning, complexity-based data segmentation, and a profitability-aware evaluation framework.

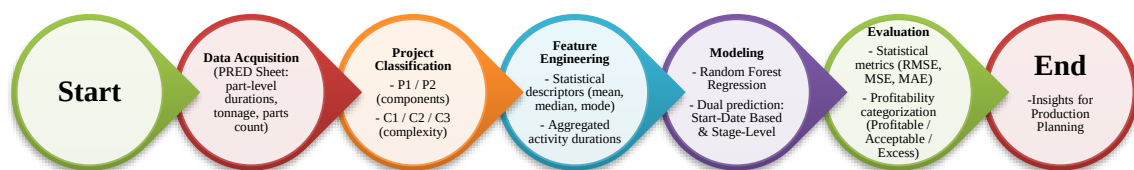


Fig. 1. Methodology flowchart

Fig. 1. Methodology flowchart showing the sequence from classification-based project grouping to feature engineering, Random Forest regression modeling, and profitability-oriented evaluation.

4.1 Model Selection

A Random Forest Regressor was selected for its proven effectiveness in capturing non-linear dependencies and managing overfitting. Unlike linear models that assume additive effects, Random Forests can capture the complex interactions that arise from fabrication sequences, variable

workloads, and project heterogeneity. Its robustness to outliers and capability to operate with minimal preprocessing make it suitable for real-world industrial datasets. Implementation was carried out using the Scikit-learn Python package.

Random Forest Regression was chosen not only based on evidence from the literature but also due to its methodological suitability for the nature of fabrication data. The dataset was high-dimensional, with a mixture of categorical and continuous engineered features. Random Forest can handle such datasets effectively without extensive preprocessing, was not sensitive to feature scaling, and captures non-linear dependencies that linear models cannot address. Furthermore, it mitigates overfitting through ensemble averaging, making it robust for real-world production data.

The predictive performance observed in this study (Sections 4.6.1–4.6.5) further confirms that Random Forest aligns with the requirements of multi-stage fabrication forecasting, producing low error rates and profitability-consistent outputs across complexity tiers. These results validate Random Forest as a methodologically appropriate and empirically effective choice for this problem setting.

4.2 Feature Inputs

The input feature set was derived from the PRED Sheet and included a combination of static, dynamic, and engineered features. Static fields such as job code, number of parts, member weights, and total tonnage formed the base attributes. Dynamic activity durations and completion percentages extracted from automated spreadsheet formulas enhanced the temporal granularity. Features were grouped into three primary types:

- Project attributes: Total parts, tonnage, dominant member types (e.g., columns, rafters).
- Activity durations: Mean, median, mode for each of the ten production stages.
- Completion indicators: Real-time percentages derived from daily updates.

Categorical variables were label encoded. Due to the decision-tree architecture of the Random Forest model, feature normalization or scaling was not required.

4.3 Training and Validation Strategy

To preserve stratified project characteristics, model training was performed separately for each grouping category (P1C1–P2C1, P1C2–P2C2, P1C3–P2C3). Within each class, data was split into 80% training and 20% testing sets. A 5-fold cross-validation scheme was employed during hyperparameter tuning. Hyperparameters tuned include:

- Number of trees (`n_estimators`)
- Maximum depth of trees (`max_depth`)
- Minimum samples required to split a node (`min_samples_split`)

Grid search optimization was used, with RMSE as the scoring metric to ensure penalty on large deviations.

4.4 Prediction Targets

Two target variables were defined to satisfy both managerial and operational forecasting needs:

- Overall Project Duration: Measured in working days from the project's start date to final dispatch.
- Activity-Level Durations: Time required for specific fabrication stages such as Cutting, Welding, and Painting.

This dual target approach enabled both macro-level (project-wide) and micro-level (activity-specific) predictive insight.

4.5 Evaluation Metrics

The model was evaluated using traditional regression metrics:

- Root Mean Square Error (RMSE) – Highlights model sensitivity to large prediction errors.
- Mean Squared Error (MSE) – Reflects the average squared deviation across all samples.

Additionally, a custom classification scheme was developed to assess predictions based on practical thresholds for profitability and schedule tolerance. Predictions were categorized as:

- Profitable ($\leq -5\%$ deviation from actual)
- Acceptable ($\pm 5\%$ deviation)
- Excess ($> +5\%$ overestimation)
- Shortfall ($> +5\%$ underestimation)

After obtaining predicted and actual fabrication durations for a given part, the percentage deviation is calculated using the formula: $(\text{"Predicted"} - \text{"Actual"}) / \text{"Actual"} \times 100$. For instance, if the model predicts 12 days and the actual duration is 10 days, the percentage deviation is $(12 - 10) / 10 \times 100 = 20\%$. According to the study's classification criteria, predictions with a deviation between 0% and +10% are considered Profitable; those exceeding +10% are classified as Excess; and deviations between 0% and -10% indicate More Profitable scheduling. In this example, the 20% deviation places the prediction in the Excess category, signaling an overestimation of time that could lead to inefficient resource allocation in project planning.

4.6 Model Performance Summary

4.6.1 Statistical Error Analysis

This subsection presents a detailed analysis of the model's prediction performance using Root Mean Square Error (RMSE) and Mean Squared Error (MSE), computed for total project durations and activity-level forecasts across the three project complexity groups—P1C1–P2C1 (low complexity), P1C2–P2C2 (medium complexity), and P1C3–P2C3 (high complexity).

4.6.1.1 RMSE-Based Evaluation by Classification

Low Complexity (P1C1–P2C1): The model demonstrated excellent predictive performance. The RMSE for total duration from Start Date (TSD) was 2.12 days, and activity-level RMSEs for deterministic stages like Cutting (SCA, PCA) and Splicing (SSA, PSA) were 0.00, indicating high confidence and standardization. Mid-stream processes such as SAW Welding (SSWA: 2.27), Fit-up (SFA: 2.27), and Welding (SWA: 0.87) had relatively low RMSE values. Finishing activities showed higher variability: Painting average from SD (SPA: 7.69) and Finish Paint average from SD (SFPA: 7.67) highlighting their dependency on external variables. Median RMSEs were closely aligned, reinforcing stable model performance, as shown in Fig. 2 (a) which depicts RMSE for Start-Date-Based and Activity-Level Predictions in Low-Complexity Projects (P1C1–P2C1).

Medium Complexity (P1C2–P2C2): With increased fabrication diversity, RMSE values moderately increased. The RMSE for TSD rose to 2.50 days. SAW Welding (SSWA: 3.06) and Drilling (SDA: 3.55) became more variable. Painting (SPA: 8.93), Finish Paint (SFPA: 8.97), and Grinding average from SD (SGA: 2.78) continued to exhibit significant dispersion. However, base activities remained stable (Cutting and Splicing: 0.00). Median RMSEs again mirrored average values, confirming that errors were not driven by extreme outliers, as detailed in Fig. 2 (b) which is RMSE for Start-Date-Based and Activity-Level Predictions in Medium-Complexity Projects (P1C2–P2C2).

High Complexity (P1C3–P2C3): Although the TSD RMSE was comparable (2.40 days), increased component variability led to sharper divergence in certain activities. SAW Welding average from SD (SSWA: 2.62), Fit-up average from SD (SFA: 3.94), Welding average from SD (SWA: 4.35), and Grinding (SGA: 6.27) saw higher RMSEs. Notably, Primer (SPA: 9.81) and Finish Paint (SFPA: 9.97) recorded the highest variability across all activities. In contrast, Cutting and Splicing remained predictable with 0.00 RMSE. The median RMSEs, such as SPM_e and SFP_m_e, were significantly lower (~ 1.00), demonstrating that while outliers increased the mean error, core predictions remained centered, as shown in Fig. 2 (c) RMSE for Start-Date-Based and Activity-Level Predictions in High-Complexity Projects (P1C3–P2C3).

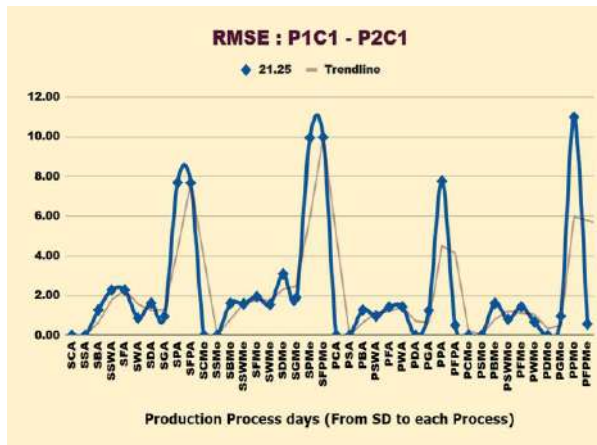


Fig. 2 (a). RMSE for Start-Date-Based and Activity-Level Predictions in Low-Complexity Projects (P1C1-P2C1)

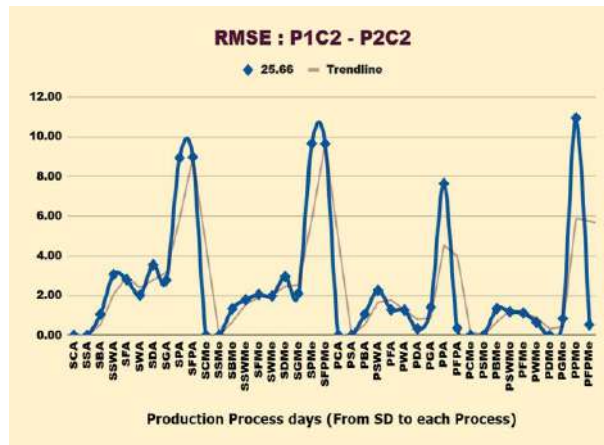


Fig. 2 (b). RMSE for Start-Date-Based and Activity-Level Predictions in Medium-Complexity Projects (P1C2-P2C2)

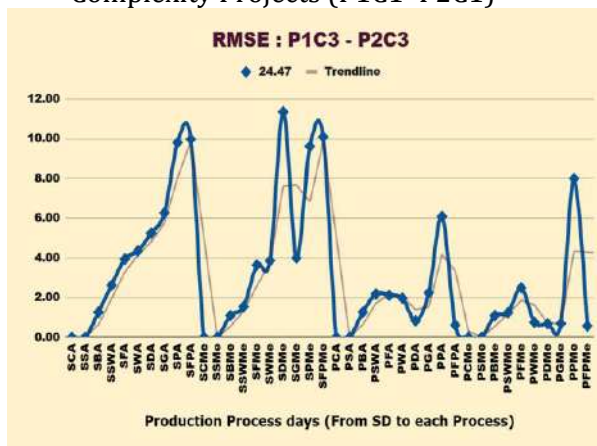


Fig. 2 (c). RMSE for Start-Date-Based and Activity-Level Predictions in High-Complexity Projects (P1C3-P2C3)

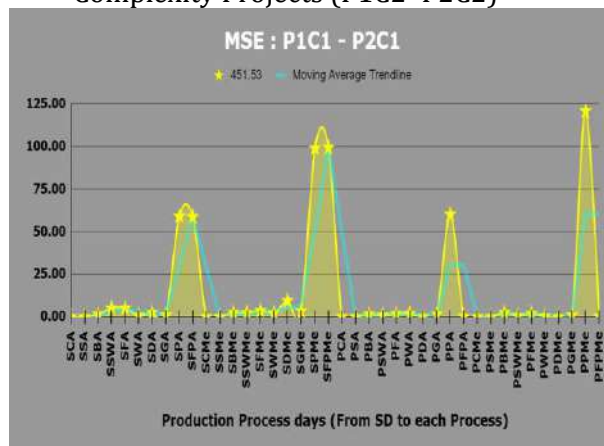


Fig. 3 (a). MSE for Start-Date-Based and Activity-Level Predictions in Low-Complexity Projects (P1C1-P2C1)

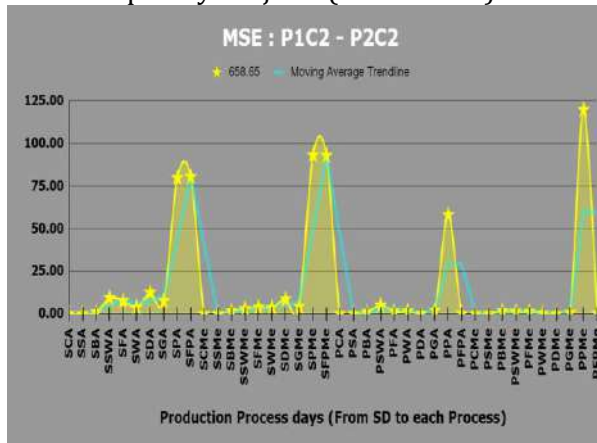


Fig. 3 (b). MSE for Start-Date-Based and Activity-Level Predictions in Medium-Complexity Projects (P1C2-P2C2)

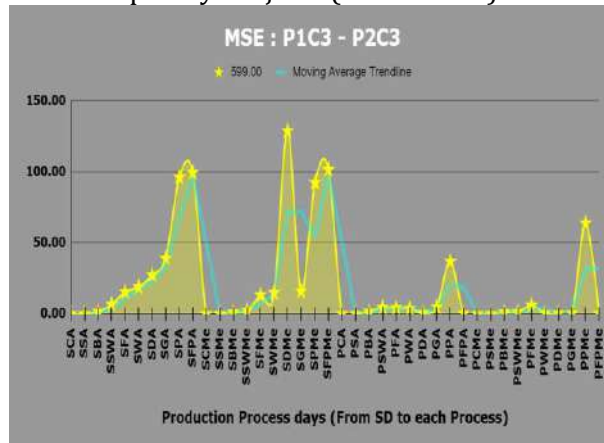


Fig. 3 (c). MSE for Start-Date-Based and Activity-Level Predictions in High-Complexity Projects (P1C3-P2C3)

4.6.1.2 MSE-Based Evaluation by Classification

Low Complexity (P1C1-P2C1): The MSE for (TSD) was 4.52 days². SAW Welding (SSWA: 5.17), Grinding (SGA: 0.90), and Painting activities like SPA (5.91) and SFPA (5.88) showed notable squared errors. Other stages remained within 2.5 days², indicating the model's strong resistance to large deviations in simpler projects, from Fig. 3 (a) which shows MSE for Start-Date-Based and Activity-Level Predictions in Low-Complexity Projects (P1C1-P2C1).

Medium Complexity (P1C2–P2C2): The MSE for TSD increased to 6.50 days², with elevated squared errors in SAW Welding (SSWA: 9.38), Grinding (SGA: 7.70), and Painting stages (SPA: 7.90, SFPA: 8.00). Fit-up and Welding also crossed the 4.0 mark, indicating increased forecast instability as task complexity and variability grew, as presented in Fig. 3 (b) - MSE for Start-Date-Based and Activity-Level Predictions in Medium-Complexity Projects (P1C2–P2C2).

High Complexity (P1C3–P2C3): The MSE for TSD dropped slightly to 5.00 days², potentially due to overcorrected predictions. Activity-level MSEs for Saw Welding mean average (SSWA: 6.89) and Grinding average (SGA: 3.90) remained significant. Interestingly, Painting stages had lower MSE values (SPA: 0.90, SFPA: 0.90) possibly due to conservative adjustments in response to previous overestimations. The pattern supports that model variance was concentrated in a limited set of outlier tasks, as depicted in Fig. 3 (c) which represents MSE for Start-Date-Based and Activity-Level Predictions in High-Complexity Projects (P1C3–P2C3).

As shown in Figs. 2 (a), 2 (b) and 2 (c), the RMSE values for both start-date-based and activity-level predictions differ across low, medium, and high-complexity projects, respectively. Similarly, MSE results for start-date-based and activity-level predictions are shown in Fig. 3 (a) for low-complexity, Fig. 3 (b) for medium-complexity, and Fig. 3 (c) for high-complexity projects. This explains that the Random Forest model consistently delivered high predictive accuracy in low-complexity projects, with moderate performance in medium and high-complexity settings. Errors increased notably in stages susceptible to variability such as SAW Welding, Grinding, and Painting. Median RMSE values were generally lower than their means, reflecting resilience to outliers. This group-based insight confirms that while deterministic activities maintain high fidelity, finishing operations remain sensitive to contextual fluctuations.

Overall, both RMSE and MSE metrics affirm the model's robustness and applicability in operational forecasting. Its performance supports deployment in industrial PEB scheduling environments with potential future improvements through integration of real-time production signals and environmental parameters.

4.6.2 Classification-Wise Prediction Performance

To assess the consistency and adaptability of the Random Forest model across different fabrication project types, three set of pairs were analyzed: P1C1 vs P2C1, P1C2 vs P2C2, and P1C3 vs P2C3. Each pair represented a balance of structural complexity and production scale. The goal was to evaluate model robustness in predicting both total project duration (from Start Date to dispatch) and individual activity-wise durations. The predictions were evaluated using a structured tabular framework, the sample table template as shown in Figs. 4 (a–c). For each classification, four sub-tables were generated:

- Original values (actual durations),
- Predicted values from the Random Forest model,
- Absolute error (difference between actual and predicted),
- Cumulative interpretation: percentage-based summary and qualitative categorization.

The comparative analysis of original data, ML predictions, error heatmaps, and profitability classification are presented in Fig. 4 (a) for low-complexity. The model demonstrated excellent agreement with actual values. Most predictions remained within an absolute deviation of $\pm 5\%$, and the cumulative summary showed that 95% of the outputs fell under profitable, more profitable, or acceptable categories. Only one instance approached a shortfall boundary, suggesting conservative tendencies, which are often favorable in production planning.

The comparative analysis of original data, ML predictions, error heatmaps, and profitability classification are presented in Fig. 4 (b) for medium-complexity. The projects performance remained consistent despite increased complexity. The average absolute error was 2.03%, and all predictions stayed within tolerable thresholds. The model exhibited stability in forecasting mid-range projects, balancing precision with production-safe margins. The comparative analysis of original data, ML predictions, error heatmaps, and profitability classification are presented in Fig. 4 (c) for High-complexity.

Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe	Original Data		
1	P29	45.20	46	1	1	4	8		2	2	1	4	2			
2	P36	42.78	33	1	1	2	3		3	2	3	3	2			
3	P43	99.08	70	1	1	3	5		2	2	3	28	2			
4	P46	90.13	40	1	1	4	6		2	2	2	5	2			
5	P51	42.97	20	1	1	2	5		3	2	2	6	1	ML Predictions		
Sl.No.	Cod e	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe			
1	P29	45.2	42	1	1	4	7		2	2	1	5	2			
2	P36	42.78	36	1	1	2	4		3	2	3	5	2			
3	P43	99.08	59	1	1	3	5		2	2	3	21	2			
4	P46	90.13	40	1	1	4	6		2	2	2	7	2			
5	P51	42.97	28	1	1	3	6		3	2	2	6	1	Difference %		
Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe			
1	P29	45.20	-8.63%	0.00%	-0.03%	-1.13%	-1.00%		0.42%	0.00%	0.86%	1.55%	-0.24%			
2	P36	42.78	7.22%	0.00%	0.00%	1.68%	2.81%		-0.56%	0.00%	-0.94%	4.68%	-0.21%			
3	P43	99.08	-11.61%	0.00%	-0.44%	0.41%	0.51%		0.13%	0.00%	-0.31%	-7.45%	-0.09%			
4	P46	90.13	-0.52%	0.00%	0.00%	-0.71%	-0.33%		0.22%	0.00%	0.06%	2.01%	-0.17%			
5	P51	42.97	19.50%	0.00%	0.00%	1.59%	1.53%		-0.70%	0.00%	-0.21%	0.26%	0.79%	Cumulative Average to see overall trends		
Excess Prediction (above 5%)		min	7.22%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%			0.18%
		max	19.50%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%			0.48%
		Total %	40.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%			0.98%
Shortfall Prediction (below -5%)		min	-11.61%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	-7.45%	0.00%			-1.17%
		max	-8.63%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	-7.45%	0.00%			-1.10%
		Total %	40.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	20.00%	0.00%			3.90%
Profit (P) : (0 to 5%)			0.00%	0.00%	0.00%	60.00%	60.00%		60.00%	0.00%	40.00%	80.00%	20.00%			43.90%
More Profit : (0 to -5%)			20.00%	100.00%	100.00%	40.00%	40.00%		40.00%	100.00%	60.00%	0.00%	80.00%			51.22%
Acceptable P : (-5 to 5%)			20.00%	100.00%	100.00%	100.00%	100.00%		100.00%	100.00%	100.00%	80.00%	100.00%	95.12%		

Fig. 4 (a). Comparison of Original Data, ML Predictions, Differences Heatmap and Profitability Classification (Acceptable, Excess, Shortfall) for Low-Complexity Projects (P1C1 vs. P2C1)

Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe	Original Data	
1	P29	45.20	46	1	1	4	8		2	2	1	4	2		
2	P31	73.82	77	1	1	2	11		2	2	2	7	2		
3	P36	42.78	33	1	1	2	3		3	2	3	3	2		
-	-	-	-	-	-	-	-	-	-	-	-	-	-		
Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe	ML Predictions	
1	P29	45.20	42	1	1	3	7		2	2	1	5	2		
2	P31	73.82	65	1	1	2	9		2	2	2	6	2		
3	P36	42.78	40	1	1	2	4		3	2	3	6	2		
-	-	-	-	-	-	-	-	-	-	-	-	-	-		
Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PWMe	PDMe	PGMe	PPMe	PFPMe	Difference %	
1	P29	45.20	-8.01%	0.00%	-0.03%	-1.42%	-0.95%		0.51%	0.00%	0.95%	1.11%	-0.27%		
2	P31	73.82	-16.36%	0.00%	-0.02%	-0.03%	-1.91%		0.15%	0.00%	0.07%	-0.89%	-0.01%		
3	P36	42.78	15.40%	0.00%	0.00%	1.32%	3.23%		-0.61%	0.00%	-0.65%	8.09%	-0.23%		
-	-	-	-	-	-	-	-	-	-	-	-	-	-		
Excess Prediction (above 5%)	min		15.40%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	8.09%	0.00%	Cumulative Average to see overall trends	0.83%
	max		18.15%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	8.09%	0.00%		0.90%
	Total %		33.33%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	16.67%	0.00%		2.03%
Shortfall Prediction (below -5%)	min		-16.36%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	-8.93%	0.00%		-1.45%
	max		-8.01%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	-8.93%	0.00%		-1.19%
	Total %		50.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	16.67%	0.00%		4.47%
Profit (P) : (0 to 5%)			16.67%	0.00%	0.00%	50.00%	66.67%		66.67%	0.00%	50.00%	33.33%	16.67%		41.87%
More Profit : (0 to -5%)			0.00%	100.00%	100.00%	50.00%	33.33%		33.33%	100.00%	50.00%	33.33%	83.33%		51.63%
Acceptable P : (-5 to 5%)			16.67%	100.00%	100.00%	100.00%	100.00%		100.00%	100.00%	100.00%	66.67%	100.00%		93.50%

Fig. 4 (b). Comparison of Original Data, ML Predictions, Differences Heatmap and Profitability Classification (Acceptable, Excess, Shortfall) for Medium-Complexity Projects (P1C2 vs. P2C2)

Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PFMe	PWMe	PDMe	PGMe	PPMe	PFPMe	Original Data
1	P24	62.58	28	1	1	2	4		2	1	1	2	8	2	
2	P31	73.82	77	1	1	2	11		2	2	2	2	7	2	
3	P32	100.99	73	1	1	2	5		8	3	3	3	11	2	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	ML Predictions
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PFMe	PWMe	PDMe	PGMe	PPMe	PFPMe	Difference %
1	P24	62.58	34	1	1	2	5		3	2	1	2	7	2	
2	P31	73.82	63	1	1	2	9		2	2	2	2	6	2	
3	P32	100.99	59	1	1	2	5		6	3	3	3	9	2	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Difference %
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
Sl.No.	Code	Total MT	TSD	SCA	SSA	SBA	SSWA		PFMe	PWMe	PDMe	PGMe	PPMe	PFPMe	Difference %
1	P24	62.58	10.21%	0.00%	-0.08%	0.38%	0.66%		1.15%	0.80%	0.54%	0.22%	-0.91%	-0.16%	
2	P31	73.82	-18.38%	0.00%	-0.02%	-0.31%	-2.33%		0.54%	0.22%	-0.07%	0.20%	-1.04%	-0.11%	
3	P32	100.99	-13.86%	0.00%	-0.04%	0.48%	0.24%		-1.79%	-0.31%	-0.50%	-0.27%	-1.73%	-0.12%	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Cumulative Average to see overall trends
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
Excess Prediction (above 5%)	min	5.14%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	1.84%
	max	14.94%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2.54%
	Total %	50.00%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	8.54%
Shortfall Prediction (below -5%)	min	-18.38%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	-7.35%	0.00%	-3.52%
	max	-11.28%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	-7.35%	0.00%	-2.68%
	Total %	37.50%	0.00%	0.00%	0.00%	0.00%	0.00%		0.00%	0.00%	0.00%	0.00%	12.50%	0.00%	6.71%
Profit (P) : (0 to 5%)		12.50%	0.00%	0.00%	62.50%	62.50%	62.50%		62.50%	62.50%	50.00%	62.50%	50.00%	25.00%	42.07%
More Profit : (0 to -5%)		0.00%	100.00%	100.00%	37.50%	37.50%	37.50%		37.50%	37.50%	50.00%	37.50%	37.50%	75.00%	42.68%
Acceptable P : (-5 to 5%)		12.50%	100.00%	100.00%	100.00%	100.00%	100.00%		100.00%	100.00%	100.00%	100.00%	87.50%	100.00%	84.76%

Fig. 4 (c). Comparison of Original Data, ML Predictions, Differences Heatmap, and Profitability Classification (Acceptable, Excess, Shortfall) for High-Complexity Projects (P1C3 vs. P2C3)

The projects involved higher variability in structure, sequencing, and task durations. Yet, the model maintained reliable predictive behavior. The average cumulative deviation stood at 1.84%, and most outputs remained within acceptable or better classifications. Only one project approached a mild shortfall, reinforcing the model's generalization capability across fabrication complexities. Overall, the structured comparison across these classification pairs validates the model's ability to produce accurate and consistent predictions across diverse project types. This forms the baseline for deeper trend analysis (Section 4.6.3) and classification-based profitability assessments (Section 4.6.4)

4.6.3 Profitability Classification Summary

To interpret the real-world scheduling implications of the model's predictions, a profitability-based evaluation framework was applied. Each predicted duration was compared to its actual counterpart and categorized as Excess (prediction >5% longer), Shortfall (prediction >5% shorter), or Acceptable (within $\pm 5\%$). Within the Acceptable range, further refinement distinguished between Profitable (0 to -5%) and More Profitable ($<-5\%$) predictions. Figs. 5 (a-c) and Figs. 6 (a-c) present these classifications for start-date-based and activity-specific predictions, respectively, across the three complexity tiers.

4.6.3.1 Start Date Based Predictions

For start-date-based predictions shown in Fig. 5 (a), low-complexity projects (P1C1-P2C1) revealed a perfectly balanced outcome: 40% of predictions were Excess and another 40% were Shortfall, with only 20% falling in the Acceptable range. While this highlights limitations in predicting total project timelines from the start date alone, the fact that prediction errors were evenly distributed suggests there was no systemic bias toward over- or underestimation in these simpler projects.

In Fig. 5 (b), for medium-complexity projects (P1C2-P2C2), start-date-based predictions became less reliable. A total of 50% of predictions fell under the Shortfall category, and 33.3% were Excess,

leaving only 16.7% as Acceptable. This trend indicates a notable increase in prediction dispersion, which may be attributed to variability in part sequencing or minor overlaps in mid-stage activities.

The pattern continued in Fig. 5 (c) for high-complexity projects (P1C3-P2C3), where the model's prediction quality further degraded. Half the predictions were Excess, and 37.5% were Shortfall, with a mere 12.5% falling within the acceptable margin. This steep drop in predictive reliability highlights the challenge of accurately estimating total project timelines in projects characterized by component diversity, nonlinear production flows, and extended finishing operations.

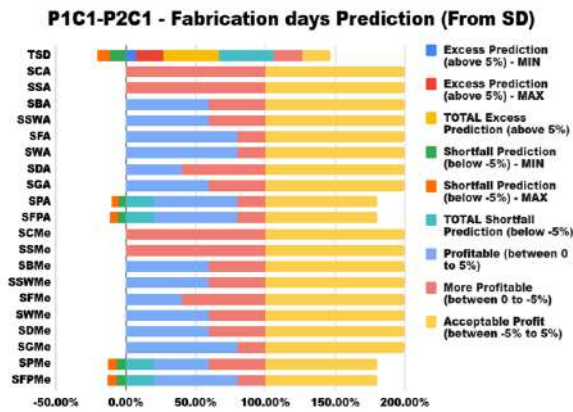


Fig. 5 (a). SD Based Forecasts for Low-Complexity (P1C1 vs. P2C1), including excess, shortfall, and profitability ranges

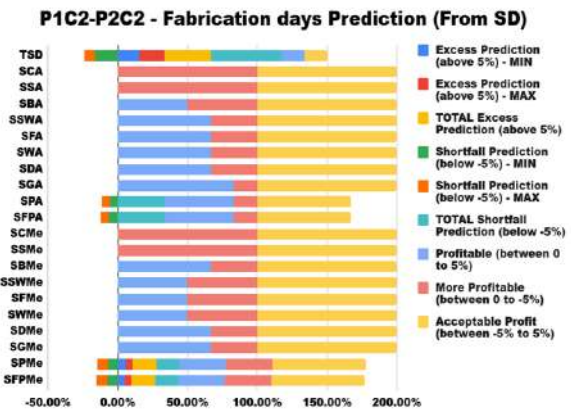


Fig. 5 (b). SD Based Forecasts for Medium-Complexity (P1C2 vs. P2C2), including excess, shortfall, and profitability ranges

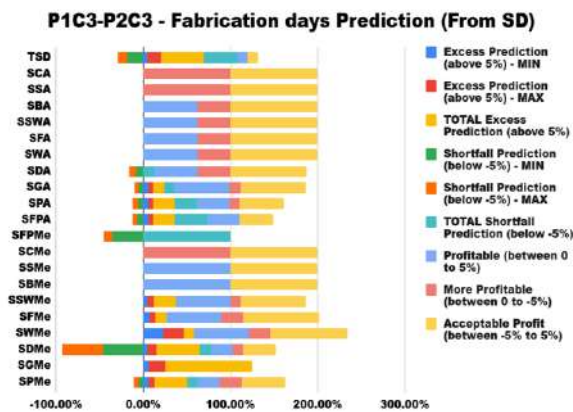


Fig. 5 (c). SD Based Forecasts for High-Complexity (P1C3 vs. P2C3), including excess, shortfall, and profitability ranges

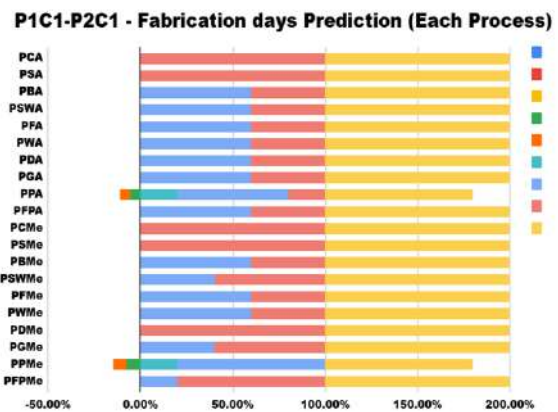


Fig. 6 (a). AD Forecasts for Low-Complexity (P1C1 vs. P2C1), including excess, shortfall, and profitability ranges

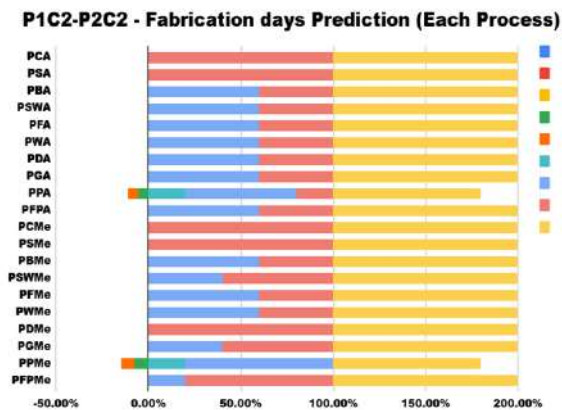


Fig. 6 (b). AD Forecasts for Medium-Complexity (P1C2 vs. P2C2), including excess, shortfall, and profitability ranges

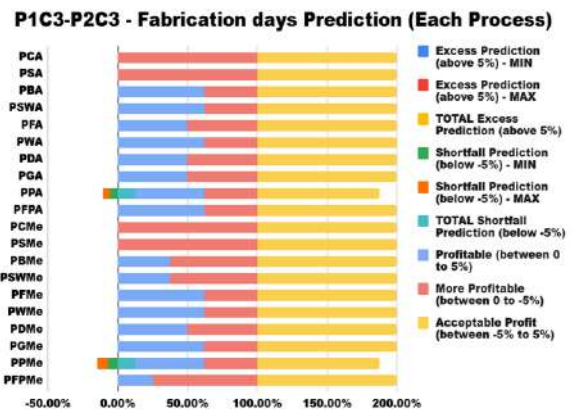


Fig. 6 (c). AD Forecasts for High-Complexity (P1C3 vs. P2C3), including excess, shortfall, and profitability ranges

4.3.6.2 Activity Specific Predictions

By contrast, the activity-specific classification results shown in Fig. 6 (a) for low complexity projects in P1C1–P2C1 projects were markedly better. All fabrication activities except for a few painting stages achieved 100% Acceptable classification, with no instances of Excess or Shortfall. For example, Cutting (PCA) and Splicing (PSA) were fully consistent, while even more variable tasks like Painting (PPA) only registered a 20% Shortfall. This confirms that for low-complexity jobs, activity-wise modeling captures fabrication durations with exceptional accuracy.

In medium-complexity projects (P1C2–P2C2), the results in Fig. 6 (b) reaffirm the advantage of activity-specific forecasting. While Painting (PPA) showed a 16.7% Shortfall, and Painting Median (PPMe) had both Excess and Shortfall at 16.7% each, all other fabrication stages maintained 100% Acceptable classification. Profitable and More Profitable segments were well-balanced, indicating that even under increased part variety and intermediate complexity, the model retains practical usefulness at the activity level.

Finally, in high-complexity projects (P1C3–P2C3), Fig. 6 (c) shows some dispersion but still maintains remarkable control compared to the corresponding start-date-based predictions. Most core fabrication tasks such as Cutting (PCA), Splicing (PSA), SAW Welding (PSWA), and Primer (PFPA) showed no Excess or Shortfall and remained within the Acceptable range. Even in the more variable stages like Painting (PPA) and its median values (PPMe), only 12.5% fell into the Shortfall range, while the rest were well-distributed between Profitable and More Profitable. These comparisons establish that while overall project duration forecasts tend to suffer from growing variance as complexity increases, activity-level prediction classification is consistently stable, accurate, and informative across all complexity tiers. This reinforces the conclusion that micro-level prediction granularity better accommodates structural variation and operational uncertainty inherent in complex Pre-Engineered Building projects.

4.6.4 Profitability Trend Analysis

To gain insight into how predictive performance shifts with increasing project complexity, a trend-based analysis of classification outcomes was conducted. Figs. 7 and 8 summarize the proportion of predictions falling into Excess, Shortfall, and Acceptable Profit categories across the three classification groups: P1C1–P2C1 (low), P1C2–P2C2 (medium), and P1C3–P2C3 (high complexity).

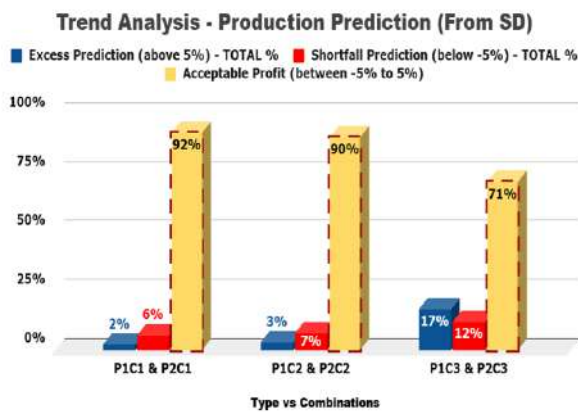


Fig. 7. Profitability Trend analysis of production (From SD)

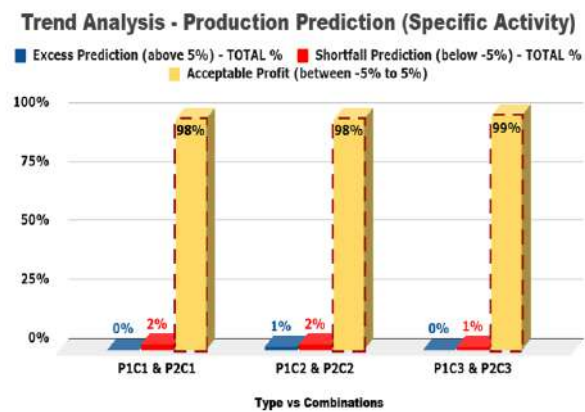


Fig. 8. Profitability trend analysis of production (Activity Based)

In Fig. 7, which visualizes the trends for start-date-based predictions, a clear decline in predictive reliability is observed as complexity increases. In the low-complexity group (P1C1–P2C1), Acceptable predictions accounted for 92%, with only 2% Excess and 6% Shortfall. This demonstrates strong schedule-aligned forecasting in standardized fabrication environments. However, in the medium-complexity group (P1C2–P2C2), Acceptable classification slightly dropped to 90%, while Excess and Shortfall rose to 3% and 7%, respectively. The decline becomes much steeper in the high-complexity group (P1C3–P2C3), where Acceptable predictions fall to just 71%, and Excess and Shortfall rise sharply to 17% and 12%, respectively. This pattern indicates

the compounding effects of fabrication variability and inter-task dependencies on the model's cumulative timeline accuracy.

Conversely, Fig. 8, which illustrates the trend for activity-specific predictions, presents a striking contrast. In low-complexity projects, the model achieved 98% Acceptable predictions, with only 2% Shortfall and no Excess. In medium complexity, Acceptable predictions remained constant at 98%, while Excess and Shortfall slightly increased to 1% and 2%, respectively. Remarkably, in high-complexity projects, Acceptable classification further improved to 99%, with Shortfall dropping to 1% and no Excess predictions recorded. These trends reinforce a key operational insight: as project complexity increases, start-date-based predictions become increasingly volatile and prone to deviation, while activity-level predictions maintain high accuracy and profitability alignment. The ability of stage-focused modeling to isolate task durations independently of project-wide variability makes it a more robust strategy for forecasting in complex Pre-Engineered Building (PEB) fabrication workflows.

4.6.5 Profitability Consolidation Across Prediction Strategies

To consolidate the classification analysis and highlight the consistency of the predictive model across varying complexity tiers, Figs 9 and 10 present grouped bar charts showing the overall profit classification trends for both start-date-based and activity-specific prediction strategies. In Fig. 9, which illustrates results from start-date-based predictions, the overall profitability classification (i.e., combined share of Profitable and More Profitable predictions) shows a declining trend as project complexity increases. In the low-complexity group (P1C1–P2C1), the model achieved a strong 98% profitability classification, which slightly declined to 97% for medium-complexity projects (P1C2–P2C2). However, for the high-complexity group (P1C3–P2C3), profitability dropped notably to 83%, confirming that aggregate predictions become more susceptible to deviation as fabrication workflows become structurally diverse and less deterministic.

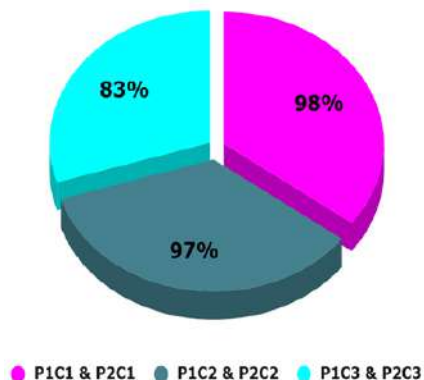


Fig. 9. Overall profit analysis for production (From SD)

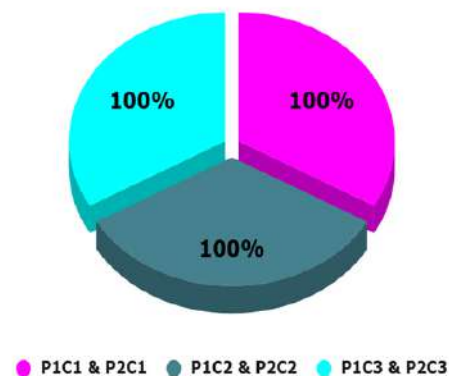


Fig. 10 Overall profit analysis for production (Activity Based)

In contrast, Fig. 10, which summarizes profitability classification for activity-specific predictions, demonstrates remarkable stability and robustness across all complexity levels. The model consistently achieved 100% profitability classification for low, medium, and high-complexity projects alike. This means that every activity-level prediction—regardless of project scale or structural heterogeneity—remained within the Profitable or More Profitable classification zones.

These findings visually reinforce the earlier statistical and trend-based conclusions: activity-specific modeling ensures highly profitable and stable forecasts, even in projects characterized by fabrication variability, concurrent workflows, and extended finishing durations. On the other hand, start-date-based predictions, while initially strong, tend to degrade under project complexity, underscoring the limitations of aggregated forecasting models for detailed production planning.

5. Discussion

The predictive performance analysis of this study provides both theoretical and empirical validation for the structured application of machine learning in real-time fabrication forecasting

for PEB. A critical comparison between start-date-based and activity-specific regression models has revealed several important insights into how fabrication workflows, model architecture, and feature behaviors interact under varying degrees of project complexity.

An important methodological consideration in this study was the relatively small sample size within each classification subset. While small data regimes typically risk overfitting, the results showed close agreement between mean and median RMSE values, and profitability classifications were balanced across Excess and Shortfall categories. These outcomes suggest that the model did not memorize isolated instances but generalized patterns effectively. The use of bootstrap aggregation within Random Forest, complexity-aware stratification, and hyperparameter tuning collectively contributed to this resilience.

The stark divergence in error behavior particularly in Root Mean Square Error (RMSE) and Mean Squared Error (MSE) between start-date-based predictions and activity-specific predictions underscores the nonlinear error propagation inherent in cumulative timeline modeling. In high-complexity projects (P1C3–P2C3), aggregated predictions exhibited both amplified residuals and greater variance, suggesting model strain under feature interactions that are not easily decomposable. This is a well-observed behavior in ensemble tree-based models like Random Forests, which excel at isolating decision boundaries for localized predictions (e.g., specific activities), but become less interpretable and less accurate when tasked with predicting long, path-dependent sequences.

In contrast, the task-specific modeling approach constrained the regression task to independent, bounded intervals (e.g., welding duration, painting lead time), which allowed the Random Forest algorithm to operate with higher granularity and lower cumulative variance. The predictive fidelity across even high-complexity projects with significant concurrent tasks and variability in fabrication sequencing reinforces the ensemble model's capacity to effectively partition feature space when provided with stage-isolated data.

From a feature behavior standpoint, the engineered inputs particularly the statistical descriptors such as stage-level mean, median, and mode were key contributors to model generalization. These summary statistics act as stabilizing factors in the presence of noise and outliers. In addition, the classification-based stratification of the dataset into P1C1–P2C3 groups significantly improved model bias-variance trade-off by ensuring homogeneity within training samples. This stratification aligns with complexity theory in project scheduling, where performance is known to degrade in heterogeneous or poorly segmented datasets.

The classification performance summary, visualized through Fig. 4 to Fig. 10, further reinforces the limitations of cumulative prediction models under fabrication variability. The downward trend in profitability and acceptable prediction rates for start-date-based models highlights an exponential sensitivity to component diversity and flow disruptions. Meanwhile, the flat or improving trend in activity-level classifications demonstrates the superior scalability of stage-based models. Notably, no excess classifications were recorded in activity-level predictions for high-complexity projects—a rare behavior in predictive modeling that indicates excellent overfit control and structural resilience.

These findings have implications for both model design and production control. First, they validate the use of ensemble regressors with classification-aware preprocessing for industrial applications involving multistage workflows. Second, they show that disaggregating predictive tasks across operational stages provides not only statistical benefits (e.g., reduced error, variance), but also operational advantages (e.g., task-specific insights, bottleneck detection). Lastly, the introduction of a profitability-aware classification framework represents a methodological advancement, aligning predictive accuracy with actionable performance categories rooted in cost, schedule tolerance, and delivery reliability.

This study has several limitations that should be acknowledged. First, the dataset comprises 34 completed projects, which, while carefully preprocessed and stratified, remains relatively small and may limit the generalizability of the predictive models to other PEB projects with different characteristics. Second, model validation was performed using historical data from past projects,

without external prospective validation. These limitations restrict the extent to which the models' predictive accuracy can be assured in new or evolving project contexts. Future research should aim to augment the dataset with additional and more diverse projects and validate the approach prospectively to strengthen robustness and applicability.

5.1 Break-Even Point (BEP)–Based Evaluation of Fabrication Efficiency

To gain a clearer understanding of how individual fabrication stages contribute to overall schedule variability, we evaluated performance using a Break-Even Point (BEP) framework. In this context, the BEP refers to the typical (median) number of days a given process takes across all recorded parts. By design, the median serves as a more robust baseline than the mean, as it reduces sensitivity to extreme outliers and rare delays.

Each fabrication stage was assessed by comparing its mean duration to its BEP. When the mean exceeded the BEP, the stage was flagged for review, with the degree of deviation used to classify the process as efficient, marginally overrun, or significantly delayed. This evaluation helps establish which stages are consistently under control and which ones may require deeper analysis or operational adjustment.

Table. 5 Fabrication Stage Performance Compared to Break-Even Point

Fab Stage	No. of Parts	BEP (Median Days)	Mean Days	Deviation from BEP	Efficiency Status
1. Cutting	31,547	1	1	0	Efficient
2. Splicing	9,473	1	1	0	Efficient
3. Built-Up Section	3,773	2	2	0	Efficient
4. SAW Welding	3,773	2	4	+2	Slight Overrun
5. Fit-up	24,853	2	3	+1	Slight Overrun
6. Welding	22,473	2	2	0	Efficient
7. Drilling	7,670	1	2	+1	Slight Overrun
8. Cleaning & Grinding	31,547	2	4	+2	Moderate Delay
9. Primer	31,547	2	14	+12	Significant Delay
10. Finish Paint	30,840	1	1	0	Efficient

The analysis and the findings are summarized in Table 5 and infers that the majority of fabrication stages particularly Cutting, Splicing, Welding, and Finish Paint operated at or below their BEP, suggesting strong control and consistent execution. A few stages, including Fit-up, SAW Welding, and Drilling, showed mild deviations (+1 to +2 days), which may be acceptable given typical production fluctuations. However, two stages stood out as clear bottlenecks. Primer application averaged 14 days compared to a BEP of 2, representing a substantial delay that could impact downstream scheduling. Cleaning and Grinding also exceeded its BEP by two full days on average, suggesting a need for either process streamlining or capacity realignment. These findings support the motivation for stage-level modeling in this study. By identifying where delays cluster and quantifying how far actual durations deviate from standard expectations, we can target high-variance stages for enhanced machine learning–based forecasting and optimization. Moreover, BEP-based classifications provide a domain-aligned reference point that can be incorporated into profitability-aware ML evaluation metrics. From a theoretical perspective, this study confirms the value of treating time-to-completion not as a monolithic regression target, but as a composition of bounded sub-tasks influenced by domain-specific complexity. Practitioners and researchers in smart manufacturing and construction informatics can extend this approach to integrate uncertainty quantification, real-time sensor feedback, or reinforcement learning-based rescheduling in future work.

Table: 6 Comparative Summary of Start-Date-Based (SD) and Activity-Based (AD) Forecasting Results across Complexity Tiers, including Error Metrics, Profitability Rates, and Break-Even Status

Complexity Tier	SD Based RMSE (Avg)	AD Based RMSE (Avg)	SD Based MSE (Avg)	AD Based MSE (Avg)	SD Based Profitability Rate	AD Based Profitability Rate	Break Even Status
P1C1–P2C1 (Low)	2.12	1.85	4.49	3.98	98	100	Achieved
P1C2–P2C2 (Medium)	2.5	2.14	6.25	5.11	97	100	Achieved
P1C3–P2C3 (High)	3.1	2.72	9.61	7.39	83	100	Achieved

To provide a consolidated view of both technical accuracy and business alignment, Table 6 summarizes start-date-based (SD) and activity-based (AD) forecasting results across all complexity tiers. The table integrates RMSE, MSE, profitability rates, and break-even status into a single comparative framework, highlighting the consistent advantage of activity-level predictions. This consolidated comparison is provided in Table 6, shows activity-based (AD) predictions consistently yield lower RMSE and MSE values across all complexity tiers compared to start-date-based (SD) predictions. Notably, while SD-based profitability rates decline with higher project complexity (98% → 97% → 83%), AD-based predictions maintain a 100% profitability rate across all tiers. Furthermore, the break-even status is achieved in every case, underscoring the robustness of the proposed framework.

6. Conclusion

This study developed a machine learning–driven approach for predicting fabrication durations in Pre-Engineered Building (PEB) projects using real-time production data from a structured PRED system. Leveraging an ensemble Regressor (Random Forest), the model was trained on a stratified dataset segmented by project complexity and structural configuration (P1C1–P2C3). Input features included total tonnage, number of parts, and statistical descriptors (mean, median, mode) of task durations, engineered from part-level fabrication records. Two target prediction strategies were evaluated start-date-based and activity-specific. The former suffered increased RMSE and MSE as project complexity rose, while the latter maintained low error rates and exceptional prediction consistency. Specifically, activity-level models achieved over 99% accuracy in profitability-aligned predictions across all complexity tiers, with no Excess in high-complexity projects. These results confirm that granular, task-isolated modeling significantly improves forecast robustness, especially under conditions of sequencing variability and concurrent task execution.

The integration of group-aware preprocessing, ensemble learning, and profitability-based evaluation establishes a scalable and interpretable solution for predictive scheduling in industrial fabrication. By aligning forecast accuracy with profitability classes—Profitable, Acceptable, and Excess—the framework translates technical predictions into direct financial implications. This enables fabrication managers to anticipate schedule risks, detect bottlenecks early, and optimize resource allocation, ultimately strengthening competitiveness in cost- and time-sensitive markets.

Technically, the results highlight the value of statistical feature engineering in stabilizing predictions, as descriptors such as mean and median mitigated the influence of noise and outliers. The classification-based stratification further ensured balanced learning across projects of differing complexity, minimizing bias–variance trade-offs. Together, these design choices explain the framework’s superior performance compared to conventional approaches.

The demonstrated robustness of activity-level forecasts provides actionable insights for PEB firms by reducing uncertainty in project timelines and profitability planning. This makes the framework a valuable decision-support tool for managers seeking to balance operational efficiency with financial performance in competitive fabrication environments. Future work may incorporate

repeated cross-validation, regularization techniques, or blended ensemble approaches to further reinforce generalization.

While the results are promising, further research may expand on this foundation. Future work could explore repeated cross-validation, regularization techniques, or blended ensemble approaches to strengthen generalization. Integration with IoT-enabled real-time feedback systems may enable adaptive rescheduling under disruptions, while reinforcement learning or probabilistic modeling could improve handling of uncertainty in highly variable projects. Finally, extending the framework to include supply chain and resource allocation considerations would allow for an even more holistic view of PEB project planning.

References

- [1] Ramesh V, Pandulu G, Khanam N, Lian O C. Compressive strength prediction of ecofriendly recycled aggregate concrete: A machine learning approach. *Res. Eng. Struct. Mater.*, 2025; 11(6): 2965-2982. <http://dx.doi.org/10.17515/resm2025-595me1212rs>
- [2] Raju SMTU, Sarker A, Das A, Islam MM, Al-Rakhami MS, Al-Amri AM, Mohiuddin T, Albogamy FR. An approach for demand forecasting in steel industries using ensemble learning. *Complexity*. 2022;9928836. <https://doi.org/10.1155/2022/9928836>
- [3] Zermane H, Madjour H, Bouzghaya MA. Prediction of the amount of raw material in an Algerian cement factory. *Eurasia Proc Sci Technol Eng Math*. 2022;19:41-46. <https://doi.org/10.55549/epstem.1218718>
- [4] Zermane H, Madjour H, Ziar A, Zermane A. Forecasting material quantity using machine learning and time series techniques. *J Electr Eng*. 2024;75(3):237-248. <https://doi.org/10.2478/jee-2024-0029>
- [5] Raja Gopal G ST. Analysis of factors affecting labour productivity in construction. *Int J Recent Sci Res*. 2016;7(6).
- [6] Salihu AA, Ibrahim Y, Muhammad S. Impact of labour productivity factors on construction project cost and time. *LAUTECH J Civ Environ Stud*. 2022;8(1). <https://doi.org/10.36108/laujoces/2202.80.0110>
- [7] Soham M, Rajiv B. Critical factors affecting labour productivity in construction projects: Case study of South Gujarat region of India. *Int J Eng Adv Technol*. 2013;(2).
- [8] Shinde PP, Shah SA. A review of machine learning and deep learning applications. *IEEE*; 2018. <https://doi.org/10.1109/ICCUBE.2018.8697857>
- [9] Yusoff MIM. Machine learning: An overview. *Open J Model Simul*. 2024;12(3):89-99. <https://doi.org/10.4236/ojmsi.2024.123006>
- [10] Tao F, Qi Q, Liu A, Kusiak A. Data-driven smart manufacturing. *J Manuf Syst*. 2018;48:157-169. <https://doi.org/10.1016/j.jmsy.2018.01.006>
- [11] Abdulfattah BS, Abdelsalam HA, Abdelsalam M, Bolpagni M, Thurairajah N, Perez LF, Butt TE. Predicting implications of design changes in BIM-based construction projects through machine learning. *Automation in Construction*. 2023;155:105057. <https://doi.org/10.1016/j.autcon.2023.105057>
- [12] Fahle S, Prinz C, Kuhlenkötter B. Systematic review on machine learning (ML) methods for manufacturing processes - Identifying artificial intelligence (AI) methods for field application. *Procedia CIRP*. 2020;93:413-418. <https://doi.org/10.1016/j.procir.2020.04.109>
- [13] Saini S, Sharma N, Chopra R K, Mehra M, Kumar R, Bhardwaj S, Kumar D. Data-driven prediction of mechanical properties in friction stir processed Al6061-Alumina composite using enhanced machine learning models. *Res. Eng. Struct. Mater.*, 2025; 11(5): 2097-2117. <http://dx.doi.org/10.17515/resm2024.438me0907rs>
- [14] Sadatnya A, Sadeghi N, Sabzekar S, Khanjani M, Tak AN, Taghaddos H. Machine learning for construction crew productivity prediction using daily work reports. *Automation in Construction*. 2023;152:104891. <https://doi.org/10.1016/j.autcon.2023.104891>
- [15] Villegas-Ch W, Navarro AM, Sanchez-Viteri S. Optimization of inventory management through computer vision and machine learning technologies. *Intell Syst Appl*. 2024;24:200438. <https://doi.org/10.1016/j.iswa.2024.200438>
- [16] Ali SM, Paul SK, Ahsan K, Azeem A. Forecasting of optimum raw material inventory level using artificial neural network. *International J. of Ops. and Quant. Management*, 2011;17(4): 333-348.
- [17] Duymaz Ş. The application of machine learning algorithms in the estimation of production lead times: A case study of a steel construction manufacturing company. *J Adv Manuf Eng*. 2024;21-28. <https://doi.org/10.14744/ytu.jame.2024.00004>
- [18] You Z, Wu C. A framework for data-driven informatization of the construction company. *Adv Eng Inform*. 2019;39:269-277. <https://doi.org/10.1016/j.aei.2019.02.002>
- [19] Elbeltagi E, Ammar M, Sanad H, Kassab M. Overall multiobjective optimization of construction projects scheduling using particle swarm. *Eng Constr Archit Manag*. 2016;23(3):265-282. <https://doi.org/10.1108/ECAM-11-2014-0135>

- [20] Kang Z, Catal C, Tekinerdogan B. Machine learning applications in production lines: A systematic literature review. *Comput Ind Eng.* 2020;149:106773. <https://doi.org/10.1016/j.cie.2020.106773>
- [21] Tiwari S. Artificial intelligence implications in engineering and production. *ASEC.* 2022;31(1):16. <https://doi.org/10.3390/ASEC2022-13823>
- [22] Li Y, Carabelli S, Fadda E, Manerba D, Tadei R, Terzo O. Machine learning and optimization for production rescheduling in Industry 4.0. *Int J Adv Manuf Technol.* 2020;110(9-10):2445-2463. <https://doi.org/10.1007/s00170-020-05850-5>
- [23] Guo K, Yang Z, Yu CH, Buehler MJ. Artificial intelligence and machine learning in design of mechanical materials. *Mater Horiz.* 2021;8(4):1153-1172. <https://doi.org/10.1039/D0MH01451F>
- [24] Gao Y, Wang J, Xu X. Machine learning in construction and demolition waste management: Progress, challenges, and future directions. *Automation in Construction.* 2024;162:105380. <https://doi.org/10.1016/j.autcon.2024.105380>
- [25] Bojer CS. Understanding machine learning-based forecasting methods: A decomposition framework and research opportunities. *Int J Forecasting.* 2022;38(4):1555-1561. <https://doi.org/10.1016/j.ijforecast.2021.11.003>
- [26] Jin B, Xu X. Machine learning-based scrap steel price forecasting for the Northeast Chinese market. *Int J Empir Econ.* 2024. <https://doi.org/10.1142/S2810943024500112>
- [27] Hwang SH, Mangalathu S, Shin J, Jeon JS. Estimation of economic seismic loss of steel moment-frame buildings using a machine learning algorithm. *Eng Struct.* 2022;254:113877. <https://doi.org/10.1016/j.engstruct.2022.113877>
- [28] Guo S, Yu J, Liu X, Wang C, Jiang Q. A predicting model for properties of steel using the industrial big data based on machine learning. *Comput Mater Sci.* 2019;160:95-104. <https://doi.org/10.1016/j.commatsci.2018.12.056>
- [29] Kawakami K, Kobayashi H, Nakata K. Seasonal inventory management model for raw materials in steel industry. *INFORMS J Appl Anal.* 2021;51(4):312-324. <https://doi.org/10.1287/inte.2021.1073>

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RK-OBGRNet-based high-precision sensing model for landslide detection and geological disaster emergency response

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Article Info	Abstract
Article History: Received 17 Sep 2025 Accepted 26 Nov 2025 Keywords: Geological disasters, emergency response; Early warning systems, Landslide hazards; Sensing and monitoring; Runge Kutta Optimized Backpropagate Gated Layered RecurrenceNet (RK-OBGRNet)	Geological disasters, such as landslides, are vital in terms of life and property preservation and requires timely and accurate monitoring to adequately react to them and respond appropriately. Such catastrophes are to be monitored and acted on by advanced sensor technology that need advanced sensing technologies to operate in-person to detect and track them. This work utilizes satellite remote sensing data with several environmental and geological variables to produce high-precision landslide influence factors. These are the foundations of advanced sensors and monitoring techniques in geological disaster response. The RK-OBGRNet, an integrated recurrence and monitoring model for landslides which uses remote sensing images, is designed to detect and monitor landslides. The preprocessing methods for satellite images are z-score normalization, Fourier Transform, FT which improves the quality of the satellite images by eliminating noise and making data synchronized to the analyzer. It utilizes feature extraction such as principal component analysis to identify characteristics of landslides that dominate their presence. The impact of model parameters on landslide detection accuracy has been analyzed. The performance of RK-OBGRNet is better compared to the RNN-Autoencoder and Cascade R-CNN with improved accuracy of 95%, precision of 94%, recall of 93%, and F1-Score of 92%. The results indicate that RK-OBGRNet achieves better and demonstrates its effectiveness for high-precision landslide detection. This research provides valuable insights for enhancing geological disaster monitoring and emergency response. It offers a reference for applying high-precision sensing technologies to early warning systems and rapid response strategies in the management of landslide hazards.

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1. Introduction

In disaster management and disaster risk reduction (DRR), the majority of nations utilize the single-hazard approach because hazards are viewed and handled as distinct, separate events. A proactive approach to disaster risk reduction by minimizing vulnerability and strengthening vulnerability against natural hazards. But occasionally, several danger categories overlap and interact in the ways listed below: Human activities can cause natural hazards by generating one or more hazard events, by generating natural hazards, intensifying the triggering of natural hazards, establishing networks of interconnections between hazards, and generating two or more hazard events simultaneously [1]. Research on emergency planning and Geographic Information System (GIS) based emergency command systems for geologic hazards is essential. A system that captures, stores, analyzes, and visualizes geographic and spatial data for decision-making and planning.

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All levels of emergency management agencies have noticed improvements in rescue operations and response times due to emergency plans and command systems [2]. Geological catastrophes are frequent natural occurrences that have a significant impact on human existence because of their abruptness and extreme destructiveness. Therefore, multiple investigations both domestically and internationally have developed catastrophe emergency management programs, for the sake of human survival and the normal development of human civilization [3]. As the social economy expands, large-scale projects, like the railways, may be constructed. Therefore, an increasing number of economies at risk from the hidden dangers of geological disasters, especially long-term landslide threats. However, earthquakes and geological hazard chains may become more apparent when the effects of extreme weather, such as typhoons, drought, and heavy rain, worsen [4]. Reducing catastrophe losses after natural disasters occur is mostly dependent on decision-making based on big data, both geographical and non-spatial, to enable quick emergency response and rescue. A disaster emergency management system's primary component, GIS technology, aids decision-makers in rapidly integrating, processing, and analyzing catastrophe data [5]. The shortest distance, the shortest time, and the highest rescue effect are the goals of emergency resource dispatch models, which typically have particular features. To disperse resources for prompt rescue, emergency management departments typically set up many resource centers. The quantity and kind of vehicles at each resource center are unknown due to the abruptness of geological disasters. Furthermore, roadways can sustain damage and new places can be affected by the effects of secondary catastrophes [6]. The RK-OBGRNet model is aided by GIS systems for integration of spatial data, multi-hazard management principles for the integrated response to disasters, and satellite communication for real-time transmission of data.

Emergency communication greatly benefits from satellite networks, while the conventional approach mostly uses a single satellite vehicle or satellite portable station to build the network. While this plan can help with communication issues to a certain degree, satellite vehicles have limited network coverage and poor mobility in crises. The role of satellite vehicles is limited by the aforementioned issues [7]. Seismological monitoring has been made possible by advances in seismology, and the Global Seismographic Network (GSN) offers almost consistent worldwide monitoring. The network, which has more than 150 stations globally, reduces the number of fatalities and financial losses caused by significant earthquakes [8]. Meteorology, geology, and seismology are among the sciences that apply technological methods to avoid and lessen natural disasters. GPS measures crustal movements, weather radar detects precipitation, satellites collect data, and autonomous weather stations track data in real-time [9]. Imaging spectroscopy, another name for hyperspectral imaging, is a technique that uses space-based, airborne, or unmanned aerial systems or lab-based imaging systems to capture images of geologic or outdoor environments. Each pixel of the images contains a reflective/emissive spectrum of the material present. Each pixel can be analyzed to identify items in the obtained image or scene, given context, and a reference library [10]. One of the primary trends in global catastrophe prevention and reduction is community-based disaster risk management (CBDRM) extensively adopted and used by national, international, and local organizations. A participatory process of participatory participation of local communities in how they identify, assess, and manage disaster risk to reduce vulnerability. In recent years, some nations have embraced the development of community-centered disaster reduction strategies to construct the policies, plans, and schemes of CBDRM to address the increasing difficulties of catastrophe risks [11]. Geological disasters, including collapses, landslides, and debris flows, happen all over the world and have a major negative impact on property, human life, the environment, and sustainable economic growth, particularly in developed countries [12]. Runge-Kutta optimization enhances convergence and stability through the optimization of model parameters, and gated layers increase the temporal learning capability, providing dynamic adjustment towards geological information over time. The limitations include,

- Handling real-time, high-dimensional data: Processing high-dimensional data in real-time remains computationally intensive, requiring advanced optimization methods that can be prone to resource constraints.

- Optimizing sensor placement through Runge Kutta: Runge Kutta optimization may struggle with non-stationary or unpredictable data, potentially leading to suboptimal sensor placement in rapidly changing environments.
- Using gated layers for better temporal data learning: Gated layers can become prone to overfitting when trained on insufficient or noisy data, limiting their generalization ability in diverse real-world disaster scenarios.

The integration of high-precision sensing and monitoring technologies in geological disaster emergency response plays a crucial role in promoting climate-responsive and sustainable infrastructure development. With the increasing frequency of extreme climatic events, such as intense rainfall, drought, and temperature fluctuations, the stability and resilience of smart construction systems have become vital concerns. The proposed RK-OBGRNet-based sensing framework not only enhances the accuracy of geological disaster prediction but also contributes to climate-adaptive infrastructure planning by enabling real-time monitoring of environmental parameters such as soil moisture, rainfall intensity, and ground deformation. Through intelligent sensor deployment, AI-driven analytics, and adaptive learning, the system supports early warning and proactive maintenance strategies that reduce the vulnerability of critical infrastructure to climate-induced geohazards. By integrating geological sensing with climate-responsive design principles, this research provides a pathway toward smart, sustainable, and resilient construction ecosystem that can dynamically adapt to evolving environmental conditions and ensure long-term structural stability under the influence of changing climate patterns.

1.1. Objective of the Research

In research focused on landslide detection, the aim is to develop high-precision sensing and monitoring systems to enhance emergency response for geological catastrophes. In improving accuracy as well as the efficiency in landslide monitoring and early warnings, this research explores the combination of satellite remote sensing data, complex pre-processing methods, and the use of the RK-OBGRNet model.

1.2. Key Contribution

- For continuous monitoring, high-precision landslide influencing variables in this research are produced by the combination of both geology and environmental data from satellite remote sensing.
- The suggested RK-OBGRNet model enhanced by applying Runge Kutta, proves to have surpassed conventional methods for landslide detection.
- This research helps enhance emergency response tactics and raise the level of effectiveness of disaster management through offering a useful tool for landslide early warning systems.
- The RK-OBGRNet model greatly enhances the accuracy of disaster detection via sophisticated optimization methods and deep learning, providing an effective tool for early warning systems. Its ability to incorporate dynamic sensor placement and temporal learning maximizes real-time monitoring ability, pushing forward disaster management practices.

1.3 The Remaining Research

Phase 2 includes the literature review, Phase 3 discusses the methodology, Phase 4 evaluates the experimental findings and the discussion, and Phase 5 establishes the conclusions.

1.4 Literature Review

The Sichuan-Tibet Railway, a complex and elaborate project across the Qinghai-Tibetan Plateau, was at risk of disasters because its terrain was extremely difficult. The Sichuan-Tibet Fund special project focused on five studies: geological structure, hazard-inducing processes, tunnel engineering and disaster identification to address these problems and advance technology [13]. The probability of landslides, intensity, and location as well as time and place, are considered as the predictive factors for emergency rescue risk in a landslide catastrophe scenario [14]. For a country on the coast, it examined the effects of risk perception and protective motive theory in disaster preparedness in China. In their analysis, people's desire to participate in protective activities was greatly influenced by their own assessment of coping and risk attitudes and this is reflected in the

need to do more than just managing risk [15]. Real-time data collection and enhanced agency collaboration in emergency management are made possible by the Internet of Emergency Services (IoES). It discussed the difficulties, possibilities, risks, restrictions, and possible effects on the public safety of sensors and Internet of Things (IoT) devices [16]. Forecasting and monitoring were necessary to reduce the dangers associated with large-scale landslides in Zhouqu County, Gansu Province, China. Global Navigation Satellite System monitoring and unmanned aerial vehicle imaging were combined techniques for threat reference and emergency decision-making [17]. With an emphasis on their suddenness, mass occurrence, induction, geographical variation, and social effect, the research investigated the reasons for frequent geological disasters in mined regions. It created a risk assessment system with 10 indicators and emphasized the relationship between environmental geology and breeding, inducing, and forming geology [18]. Public involvement in disaster mitigation programs in both model and nonmodel towns in a geologically vulnerable area was examined. The findings indicated that while catastrophe experience and perceived behavioral control impacted the behavior of nonmodel societies, model communities participated more in evacuation drills and self-help skills training [19]. Earthquake catastrophes in China, an earthquake-prone region, result in direct loss of life and property, secondary natural disasters, and societal repercussions. Researching the main earthquake risks aids in mitigation prediction and prevention. The special issue compiles the most recent findings in research on significant earthquake hazards, such as active structure evolution, occurrence mechanisms, prediction, risk assessment, mitigation, emergency response, and rescue following big earthquakes [20]. For catastrophe scenarios, remote sensing was essential; however, user applications and science restrict its potential. Through the use of social surveys and case studies, it investigated the non-technical aspects that affect RSES. The objective was to enhance RSES administration and comprehension, provide a framework and software for authorities, and direct RSES procedures during crises [21]. The research examined the safety of pipelines that were suspended as a result of natural disasters, with a particular emphasis on plastic deformation and nonlinear finite element techniques (FEM). When the hung length surpasses 50 meters, irreversible plastic stresses happen, reaching 2% at 340 meters. It suggested promoting the sustainable expansion of oil and natural gas pipelines in areas that frequently encounter natural disasters [22]. To lower the expenses of disaster management and the number of fatalities, the Chinese government implemented the Public Participation Monitoring and Warning (PPMW) system. The method, which was used in landslides in Boli village, organizes inhabitants to evacuate ahead of time and distributes timely emergency information. For other nations dealing with comparable circumstances, the system can develop into an inexpensive catastrophe risk management tool [23]. The Yellow River Basin (YRB) was vulnerable to significant calamities due to its diverse climate, fast evolution, and active geological processes. Ecosystems can be destroyed by these catastrophes, which can have serious repercussions. The research examined the YRB's geological processes, significant catastrophe impacts, and risk mitigation studies, highlighting important scientific issues and outlining potential directions for further research based on earth system science ideas [24,25]. Kadiyala et al. (2025) presents a cloud-integrated IoT-based healthcare monitoring and emergency response system using deep learning. RK-OBGRNet adopts similar strategies by integrating satellite IoT sensors with cloud computing for real-time geological monitoring and applying deep learning algorithms for analyzing spatiotemporal data. This integration improves real-time monitoring, enhances disaster prediction accuracy, and ensures scalability for various disaster scenarios, offering a robust solution for geological hazard management [26]. It incorporates ConvLSTM networks and satellite images to enhance predictive modeling for natural hazards such as floods, landslides, and earthquakes, describing its capability in learning spatiotemporal relationships between remote sensing data. It presents a modern example of AI/ML use in disaster sensing, which might be contrasted with the models such as U-Net. Adding this study in your literature review provides a critical evaluation of the development in disaster management technologies [27].

2. Methodology

The technique includes RK-OBGRNet optimization for real-time landslide detection, sensor placement, improved geological disaster monitoring, PCA for feature extraction, and data preprocessing using Z-Score normalization and Fourier Transform. Primary geological disaster

data, including information on landslides, tsunamis, floods, and earthquakes, were directly collected through satellite imagery. Figure 1 represents the proposed methodology flow.

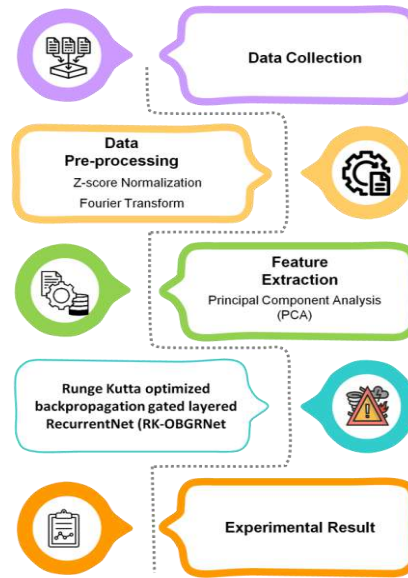


Fig. 1. Flow of proposed methodology

2.1 Data Preprocessing Using Z-Score Normalization and Fourier Transform (FT) For Geological Disaster Monitoring

2.1.1. Z-Score Normalization

In the preprocessing stage of the geological disaster monitoring system, the Z-Score normalization method is used to normalize the data collected from different high-precision sensing technologies, such as seismic activity, soil movement, and atmospheric pressure. The technique is useful in real-time disaster response systems, as it ensures all the features are on the same scale and, hence, improves the accuracy and performance of DL models. Equation (1) follows for standardizing the data:

$$Z = N - \frac{\text{mean}(N)}{\text{standard deviation}(N)} \quad (1)$$

By removing the biasing in units of data, there is an improvement of RK-OBGRNet models regarding predictions about geological risk hazards and emergency response. This implies with the processing of various source data, it ensures the model can perform, and generate timely results, which will lead to improving tactics for response and disaster management.

2.1.2. Fourier Transform (FT)

The application of FT techniques improves the procedure for the feature extraction of geocatastrophe monitoring. Combined with the data of high accuracy, FT enables real-time processing of frequency terms in data signals related to environmental and seismic events. It converts time-domain information to frequency-domain representation as it unfolds critical frequency content, such as ground acceleration or angular velocity; which is required to understand geologic activity, such as an earthquake, landslides, or volcanic activity. These are frequency components on which traditional correct identification and prediction of disaster scenarios are based. The mathematical expression of a constant FT of a time-domain signal $Y(s)$, which is an expression of geological monitoring equipment or seismic data, is given by equation (2):

$$y(e) = \int_{-\infty}^{\infty} Y(s) f^{-j2\pi ds} ds \quad (2)$$

Here, $y(e)$ corresponds to the signal in frequency domains, $y(s)$ or the signal in a time domain, and symbols c and j correspond for frequency and imaginary units respectively. This domain transformation is required in order to produce elements according to frequency analysis in order to characterize behavior geology in a natural catastrophe. Subsequently after the conversion formula in equation (3), the average of converted signal with respect to mean frequency can be calculated as an imperative component of disaster management surveillance:

$$\text{Mean Frequency} = \frac{1}{n} \sum_{k=0}^{n-1} |y[k]| \quad (3)$$

The equation represents the amplitude of each frequency component by $Y[k]$ whereas n is the number of frequency samples. The proposed solution can improve on monitoring knowledge accuracy through the study of such frequency aspects, therefore, facilitating real-time monitoring and prediction of geological disasters. Z-score normalization was used over min-max scaling because it is able to only normalize data and does not eliminate bias, and all features are equally contributing and their range is not limited hence more stable with other data distributions. The application of Fourier Transform (FT) was based on the fact that it is able to convert time-domain signals into frequency-domain characteristics, which allows a researcher to study a periodic nature and anomalies typical with geological events, including seismic occurrences. This mixture enhances the model to effectively handle dynamic and noisy geological data.

2.2 Feature Extraction Using Principal Component Analysis (PCA)

PCA is a significant preprocess technique, which can be applied to data of high-resolution sensor systems. Typically, such geological factors are the monitoring of the climatic conditions, soil movement, and seismic activity, which generate large amounts of information and have a high level of dimensionality. This is easy to understand and can easily be facilitated to achieve the real-time analysis of response to disasters because the most salient features of this data created by the process of PCA are removed. The initial procedure is the normalization of data matrix Z that is a representation of geological measurements. Principal Component Analysis (PCA) is important in lowering the dimensionality of geological data, determining the most significant features for precise disaster forecasting by recognizing major patterns in seismicity, soil movement, and other environmental conditions. In this case, parameters might be on varied scales e.g., pressure in Pascal, displacement in meters hence the standardization procedure would ensure that all features contribute equally to this research. Subtracting the mean of each feature and dividing by the standard deviation yields the standardized data matrix, or Z_{stand} in equation (4):

$$Z_{stand} = \frac{Z - \text{mean}(Z)}{\text{std}(Z)} \quad (4)$$

The matrix of covariance B is calculated to comprehend the connections among the dataset's characteristics. Understanding the correlations and fluctuations between feature pairs is provided by the covariance matrix, which depicts how fluctuate together. The following standardized data is used to compute it in equation (5):

$$X = \frac{1}{p-1} Y_{stand}^B Z_{stand} \quad (5)$$

The standardized form of the data matrix is called Y_{stand} , and p is the number of observations in the dataset. To determine the most significant directions of variation in the data, the covariance matrix is essential. The covariance matrix A is used to compute the eigenvalues and eigenvectors. The associated eigenvectors characterize the directions of maximal variance, whereas the eigenvalues, represented by λ , show the variance along with each primary component. The following equation (6) is an expression for the connection between eigenvalues and eigenvectors:

$$Xa = \lambda a \quad (6)$$

This equation demonstrates how each eigenvalue λ scales the corresponding eigenvector a . The primary components, or the directions in which the data fluctuates are represented by the eigenvectors. Components with eigenvalues greater than one are typically retained when using the Kaiser criterion, as they account for the majority of the variation in the data. The following is how top T components are chosen in equation (7):

$$\text{Top } T \text{ components } a_1, a_2, \dots, a_T \quad (7)$$

These particular elements are the most crucial for comprehending the main causes of geological disasters. The regular data Z_{stand} must be predictable into the new coordinate system that the chosen primary components have specified. While keeping the most significant attributes, this alteration lowers the dimensionality of the data. The covered dataset Z_{PCA} is provided by the equation (8):

$$Z_{PCA} = Z_{stand} U_T \quad (8)$$

U_T is the matrix of the selected eigenvectors. By reducing the dataset and emphasizing its prime components, this revolution facilitates data analysis and understanding. The most important elements of the concentrated data can be highlighted while maintaining critical information by reconstructing it to roughly be similar to the dataset. During the renewal process, the following equations (9& 10) are used:

$$F = U_T^B \quad (9)$$

$$V = \text{mean}(Z) + U_T f \quad (10)$$

The reconstructed data is denoted by F in these equations, and the reconstructed dataset with the mean added back is denoted by V . By using PCA on the data collected by high-precision sensing systems, the research determines the main causes of geological catastrophes, such as changes in the environment, shifting seismic activity, or soil displacement. PCA was employed to compress data dimensionality while retaining essential features, maximizing high-precision sensor data fusion, and enabling effective analysis in dynamic geological monitoring systems. PCA outputs, which constitute important geological features, were incorporated in the RK-OBGRNet model to enhance the predictive performance by enabling the modeling process with a compressed, but meaningful, feature set. The model relies on the assumption of frequent satellite image data availability with high-frequent updates, and the data-collecting sensors are usually high-precision geological sensors that can sense seismic activity, soil movement, and changes in atmospheric pressure.

2.3 Landslide Detection and Monitoring Using Runge Kutta Optimized Backpropagate Gated Layered RecurrenceNet (RK-OBGRNet) from Remote Sensing Images

The images of remote sensing are used in landslide detection and monitoring landslide occurrences. Nevertheless, the accuracy of the prediction can be increased by employing the design of the OBGRNet, which incorporates the usage of backpropagation and the gated recurrent layers; however, the efficiency of the model can be optimized using RK. This improves the accurate geographical and time nature of the landslide processes in real-time monitors. RK-OBGRNet is particularly useful in geological disaster observation, as it is capable of work with high-dimensional time-series, adapting to changes in disasters with gated layers, RK optimization of sensor placement positioning, and real-time observation at high precision.

2.3.1 Backpropagation

The Backpropagation Algorithm is used to optimize sensor data, predict danger and validate the results through error functions, including the Mean Squared Error (MSE) when high precision

sensing data is required to respond to geological disasters. Minimizing prediction accuracy through error correction Backpropagation can be used to minimize prediction errors, gated layers allow the learning of real-time temporal predictions and RK optimization can be used to maximize the performance of sensors by the prediction of geological occurrences.

Forward Pass Calculations: The weather and cloud cover are external factors that are known to distort satellite remote sensing data and this may affect the prediction by a model. RK-OBGRNet responses to this by incorporating preprocessing techniques such as Fourier Transform and PCA in order to eliminate noise and capture useful information in noisy satellite pictures. The normalized input variables are received by the input layer and are positioned based on geological features (ground displacement or seismic). Each input is given significance by initializing the weights X_j , l , and i at random. The output W_1 for the first hidden layer is computed as follows in equation (11):

$$Z_{1,1} = e(net_{1,1}) \quad (11)$$

Where the first layer's input is in equation (12):

$$net_{1,1} = \sum_{j=1}^n X_{j,1,1} \quad (12)$$

- **Activation Function:** The Sigmoid activation function is used to normalize the output of each neuron in equation (13):

$$\hat{Z}_c = \frac{1}{1 + f^{\sum_i (net_{l,i} + g_i)}} \quad (13)$$

Where, the predictable output, such as the anticipated danger of a geological disaster, is represented by $\hat{Z}_c$.

- **Error Function and Validation:** The discrepancy between the expected and intended output Z_b . The MSE is used to measure $\hat{Z}_c$ in equation (14):

$$MSE = \frac{1}{m} \sum_{c=1}^m (Z_b - \hat{Z}_c)^2 \quad (14)$$

To guarantee accurate forecasts for geological disaster management, this error function is utilized to assess the model's correctness.

2.3.2 Gated Layered

Multi-layered sensing systems that are able to analyze are a gated layered approach. selective choice of important data in real-time. This method prioritizes and filters data of different layers of sensors, based on urgency and relevance, when there is a geological crisis, e.g. an earthquake, landslide, or volcanic eruption, using gated mechanisms. It enhances the accuracy and efficiency of monitoring systems by integrating the state-of-the-art sensors with intelligent data filtering mechanisms, thereby making prompt and informed decisions respectively on disaster response and mitigation.

2.3.3 Runge Kutta (RK) Optimization

RK optimization algorithm is a strong optimization technique, which applies the concepts of the Runge-Kutta algorithm in solving ordinary differential equations. The RK in the geological disaster emergency response is applied to enhance the precision of real-time sensor selection and location to sense and monitor. This is because the estimation of gradients was made through the RK technique to guide the search of optimal design and placement of the sensors. This is to trade exploration and exploitation with the solution space to attain high-precision sensing information regarding early warning and disaster relief. RKOBGRNet framework enables an easy coordination of sensing units by integrating resilient communication schemes, which adjusts the placement of sensors and transmit data in response to real-time feedback and monitoring requirements.

Backpropagation, RK optimization, and gated layers provide a high level of synergy by combining the three mechanisms, namely, error correction, dynamic sensor placement, and temporal learning. Backpropagation is used to optimize model accuracy, RK optimization is used to optimize locations of response sensors in real time and gated layers enhance temporal behavior of geological processes to model performance, in comparison with traditional models.

- Search Mechanism to Disaster Monitoring:

The RK optimization identifies regions of the solution space that represent the zones of interest that include seismic activity or prone landscape areas or areas that are at risk of flooding in geological disaster monitoring. The system is operated to guarantee a balance between exploration (of finding new potentially valuable sensor placements) and exploitation (finer-tuning of existing sensor placements based upon the information gathered), with initial sensor placements being randomly assigned to these areas. As examples of the RK optimization of geological sensing, the following equations (15 and 16) are illustrations of the search mechanism (SM):

$$SM = \frac{1}{6} w_{RK} \quad (15)$$

$$w_{RK} = l_1 + 2. l_2 + 2. l_3 + l_4 \quad (16)$$

Where, w is the positional adjustment of sensors inside the monitoring network and l_1, l_2, l_3 , and l_4 are coefficients obtained through the RK approach.

- Solution Update in Disaster Monitoring Systems:

The initial inputs of the RK optimization method are a randomly distributed population of sensors, which are potential locations of monitoring. The locations of these sensors are updated at every cycle with the RK technique. The sensor network is altered to suit the real-time information of geological processes such as earthquakes, landslides, or floods. The sensor location updating process is regulated by the following equations (17 & 18) that ensure the exploration of the new territories as well as the usage of the existing information to manage the disaster:

$$\text{if } rand < 0.5 \quad w_{m+1} = (w_d + q.SF.h.w_d) + SF.SM + \mu.randm.(w_n - w_d) \quad \text{exploration} \quad (17)$$

$$\text{else } w_{m+1} = (w_n + q.SF.h.w_n) + SF.SM + \mu.randm.(w_q1 - w_q2) \quad \text{exploitation} \quad (18)$$

With SF the adaptive scaling factor that varies the search intensity in accordance with the iteration progression, and q and h random integers controlling exploration and exploitation respectively equation (18).

- Enhanced Solution Quality (ESQ) in Monitoring Systems:

To ensure that the sensor placement quality is steadily improved with the iteration progression, the RK algorithm ensures that q and h are random integers that regulate exploration and exploitation, respectively. This enhancement called the Enhanced Solution Quality (ESQ) is ensured by a feedback mechanism which entails real-time sensor data, which ensures that the system keeps on improving its search to the best monitoring locations. The ESQ process of sensor placement are the equations below (19-21):

$$\text{if } rand < 0.5 \quad (19)$$

$$\text{if } x < 1 \quad w_{new2} = w_{new1} + q.x.|w_{new1} - w_{avg}| + randm \quad (20)$$

$$\text{else } w_{new2} = (w_{new1} - w_{avg}) + q.x.|v.w_{new1} - w_{avg}| + randm \quad (21)$$

Where x is a parameter which is dynamically modified to control the work of the algorithm. Ensuring the new sensor placement points are moved towards the region of high-risk activities like the fault lines or floodplains by continuously setting the average location of sensors w_{avg} to optimally and efficiently improve sensor network. The program experiment with additional configurations in the event that the addition of further sensors does not improve the data collecting (i.e., the fitness function is not maximized). Improvement of sensor location is achieved by shifting the sensor in the direction which is shown by the subsequent equation (22):

$$\begin{aligned} \text{if } rand < x \quad w_{new3} \\ = (w_{new2} - rand \cdot w_{new2}) + SF \cdot (rand \cdot w_{RK} + u \cdot w_a - w_{new2}) \end{aligned} \quad (22)$$

To ensure the algorithm continues to search regions which have the highest probability of locating an important geological process, say a tremor, change in the surface, or rising water levels, u is a random integer that determines the search direction. The hybrid RK-OBGRNet is represented in algorithm 1.

- Algorithm 1: Hybrid RK-OBGRNet

Main Loop: Iteration over – optimization and sensor placement

for each iteration in range(MaxIterations):

Backpropagation (BPA) Step to adjust weights based on prediction error

Input data received (e.g., remote sensing images or sensor data)

input_data = normalize(remote_sensing_data)

hidden_layer_output = activate(input_data) Forward pass

predicted_output = forward_pass(hidden_layer_output)

Calculate error using MSE (Mean Squared Error)

error = calculate_MSE(predicted_output, true_output)

Backpropagate error to adjust weights and improve prediction accuracy

backpropagate(error)

Gated Layered Approach: Real – time data filtering based on geological events

if geological_event_detected(): # Check if a disaster event (e.g., landslide) is detected

if data_urgency_level() > threshold:

Prioritize data from critical sensors based on urgency

filtered_data = filter_critical_data(input_data)

Else:

Process all available data (non – urgent)

filtered_data = filter_all_data(input_data)

Else:

filtered_data = input_data # No event detected, use normal data

Runge – Kutta Optimization (RK) for sensor placement

for the sensor in sensors:

Step 1: Update sensor placement using the RK search mechanism

search_mechanism = calculate_search_mechanism(sensor_position)

Step 2: Exploration vs Exploitation (Balance between new and existing placements)

if random() < 0.5:

Exploration: Search for new sensor placements

sensor_position = exploration(sensor_position, search_mechanism)

Else:

Exploitation: Refine current sensor placements

sensor_position = exploitation(sensor_position, search_mechanism)

Step 3: Update sensor placement with Enhanced Solution Quality (ESQ)

if random() < 0.5:

Enhance solution quality to avoid local minima

sensor_position = enhanced_solution_quality(sensor_position)

Sensor Network Update: Adjust sensor locations based on real – time data

For sensor in sensors:

Update sensor position based on new geological event data

sensor_position = update_sensor_position(sensor_position)

Check if optimization has converged (no significant improvement in results)

if check_convergence():

break Exit loop if optimization converges

The system incorporates gated layers for real-time data prioritization during geological events, backpropagation for error correction-based model prediction optimization, and RK optimization for dynamic sensor placement. This combination of RK-OBGRNet methods improves the reaction to geological risks by increasing forecast accuracy, fine-tuning sensor placements, and guaranteeing effective disaster monitoring.

3. Case Studies

3.1 Case Study 1: Wayanad Landslide Disaster (2024)

This paper presents a detailed investigation into the 2024 Wayanad landslide in India, determining the major factors (geological instability, heavy rainfall) and far-reaching effects on indigenous communities. The research indicates how RK-OBGRNet's dynamic sensor deployment and real-time monitoring could have enhanced early warning systems, saved lives and minimized damage. The model's potential to learn temporal relationships and sensor placement optimization would give more effective disaster response plans in other high-risk territories [28].

3.2 Case Study 2: Pettimudi Landslide Case Study (2020)

This research examines the environmental and social effects of the Pettimudi landslide in Kerala that occurred in 2020 with special focus on how geographic and human factors contribute to disaster management. Incorporating RK-OBGRNet in such a case would have been able to yield improved spatial and temporal analysis of vulnerability in the region, providing early warning mechanisms to avoid such a massive loss. The model's high-level feature learning and temporal ability would improve the predictive power for landslide events, with real-time guidance for enhanced evacuation and resource deployment [29].

4. Experimental Results

The recommended fix was implemented on a desktop computer running Windows 10 with a 64-bit processor, an 18th-generation Intel Core i7 CPU, and a 4 GB GPU driver that supports CUDA and other components. The suggested structure was constructed using Keras with TensorFlow software capabilities and Python 3.5 software tools for the backend toolkits. The accuracy, precision, recall, and F1-Score of this research's comparison to the traditional models monitoring technology in geological disaster emergency response RNN-Auto encoder [30], and Cascade R-CNN [31] clarify the extent to which the proposed RK-OBGRNet model accurately monitors the geological disaster emergency response. The data used here is the satellite image obtained using Landsat-8 and Sentinel-2 satellites, targeted for geological disaster observation. Landsat-8 offers a spatial resolution of 30 meters with a temporal resolution of 16 days, whereas Sentinel-2 offers a higher spatial resolution of 10 meters and a temporal resolution of 5 days. The data consists of around 5,000 tagged images in different disaster situations, including landslides, earthquakes, and floods, with ground truth information hand-marked on the basis of past records and field observations. The RK-OBGRNet model integrates convolution layers for feature extraction, gated recurrent units (GRUs) for recognizing temporal relationships in sequential data, and Runge-Kutta optimization for dynamic sensor placement and real-time adaptation of data. The hyperparameters for the model are a learning rate of 0.001, 10 layers, the Adam optimizer, 50 epochs, and a batch size of 32, selected based on initial experiments to balance performance and training time.

4.1. Accuracy Loss

Accuracy loss, as it relates to high-precision sensing monitoring for geological catastrophes, is the deterioration in the model's capacity to accurately identify landslides as a result of inconsistent data or model restrictions. Figure 2 represents the accuracy loss.

4.2. Confusion Matrix

The RK-OBGRNet model's accuracy in landslide detection is evaluated using a confusion matrix, which displays true positives, false positives, true negatives, and false negatives in monitoring. The confusion matrix, which displays the real labels vs, expected labels for landslides, earthquakes, floods, and tsunamis, with color intensity representing occurrences, displays the categorization

performance of geological catastrophe categories. Figure 3 displays the confusion matrix for geological disaster classification.

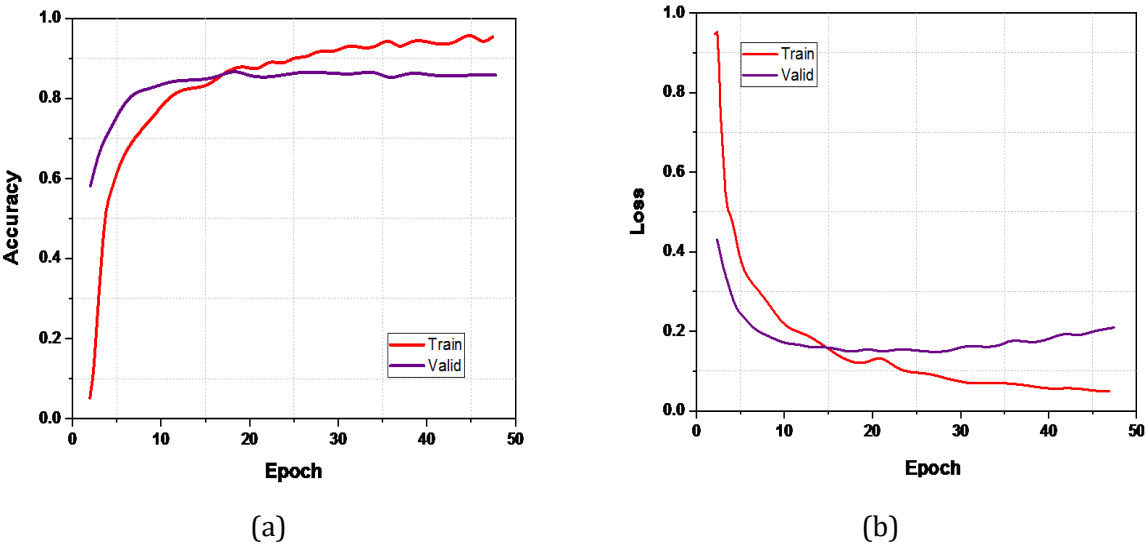


Fig. 2. a) Accuracy and b) Loss

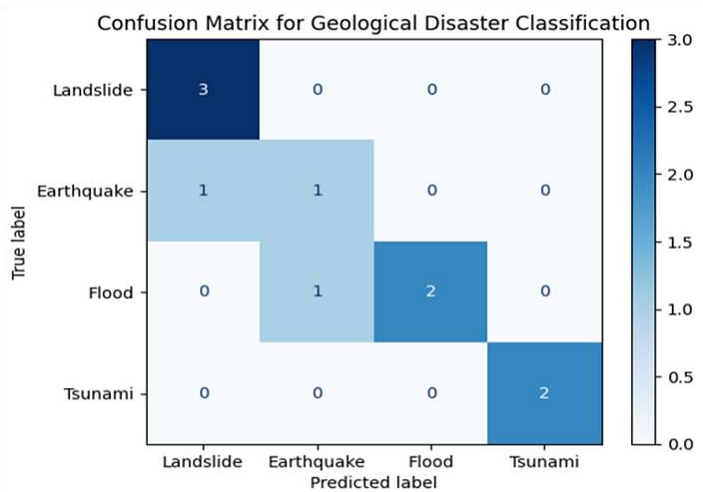


Fig. 3.: Confusion matrix

4.3. Accuracy

The degree to which sensor data, such as from environmental, displacement, or seismic sensors reflects real-world circumstances during geological events is known as accuracy. It is essential to ensure that disasters like earthquakes, landslides, and floods are reliably detected and monitored. This enables prompt warnings and efficient decision-making in emergency response operations. Table 1 and Figure 4 illustrate the comparison results of accuracy.

Table 1. Outcomes of accuracy

Method	Accuracy (%)
RNN-Autoencoder [25]	68%
RK-OBGRNet [Proposed]	95%

For geological disaster monitoring, the RK-OBGRNet technique outperforms the RNN-Autoencoder method, which achieves 68% accuracy, with an accuracy of 95%. These notable enhancements show how well the RK-OBGRNet model detects and tracks geological hazard occurrences using remote sensing data.

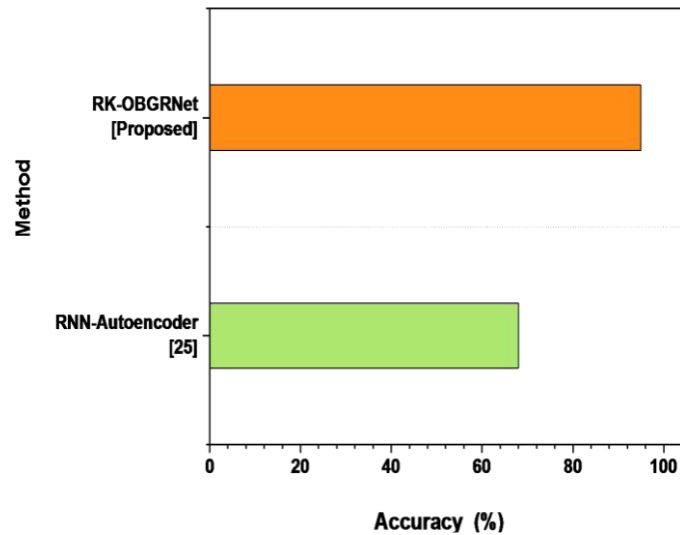


Fig. 4. Analysis of accuracy

4.4. Precision

The precision of identifying and quantifying geological occurrences is referred to as precision in high-precision sensing monitoring technology for geological disorder response. It guarantees low sensor reading error, precise position determination, and trustworthy forecasts, facilitating prompt, well-informed disaster management decision-making and improving response efficiency. Table 2 and Figure 5 depict the comparison results of precision.

Table 2: Outcomes of precision

Method	Precision (%)
RNN-Autoencoder [30]	70%
Cascade R-CNN [31]	93.15%
RK-OBGRNet [Proposed]	94%

The suggested RK-OBGRNet outperforms the RNN-Autoencoder (70%) and Cascade R-CNN (93.15%) with a precision of 94%. This illustrates how well RK-OBGRNet detects geological catastrophes and how well it optimizes high-precision sensing and monitoring systems for emergency response in areas affected by geological disasters.

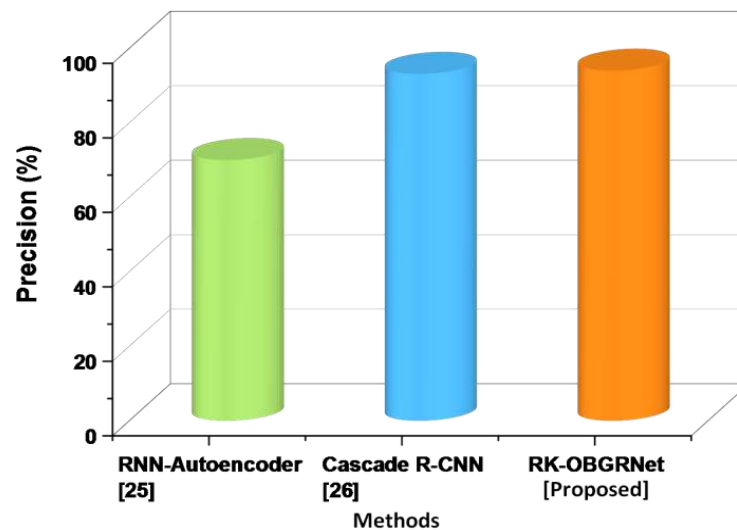


Fig. 5. Analysis of precision

4.5. Recall

Recall refers to a sensory monitoring system's capacity to accurately identify and detect real catastrophe occurrences, such as earthquakes or landslides, in the context of geological disaster emergency response. High recall guarantees low false negatives and enhances the dependability of the system for early warning as well as effective catastrophe mitigation techniques. Table 3 and Figure 6 represent the comparison results of recall.

Table 3. Outcomes of Recall

Method	Recall (%)
RNN-Autoencoder [30]	72%
Cascade R-CNN [31]	89.32%
RK-OBGRNet [Proposed]	93%

From the numerical result, it indicates that RK-OBGRNet performs better than the Cascade R-CNN by 89.32% and then RNN-Autoencoder by 72%. The best value of the recall rate reaches up to 93%. Hence, it presents how RK-OBGRNet exhibits superior capability for accurately identifying and responding to geological disaster occurrences.

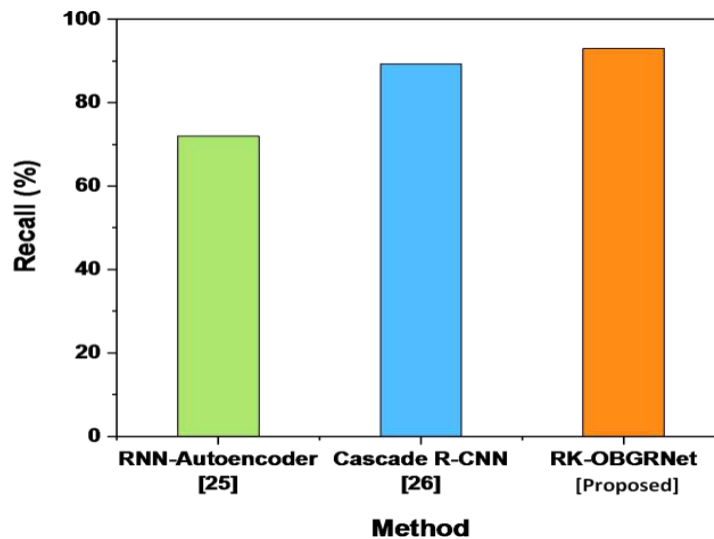


Fig. 6. Analysis of recall

4.6. F1-Score

In high-precision sensing related to geological disaster monitoring, the F1-Score balances precision and recall of estimating its accuracy. It is considered crucial for evaluating how efficiently sensor networks and precision models identify and react to events leading to geological disasters, reducing false alarms while being responsive toward actual events. Table 4 and Figure 7 show the comparison results in terms of F1-score.

Table 4. Outcomes of F1-Score

Method	F1-Score (%)
RNN-Autoencoder [30]	71%
RK-OBGRNet [Proposed]	92%

With an F1-Score of 92% compared to 71% for the RNN-Autoencoder, the suggested RK-OBGRNet model is better than the RNN-Auto encoder model and thus, suitable for geological disaster monitoring. This means that regarding achieving equilibrium between recall and precision, the suggested model offers much higher accuracy in disaster warning and response.

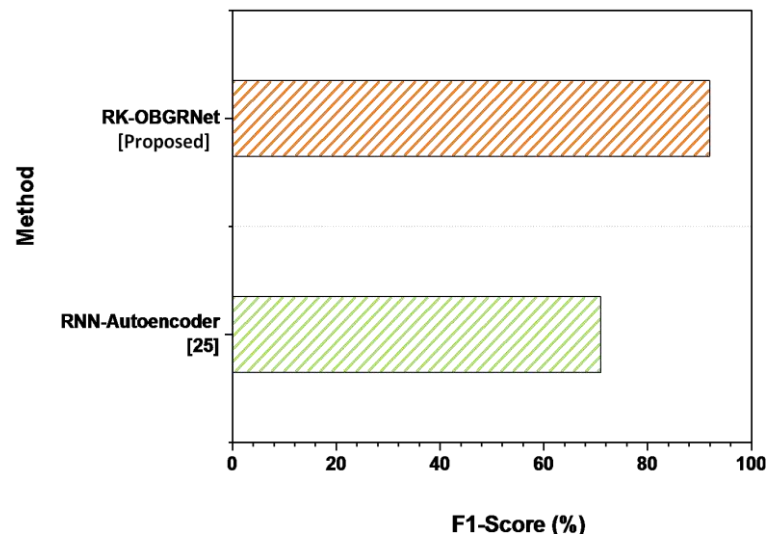


Fig. 7. Analysis of F1-score

4.7. Discussion

Despite its utility for anomaly detection in monitoring geological catastrophes, the RNN-Autoencoder [30] has limitations concerning accuracy and online adaptability. It performs less than other models when it is concerned with predicting and responding to changing geological hazards, as it fails to handle complex spatial-temporal relationships and environmental changes. The computational complexity and difficulty in processing real-time, high-resolution remote sensing data make the approach of Cascade R-CNN [31,32] unsuitable for geological hazard monitoring, even though it performs well for object recognition. This might cause delays in critical disaster reaction situations. The proposed RK-OBGRNet method based on RK optimization of dynamic sensor placement, the application of back propagation to improve accuracy, and the Gated Layered methods of real-time adaptation help to overcome the drawbacks and cope successfully with the complicated spatial-temporal data, which will guarantee the faster and more precise responses to disasters.

5. Conclusion

Geological catastrophes such as landslides are very dangerous to human life and assets, therefore, needs prompt and precise monitoring to control instances of emergencies. Complex sensory technology was crucial in the real-time identification and control of such incidences. This study combines the information of satellite remote sensing and various geological and environmental variables to design high-quality landslide factors that influence factors. On the basis of these features, it was predicted that advanced sensing and monitoring of geological catastrophe emergency response would be created. The RK-OBGRNet was used to identify and monitor landslides using remote sensing photographs in a model. In order to enhance the quality of the satellite images, preprocessing techniques such as FT and z-score normalization are applied to remove noise, and to give the same data to perform the analysis. PCA and other feature extraction methods were embraced in order to distinguish the significant features that characterize presence or absence of landslides. The effect of the model parameters on the accuracy of landslide prediction was assessed. RK-OBGRNet is better than RNN-Autoencoder and Cascade R-CNN with a higher F1-Score of 92%, accuracy of 95%, precision of 94% and recall of 93%. As per the outcomes, RK-OBGRNet got the lowest overall error and it was shown to be applicable in identification of landslides with high precision. On the theoretical level, this study can be used in forming adaptive, real-time, and data-driven solutions in disaster management as they would enhance the effectiveness of early warning systems and response time, which could transform the exercise of disaster risk reduction. RK-OBGRNet is perfectly applicable to existing disaster response systems due to extreme weather conditions, sparsity of data, and real-time adaptation, which may affect the precision of the model during dynamic and large-scale disasters. High computational demands in it would inhibit its scalability especially in systems with resource constraints. The unpredictability

of data based on the quality of satellite images and capacity of the sensors may lower predictions and this would be challenging to the model strength in its real application. The study assists in the improvement of emergency reaction and geological disasters observation. It offered a blueprint on how the management of landslide risks could be done through the combination of high-precision monitoring devices and early warning systems and rapid response strategies. One of the limitations is that it depends on satellite data, which has weather fluctuations. Future studies could explore the combining deep learning with real-time sensor networks can enhance predictive accuracy.

References

- [1] Mavroulis S, Mavrouli M, Kourou A, Thoma T, Lekkas E. Multi-hazard emergency response for geological hazards amid the evolving COVID-19 pandemic: good practices and lessons learned from earthquake disaster management in Greece. *Sustainability*. 2022 Jul;14(14):8486. <https://doi.org/10.3390/su14148486>
- [2] Liu X, Liu Z, Liu Y, Tian J. Integration of a geo-ontology-based knowledge model and spatial analysis into emergency response for geologic hazards. *Nat Hazards*. 2021 Aug;108(2):1489-514. <https://doi.org/10.1007/s11069-021-04742-5>
- [3] Qian X. Regional geological disasters emergency management system monitored by big data platform. *Processes*. 2022 Dec;10(12):2741. <https://doi.org/10.3390/pr10122741>
- [4] Jiang W, Jiao Q, Luo Y, Li Y, Li Q, Li B. Risk investigation of landslide hazard and disaster emergency based on multi-platform remote sensing techniques. In: *IGARSS 2020 - 2020 IEEE International Geoscience and Remote Sensing Symposium*. Waikoloa (HI): IEEE; 2020. p. 5211-4. <https://doi.org/10.1109/IGARSS39084.2020.9324550>
- [5] Yang Z, Li J, Hyppä J, Gong J, Liu J, Yang B. A comprehensive and up-to-date web-based interactive 3D emergency response and visualization system using Cesium Digital Earth: taking landslide disaster as an example. *Big Earth Data*. 2023;7(4):1058-80. <https://doi.org/10.1080/20964471.2023.2172823>
- [6] Fang W, Haixiang G, Jinling L, Mingyun G, Wenwen P. Multi-objective emergency scheduling for geological disasters. *Nat Hazards*. 2021;105:1323-58. <https://doi.org/10.1007/s11069-020-04356-3>
- [7] Zhou J, Zhou C, Kang Y, Tu S. Integrated satellite-ground post-disaster emergency communication networking technology. *Nat Hazards Res*. 2021;1(1):4-10. <https://doi.org/10.1016/j.nhres.2020.12.002>
- [8] Zhao R, Liu X, Xu W. Integration of coseismic deformation into WebGIS for near real-time disaster evaluation and emergency response. *Environ Earth Sci*. 2020;79(18):414. <https://doi.org/10.1007/s12665-020-09153-6>
- [9] Xu C, Xue Z. Applications and challenges of artificial intelligence in the field of disaster prevention, reduction, and relief. *Nat Hazards Res*. 2024;4(1):169-72. <https://doi.org/10.1016/j.nhres.2023.11.011>
- [10] Krekeler MP, Burke M, Allen S, Sather B, Chappell C, McLeod CL, et al. A novel hyperspectral remote sensing tool for detecting and analyzing human materials in the environment: a geoenvironmental approach to aid in emergency response. *Environ Earth Sci*. 2023;82(4):109. <https://doi.org/10.1007/s12665-023-10761-1>
- [11] Xu Y, Qiu X, Yang X, Lu X, Chen G. Disaster risk management models for rural relocation communities of mountainous southwestern China under the stress of geological disasters. *Int J Disaster Risk Reduct*. 2020;50:101697. <https://doi.org/10.1016/j.ijdrr.2020.101697>
- [12] Zhang D, Feng D. Mine geological disaster risk assessment and management based on multisensor information fusion. *Mob Inf Syst*. 2022;2022(1):1757026. <https://doi.org/10.1155/2022/1757026>
- [13] Cui P, Ge Y, Li S, Li Z, Xu X, Zhou GG, et al. Scientific challenges in disaster risk reduction for the Sichuan-Tibet Railway. *Eng Geol*. 2022;309:106837. <https://doi.org/10.1016/j.enggeo.2022.106837>
- [14] Wang M, Wang W, Dai C, Ma C, Luo Y, Xu M. Risk analysis and evaluation of emergency rescue in landslide disaster. *Nat Hazards*. 2024;1-27. <https://doi.org/10.1007/s11069-024-06811-x>
- [15] Hu SY, Yu M, Que T, Fan G, Xing HG. Individual willingness to prepare for disasters in a geological hazard risk area: an empirical study based on the protection motivation theory. *Nat Hazards*. 2022;1-25.
- [16] Damaševičius R, Bacanin N, Misra S. From sensors to safety: Internet of Emergency Services (IoES) for emergency response and disaster management. *J Sens Actuator Netw*. 2023;12(3):41. <https://doi.org/10.3390/jsan12030041>
- [17] Guo FY, Meng XM, Qi TJ, Dijkstra T, Thorkildsen JK, Yue DX, et al. Rapid onset hazards, fault-controlled landslides, and multi-method emergency decision-making. *J Mt Sci*. 2022;19(5):1357-69. <https://doi.org/10.1007/s11629-021-6941-x>
- [18] Yan B, Liu J, Qi Q, Liu W, Li X. Study on chain relationship and risk assessment model of coal mine geological disasters. *Arab J Geosci*. 2022;15(9):900. <https://doi.org/10.1007/s12517-022-10184-5>

- [19] Que T, Wu Y, Hu S, Cai J, Jiang N, Xing H. Factors influencing public participation in community disaster mitigation activities: A comparison of model and nonmodel disaster mitigation communities. *Int J Environ Res Public Health*. 2022;19(19):12278. <https://doi.org/10.3390/ijerph191912278>
- [20] He Z, Wang D, Fang L, Ren Z, Xu X. Preface to the special issue on major earthquake hazards and disaster reduction. *Nat Hazards Res*. 2021;1(2):33-5. <https://doi.org/10.1016/j.nhres.2021.06.006>
- [21] Zheng X, Wang F, Qi M, Meng Q. Planning remote sensing emergency services: Bridging the gap between remote sensing science and emergency practice in China. *Saf Sci*. 2021;141:105346. <https://doi.org/10.1016/j.ssci.2021.105346>
- [22] Yu J, Chen C, Li C. Safety analysis and emergency response of suspended oil and gas pipelines triggered by natural disasters. *Sustainability*. 2022;14(24):17045. <https://doi.org/10.3390/su142417045>
- [23] Wu SN, Lei Y, Cui P, Chen R, Yin PH. Chinese public participation monitoring and warning system for geological hazards. *J Mt Sci*. 2020;17(7):1553-64. <https://doi.org/10.1007/s11629-019-5933-6>
- [24] Lan H, Peng J, Zhu Y, Li L, Pan B, Huang Q, et al. Research on geological and surficial processes and major disaster effects in the Yellow River Basin. *Sci China Earth Sci*. 2022;65(2):234-56. <https://doi.org/10.1007/s11430-021-9830-8>
- [25] Brighenti F, Caspani VF, Costa G, Giordano PF, Limongelli MP, Zonta D. Bridge management systems: A review on current practice in a digitizing world. *Eng Struct*. 2024;321:118971. <https://doi.org/10.1016/j.engstruct.2024.118971>
- [26] Kadiyala B, Alavilli SK, Nippatla RP, Boyapati S, Vasamsetty C. Cloud-Integrated IoT-based healthcare monitoring and emergency response system with deep learning. *Int J Recent Adv Multidiscip Res*. 2025;12(03):10877-81.
- [27] Akash R, Mouli KV, Vidhya V. Leveraging ConvLstm and satellite imagery for predictive modeling of floods, landslides, and earthquakes. *i-Manager's J Future Eng Technol*. 2025;20(2). <https://doi.org/10.26634/jfet.20.2.21545>
- [28] Wang Q, Xu C, Ma J, Huang Y, Zhang S, Gao H. Preliminary analysis of the July 30, 2024, Wayanad landslide disaster in India: causes and impacts. *Nat Hazards Res*. 2025. <https://doi.org/10.1016/j.nhres.2025.04.005>
- [29] Irshad SM, Solaman SC. Identity, space and disaster: a case study of Pettimudi landslide in Kerala. *Sociol Bull*. 2022;71(3):437-53. <https://doi.org/10.1177/00380229221094785>
- [30] Infante D, Di Martire D, Confuorto P, Tessitore S, Tòmas R, Calcaterra D, et al. Assessment of building behavior in slow-moving landslide-affected areas through DInSAR data and structural analysis. *Eng Struct*. 2019;199:109638. <https://doi.org/10.1016/j.engstruct.2019.109638>
- [31] Tang J, Yang R, Dai Q, Yuan G, Mao Y. Research on feature extraction of meteorological disaster emergency response capability based on an RNN autoencoder. *Appl Sci*. 2023;13(8):5153. <https://doi.org/10.3390/app13085153>
- [32] Wang L, Xu Y, Chen Q, Wu J, Luo J, Li X, et al. Research on remote-sensing identification method of typical disaster-bearing body based on deep learning and spatial constraint strategy. *Remote Sens*. 2024;16(7):1161. <https://doi.org/10.3390/rs16071161>

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The wear of polytetrafluoroethylene helical gears under dry and lubricated conditions

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Article Info

Abstract

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The wear and thermal characteristics of Polytetrafluoroethylene (PTFE) helical gears were experimentally investigated using a rig test under dry and lubricated conditions. The study employed two gear ratios (1:1 and 0.75:1) at constant speed and torque to determine the effect of ratio on wear. Both wear and temperature increased for the driving and driven gears in all test environments. The experimental results highlighted the critical role of lubrication: significant surface wear began much earlier in dry conditions (after 100×10^4 cycles) than in lubricated conditions (after 120×10^4 cycles). This confirms that lubrication significantly improves the wear resistance of PTFE gears.

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1. Introduction

Industrial development fundamentally depends on machines that have high capability and reliability in their components. Traditionally, metal gears have been the go-to choice for many applications. Over time, polymer gears have become increasingly popular due to their unique benefits, such as being lightweight, producing less vibration, reducing noise, offering high corrosion resistance, and being self-lubricating. While metal gears are prone to issues like distortion, cracking, and rupture under stress, polymer gears offer a promising alternative. These types of failures are a concern in any industry, but especially in the fast-paced world of manufacturing, where downtime or performance issues can lead to costly delays. Back in the 1700s, testing machines were first introduced, using simple methods like pulling, bending, and twisting materials to understand how they behave under stress [1]. Despite these advances in testing, the goal remains the same: to prevent failures and ensure the longevity and efficiency of machine elements. Today, avoiding gear failure is especially important in industries like automotive, printing, and food, pharmaceuticals, and beverages, where production depends on the smooth operation of every part. Metals can corrode or contaminate products, posing health risks to humans and animals. This is particularly problematic in food and pharmaceutical production, where safety and hygiene are paramount. Polymer gears offer a much-needed solution. They are not only lighter and quieter, but they also resist corrosion and require less maintenance due to their self-lubricating properties. Materials like MC blue, polytetrafluoroethylene (PTFE), and polyoxymethylene (POM) are often used to make these gears, either on their own or in combination. These materials are perfect for use in industries that demand high standards of hygiene and safety, like food and pharmaceuticals. Nearly every mechanical device has some form of transmission elements. For example, motor vehicles use gears to transfer power, washing machines rely on pulleys to rotate the drum,

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computer printers have gears, cams, and pulleys to move paper and adjust the printing heads, and even photocopiers and ATMs are full of various power transfer components [2-3]. In industries like food and pharmaceuticals, where equipment is exposed to harsh environments, it's vital to choose the right type of steel to avoid corrosion. Even with careful selection, corrosion can still happen, and it can have serious consequences. Not only can it affect the quality of food and medicine, but it can also change their taste, texture, or safety. Plus, corrosion can damage the equipment used in production, potentially causing costly breakdowns and slowing down manufacturing. The risk of corrosion is present during different stages of production, like processing raw materials, making food or medicine, and packaging the final products. For example, in food production, equipment can come into contact with acidic substances, chloride solutions, and protein-rich media, especially at high temperatures, all of which increase the chances of corrosion. On the other hand, drug production involves oxidizing environments and the use of high-purity water and steam [4-5]. Wear process of gears specially made by polymers have been studied, such as, polyformaldehyde gears [6-13]. Studies on polymer gears regarding durability [14-15], composite gear design [16-18], the effects of temperature and lubrication [19-22], and acoustic noise [23] have been the focus of many researchers. While there have been some experimental studies on the topic, the wear process of polymer gears still presents an interesting area for further research, especially when using polymers produced in Indonesia. In this study, we looked at how polytetrafluoroethylene (PTFE) helical gear pairs wear under both dry and oil-lubricated conditions. The findings from this experiment give us a better understanding of the wear mechanisms and show how polymer gears' load capacity can be improved.

2. Wear Test Rig for Polytetrafluoroethylene Helical Gears

2.1. Testing Methodology

The experiment was conducted using a gear test that comply with ASTM D5182-19 as shown in Fig. 1. The drive gear is directly connected to the electromotor using a shaft. The driven gear is connected to a shaft equipped with a controlled loading instrument. The test rig allows the adjustment of the center distance to accommodate gears of different sizes or modules. The specimens were set in the box, which was filled empty or oil in order to realize the continuous dry and lubricated conditions. The experiments were performed with a rotational speed of 2400 rpm, while the output torque is maintained 5.8 N.m by adjusting the load on a disk brake system.

In this study, the testing procedure was carried out as shown in Fig. 3. The testing was carried out in 2 stages. The first stage is durability testing, in which in this stage, the gear is not removed to measure the weight and scan tooth profile, but continues to run, only temperature measurements are taken per 20×10^4 cycles. The next stage is worn testing, at this stage the gear is removed per 20×10^4 cycles to measure the initial and final gear weights then the lost weight can be calculated and the profile of each tooth scanned.

2.2. Gear samples

To observe the effect of gear ratio on wear under overdrive conditions which are commonly found in pharmaceutical and food industry machines, three bevel gear geometries with different nominal values are used. The gear ratios are 1:1 and 0.75:1. For simplicity, the geometry of the driving gear is maintained the same for both driven gears while the driven gear is adjusted. The gear geometries and mechanical properties are shown in Table 1 while sample gear photos are shown in Fig. 2.

Table 1. Helical gear geometry and properties

Gear	Material	Dimensions				Module (mm)	Number of teeth	Yield strength (MPa)	Density (kg/m ³)
		DO (mm)	DP (mm)	Width (mm)	Angle (°)				
Drive	PTFE	100	96	25	12	2	48	10,47	2200
Driven-1	PTFE	100	96	25	12	2	48	10,47	2200
Driven-2	PTFE	75	71	25	12	2	36	10,47	2200

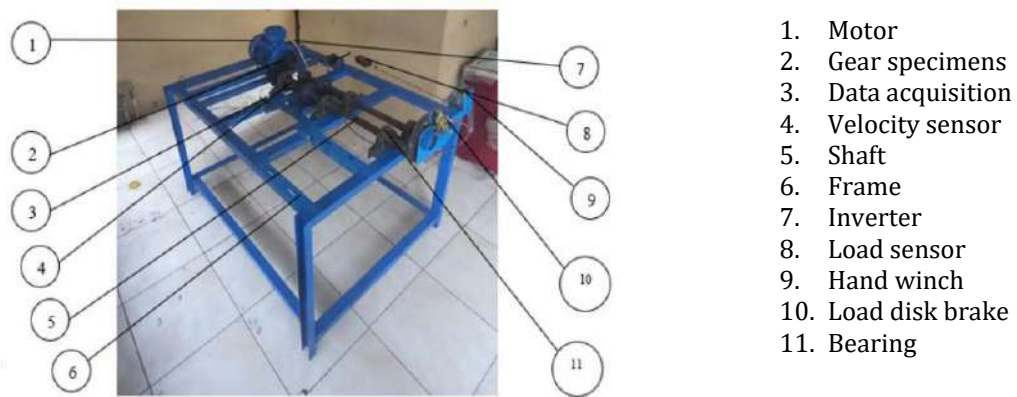


Fig. 1. Gear test rig

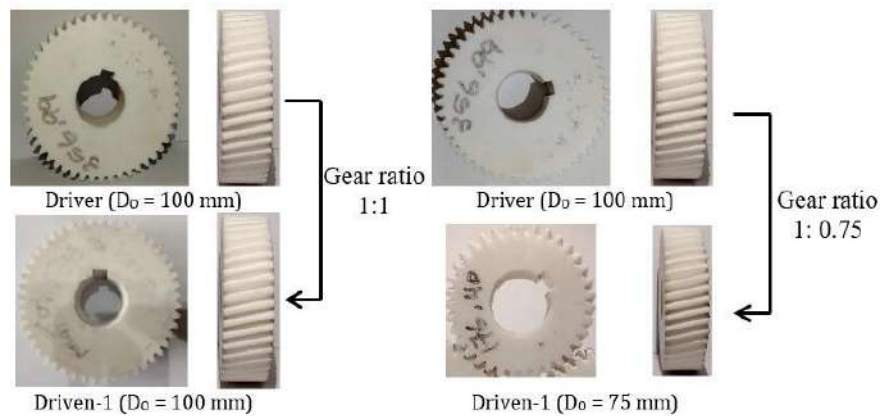


Fig. 2. Gear samples

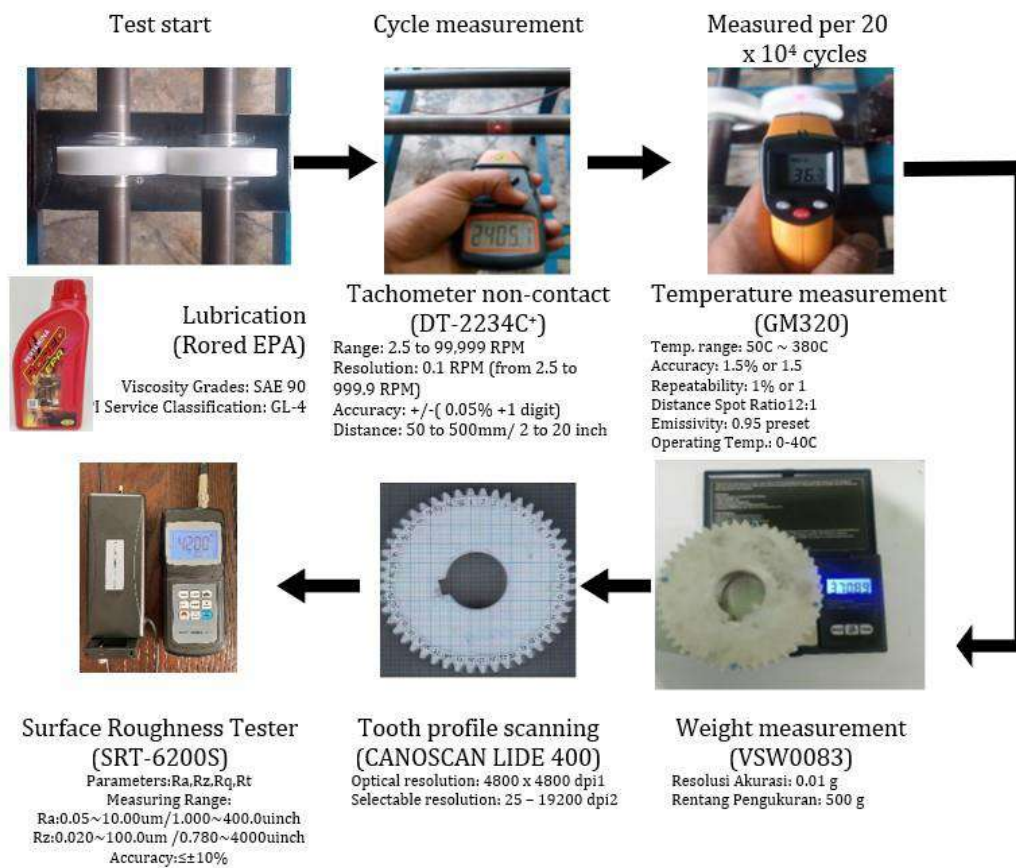
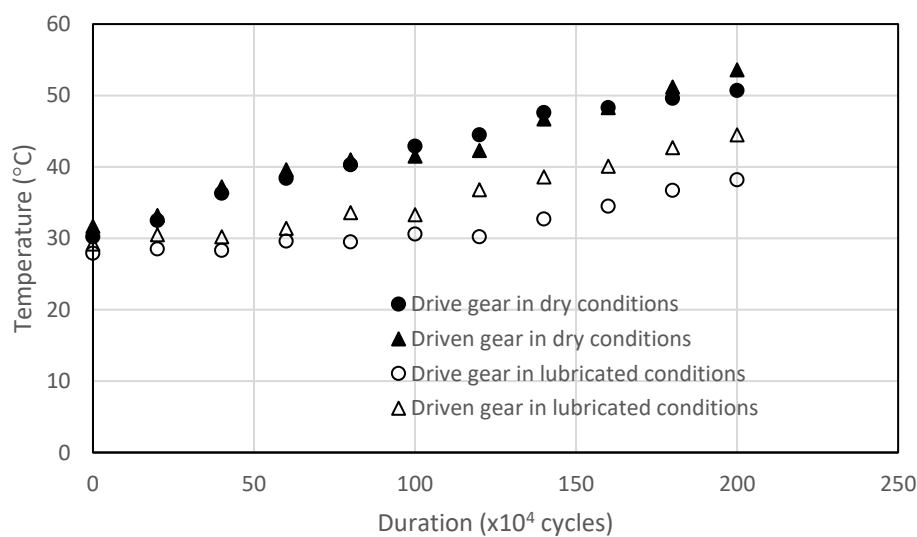


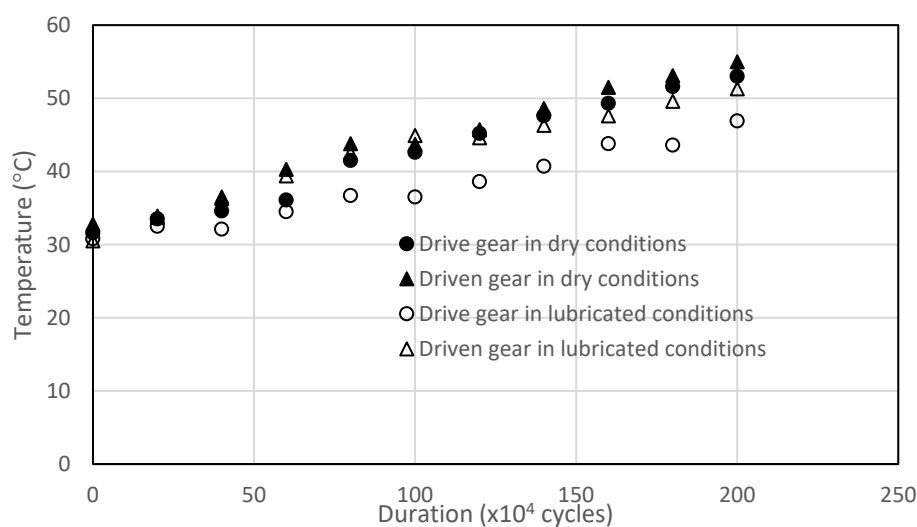
Fig. 3. Testing methodology

4. Results and Discussion

Figure 4 shows the results of temperature measurements on the gear operated in dry and lubricated conditions. A total of 11 hours of gear running continuously without any breaks. Starting from 30°C -31°C of room temperature, the maximum temperature is reached at 55,6°C on driven gear operated in dry condition. By comparing Fig. 4(a) and Fig. 4(b), the lower gear ratio does not show significant temperature difference. The complete results of the temperature and gear weight measurements for gear ratios 1:1 and 0.75:1 is shown in Table 1 and Table 2 respectively. Figure 5 presents the wear accumulation for PTFE helical gears across different gear ratios. A significant difference was observed between the dry and lubricated conditions, with the drive gear exhibiting greater wear than the driven gear in both cases. For the 1:1 ratio (Fig. 5a), the final wear accumulation reached 0.38 g in dry conditions and 0.18 g in lubricated conditions. Conversely, for the 0.75:1 ratio, the maximum accumulation was 0.42 g (dry) and 0.24 g (lubricated). This demonstrates that a lower gear ratio increases the overall accumulation of tooth wear, though the notable difference between dry and lubricated results persists.

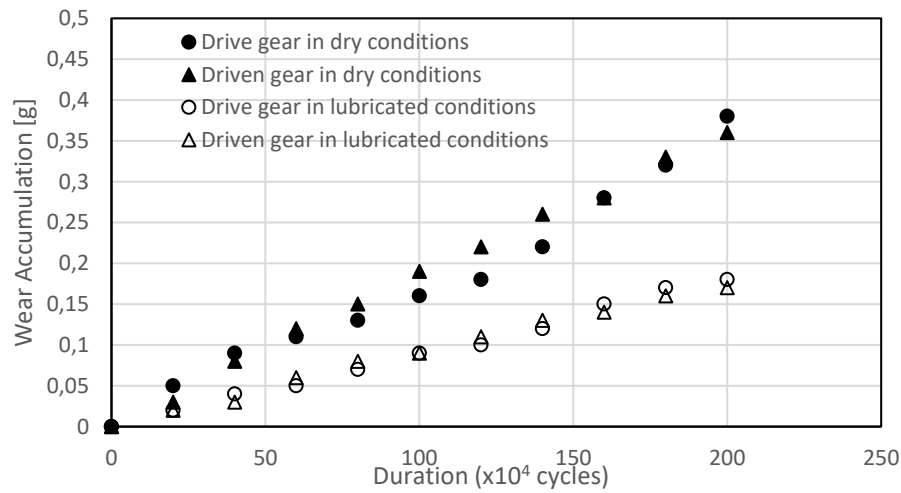


(a) Temperature of helical gear with a ratio 1:1

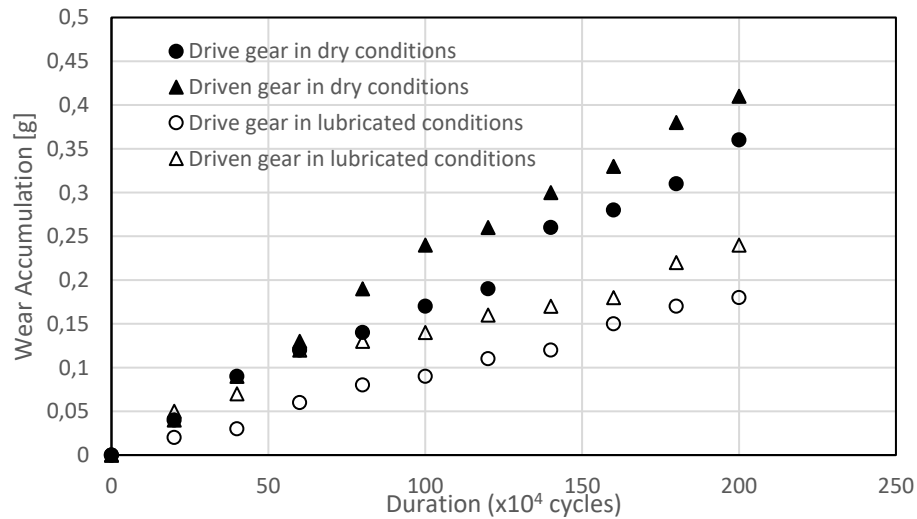


(b) Temperature of helical gear with a ratio 0.75:1

Fig. 4. Surface temperature of PTFE helical gear with different gear ratio.



(a) Wear accumulation of helical gear with a ratio 1:1



(b) Wear accumulation of helical gear with a ratio 0.75:1

Fig. 5. Wear accumulation of PTFE helical gear with different gear ratio

The PTFE gears were monitored for permanent damage every 20×10^4 cycles by scanning the changes in their geometric profiles. This damage initiates through material tearing caused by mating tooth engagement, resulting in a characteristic rough, torn surface (Fig. 6). Figure 6 summarizes the wear accumulation, demonstrating a significant disparity between dry and lubricated environments, and showing that the driven gear accumulated slightly more wear than the drive gear overall. Significant surface wear became clearly visible after 100×10^4 cycles. At this point, the severity of the dry condition was evident: the driven gear teeth (Fig. 6b) were torn to almost half their original shape. While damage was also clear under lubricated conditions (Fig. 6c and 6d), the resulting surface profile was noticeably smoother.

To gain detailed insight into the surface damage of the PTFE gear, damaged and undamaged teeth were sectioned and observed using Scanning Electron Microscopy (SEM). The SEM results in Fig. 7 clearly distinguish between the undamaged and worn tooth profiles. Figure 7(a) displays the initial roughness from the machining process (fine parallel lines, marked by the red circle) on a clean section. Conversely, the damaged section (Fig. 7b) shows the obliteration of these initial marks in the red-circled area, replaced by an entirely modified, smooth, worn, or polished surface, which confirms the extent of material degradation. This observation is consistently supported by images captured at both $30 \mu\text{m}$ and $20 \mu\text{m}$ resolutions.

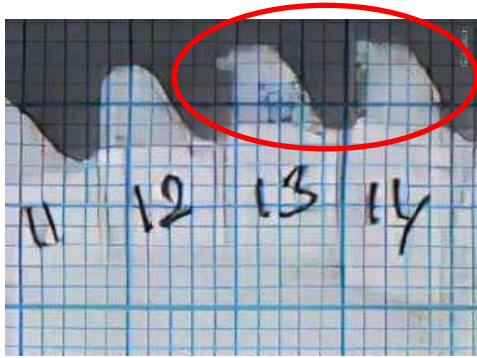
The surface roughness test was conducted for each condition after 200×10^4 cycles of wear test. The top land of tooth gear of PTFE gear was tested. The average data of roughness test results is presented in Table 4. Initial surface roughness measurements under dry, 1:1 ratio conditions were $4.30 \mu\text{m}$ (driving) and $4.43 \mu\text{m}$ (driven). After approximately 11 hours of wear loading, the roughness decreased significantly to $2.66 \mu\text{m}$ (driving) and $2.56 \mu\text{m}$ (driven), representing a 40-45% reduction. A similar pattern of surface roughness reduction (38-45%) was observed when the gears were tested under lubricated conditions across both gear ratios. Overall, lubrication was found to have a positive impact, further reducing the final surface roughness value by an additional 14-16% compared to the dry condition.

Table 2. Experimental results of helical gears with a ratio of 1:1

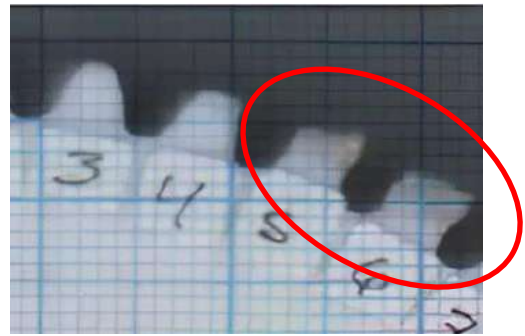
Test duration ($\times 10^4$)	Dry conditions				Lubricated conditions			
	Temperature ($^{\circ}\text{C}$)		Weight (g)		Temperature ($^{\circ}\text{C}$)		Weight (g)	
	Drive	Driven	Drive	Driven	Drive	Driven	Drive	Driven
0	30.2	31.7	369.14	373.70	27.9	29.2	368.34	373.30
20	32.5	33.2	369.09	373.67	28.5	30.5	368.32	373.28
40	36.3	37.2	369.05	373.62	28.3	30.2	368.30	373.27
60	38.4	39.6	369.03	373.58	29.6	31.4	368.29	373.24
80	40.3	41.0	369.01	373.55	29.5	33.6	368.27	373.22
100	42.9	41.5	368.98	373.51	30.6	33.3	368.25	373.21
120	44.5	42.3	368.96	373.48	30.2	36.8	368.24	373.19
140	47.6	46.7	368.92	373.44	32.7	38.6	368.22	373.17
160	48.3	48.3	368.86	373.42	34.5	40.1	368.19	373.16
180	49.6	51.2	368.82	373.37	36.7	42.7	368.17	373.14
200	50.7	53.6	368.76	373.34	38.2	44.5	368.16	373.13

Table 3. Experimental results of helical gears with a ratio of 0.75:1.

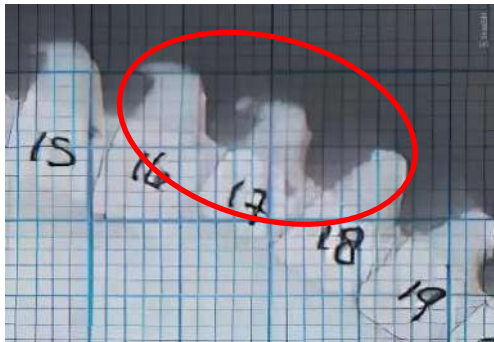
Test duration ($\times 10^4$)	Dry conditions				Lubricated conditions			
	Temperature ($^{\circ}\text{C}$)		Weight (g)		Temperature ($^{\circ}\text{C}$)		Weight (g)	
	Drive	Driven	Drive	Driven	Drive	Driven	Drive	Driven
0	31.6	32.8	368.73	182.42	30.8	30.5	368.14	179.53
20	33.5	33.9	368.69	182.38	32.5	33.7	368.12	179.48
40	34.6	36.5	368.64	182.33	32.1	36.3	368.11	179.46
60	36.1	40.3	368.61	182.29	34.5	39.4	368.08	179.41
80	41.5	43.8	368.59	182.23	36.7	42.5	368.06	179.4
100	42.6	43.7	368.56	182.18	36.5	44.9	368.05	179.39
120	45.2	45.7	368.54	182.16	38.6	44.6	368.03	179.37
140	47.6	48.6	368.47	182.12	40.7	46.3	368.02	179.36
160	49.3	51.5	368.45	182.09	43.8	47.6	367.99	179.35
180	51.6	53.1	368.42	182.04	43.6	49.6	367.97	179.31
200	53.0	55.0	368.37	182.01	46.9	51.3	367.96	179.29



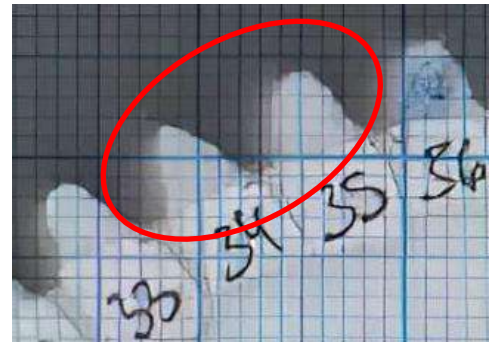
(a) After 100×10^4 cycles for drive gear



(b) After 100×10^4 cycles for driven gear

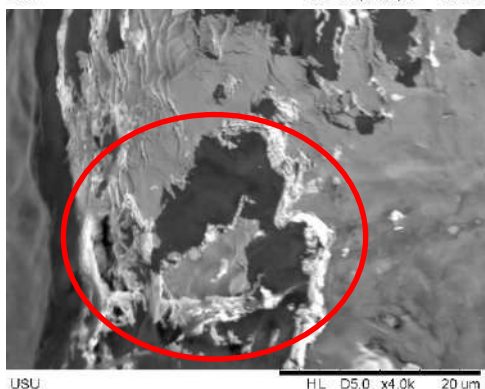


(c) After 120×10^4 cycles for drive gear

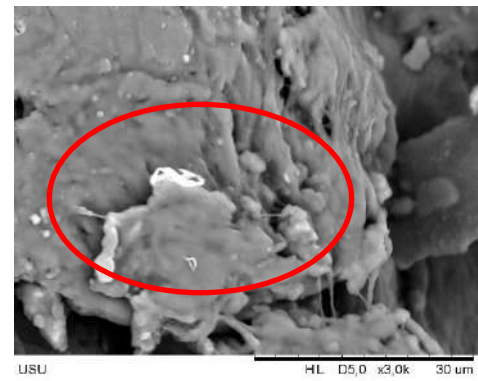


(d) After 120×10^4 cycles for driven gear

Fig. 6. Result of tooth profile scanning for dry and lubricated conditions.



(a) Undamaged teeth



(b) Damaged teeth

Fig. 7. SEM test results for damaged and undamaged teeth on gears.

Table 4. Surface Roughness Test of helical gears.

No	Wear test conditions	Gear Ratio	Initial wear test	[Ra	After wear test	[Ra
			(μm)]		(μm)]	
			Drive	Driven	Drive	Driven
1	Dry	1 : 1	4.30	4.43	2.66	2.56
2		0.75 : 1	4.41	4.47	2.72	2.45
3	Lubricated	1 : 1	4.38	4.45	3.96	3.81
4		0.75 : 1	4.32	4.50	3.91	3.74

5. Conclusions

The experimental investigation into the wear behavior of PTFE helical gear pairs under dry and lubricated conditions yielded several key findings. First, temperature significantly affects PTFE gear wear, though the overdrive (0.75:1) and fixed (1:1) ratios exhibited no observable temperature difference. Second, wear accumulation increased as the gear ratio was reduced. The lower 0.75:1 ratio resulted in higher maximum wear (0.42 g dry; 0.24 g lubricated) compared to the 1:1 ratio (0.38 g dry; 0.18 g lubricated), confirming that lubrication's protective benefit persists despite the increased mechanical wear at the lower ratio. Finally, SEM analysis clearly confirmed the extent of material degradation: the initial machining marks on undamaged teeth were entirely obliterated and replaced by a modified, smooth, worn surface on the damaged sections.

Acknowledgement

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References

- [1] Budynas RG, Nisbett JK. Shigley's mechanical engineering design. 11th ed. New York: McGraw-Hill; 2019.
- [2] Beer FP, Johnston ER Jr, DeWolf JT. Mechanics of materials. 7th ed. New York: McGraw-Hill; 2014.
- [3] Groover MP. Fundamentals of modern manufacturing: materials, processes, and systems. 7th ed. Hoboken: Wiley; 2020.
- [4] Groover MP. Automation, production systems, and computer-integrated manufacturing. 5th ed. Boston: Pearson; 2018.
- [5] Kemp AW. Industrial mechanics. 4th ed. Alsip (IL): American Technical Publishers; 2018.
- [6] Bravo A, Koffi D, Toubal L, Erchiqui F. Life and damage mode modeling applied to plastic gears. Eng Fail Anal. 2015;58(Pt 1):113-133. <https://doi.org/10.1016/j.engfailanal.2015.08.040>
- [7] Yu G, Liu H, Mao K, Zhu C, Lu Z. Examination on the wear process of polyformaldehyde gears under dry and lubricated conditions. Friction. 2021;9(3):538-550. <https://doi.org/10.1007/s40544-020-0362-7>
- [8] Siregar RA, Umurani K, Mukhlis. Studi eksperimen terhadap keausan pada roda gigi cacing komposit. JRMME. 2019;2(2). <https://doi.org/10.30596/rmme.v2i2.3670>
- [9] Mertens AJ, Senthilvelan S. Effect of mating metal gear surface texture on the polymer gear surface temperature. Mater Today Proc. 2015;2:1763-1769. <https://doi.org/10.1016/j.matpr.2015.07.017>
- [10] Mao K, Li W, Hooke CJ, Walton D. Friction and wear behaviour of acetal and nylon gears. Wear. 2009;267:639-645. <https://doi.org/10.1016/j.wear.2008.10.005>
- [11] Li W, Wood A, Weidig R, Mao K. An investigation on the wear behaviour of dissimilar polymer gear engagements. Wear. 2011;271:2176-2183. <https://doi.org/10.1016/j.wear.2010.11.019>
- [12] Wright NA, Kukureka SN. Wear testing and measurement techniques for polymer composite gears. Wear. 2001;251:1567-1578. [https://doi.org/10.1016/S0043-1648\(01\)00793-1](https://doi.org/10.1016/S0043-1648(01)00793-1)
- [13] Mao K. A numerical method for polymer composite gear flash temperature prediction. Wear. 2007;262:1321-1329. <https://doi.org/10.1016/j.wear.2007.01.008>
- [14] Kim CH. Durability improvement method for plastic spur gears. Tribol Int. 2006;39:1454-1461. <https://doi.org/10.1016/j.triboint.2006.01.020>
- [15] Düzcükoğlu H. PA 66 spur gear durability improvement with tooth width modification. Mater Des. 2009;30:1060-1067. <https://doi.org/10.1016/j.matdes.2008.06.037>
- [16] Senthilvelan S, Gnanamoorthy R. Effect of gear tooth fillet radius on the performance of injection molded Nylon 6/6 gears. Mater Des. 2006;27:632-639. <https://doi.org/10.1016/j.matdes.2004.12.015>
- [17] Mao K. A new approach for polymer composite gear design. Wear. 2007;262:432-441. <https://doi.org/10.1016/j.wear.2006.06.005>

- [18] Düzcükoğlu H. Study on development of polyamide gears for improvement of load-carrying capacity. Tribol Int. 2009;42(8):1146-1153. <https://doi.org/10.1016/j.triboint.2009.03.009>
- [19] Kalin M, Kupec A. The dominant effect of temperature on the fatigue behaviour of polymer gears. Wear. 2017;376-377:1339-1346. <https://doi.org/10.1016/j.wear.2017.02.003>
- [20] Yousef S, Visco AM, Galtieri G, Njuguna J. Wear characterizations of polyoxymethylene (POM) reinforced with carbon nanotubes using the paraffin oil dispersion technique. JOM. 2015;68(1):288-299. <https://doi.org/10.1007/s11837-015-1674-3>
- [21] Hiral HP, Piyush PG. Experimental investigation and prediction of wear behavior of cotton fiber polyester composites. Friction. 2017;5(2):183-193. <https://doi.org/10.1007/s40544-017-0145-y>
- [22] Lu ZH, Liu HJ, Zhu CC, Song HL, Yu GD. Identification of failure modes of a PEEK-steel gear pair under lubrication. Int J Fatigue. 2019;125:342-348. <https://doi.org/10.1016/j.ijfatigue.2019.04.004>
- [23] Hoskins TJ, Dearn KD, Kukureka SN, Walton D. Acoustic noise from polymer gears-A tribological investigation. Mater Des. 2011;32(6):3509-3515. <https://doi.org/10.1016/j.matdes.2011.02.041>

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Performance of geopolymer concrete flat slab with various transverse reinforcement in resisting punching shear

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Abstract

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A flat slab is an efficient structure in resisting gravity loads by transferring the load from the slab to the columns and then to the foundations. Various normal concrete (NC) flat slab reinforcements against punching shear have been developed, including drop panels, steel plates, aluminum plates, additional longitudinal bar reinforcement, steel tubes, and transverse reinforcement. However, previous studies have not discussed the effect of the transverse reinforcement ratio that impacts capacity and ductility. In contrast, the performance of flat slabs with environmentally friendly concrete, such as geopolymer concrete (GC), has not been discussed. This study aims to analyze the performance of GC flat slabs reinforced with transverse reinforcement. The novelty of this study is extending the NC flat slab test results to GC flat slab numerical models with different transverse reinforcement. The performance of NC and GC flat slabs subjected to monotonic downward vertical loads is analyzed using a finite-element-based software. The issues discussed include shear load, deflection, stress contour, displacement contour, ductility, stiffness, energy absorption, and punching shear capacity. The results show that transverse reinforcement increases the flat slab capacity to withstand punching shear loads. The GC materials enhance shear strength. However, excessive transverse reinforcement results in brittle behavior and reduces deformation, ultimate load, and ultimate deflection.

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1. Introduction

Geopolymer concrete is environmentally friendly because it does not contain Portland cement as a binder and its manufacturing process requires low energy [1, 2], thereby supporting sustainable, environmentally friendly construction activities [3, 4]. The binder during the hydration process in geopolymer concrete is a compound rich in silica and alumina, such as fly ash or rice husk ash [5-7], which is activated with alkaline liquid [8-10]. It supports reducing industrial and organic waste, which helps prevent climate change [2]. In contrast to geopolymer concrete, conventional concrete contains a binder—Portland cement—which produces carbon dioxide emissions [11], increases global temperatures [12], and creates pollution [13]. Geopolymer concrete has high durability [1, 14, 15], is more fire resistant than conventional concrete, and can be used as an insulator [16]. Geopolymer concrete has the potential to serve as a material for forming structural elements, as its mechanical properties can match or exceed those of conventional concrete. Geopolymer concrete achieves a compressive strength of 12.5-31.3 MPa [14], 28.5-50.1 MPa [6], and meets the minimum compressive strength requirements for structural elements of 17.5 MPa [17, 18]. The interfacial shear capacity of geopolymer concrete ranges from 2.15 to 2.5 MPa [19], enabling it to form column and beam structural elements under axial and bending loads [20, 21], as well as other structural elements, such as slabs and foundations. High-quality geopolymer concrete reaches a compressive

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strength of 78.2-81.17 MPa [20], 163-210 MPa [21], and has the potential to be used as a material for forming structural elements that can withstand high compressive loads, namely foundations, bridge pillars, bridge girders, and prestressed beams [22, 23].

Research on geopolymer concrete structural elements generally focuses on beams and columns to investigate their flexural and compressive performance [23-28]. The performance of flat slabs using geopolymer concrete materials and resisting punching shear loads has not been comprehensively studied and thus requires further investigation. Flat slab is one of the structural element systems for high-rise buildings that is vulnerable to seismic loads. Its function as a secondary seismic-resistant structure [29] makes it more suitable in the low-magnitude earthquake risk zones [30].

The corners of rectangular or square columns experience increased concrete strain concentrations as the ratio of the long to short sides of the column cross-section increases. The circular column cross-section reduces the concrete strain concentration and provides higher shear strength than rectangular columns [31]. The column shape affects the punching shear capacity of flat plates. The results of the analysis using the finite element method show that L-shaped, T-shaped, and Cross-shaped column cross-sections increase the punching shear capacity by 3%, 5.3%, and 22.6%, respectively, compared to columns with square cross-sections. This shows that the cross-sectional shape of the column can be effectively optimized to increase the punching shear resistance of flat plates [32]. The use of flat slabs reduces construction costs because they are built without beams. In the flat slab system, the concentrated axial load of the column is transferred directly to the flat slab and potentially causes damage due to punching shear. The performance of flat slabs in punching shear needs improvement. The transverse reinforcement system in flat slabs increases the shear capacity to withstand punching shear significantly [33-35].

Other methods for enhancing flat slab performance include shear reinforcement systems using diagonal and orthogonal aluminum sheets [36]. Aluminum sheets measuring 25 x 50 mm were installed at the mid-thickness of the slab to increase shear strength. The aluminum sheet installation position configurations were horizontal, vertical, diagonal, and hybrid. The aluminum sheet thicknesses were 1.0 mm, 1.5 mm, and 2.0-mm. Monotonic loading was gradually applied at a single location in the middle of the slab, downward until the specimen failed. The results of the investigation in this research showed that each aluminum sheet placement configuration slowed crack initiation and propagation. Aluminum sheets helped prevent the concrete cover from cracking before failure occurred. The thicker the aluminum sheet, the higher the initial cracking load in all aluminum sheet position configurations. The most significant increase in initial cracking load, 55.8%, was achieved with a flat slab using a combination of horizontal and vertical aluminum sheet configuration with a thickness of 2 mm. The most significant increase in ultimate load, 60.9%, was achieved with the flat slab reinforced with a hybrid aluminum sheet position. The highest increase in absorbed energy, 60%, was achieved by the flat slab with aluminum sheets in a combination of horizontal and vertical configurations.

Another type of reinforcement is a two-way integrity reinforcement, parallel to the column, located in the center of the flat slab and the compression zone [37]. Three flat slabs with a two-way $\phi 8$ -60 longitudinal reinforcement were reinforced with integrity reinforcement installed in the X and Y axes in the center of the slab and the compression zone. The diameter variations for the three integrity reinforcements were 10 mm, 12 mm, and 14 mm for the flat slab types 1, 2, and 3, respectively. Each slab was also reinforced with perimeter reinforcement. The monotonic load was applied to the top of a short column in the middle of a rectangular steel plate as an intermediary. Before punching shear failure, concrete plays a more dominant role in resisting shear. Upon entering the punching shear condition, the concrete cracked and crushed; thus, the longitudinal steel reinforcement and integrity reinforcement were more dominant in resisting the load. Shear capacity increases with the larger diameter of the integrity reinforcement steel bar. The difference between the concrete compressive and steel tensile strengths significantly affected punching shear capacity and post-punching resistance.

Some other methods for improving flat slab performance are elaborated in the following brief reports. Reinforcement around concrete columns enclosed by steel tubes in the form of horizontal

and vertical bearing plates enhances the performance of flat slabs in terms of punching shear capacity, ductility, and energy absorption. The gradually increasing vertical loading on the upper side of the column cross-section does not result in yielding of the horizontal bearing plates. Otherwise, the vertical bearing plates yield only at the upper and lower corners. This condition indicates that the vertical bearing plates provide a greater stiffness and contribute to resisting punching shear than the horizontal bearing plate. The use of larger vertical bearing plate dimensions or slab thickening increases resistance to punching shear. Another property of flat slabs that contributes to increased punching shear capacity is the concrete's compressive strength, which also increases ductility and energy absorption. The addition of tensile reinforcement significantly increases the punching shear capacity in flat slabs with minimal longitudinal reinforcement [38].

The use of steel plates at the column-to-flat slab connection has been shown to increase maximum load capacity and shear strength by up to 2.15 times. The rib plates connecting the column and the flat slab contribute to increased stiffness and shear performance. Some rib plates experienced premature buckling despite the application of rib plate thickening. The behavior of the flat slab reinforced with steel plates at the column-to-flat slab and rib plate connections is also influenced by the bond and deformation of the steel reinforcement [39].

The shear capacity of the flat slab-column connection was increased by using ultra-high-performance fiber-reinforced concrete material on the flat slab around the column with a side size of 273 mm to 600 mm in a square shape. High compressive strength in concrete increases the capacity of the flat slab-column connection in terms of initial stiffness, punching shear capacity, and deformation. The increase in capacity is influenced by the addition of a flat slab area with ultra-high-performance concrete containing fibers [40].

The above methods for improving flat slab performance indicate that punching shear resistance can be achieved by strengthening transverse reinforcement. The addition of shear reinforcement increases the maximum shear force that can be resisted by a flat slab in both elastic and inelastic conditions. Transverse reinforcement in thick slabs and waffle slabs can increase the shear capacity [41,42]. Thus, it can be applied to flat slabs to improve the ability to resist punching shear equivalent to or better than flat slabs with conventional concrete materials. However, research on the performance of flat slabs with geopolymer concrete materials and shear reinforcement to resist punching shear has not been comprehensively studied. An adequate transverse reinforcement provides sufficient shear capacity and enables geopolymer concrete flat slabs to resist punching shear.

This study aims to perform a performance analysis of geopolymer concrete flat slabs with transverse reinforcement to resist punching shear, with reinforcement of various diameters. The novelty of this study is extending the NC flat slab test results to the GC flat slab through numerical modeling with different transverse reinforcement configurations.

2. Materials and Methods

The research method involves investigating the mechanical properties of geopolymer concrete, including compressive strength, elastic modulus, and stress-strain relationship curves. The numerical modeling of a geopolymer concrete flat slab under monotonic loads was carried out using ANSYS, based on the finite element method, to analyze load-displacement relationships, stress and displacement contours, ductility, stiffness, energy absorption, and punching shear resistance. The transverse reinforcement was varied to investigate the flat slab's behavior in resisting punching shear under monotonic loading.

2.1. Details of Specimens and Models

In the previous research [41], the NC compressive strength and the modulus of elasticity were 30 MPa and 22500 MPa, respectively. The yield strength of steel for longitudinal reinforcement was 549 MPa for the flat slab without transverse steel reinforcement, and 615 MPa for the flat slab with transverse reinforcement of $\phi 5$ and $\phi 8$. The yield strength of transverse reinforcement of $\phi 5$ and $\phi 8$ was 698 MPa and 706 MPa, respectively [41], as shown in Table 1. In all flat slab models, the

upper longitudinal reinforcement was 16 mm in diameter, spaced at 155 mm, and connected with 16 mm hook-shaped rods at the same spacing. The lower longitudinal reinforcement diameter was 10 mm. The column reinforcement consisted of 8 longitudinal reinforcement bars with 16 mm in diameter surrounded by 10 mm in diameter stirrups with 80 mm spacing. The modeling analysis includes punching shear capacity, stress and displacement contours, ductility, strength degradation, and energy absorption [43-45].

The investigated flat slabs have dimensions of 2500 mm × 2500 mm with a thickness of 180 mm, effective depth of 162 mm, consisting of a composite of concrete and reinforcing steel. The concrete materials used in the specimens were normal concrete (NC), whereas in the numerical models, they were NC and geopolymer concrete (GC). The flexural reinforcement configuration from the previous research [41] is the same for all slabs. The upper longitudinal reinforcement has a diameter of 16 mm, spaced at 155 mm, and is connected with 16 mm-diameter steel bars in the form of hooks at the same spacing. The lower longitudinal reinforcement has a diameter of 10 mm and is placed directly below the upper reinforcing steel bars with a spacing of 155 mm. Table 1 shows the types of flat slabs. In the previous study [41], the NC flat slab specimens consisted of NC-0 and NC-5, while in this numerical study, the models consisted of three NC flat slab types (NC-0-F, NC-5-F, and NC-8-F) and three GC flat slab types (GC-0-F, GC-5-F, and GC-8-F). The NC-0 specimen and NC-0-F model do not have shear reinforcement. The NC-5 specimen and NC-5-F model were equipped with 5 ϕ 5 mm transverse reinforcement with 72 mm spacing, while the NC-8-F model used 5 ϕ 8 mm with 72 mm spacing. The performance of the flat slab with increased transverse reinforcement area is analyzed in this study. The column cross-section is 300 mm × 300 mm, with 8D16 longitudinal reinforcement and ϕ 8-80 stirrups. The clear column heights on the top and bottom sides of the flat slab are 800 mm and 600 mm, respectively. Details of the longitudinal and transverse steel bar reinforcements in all specimens [41], as well as the NC and GC models, are shown in Table 1. The GC compressive strength and elastic modulus are 41.5 MPa and 30266 MPa, respectively [46]. Details of the flat slab dimensions, longitudinal and transverse reinforcement from the top and side sections are shown in Figures 1 and 2, respectively. The reinforcement design for the flat slab to resist shear and bending, per code [17], is shown in Figure 3. Figure 4 shows the perimeter control, expressed as the dimensions and spacing of shear reinforcement, for the slab based on the code [17].

Table 1. Steel bars properties

Type of flat slab	Concrete material	Top and bottom longitudinal steel bar	Transverse steel bar	Transverse steel bar ratio	Longitudinal steel bar yield strength [41], f_{yl}	Transverse steel bar yield strength [41], f_{yt}
		(mm)	(mm)	(%)	(MPa)	(MPa)
NC-0 & NC-0-F	NC	ϕ 16 & ϕ 10	-	0	549	-
NC-5 & NC-5-F	NC	ϕ 16 & ϕ 10	ϕ 5-72	0.17	615	698
NC-8-F	NC	ϕ 16 & ϕ 10	ϕ 8-72	0.44	615	706
GC-0-F	GC	ϕ 16 & ϕ 10	-	0	549	-
GC-5-F	GC	ϕ 16 & ϕ 10	ϕ 5-72	0.17	615	698
GC-8-F	GC	ϕ 16 & ϕ 10	ϕ 8-72	0.44	615	706

In the experiment, a monotonic load was applied to the top of the column cross-section, which was covered with a steel plate serving as an intermediary between the horizontal loading frame and the column, as shown in Figure 5. The load cells pressed four spreader beams above the flat slab, resulting in uniform loads that caused the flat slab to move downwards. This condition persisted until the flat slab structure failed [41].

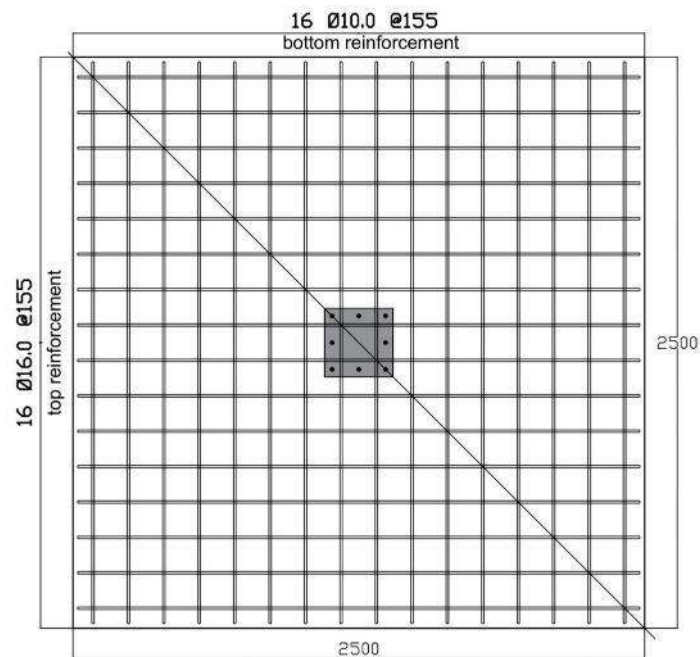


Fig. 1. Plan of flat slab details [41, redrawn]

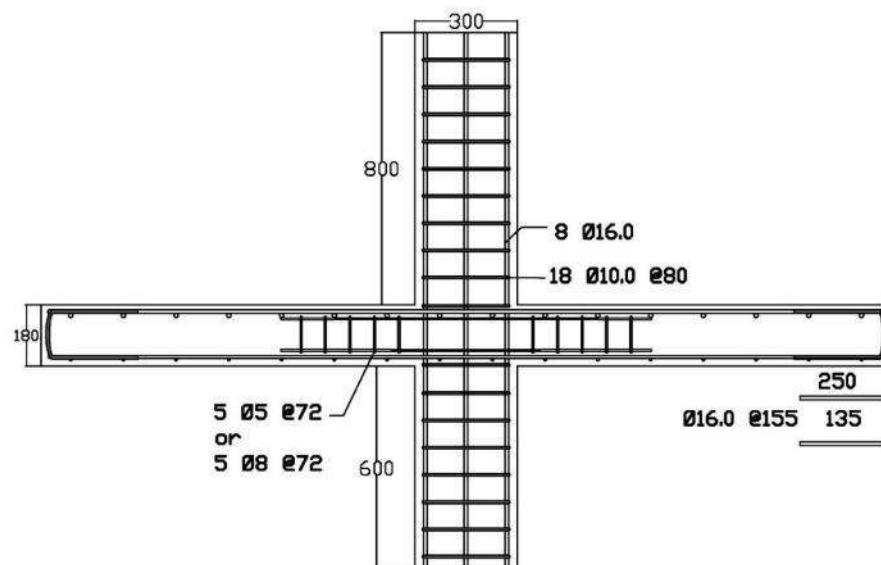


Fig. 2. Section of flat slab details [41, redrawn]

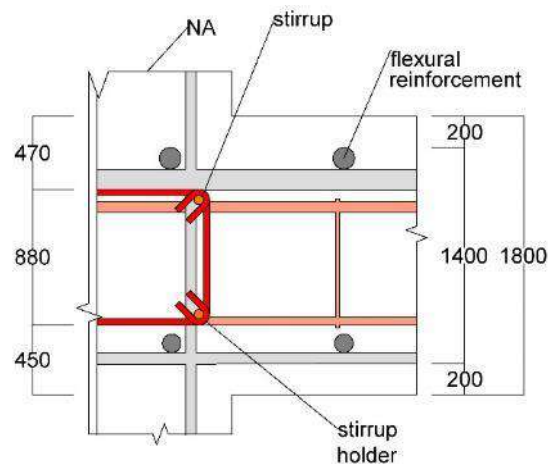


Fig. 3. Details of the transverse reinforcement [41, redrawn]

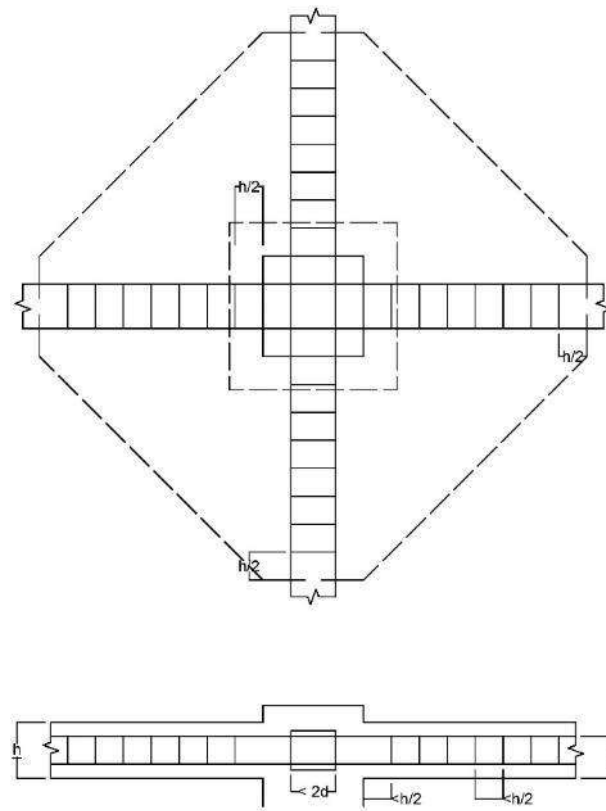


Fig. 4. Perimeter control and details for a flat slab [17, redrawn]

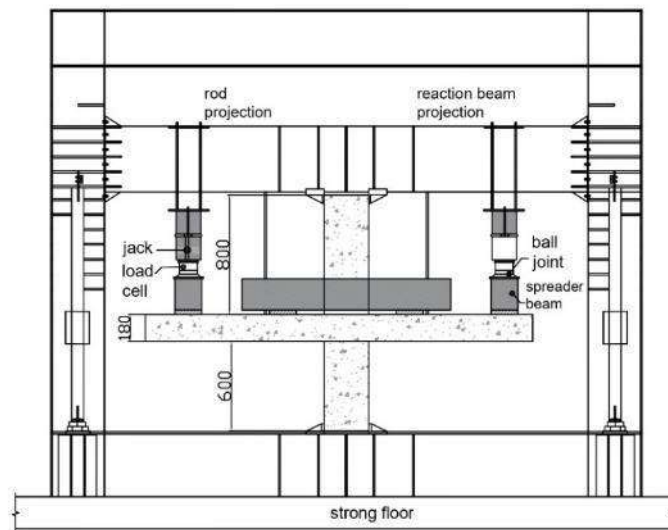


Fig. 5. Loading setup [41, redrawn]

2.2. Finite Element Modeling

The flat slab numerical model was developed using the finite element method under monotonic loading. The geometry, dimensions, details of the steel longitudinal and transverse reinforcement, concrete compressive strength, yield and ultimate strengths of the steel reinforcement, and monotonic loads were derived from previous research [41], as shown in Figure 1-5. Verification of the NC flat slab models was carried out by comparing the hysteretic curves of the load-displacement correlations from the numerical analysis with the experimental results of [41], with a tolerance of 10% to ensure accuracy [47,48].

The load-control method, with the exact loading location on the column and the spreader beams above the flat slab, was used in numerical modeling, where the compressive axial load was increased by 40 kN at each load step until failure. The position of the Linear Variable Differential

Transducer (LVDT) in the experiment [41], which became the reference for the flat slab displacement data in this study, was at a distance of 1050 mm from the center to the edge of the flat slab. The model for a flat slab and top-column structural system that supports uniform loads from four spreader beams and concentrated loads on the top of the column cross-section is shown in Figure 6. Figure 7 shows the flat slab structure connected to the columns. Flat slab finite element modeling was conducted by defining concrete and reinforcing steel as separate, discrete elements.

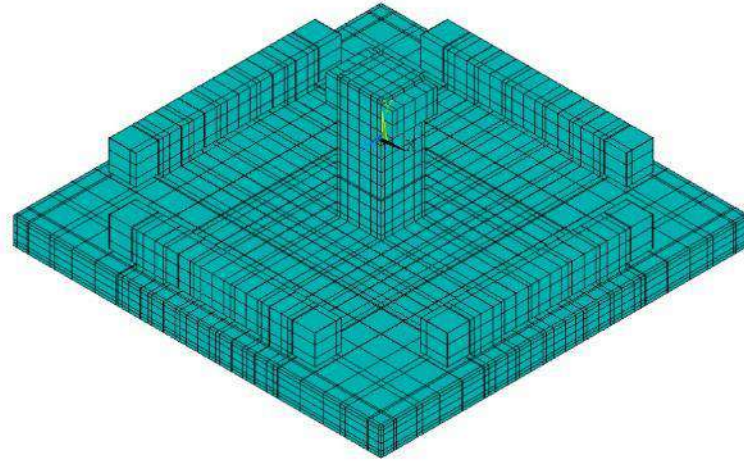


Fig. 6. The top side of a flat slab model

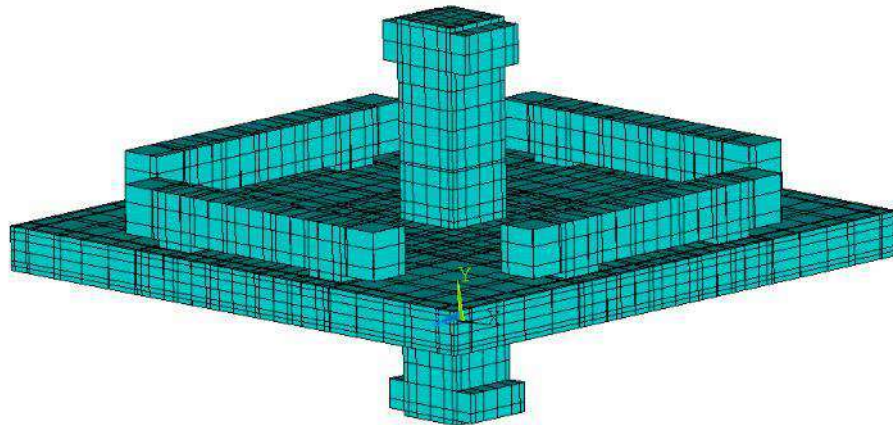


Fig. 7. Three-dimensional side view of the flat slab model

2.2.1 Concrete Element

The concrete is modeled using three-dimensional SOLID65 discrete elements with 8 nodes (8-node brick elements) in this study, as shown in Figure 8. Each node has three degrees of translational freedom in the X, Y, and Z axes. SOLID65 elements can be modeled as elements that experience cracking due to tensile stress, crushing due to compressive stress, plastic deformation, and creep.

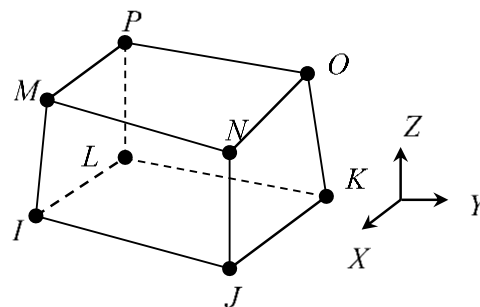


Fig. 8. SOLID65 element

2.2.1.1 Cracking Behavior of Concrete Elements

The material stiffness matrix under isotropic material conditions can be derived as in equation (1).

$$D^c = \frac{E}{(1+\nu)(1-2\nu)} \cdot \begin{bmatrix} (1-\nu) & \nu & \nu & 0 & 0 & 0 \\ \nu & (1-\nu) & \nu & 0 & 0 & 0 \\ \nu & \nu & (1-\nu) & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{(1-2\nu)}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{(1-2\nu)}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{(1-2\nu)}{2} \end{bmatrix} \quad (1)$$

Where; E : modulus of elasticity of concrete (MPa), ν : Poisson's ratio of concrete

The material stiffness from the stress-strain relationship for a cracked material in one direction is given by equation (2).

$$D_c^{ck} = \frac{E}{(1+\nu)} \begin{bmatrix} R^t(1-\nu) & 0 & 0 & 0 & 0 & 0 \\ \frac{E}{(1-\nu)(1-\nu)} & 1 & \frac{0}{\nu} & 0 & 0 & 0 \\ 0 & \frac{\nu}{(1-\nu)(1-\nu)} & 1 & 0 & 0 & 0 \\ 0 & \frac{(1-\nu)(1-\nu)\beta_t}{2} & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{2} & \frac{\beta_t}{2} \\ 0 & 0 & 0 & 0 & 0 & \frac{\beta_t}{2} \end{bmatrix} \quad (2)$$

The notation ck represents the stress-strain relationship referring to a coordinate system parallel to the principal stress axis with the X^{ck} axis perpendicular to the crack plane. R^t is the slope (secant modulus) as in Figure 9. The notation f_t represents the tensile stress and T_c is the multiplier for tensile stress relaxation.

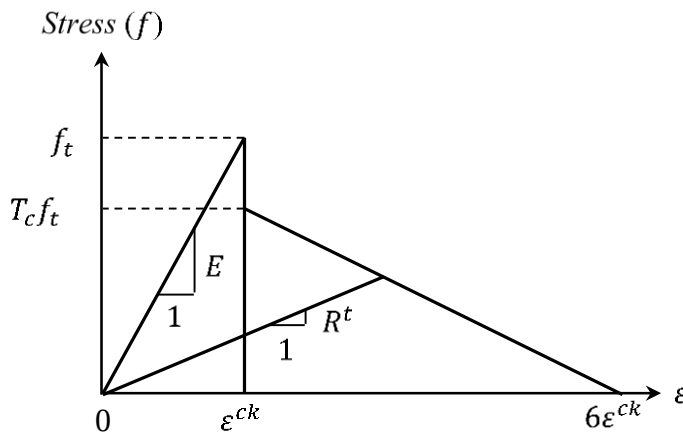


Fig. 9. Stress-strain diagram for cracked concrete material

Suppose the crack closes due to unloading conditions. In that case, the compressive stress perpendicular to the crack plane will be transferred to the crack, and there is only a shear transfer coefficient β_c , so that the stiffness matrix becomes as in equation (3).

$$D_c^{ck} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & \nu & 0 & 0 & 0 \\ \nu & (1-\nu) & \nu & 0 & 0 & 0 \\ \nu & \nu & (1-\nu) & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{\beta_c(1-2\nu)}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{\beta_c(1-2\nu)}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{\beta_c(1-2\nu)}{2} \end{bmatrix} \quad (3)$$

The stress-strain relationship for concrete that cracks in two directions is as in equation (4).

$$D_c^{ck} = E \begin{bmatrix} \frac{R^t}{E} & 0 & v & 0 & 0 & 0 \\ \frac{R^t}{E} & \frac{R^t}{E} & v & 0 & 0 & 0 \\ 0 & \frac{R^t}{E} & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{\beta_t}{2(1+v)} & 0 & 0 \\ 0 & 0 & 0 & \frac{\beta_t}{2(1+v)} & \frac{\beta_t}{2(1+v)} & 0 \\ 0 & 0 & 0 & 0 & \frac{\beta_t}{2(1+v)} & \frac{\beta_t}{2(1+v)} \end{bmatrix} \quad (4)$$

If the crack closes in two directions, then the stress-strain relationship is as in equation (3). The stress-strain relationship of concrete cracked in three directions is in accordance with equation (4). If the crack closes in three directions, then the stress-strain relationship is as in equation (3). The state of the crack is open or closed, depending on the strain value ε_{ck}^{ck} , which is called the crack strain. For the condition of the crack in the X-axis direction, the strain value is obtained by equation (5).

$$\varepsilon_{ck}^{ck} = \begin{cases} \varepsilon_X^{ck} + \frac{v}{1-v} \varepsilon_Y^{ck} + \varepsilon_Z^{ck} & \text{if no crack occurs} \\ \varepsilon_Y^{ck} + v \varepsilon_Z^{ck} & \text{if the crack is in the Y-axis direction} \\ \varepsilon_X^{ck} & \text{if the crack is in the Y and Z axis direction} \end{cases} \quad (5)$$

where ε_X^{ck} , ε_Y^{ck} , ε_Z^{ck} are the three components of normal strain in the crack direction.

If ε_{ck}^{ck} is less than zero, then the crack is assumed to be closed, whereas if ε_{ck}^{ck} is equal to or greater than zero, then the crack is assumed to be open. When the first crack occurs, the crack is assumed to be open for the next iteration.

2.2.1.2 Concrete Destruction Behavior and Damage Criteria

If a material is crushed by uniaxial, biaxial, or triaxial compressive forces, it is considered to be crushed. In the SOLID65 element in ANSYS, if crushing occurs, the material's stiffness contribution can be ignored. Concrete material that undergoes cracking and deterioration is modeled using the CONC type in SOLID65 elements. The failure criteria resulting from multiaxial stress are given by equation (6).

$$\frac{F}{f_c} - S \geq 0 \quad (6)$$

Where; F : a symbol to state the principal stress; S : failure area expressed in terms of principal stress and five parameters (f_t , f_c , f_{cb} , f_1 , and f_2); σ_{xp} , σ_{yp} , σ_{zp} : principal stresses in the direction of the principal axis; f_t : ultimate uniaxial tensile strength (MPa); f_c : Ultimate uniaxial compressive strength (MPa); f_{cb} : Ultimate biaxial compressive strength (MPa); f_1 : Ultimate biaxial compressive strength plus hydrostatic stress (MPa); f_2 : Ultimate uniaxial compressive strength plus hydrostatic stress (MPa).

The behavior of concrete material is modeled using Multilinear Isotropic Hardening (MISO) in the ANSYS software, as shown in Fig. 10. MISO uses the von Mises yield criterion and isotropic work hardening. The slope of the first segment of the curve (initial slope) represents the material's modulus of elasticity. Subsequent slopes must be greater than zero and less than the elastic modulus. The stress-strain curve can be defined up to 100 points. The response of concrete under loading is determined by its nonlinear behavior. Typical stress-strain behavior of concrete subjected to uniaxial loading is shown in Fig. 11.

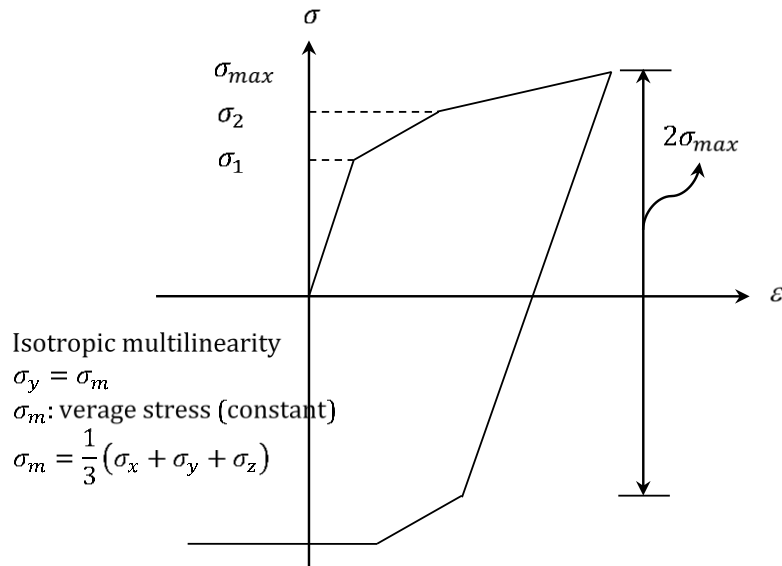


Fig. 10. Isotropic multilinear stress-strain diagram [49]

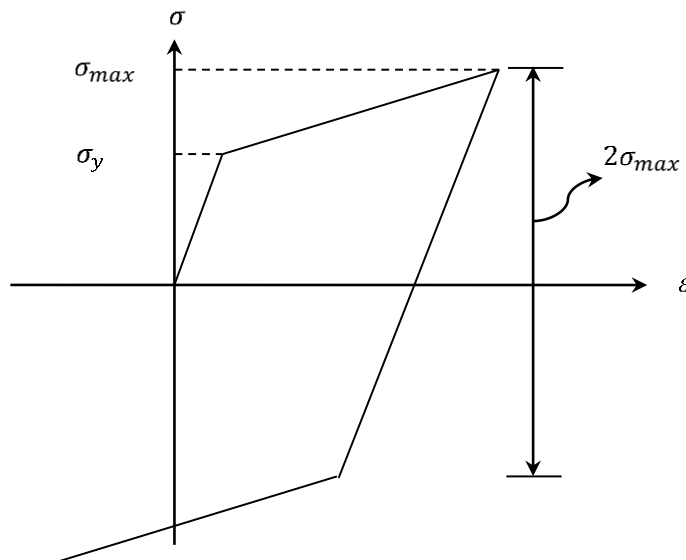


Fig. 11. Kinematic bilinear stress-strain diagram [49]

2.2.2 Steel Bar Element

In this study, the longitudinal and transverse reinforcing steel element models use 2-nodes LINK180 discrete bar elements in the ANSYS software. The axial forces at the ends of the LINK180 bars are as shown in Figure 12. The reinforcing steel is assumed to be fully bonded to the concrete.

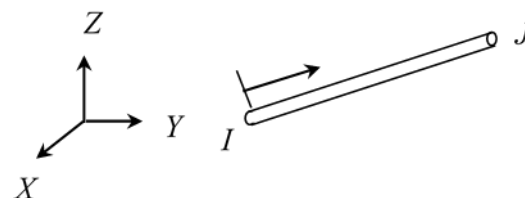


Fig. 12. LINK180 element

In these models, the element is formed through two points at the ends of the bar in the Cartesian coordinate system X, Y, Z. Each point has three degrees of freedom in the form of translations along the X, Y, and Z axes. The element does not experience bending, and the stresses are assumed to be

the same along the length of the element bar. The element stiffness matrix is expressed in equation (7), where A is the cross-sectional area of the element (mm^2), E_s is the modulus of elasticity of the steel (MPa), and L is the length of the element (mm). Multilinear Kinematic Hardening (MKIN) is used for reinforcing elements. In MKIN, the Bauschinger effect is assumed, with the total stress equal to twice the yield stress. The initial slope of the curve represents the material's elastic modulus, which is greater than zero. Subsequent slopes must be less than the elastic modulus.

$$[K] = \frac{AE_s}{L} \begin{bmatrix} 1 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ -1 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (7)$$

2.2.3 Boundary Conditions and Load

The flat slab model created in ANSYS software is identical to the specimen used in the experimental testing reported in previous research [41]. In the model, the width of the flat slab and column is in the x-axis direction, the height of the flat slab is in the y-axis direction, and the thickness of the flat slab is in the z-axis direction. In the flat slab specimen, clamps are placed to prevent the structural members from moving during testing. The middle of the column on the flat slab is given a concentrated downward load of $0.1f'_cA_g$. The top of the flat slab is provided with four spreader beams and four flexible hydraulic actuators that can move up and down. Each flat slab model has supports and loads at the exact locations as the specimen to ensure precise boundary conditions.

Monotonic loads are applied by four hydraulic actuators that press eight steel plates at the top ends of the four spreader beams. The load is then transferred from the spreader beams to the flat slab. The top of the columns is loaded through steel plates with the same load increment as the spreader beams [41]. In the numerical models, monotonic loads are applied to the top of each left and right end of the four spreader beams and the top of the columns.

2.2.4 Nonlinear Solving Solution

The solution of nonlinear equations satisfying boundary conditions using the finite element method in ANSYS software can be described as follows. When the stress-strain relationship for concrete and reinforcing steel elements is nonlinear, a solution to the nonlinear equations is required. In the structural plane, equation (8) is for linear systems.

$$[K]\{u\} = \{F^a\} \quad (8)$$

Where; $[K]$: structural stiffness matrix; $\{u\}$: degrees of freedom vector; $\{F^a\}$: load vector.

Equation (9) needs to be modified to satisfy the boundary conditions in nonlinear cases. An iterative process is required to obtain a solution to the equation. ANSYS provides some methods to solve nonlinear equations, including the Newton-Raphson method [50]. An iterative process of this method is applied to solve the nonlinear equations (9) and (10).

$$[K_i^T]\{\Delta u_i\} = \{F^a\} - \{F_i^{nr}\} \quad (9)$$

$$\{u_{i+1}\} = \{u_i\} + \{\Delta u_i\} \quad (10)$$

Where; $[K_i^T]$: structural stiffness matrix; $\{u_i\}$: degrees of freedom vector; $\{F_i^{nr}\}$: load vector

Iterations are required to obtain a convergent solution with the following workflow:

- Assume the value of $\{u_o\}$. The $\{u_o\}$ is the solution of the previous iteration step. Thus, in the first iteration, $\{u_o\} = \{0\}$.
- Obtain $[K_i^T]$, with $\{F_i^{nr}\}$ is from $\{u_i\}$ confirmation.

- Obtain $\{\Delta u_i\}$.
- Add $\{\Delta u_i\}$ to $\{u_i\}$ to obtain $\{u_{i+1}\}$

Figure 13 shows the next iteration process to solve a nonlinear equation. It provides a solution of the iteration process when the load factor $\{F_i^{nr}\}$ is equal to $\{F\}$, or less than a specific tolerance value of 0.001. In this study, the Full Newton-Raphson method was chosen for solving iterations using an ANSYS software, which built the initial stiffness matrix in each iteration.

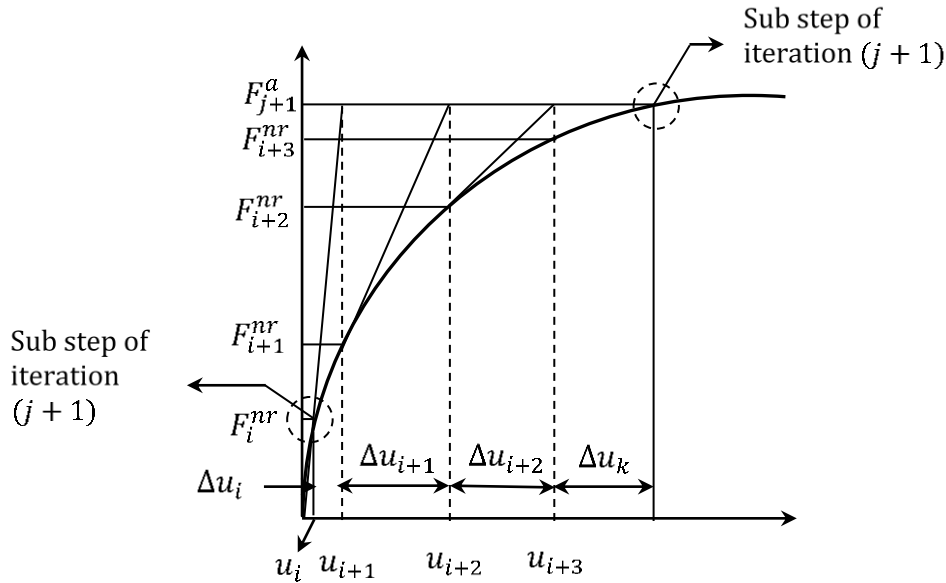


Fig. 13. Newton-Raphson iteration on one loading sub-step

2.3. Material Properties

In numerical modeling, the stress-strain relationship was based on the Hognestad equation, which has been shown to best match test results for NC and GC materials compared to other equations [51]. Equations (11) and (12) represents the stress-strain relationship according to Hognestad in ascending and descending curves, respectively. The tensile strengths of NC and GC are obtained from equation (13) [17], which yields tensile strengths closest to the test results for NC and GC [51]. The Poisson's ratios of NC and GC are assumed to be 0.2 [52] and 0.22 [53], respectively. For comparison, other equations of stress-strain in ascending and descending curves referring to Kent-Park, Saatcioglu-Razvi, and Mander are stated in equations (14-19). The Hognestad stress-strain curve is not much different compared to the Kent-Park, Saatcioglu-Razvi, and Mander curves in Figure 14.

$$f_c = f'_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_o} \right) - \left(\frac{\varepsilon_c}{\varepsilon_o} \right)^2 \right] \text{ (Hognestad; ascending)} \quad (11)$$

$$f_c = 0.85 f'_c \text{ (Hognestad; descending)} \quad (12)$$

$$f_s = 0.33 \sqrt{f'_c} \quad (13)$$

$$f_c = f'_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_o} \right) - \left(\frac{\varepsilon_c}{\varepsilon_o} \right)^2 \right] \text{ (Kent-Park; ascending)} \quad (14)$$

$$f_c = f'_c [1 - 100(\varepsilon_c - 0.002)] \text{ (Kent-Park; descending)} \quad (15)$$

$$f_c = f'_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) - \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^2 \right] \text{ (Saatcioglu-Razvi; ascending)} \quad (16)$$

$$f_c = f'_c + \left[\frac{f'_c - f_{c85}}{\varepsilon_{c0} - \varepsilon_{c85}} \right] (\varepsilon_c - \varepsilon_{c0}) \text{ (Saatcioglu-Razvi; descending)} \quad (17)$$

$$f_c = \frac{f'_c x r}{r - 1 + x^r} \text{ (Mander; ascending and descending)} \quad (18)$$

$$x = \frac{\varepsilon_c}{\varepsilon_{c0}}; r = \frac{E_c}{E_c - E_{sec}}; E_c = 5000 \sqrt{f'_c}; E_{sec} = \frac{f'_c}{\varepsilon_{c0}} \text{ (Mander)} \quad (19)$$

Where; ε_{c0} : concrete compressive strain at ultimate stress = 0.002; f'_c : concrete compressive stress (MPa); f'_c : uniaxial compressive strength (MPa); f_{c85} : 85% of uniaxial compressive strength (MPa); E_c : modulus of elasticity of concrete (MPa); f_s : tensile strength of concrete (MPa)

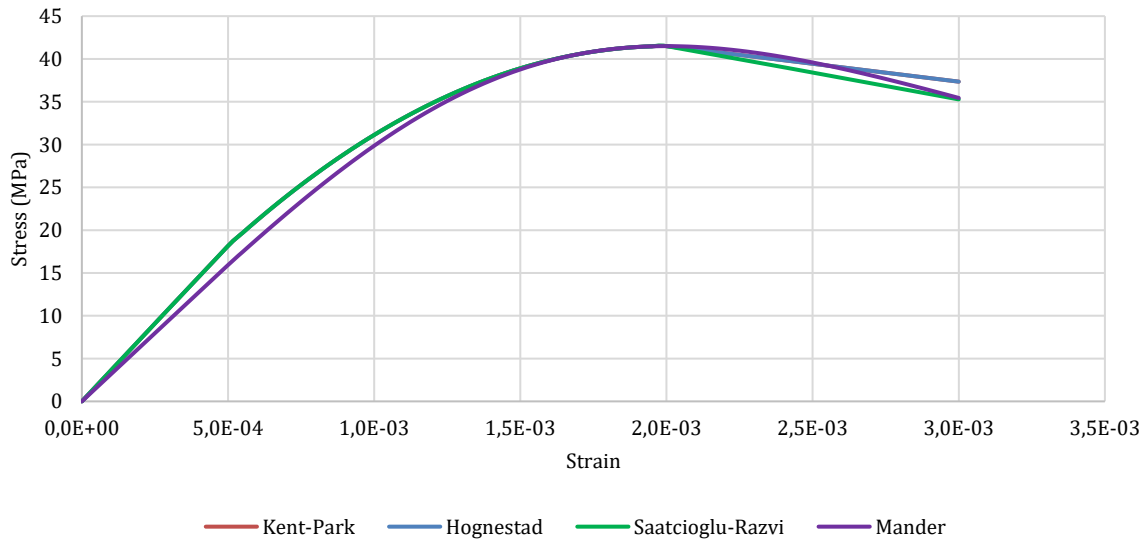


Fig. 14. GC stress-strain curves with Kent-Park, Hognestad, Saatcioglu-Razvi, and Mander equations

The fracture energy is expressed in equation (20) [35]. The maximum diameter of aggregates was 16 mm for NC [41] and 4.75 mm for GC [46]. The initial fracture energy for NC is 0.03 N/mm and 0.025 N/mm for GC [54]. Then, the fracture energies for NC and GC are 64.73 N/mm² and 67.70 N/mm², respectively.

$$G_f = G_{f0} [(f'_c/10)^{0.7}] \quad (20)$$

Where; G_f : fracture energy of concrete (N.m/mm²); G_{f0} : initial fracture energy (N/mm)

3. Results and Discussion

The load-displacement relationship curve is influenced by transverse reinforcement. The model loading process is the same as the experiment, namely by increasing the gravity load on the spreader beams in 40 kN increments [41]. Comparison of the experimental load-displacement relationship curves and flat slab-column modeling of NC-0, NC-0-F, NC-5, and NC-5-F is shown in Figure 14. The NC-0, NC-0-F, NC-5, and NC-5-F load-displacement curves almost match, due to the overlapping curves resulting from nearly the same slopes and stiffnesses. The difference in load and displacement at the initial and ultimate loading conditions is less than 10% as stated in Table 2. It indicates the accuracy of the modeling [47,48]. The highest punching shear capacity is achieved by the NC-8-F, followed by NC-5, NC-5-F, and NC-0, NC-0-F. The transverse reinforcement ratio

affects the punching shear capacity and deformability [55]. Without transverse reinforcement, NC-0 and NC-0-F cannot achieve vertical displacements as far as NC-5, NC-5-F, NC-8, and NC-8-F. NC-8 and NC-8-F exhibit more brittle properties than NC-5 and NC-5-F. Thus, their ultimate displacements are less than those of NC-5 and NC-5-F.

The NC-0 and NC-0-F load-displacement curves in Figure 15 show that the absence of transverse reinforcement results in the lowest shear load resistance, leading to the lowest maximum load and displacement compared to the NC-5, NC-5-F, NC-8, and NC-8-F counterparts. At a specific loading condition, the displacement in NC-0 is the most significant value compared to NC-5 and NC-8 due to the absence of transverse reinforcement. This behavior is also influenced by the quality of longitudinal reinforcement steel in NC-0, which is lower than that in NC-5 and NC-8, as described in Table 1. The ultimate load and displacement values in specimens NC-0 and NC-5, as well as in models of NC-0-F, NC-5-F, and NC-8-F, are shown in Table 2. The ultimate vertical loads in the NC-0-F, NC-5-F, and NC-8-F models are the same as those applied to the NC-0 and NC-5 specimens, with increments of 40 kN. The differences in the ultimate displacement values of the NC-0 and NC-5 specimens and the NC-0-F and NC-5-F numerical models are 7.909% and 1.563%, respectively. Thus, these models fulfil the accuracy criterion, with differences less than 10% [47,48]. The NC-8-F model was created to analyze the performance of flat slabs with an increased transverse reinforcement of $5\phi 8-72$. This model represented a flat slab with denser transverse reinforcement in resisting punching shear. This condition results in a higher ultimate load of 616 kN but decreases ductility [56], as evidenced by the lower ultimate displacement of 19.178 mm compared with the NC-5 and NC-5-F counterparts. Table 3 shows crack loads and displacements, as well as the initial stiffnesses of NC specimens and models. The discrepancies are less than 10% which shows modeling accuracy [47,48].

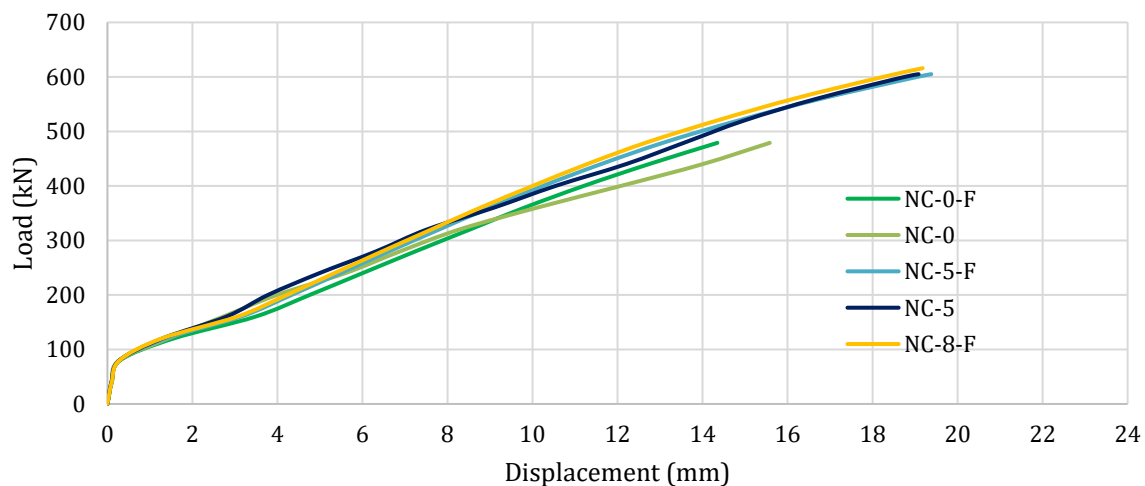


Fig. 15. Curves of load and displacement of the NC specimens and models

Table 2. Ultimate loads and displacements of NC specimens and models

Specimen			Model			
Type of flat slab	Ultimate load, F_u	Ultimate displacement, δ_u	Type of flat slab	Ultimate load, F_u	Ultimate displacement, δ_u	Difference
	(kN)	(mm)		(kN)	(mm)	(%)
NC-0	479	15.58	NC-0-F	479	14.35	7.91
NC-5	605	19.08	NC-5-F	605	19.38	1.56
-	-	-	NC-8-F	616	19.18	-

Table 3. Crack loads, displacements, and initial stiffnesses of NC specimens and models

Type of flat slab	Specimen			Type of flat slab	Model			Discrepancy (%)
	Crack load, F_{cr}	Crack displ., δ_{cr}	Initial stiffness, K_i		Crack load, F_{cr}	Crack displ., δ_{cr}	Initial stiffness, K_i	
	(kN)	(mm)	(kN/m)		(kN)	(mm)	(kN/m)	
NC-0	80	0.269	0.297	NC-0-F	80	0.294	0.272	8.57
NC-5	80	0.282	0.283	NC-5-F	80	0.288	0.277	2.10
-	-	-	-	NC-8-F	80	0.285	0.280	-

3.1. Load-displacement Curves of GC Flat Slabs

The flat slab models with GC properties were incrementally vertically loaded by 40 kN until failure. The detailed dimensions, longitudinal and transverse reinforcement of the geopolymer flat slab models were the same as those of the NC flat slab specimens as described in Table 1. The GC-0-F flat slab model only achieved an ultimate load of 679 kN, while the GC-5-F and GC-8-F flat slab models achieved an ultimate load of 805 kN and 696 kN, respectively, as shown in Figure 16. The performance of GC-0-F in resisting shear loads was affected by the absence of transverse reinforcement in the slab. GC-5-F could withstand the highest shear load compared to the other two models. The GC-8-F model achieved lower ultimate shear load and displacement than the GC-5-F model because the larger transverse reinforcement diameter imparted brittle behavior [56], leading the flat slab to collapse more rapidly.

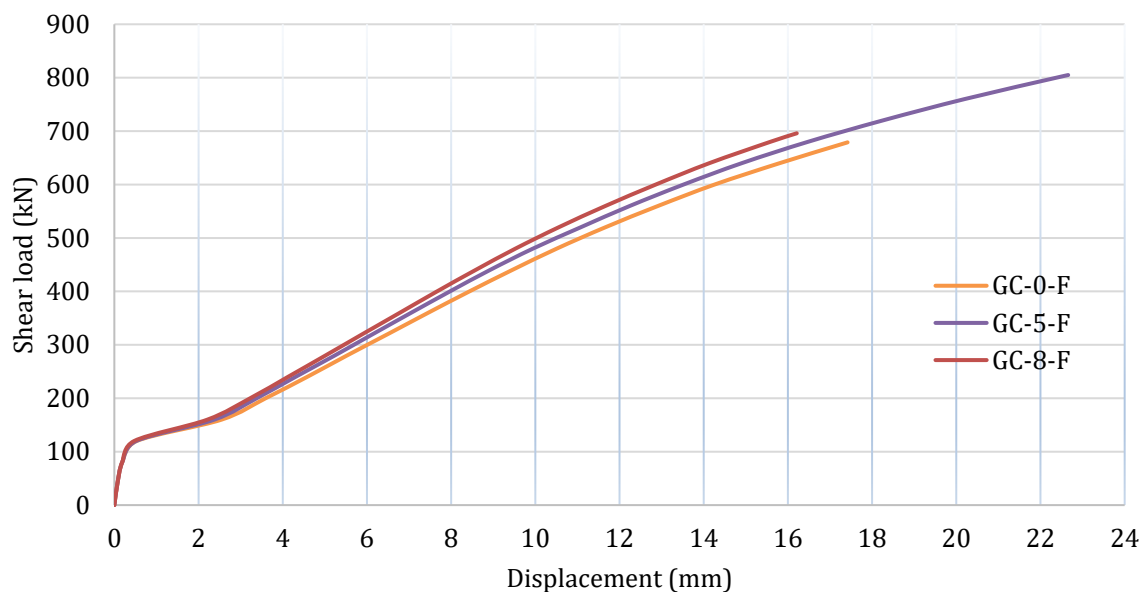


Fig. 16. Curves of load and displacement of GC-0, GC-5, and GC-8 models

Table 4 shows the crack, ultimate shear load, and displacement of GC flat slab models with different shear reinforcement configurations. All GC models cracked at a load of 120 kN, which was higher than the crack load of the NC specimens and models. The crack deflection in the GC models was greater than that experienced by the NC specimens and models. This indicates that the GC flat slab performed better under elastic conditions before cracking. Initial stiffness increases with the diameter of the transverse reinforcement. The ultimate shear load of each GC flat slab model is higher than that of an NC flat slab with the same reinforcement details. The GC-0-F, GC-5-F, and GC-8-F flat slab models achieved ultimate shear loads of 679 kN, 805 kN, and 696 kN, respectively. The $\phi 8$ reinforcement tends to produce a stiffer but less ductile response in the GC-8-F flat slab, leading to a lower ultimate shear load than in the GC-5-F flat slab. The GC-5-F model achieved the highest

ultimate shear force and displacement among the flat slab models, surpassing the GC-0-F and GC-8-F models.

The GC-0-F flat slab model without transverse reinforcement failed to resist the ultimate shear load. This behavior differs from the NC-0-F flat slab model owing to the higher compressive strength of GC compared to NC, leading to a higher ultimate load. However, the absence of transverse reinforcement reduced the shear capacity. Thus, the GC material could not resist the ultimate shear load. In the GC-0-F flat slab model, concrete crushing occurred, followed by failure at the ultimate condition [17]. The ultimate shear achieved by the GC-0-F flat slab model was 679 kN, with a difference of 200 kN compared to the NC-0-F counterpart, which only achieved an ultimate shear of 479 kN. It indicated that the higher compressive strength of GC needs to be supported by transverse reinforcement in resisting shear loads.

Table 4. Crack, ultimate load, and displacement of GC models

Type of flat slab	Model				
	Crack load, F_{cr}	Crack displacement, δ_{cr}	Initial stiffness, K_i	Ultimate load, F_u	Ultimate displacement, δ_u
	(kN)	(mm)	(kN/mm)	(kN)	(mm)
GC-0-F	120	0.530	0.227	679	17.418
GC-5-F	120	0.518	0.232	805	22.658
GC-8-F	120	0.471	0.255	696	16.211

The microstructure of GC also influences the difference in behavior of NC and GC flat slabs. The fly ash-based, low-calcium binder content in GC affects shear and bond behavior differently than in NC. Geopolymer concrete in a previous study [57] contained low-calcium fly ash, fine and coarse aggregates, and an alkaline solution. The test specimens, consisting of geopolymer blocks, were subjected to a concentrated downward vertical load. The average first cracking load and ultimate deflection of the geopolymer blocks were 22% higher than those of conventional concrete. It indicates that the inter-matrix bond in the geopolymer concrete mixture has greater adhesive strength than conventional concrete. Fly ash-based geopolymer concrete produces lightweight fine aggregate with a stronger bond than fly ash-based geopolymer concrete using quartz aggregate [58]. It is consistent with research on the relationship between density and concrete compressive strength. The lighter the concrete density, the stronger the bond [59].

The shear strength of both conventional and geopolymer concrete is significantly influenced by the concrete's compressive strength [60]. Shear cracking and the ultimate shear capacity of geopolymer blocks increase with increasing concrete strength [61]. Factors that influence the strength of the matrix bond in resisting shear are the type and content of fly ash, concrete compressive strength, sodium hydroxide (NaOH) concentration [62-69] with an optimum concentration of around 12% to 14% and molarity (solution concentration expressed in moles per liter of solution) of 10M to 15M, aggregate to binder mass ratio, mix design, curing, and coating [68].

3.2. Stress Contour of NC Flat Slabs

Stress contours are used to visualize the stress distribution on a flat slab in several colors within a specific stress range. Stress contours provide information on the location of structural elements susceptible to failure or damage based on the stress they cause, as well as the locations of maximum and minimum stresses. Compressive and tensile stresses are expressed as negative and positive values, respectively. When a flat slab is subjected to a monotonic load, the structure responds by resisting compressive or tensile stresses depending on the placement and loading location. Figure 17 (a) shows the top of the NC-0-F flat slab, including the areas around the columns and four spreader beams. The orange compressive stresses of 3-12 MPa indicate that the NC-0-F flat slab experiences compressive stresses lower than the normal concrete compressive strength of 30 MPa. Some parts of the two spreader beams and a few small yellow areas experience compressive stresses of 12-21 MPa. The red areas on the top of the columns, slab, and spreader beams

experienced stresses from 3 MPa compression to 6 MPa tension. Although the dominant stress in the red areas is compressive, the stress contours extend to 6 MPa of tensile stress. Figure 17 (b) shows the area around the columns for the NC-5-F flat slab with compressive stresses of 3-12 MPa (orange) and 12-21 MPa (yellow), which are wider than those for the NC-0-F flat slab. This is due to the NC-5-F flat slab experiencing higher shear loads and ultimate displacements than NC-0-F, resulting in higher compressive stress. The stress contour of the NC-8-F flat slab is shown in Figure 17 (c). The area around the column is fitted with shear reinforcement, resulting in an ultimate shear load that produces compressive stresses of 3-12 MPa (orange) and 12-21 MPa (yellow). The stress area in NC-8-F is similar to that of NC-5-F. It indicates that increasing the reinforcement diameter from 5 mm to 8 mm does not increase the ultimate stress.

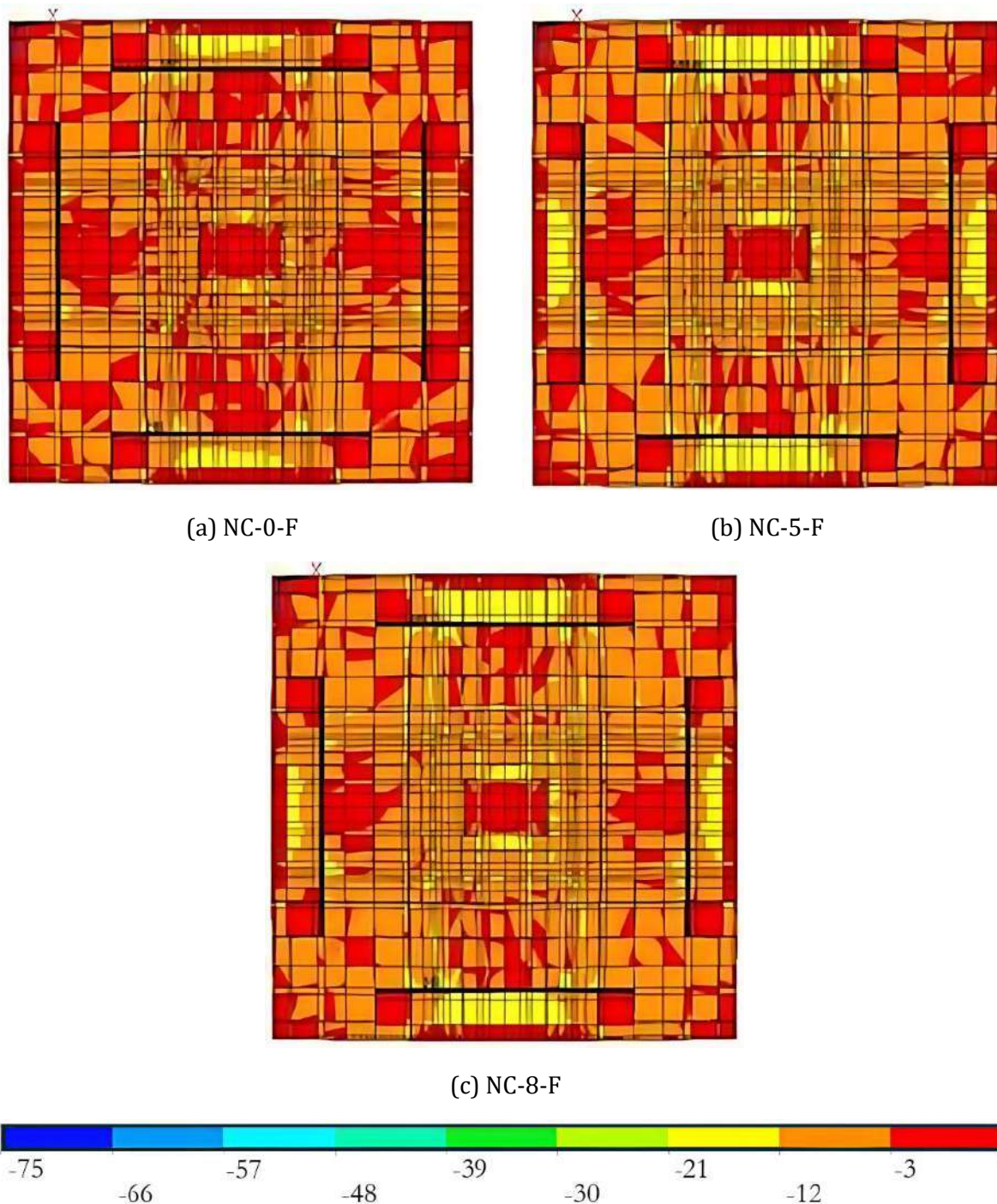
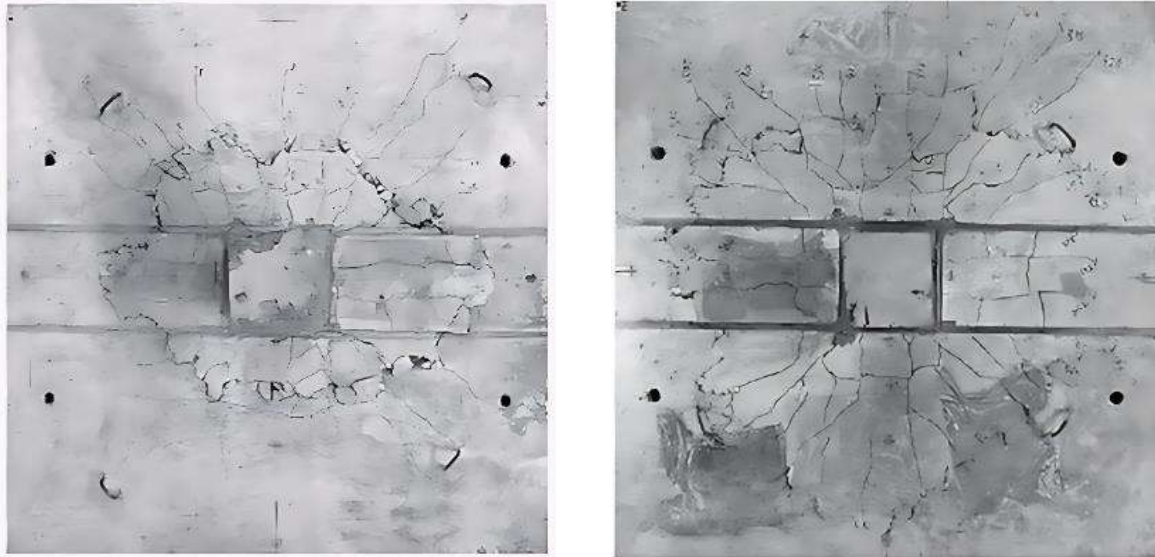


Fig. 17. Top side of stress contours of the NC flat slab models

The crack patterns in NC-0 and NC-5 specimens are shown in Figure 18. Due to the loading on the top of the column and four spreader beams placed on the flat slab, punching shear failure occurred in NC-0 and NC-5 [41]. The crack pattern that occurred in the NC-0 specimen in Figure 18(a) corresponds to the stress contour of NC-0-F in Figure 17(a). The four spreader beams and the flat slab experienced compressive stresses of 3 to 21 MPa, which led to punching shear. Specimen NC-5 experienced punching shear failure, with the area around the column formed by cracks in the flat slab in Figure 18(b) being broader than that of NC-0. This correlates to the stress contour of NC-5-F in Figure 17(b), where the compressive stress area of 12 to 21 MPa is wider than that of NC-0-F.



(a) Crack pattern of the NC-0 specimen

(b) Crack pattern of the NC-5 specimen

Fig. 18. Top side of the crack pattern of the NC flat slab specimens [41]

Figure 19(a) shows the red stress contour on the bottom side of the NC-0-F flat slab, which is predominantly subjected to tensile stress just below the column. The slab area around the column is indicated in light and dark green, with maximum compressive stresses of 21-30 MPa and 30-39 MPa, respectively. It shows that the dark green area fails due to compressive stress exceeding the normal concrete compressive strength of 30 MPa. This behavior indicates that the bottom side of the slab is more prone to failure than the top side, owing to the top side being compressed and resisted by the concrete, while the bottom side withstands tensile loads through longitudinal reinforcement [70].

Figure 19(b) shows the stress that occurs around the column on the NC-5-F flat slab with a larger area due to monotonic loads on the top side of the column and the spreader beam support indicated by light green, dark green, turquoise, and light blue with compressive stresses of 21-30 MPa, 30-39 MPa, 39-48 MPa, and 48-57 MPa, respectively. It shows that the area experiencing collapse is wider because the presence of $5\phi 5$ shear reinforcement of NC-5-F flat slab causes the ultimate shear load to be higher than that of the NC-0-F flat slab. Most of the compressive stress in the flat slab does not exceed the concrete compressive strength of 30 MPa, which is indicated by light green (21-30 MPa), yellow (12-21 MPa), and orange (3-12 MPa).

Figure 19(c) shows the stress that occurs in the area around the column in the NC-8-F flat slab with light green, dark green, a little turquoise green, and a little light blue with maximum stress of 21-30 MPa, 30-39 MPa, 39-48 MPa, and 48-57 MPa, respectively. Most of the stress contours in yellow (12-21 MPa) and orange (3-12 MPa) do not exceed the concrete compressive strength of 30 MPa. The area of the 12-21 MPa compressive stress contour (yellow) is wider on the NC-8 flat slab than on the NC-5 flat slab because the achieved ultimate shear load is higher.

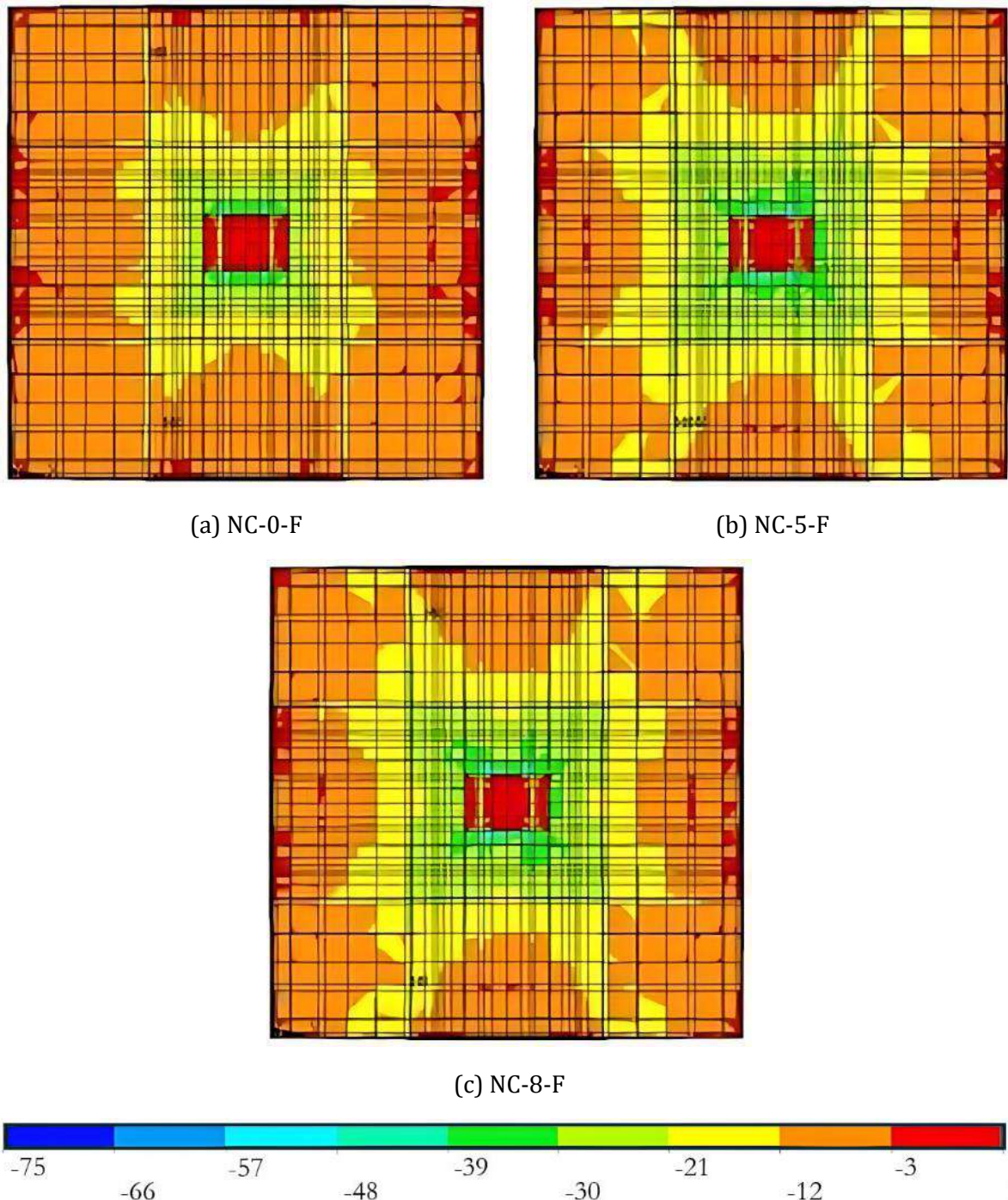


Fig. 19. Bottom side of stress contours of the NC flat slab models

3.3. Stress Contour of GC Flat Slab Models

Figure 20(a) shows the stress contours of the top side of the flat slab around the column and the spreader beam supports for the GC-0-F flat slab without shear reinforcement in the slab around the column. Compressive stresses of 1.08-9.13 MPa (orange), 9.16-17.23 MPa (yellow) are distributed and dominate the slab. The slab parts around the column experience a greater compressive stress of 17.23-25.31 MPa (light green). There is a small area that experiences a compressive stress exceeding the concrete compressive strength (30 MPa), namely 25.31-33.39 MPa (dark green). The middle of the four spreader beams experiences greater compressive stress than the ends, at 9.16-17.23 MPa (yellow) and 17.23-25.31 MPa (light green). Each end of the spreader beam and column experiences a lower compressive stress of 1.08 MPa (compression) to 7 MPa (tension). Although the red stress contour is predominantly compressive, the color display includes tensile stress.

Figure 20(b) shows the top side of the GC-5-F flat slab. The compressive stress of 9.16-17.23 MPa (yellow) in the slab occurs over a larger area than the GC-0-F flat slab. A higher compressive stress of 25.31-33.39 MPa (dark green) develops in the middle of the two spreader beams. It occurs because the ultimate shear load in GC-5-F is higher than that of the GC-0-F flat slab due to the presence of $5\phi 5$ shear reinforcement around the columns. The stress distribution on the top side of the GC-8-F flat slab in Figure 20(c) shows that the ultimate shear load is almost the same as that of GC-0-F, causing similar stress behavior. The use of $5\phi 8$ shear reinforcement around the columns causes the flat slab to become brittle and reduces the shear load and ultimate displacement [55]. The GC-8-F model experiences lower compressive stress at the ultimate condition than the GC-5-F model due to its less ductile behavior (Table 7). Since the ultimate load of the GC-8-F model is lower than that of the GC-5-F (Tables 2 and 4), its ultimate compressive stress is lower. The GC-5-F model achieves a compressive stress of 25.31 to 33.39 MPa (dark green), while the GC-8-F model only achieves a compressive stress of 17.23 to 25.31 MPa (light green) on the top side of the flat slab as shown in Figures 20(b) and (c).

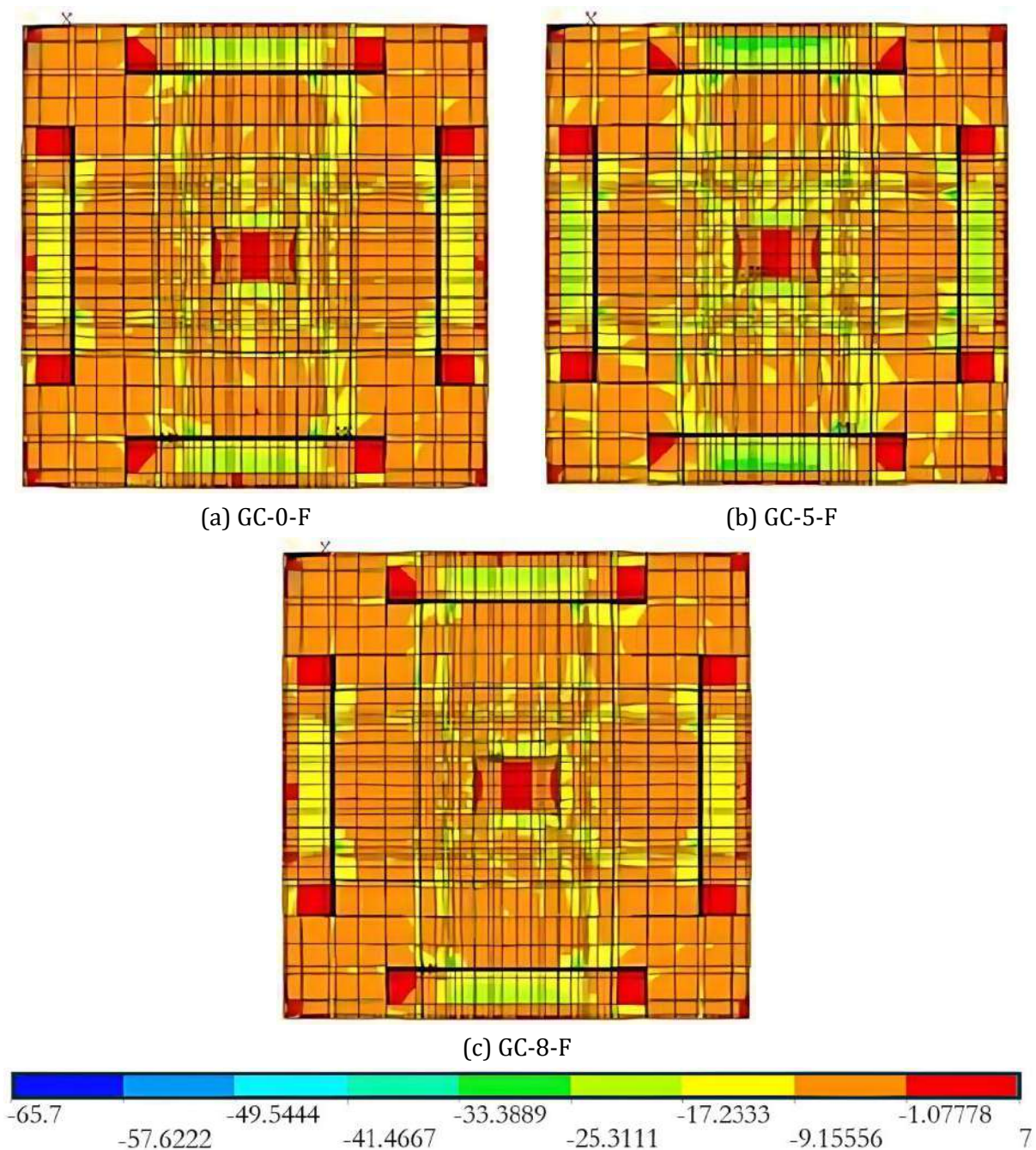


Fig. 20. Top side of stress contours of the GC flat slab models

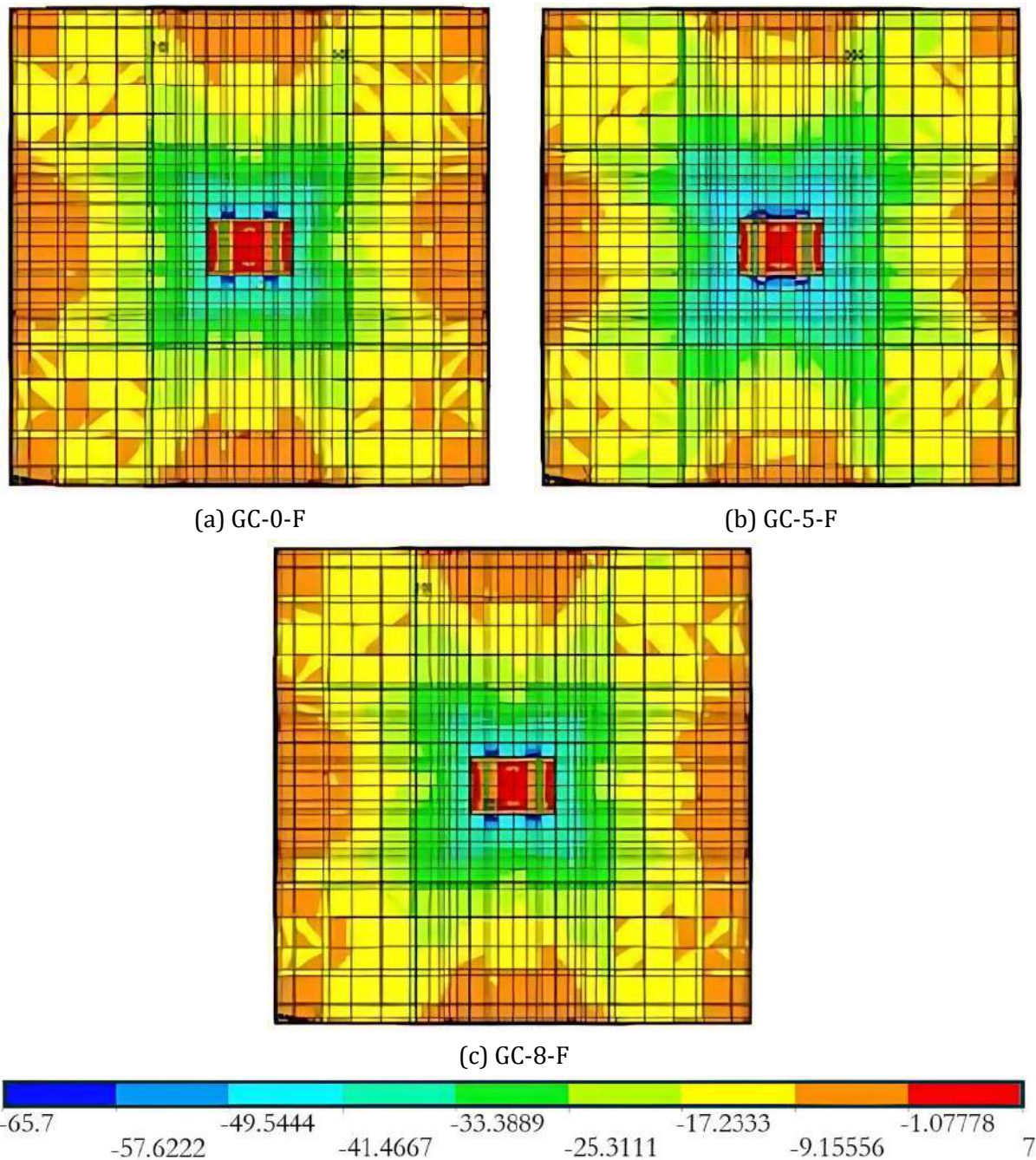


Fig. 21. Bottom side of stress contours of the GC flat slab models

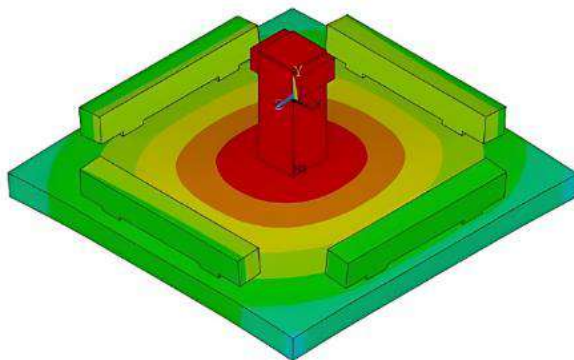
Figure 21(a) depicts the compressive stress on the underside of the GC-0-F flat slab that occurs in the area around the column. There are small areas colored turquoise, light blue, and blue with compressive stress ranges of 33.39-41.47 MPa, 41.47-49.54 MPa, and 49.54-57.62 MPa, respectively. An area of the highest compressive stress contour of 57.62-65.7 MPa appears on the slab directly below the column perimeter. This indicates that this area is experiencing significant collapse due to flat slab compressive stress exceeding the GC compressive strength of 41.5 MPa. However, most flat slabs survive failure and experience compressive stress below 41.5 MPa, as shown by the red-to-turquoise-green areas with the highest compressive stress of 41.5 MPa. The underside of the column exhibits the smallest compressive stress in red (1.08 MPa to 7 MPa-tensile stress) and orange (1.08 to 9.16 MPa).

Figure 21(b) depicts the compressive stresses that occur in the area around the wider column indicated by light blue, blue, and dark blue with values of 41.47-49.54 MPa, 49.54-57.62 MPa, and 57.62-65.70 MPa, respectively, which exceed the compressive strength of geopolymer concrete of 41.5 MPa. It indicates a wider collapse occurred owing to the achievement of a higher ultimate

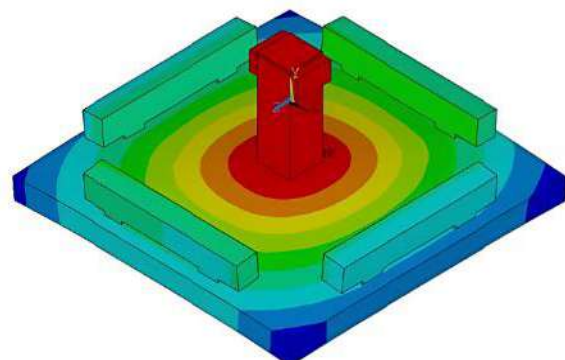
shear load than that achieved in the GC-0-F flat slab in this area. The presence of $5\phi 5$ shear reinforcement increases the shear capacity of the slab [55]. The GC-8-F flat slab shows a very similar stress contour distribution to its GC-0-F counterparts because these flat slabs achieve very close values of ultimate shear loads, as shown in Figure 21(c). The use of 8 mm diameter shear reinforcement in the GC-8-F flat slab results in less ductile behavior that restricts it from developing deformation that leads to a lower ultimate shear load than the GC-5-F flat slab. This condition also influences the stress. The bottom side of the flat slab of the GC-8-F model at the ultimate condition reaches a compressive stress of 41.47 – 49.54 MPa (light blue) and 57.62 – 65.7 MPa (dark blue) in a narrower area than that of the GC-5-F model, as shown in Figures 21 (b) and (c). It is because the ultimate load on the GC-8-F model is lower than that of the GC-5-F model. This compressive stress exceeded GC compressive strength of 41.5 MPa in the light and dark blue areas and lead to the ultimate condition.

3.4. Displacement Contours of the NC Flat Slab Models

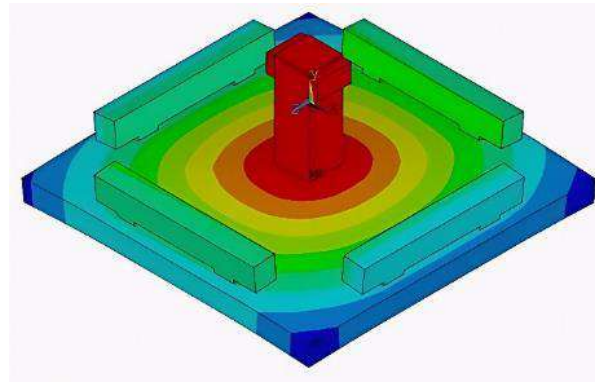
Displacement contours show the positions of the elements and nodes of the flat slabs under vertical downward monotonic loads. The achieved ultimate loads vary across flat slab models owing to the shear capacity provided by the longitudinal and transverse reinforcements. This condition affects the magnitude of the displacement expressed in the contours. The ultimate load of the NC-0-F flat slab is 479 kN with an ultimate displacement of 15.581 mm. Figure 22(a) shows the dominant downward vertical displacement on the four edges of the flat slab due to the application of monotonic loads on the four spreader beams of 14.61 to 17.58 mm (turquoise green). The displacement of the flat slab towards the center decreases because the load pressure from the spreader beam is reduced. It can be seen with displacements of 8.67 to 11.64 mm (yellow), 2.72 to 5.69 mm (orange), and +0.25 to 2.72 mm (red). The columns and the center of the flat slab experience the same range of displacements. Figure 22(b) shows the displacement contour of the NC-5-F flat slab. The presence of $\phi 5-72$ transverse reinforcement increases the ultimate load and displacement, resulting in larger displacements than the NC-0-F counterpart, which are 23.53-26.50 mm (dark blue) at the four corners of the flat slab, 20.56-23.53 mm (medium blue) at the four edges, and 17.58-20.56 mm (light blue) at the spreader beams. The magnitude of the displacement decreases as the element location approaches the center, resembling that of the NC-0-F flat slab. The NC-8-F flat slab has $\phi 8-72$ transverse reinforcement. Thus, it achieves the highest ultimate load among all other models but exhibits a lower ultimate displacement than the NC-5-F flat slab, with a difference of 1.06% (Table 2), as shown in Figure 22(c). In general, the displacement in the NC-8-F flat slab is similar to that of NC-5-F. It shows that increasing the percentage of transverse reinforcement does not result in a significant difference in the displacement contour. Otherwise, the addition of transverse reinforcement reduces ductility and leads to a lower ultimate displacement.



(a) NC-0-F under load of 479 kN



(b) NC-5-F under load of 605 kN



(c) NC-8-F under load of 616 kN

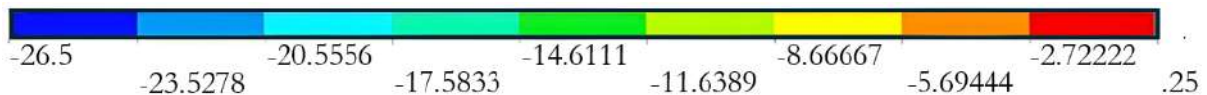
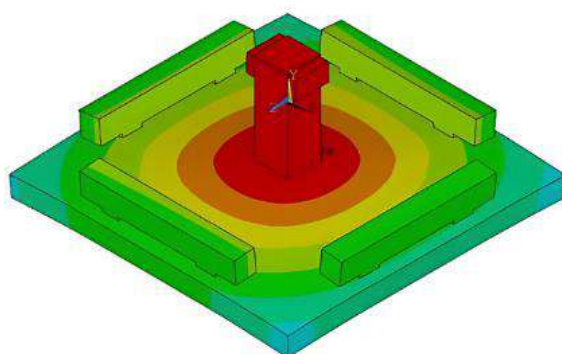


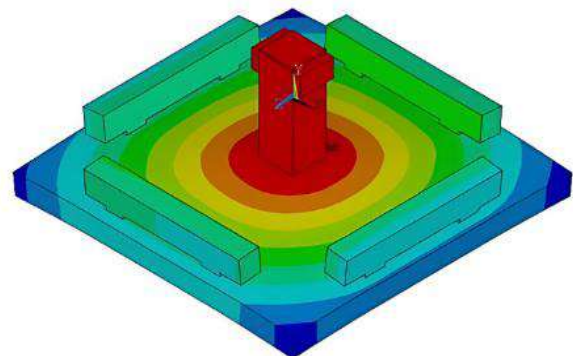
Fig. 22. Top side of displacement contours of the NC flat slab models

3.5. Displacement Contours of the GC Flat Slab Models

The downward displacement contour of GC-0-F in Figure 23(a) shows the position of the four corners of the flat slab that experienced the most significant displacement compared to other parts, namely 20.92-24.44 mm (light blue). The four edges of the flat slab performed smaller displacements, ranging from 17.39 to 20.92 mm (turquoise green). Spreader beams that received monotonic loads exhibited displacements of 13.86-17.39 MPa (dark green). The displacements decreased towards the center of the flat slab, ranging from 10.33 to 13.86 mm (light green), 6.80 to 10.33 mm (yellow), 3.28 to 6.81 mm (orange), and +0.25 (upward) to 3.28 mm (red). The displacements in the GC-0-F flat slab were greater than those of the NC-0-F counterpart owing to the presence of GC material that provided a greater capacity to withstand monotonic loads at the ultimate condition. The GC-5-F flat slab in Figure 23(b) achieved a greater ultimate displacement than the GC-0-F flat slab owing to the shear load capacity from the $\phi 5$ -72 transverse steel bars. The four corners of the GC-5-F flat slab sustained displacements of 27.97-31.50 mm (dark blue). On the side towards the center at the corners of the flat slab, the displacements reached 24.44-27.97 mm (medium blue). Towards the center of the flat slab, the displacements decreased to 20.92-24.44 mm (light blue), 17.39-20.92 mm (turquoise green) on the spreader beams, 13.86-17.39 mm (dark green), 10.33-13.86 mm (light green), 6.81-10.33 mm (yellow), 3.28-6.81 mm (orange), and +0.25-3.28 mm (red).



(a) GC-0-F under load of 679 kN



(b) GC-5-F under load of 805 kN

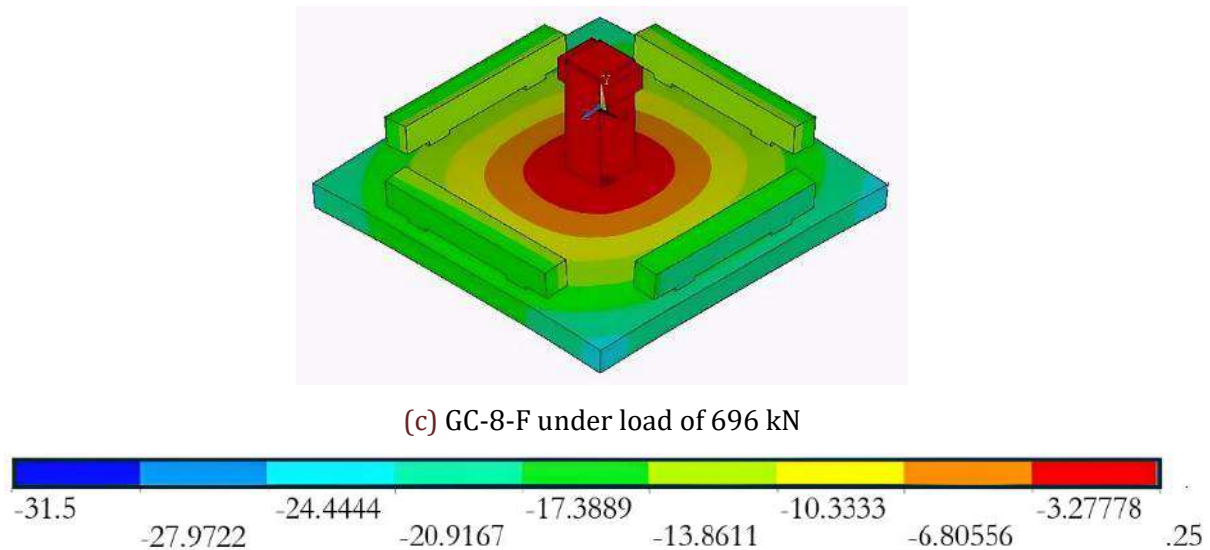


Fig. 22. Top side of the displacement contours of the GC flat slab

Figure 23(c) shows the contours of the displacements of the GC-8-F flat slab. The presence of transverse reinforcement with a larger diameter in the GC-8-F flat slab, compared with that in the GC-5-F counterpart, resulted in brittle behavior [55]. The ultimate displacement in GC-8-F was only 16.211 mm (Table 3), with a maximum displacement range of 20.92-24.44 mm (light blue) at the four corners. The four edges of the GC-8-F flat slab and spreader beams performed displacements of 17.39-20.92 mm (turquoise green). The displacements decreased toward the center and resembled those in the GC-0-F counterpart.

3.6. Ductility of NC Flat Slab Specimens

Ductility is the ratio of the ultimate displacement (Δ_u) to the yield displacement (Δ_y) [71]. Ductility represents the ability of a structure to deform and withstand loads up to the ultimate condition before failure [56,72]. The position of the yield point on the load-displacement relationship curve was determined by the method of equality of areas above and below the curve formed by the two linear lines [73]. The determination of the yield point from the load-displacement curve of the NC-0 flat slab specimen is shown in Figure 24. Table 5 shows the yield load (F_y), yield displacement (δ_y), ultimate load (F_u), ultimate displacement (δ_u), and ductility ratio. Using the same method, the yield point position of the NC-5 specimen was obtained. The ductility ratio of the NC-0 and NC-8 flat slab specimens is less than 2.0, and thus they are categorized as low-ductility [73]. The brittle nature of these specimens was due to the slab's large area, which provided adequate stiffness and resisted monotonic loads. The NC-5 flat slab specimen exhibited higher ultimate loads and displacements due to the presence of $\phi 5-72$ transverse reinforcement, which increased the shear capacity.

Table 5. Yield load, yield displacement, and ductility of NC specimens

Type of flat slab (experimental)	Yield		Ultimate		Ductility ratio μ
	Load, F_y	Displacement, δ_y	Load, F_u	Displacement, δ_u	
	(kN)	(mm)	(kN)	(mm)	
NC-0	451.433	11.271	479	15.580	1.382
NC-5	557.335	13.895	605	19.380	1.395

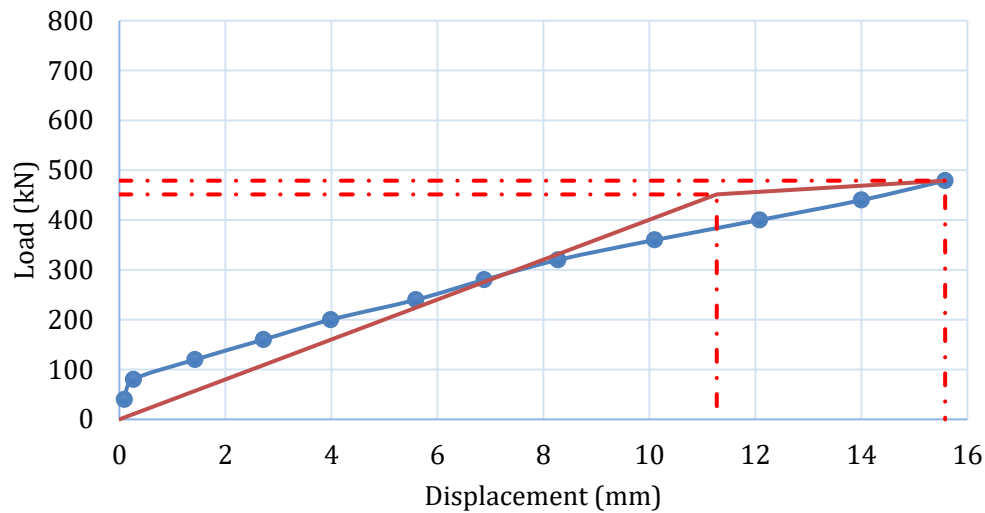


Fig. 24. Yield point determination of NC-0 specimen

3.7. Ductility of NC Flat Slab Models

Using the similarity area method [73], the yield loads and displacements were obtained for the flat slab models of NC-0-F, NC-5-F, and NC-8-F. The presence of $\phi 5-72$ transverse reinforcement increased the shear capacity, thus slowing the yield condition in the NC-5-F than the NC-0-F flat slab. Similarly, the $\phi 8-72$ transverse reinforcement in the NC-8-F model delayed the yield condition compared to NC-5-F. However, the larger shear reinforcement area led to more brittle behavior. Thus, the ultimate displacement of the NC-8-F flat slab was lower than that of NC-5-F even though its ultimate load was higher. This reinforcement resulted in a lower ductility ratio in the NC-8-F compared to the NC-5-F flat slab. All NC flat slab models exhibited ductility ratios of less than 2, as shown in Table 6; thus, they were categorized as low ductile. It was owing to the large area of the flat slab, which provided a high stiffness, resulting in low ultimate displacements. The differences in the ductility ratios of the NC-0 specimen and NC-0-F model are 9.70%, while the NC-5 specimens and NC-5-F model exhibit a 1.08% discrepancy in the ductility ratio.

Table 6. Determination of the yield point and ductility ratios of NC flat slab models

Type of flat slab (model)	Yield		Ultimate		Ductility ratio
	Load	Displacement	Load	Displacement	
	F_y (kN)	δ_y	F_u (kN)	δ_u	
NC-0-F	458.416	11.498	479	14.349	1.248
NC-5-F	562.040	14.041	605	19.381	1.380
NC-8-F	603.130	14.951	616	19.178	1.283

3.8. Ductility of GC Flat Slab Models

The ductility ratios of the GC flat slab models are shown in Table 7. Each GC flat slab ductility ratio is higher than its NC counterpart. This behavior indicates that the GC material influences flat slab properties by increasing ductility. However, all GC flat slab models are categorized as low-ductility due to ratios below 2 [73,74]. The GC-5-F flat slab model is more ductile than the GC-0-F counterpart, owing to higher ultimate loads and displacements supported by the presence of $\phi 5-72$ transverse reinforcement that provided a higher shear capacity. However, the excessive transverse reinforcement of $\phi 8-72$ resulted in a lower ductility ratio for the GC-8 flat slab than the GC-5 counterpart. This behavior is similar to NC-8-F, which exhibits less ductility compared to NC-5-F in Table 6. The transverse reinforcement of $\phi 8-72$ and the higher compressive strength of GC (41.5 MPa) compared to NC (30 MPa) decrease ductility. Thus, the GC-8-F model achieves a smaller ultimate displacement than GC-5-F, as shown in Table 7.

Table 6. Determination of the yield point and ductility ratios of NC flat slab models

Type of flat slab (model)	Yield		Ultimate		Ductility ratio μ
	Load	Displacement	Load	Displacement	
	F_y (kN)	δ_y	F_u (kN)	δ_u	
NC-0-F	458.416	11.498	479	14.349	1.248
NC-5-F	562.040	14.041	605	19.381	1.380
NC-8-F	603.130	14.951	616	19.178	1.283

Table 7. Determination of yield point and ductility ratios in GC flat slab models

Type of flat slab (model)	Yield		Ultimate		Ductility, μ	Differences with NC models (%)
	Load, F_y	Displacement, δ_y	Load, F_y	Displacement, δ_y		
	(kN)	(mm)	(kN)	(mm)		
GC-0-F	616.781	12.814	679	17.418	1.359	8.920
GC-5-F	760.042	16.301	805	22.658	1.390	0.696
GC-8-F	649.661	12.382	696	16.211	1.309	2.071

3.9. Stiffness of NC Flat Slab Specimens

Stiffness is the ratio of load to displacement at a specific loading condition. As the load increases monotonically, expressed in Time Load, the stiffness of a flat slab decreases due to extended element damage. Stiffness degradation occurs due to the reduced capacity and is expressed as the ratio of the stiffness to the yield stiffness. The curves that correlate stiffness and stiffness degradation with Time Load on NC-0 and NC-5 specimens are shown in Figures 25 and 26, respectively. The stiffness of NC-0 and NC-5 flat slabs at the 1st load step was 419.697 kN/mm and 424.513 kN/mm, respectively. At the 2nd and 3rd load steps, the stiffness dropped drastically to 297.319 kN/mm and 83.996 kN/mm for NC-0, while at NC-5, the stiffnesses were 283.297 kN/mm and 93.042 kN/mm. Since the 4th load step, the stiffness was relatively stable, although it continued to decrease through the 12th load step for NC-0 and the 16th for NC-5. At the 4th Time Load, the stiffness of NC-0 was 58.825 kN/mm. It continued to decrease to 30.742 kN/mm. The stiffness of the NC-5 flat slab at the 4th Time Load was 57.255 kN/mm. It revealed that the significant damage to the concrete occurred at the 2nd and 3rd load steps. Furthermore, concrete cracks and yielded steel bars occurred, but did not cause a drastic decrease in stiffness. It followed the load-displacement relationship curve of the NC-0 and NC-5 flat slab specimens in Figure 15.

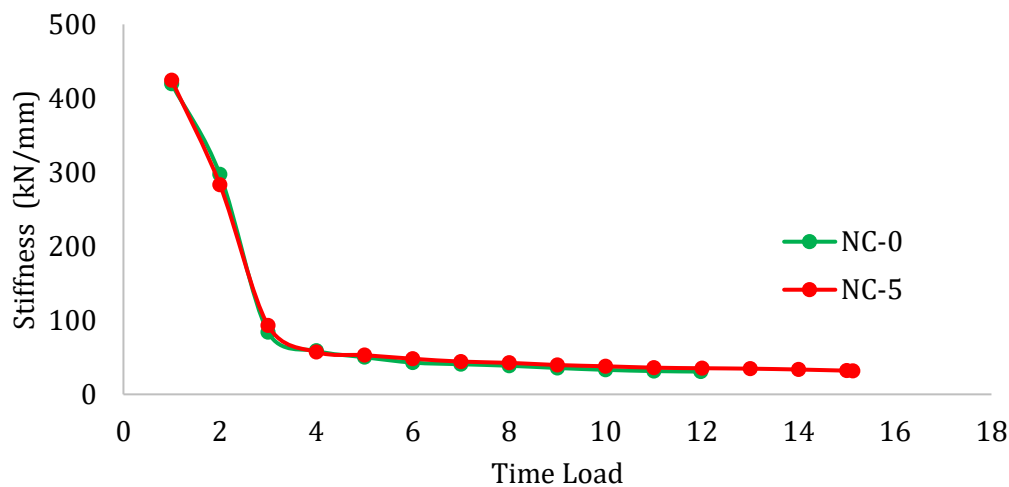


Fig. 25. Curves of stiffness of NC-0 and NC-5 specimens

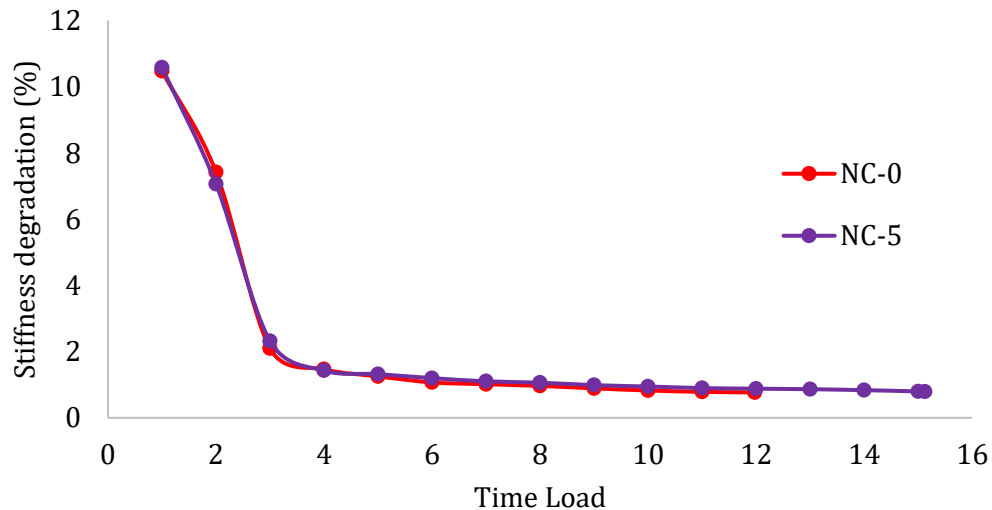


Fig. 26. Curves of stiffness degradation of NC-0 and NC-5 specimens

The behavior of the flat slab, as shown in the stiffness degradation curve, resembles the stiffness curve. Significant degradation occurred at Time Loads 1, 2, and 3, with 10.479%, 7.423%, and 2.097% in NC-0, and 10.584%, 7.063%, and 2.320% in NC-5. Furthermore, there was a gradual stiffness degradation in NC-0 from 1.469% to 0.768%. In NC-5, the stiffness degradation at Time Load 4 was 1.427% and continued to decrease slowly to 0.790%. The NC-5 flat slab exhibited greater stiffness and stiffness degradation than its NC-0 counterpart. This behavior was shown by the lower ultimate load of the NC-0 than that of the NC-5 flat slab.

3.10. Stiffness of NC Flat Slab Models

The stiffness relationship curve with Time Load of NC-0-F, NC-5-F, and NC-8-F flat slab models is shown in Figure 27. In general, the stiffness behavior of the NC-0-F flat slab model resembles the NC-0 specimen, and the NC-5-F model resembles the NC-5 specimen. The NC-0-F flat slab model has a drastically reduced stiffness of 396,660 kN/mm, 271,830 kN/mm, and 77,432 kN/mm at Time Loads 1, 2, and 3, respectively. The stiffness at Time Load 4 to 12 decreased more slowly from 46,059 kN/mm to 33,383 kN/mm. In the NC-5-F and NC-8-F flat slab models, the stiffness change behavior was relatively the same: at Time Loads 1 to 4, there was a drastic decrease, then slowed until the ultimate condition at Time Load 16. In the NC-5-F flat slab, Time Load 1, 2, 3, 4, and 16, the stiffnesses were 396.888 kN/mm, 277.345 kN/mm, 87.537 kN/mm, 51.074 kN/mm, and 31.216 kN/mm, respectively. The NC-8-F flat slab model performed stiffnesses of 397.164 kN/mm, 280.261 kN/mm, 94.286 kN/mm, 52.469 kN/mm, and 32.120 kN/mm at the same Time Loads.

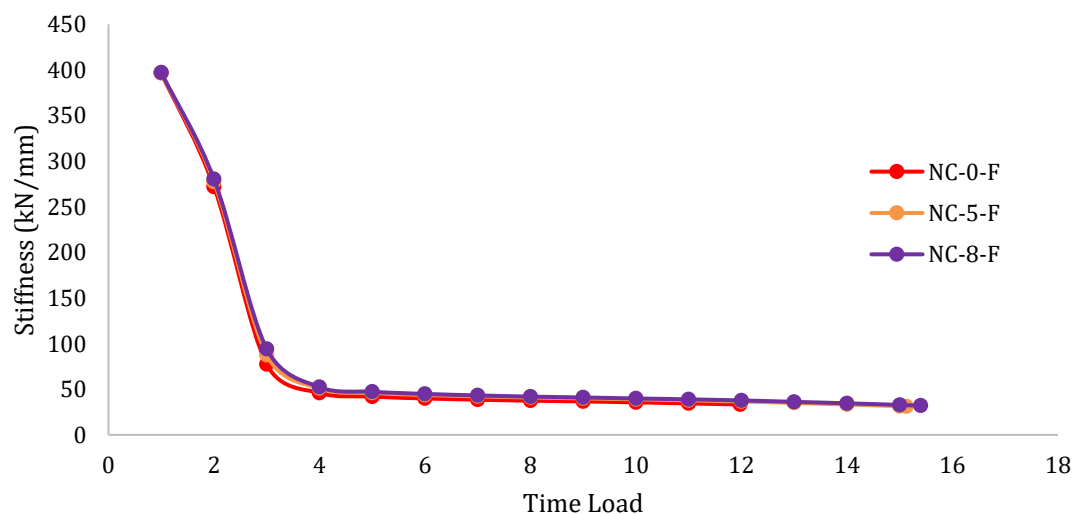


Fig. 27. Curves of stiffness of NC-0-F, NC-5-F, and NC-8-F models

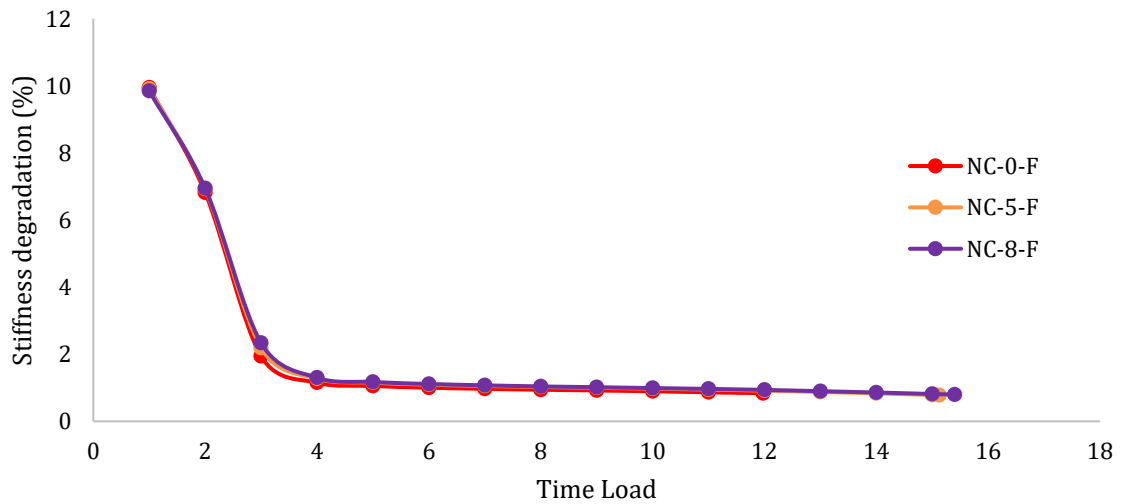


Fig. 28. Curves of stiffness degradation of NC-0-F, NC-5-F, and NC-8-F models

Overall, the stiffness of the NC-8-F flat slab was higher than that of its NC-5-F and NC-0-F counterparts. This was due to higher loads on the NC-8-F flat slab model compared to the NC-5-F and NC-0-F flat slab models, as shown in the transverse reinforcement areas. Figure 28 depicts the percentage of stiffness degradation that occurred drastically at Time Load 1 to 4 in the NC-0-F, NC-5-F, and NC-8-F models. Then, stiffness degradation slowed down at Time Load 5 until the ultimate condition at the end of loading. Stiffness degradations at Time Load 1, 2, 3, 4, and 12 in NC-0-F were 9.949%, 6.818%, 1.942%, 1.155%, and 0.837%. Stiffness degradations of NC-5-F at Time Load 1, 2, 3, 4, and 16 were 9.915%, 6.929%, 2.187%, 1.276%, and 0.780%. At the same Time Load, the degradations of the NC-8-F flat slab model were 9.846%, 6.948%, 2.337%, 1.301%, and 0.796%. The comparisons of initial stiffness, crack load, and ultimate displacement between the specimens and NC models are shown in Table 8. The difference between the NC specimens and models is less than 10%. Thus, the NC models are categorized as accurate [47,48].

Table 8. Stiffness of NC specimens and models

Specimen				Model				
Type of flat slab	Crack load, F_{cr}	Crack displ., δ_{cr}	Initial stiffness, K_i	Type of flat slab	Crack load, F_{cr}	Crack displ., δ_{cr}	Initial stiffness, K_i	Difference
	(kN)	(mm)	(kN/m)		(kN)	(mm)	(kN/mm)	(%)
NC-0	80	0.269	0.297	NC-0-F	80	0.294	0.272	8.57
NC-5	80	0.282	0.283	NC-5-F	80	0.288	0.277	2.10
-	-	-		NC-8-F	80	0.285	0.280	

3.11. Stiffness of GC Flat Slab Models

The stiffness behavior of a flat slab made of GC material was almost similar to that of its counterpart using NC material. Stiffness decreased rapidly from Time Load 1 to 4, then slowly reduced from Time Load 5 until the end of loading at the ultimate condition. The difference was that the Time Load on the GC flat slab model was greater than that on the NC model. Figure 29 shows the GC-0-F flat slab model had stiffnesses at Time Load 1, 2, 3, 4, 5, and 17 (ultimate) of 554.226 kN/mm, 429.445 kN/mm, 226.510 kN/mm, 62.816 kN/mm, 55.649 kN/mm, and 38.984 kN/mm. At Time Loads 1, 2, 3, 4, 5, and 21 (ultimate), the GC-5-F flat slab model performed stiffnesses of 554.015 kN/mm, 429.074 kN/mm, 231.593 kN/mm, 67.425 kN/mm, 59.153 kN/mm, and 35.528 kN/mm, respectively. The GC-8-F flat slab model exhibited stiffnesses at Time Loads 1, 2, 3, 4, 5, and 18 (ultimate), namely 554.288 kN/mm, 432.545 kN/mm, 254.645 kN/mm, 72.403 kN/mm, 61.975 kN/mm, and 42.934 kN/mm. The GC-8-F flat slab model was more brittle than its GC-5-F counterpart because the transverse reinforcement diameter was larger, resulting in greater stiffness at the same Time Load, while its ultimate Time Load was smaller. Figure 30 shows that

the stiffness degradation at Time Load 1 differs for the flat slab models GC-0-F, GC-5-F, and GC-8-F, namely 11.514%, 11.883%, and 10.564%. Then the stiffness at Time Load 2 decreases to 8.922%, 9.203%, and 8.244% for each flat slab. At the ultimate condition, each GC flat slab model experiences stiffness degradation of 0.810%, 0.762%, and 0.818%.

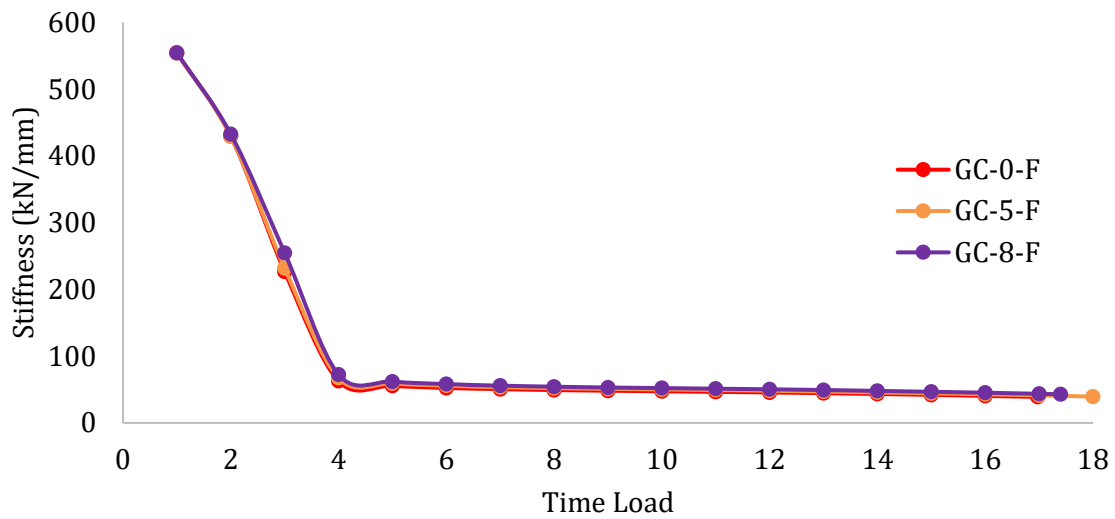


Fig. 29. Curves of stiffness of GC-0-F, GC-5-F, and GC-8-F models

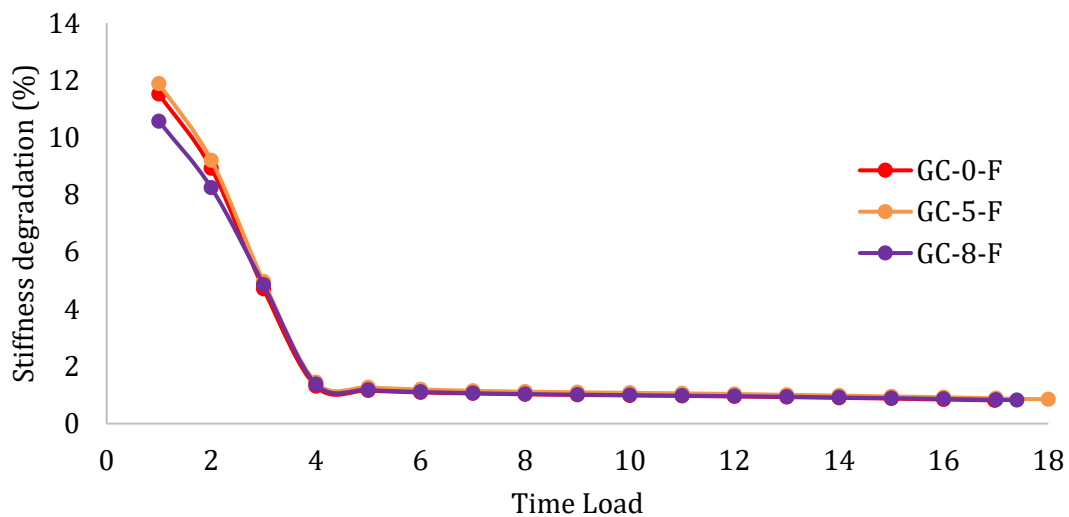


Fig. 30. Curves of stiffness degradation of GC-0-F, GC-5-F, and GC-8-F models

3.12. Energy Absorption of NC and GC Flat Slab Models

Energy absorption is distributed throughout the structure from yielding to ultimate resistance under load. Due to monotonic loading, the amount of energy absorption is calculated from the area of the load-displacement relationship curve from the yielding state to the ultimate state at the end of the loading. Figure 31 depicts the shaded area under the load-displacement curve of the NC-0 flat slab specimen from yield to ultimate state, which is the energy absorption [75]. Similarly, the energy absorption of the NC-5 specimen and the NC-0-F, NC-5-F, and NC-8-F models is presented in Table 9. The unreinforced NC-0-F flat slab model resisted a lower ultimate load than its NC-5-F and NC-8-F counterparts. The NC-5-F model achieved higher ultimate load and displacement than NC-8-F, thereby absorbing the greatest energy among all NC models. The NC-8-F flat slab model was brittle and, therefore, reached a lower ultimate load and displacement than the NC-5-F flat slab model. The difference in energy absorption between the NC-0 specimen and the NC-0-F model is because the NC-0 specimen experiences a greater deflection in the ultimate condition compared to the NC-0 model, with a difference of 1.232 mm or 7.91% as presented in Table 2. It is influenced by the lack of post-cracking tension stiffening or simplified bond behavior in the numerical modeling. The reinforcing steel bars are assumed to be fully bonded to the concrete in all flat slab models.

Thus, the NC-0-F model behaves less ductile than the NC-0 specimen. Table 10 shows that the GC-5-F flat slab model achieved the highest energy absorption at 4727 kN · mm. In comparison, the brittleness of the GC-8-F counterpart reduced its deformation capacity and resulted in a lower energy absorption of 2461.5 kN · mm. Each GC flat slab model absorbed more energy than its NC counterpart with identical transverse reinforcement details. It occurred because GC has a higher compressive strength than NC, allowing the flat slab to resist greater loads under greater displacements.

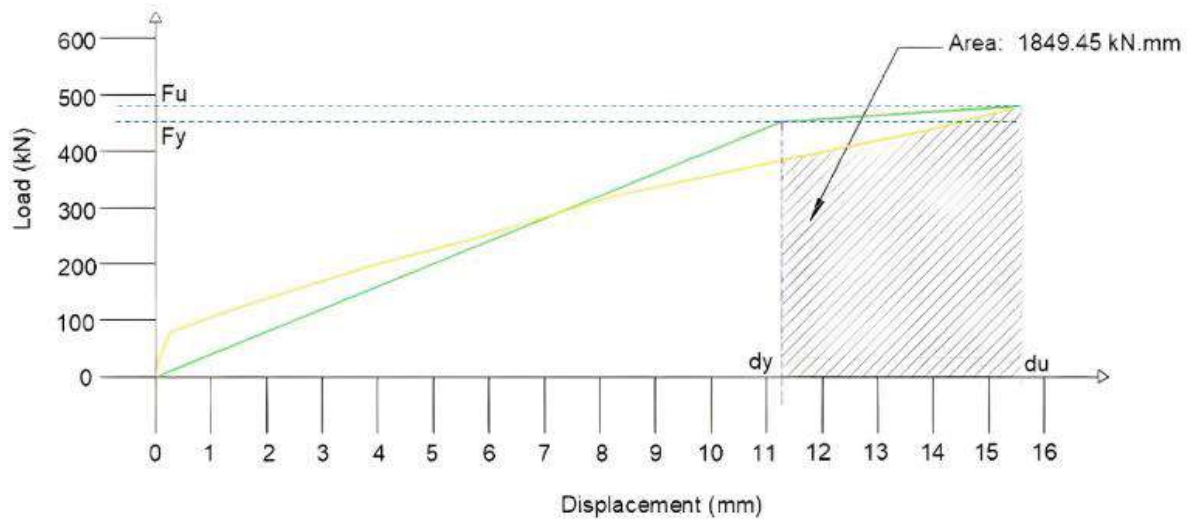


Fig. 31. The shaded area of absorption energy of the NC-0 specimen

Table 9. Energy absorption of the NC flat slab specimens and models

Type of flat slab	Energy absorption of specimen (kN · mm)	Type of flat slab	Energy absorption of model (kN · mm)	Difference (%)
NC-0	1849.45	NC-0-F	1265.66	31.57
NC-5	3528.37	NC-5-F	3043.95	13.73
NC-8	-	NC-8-F	2437.81	-

Table 10. Energy absorption of the GC flat slab model

Type of flat slab	Energy absorption (model) (kN · mm)	Difference with NC model (%)
GC-0-F	2858.05	125.815
GC-5-F	4727.00	55.292
GC-8-F	2461.50	-

3.13. Punching Shear

The effect of transverse reinforcement is analyzed by examining the nominal shear of each flat slab compared to the resisted shear. Different transverse reinforcement diameters result in various nominal shear strengths and determine the ability to resist punching shear loads. Equations (21)-(34) are used to perform the analysis. The load combination involved in this study is from Table 5.3.1 [17] and expressed by equation (14).

$$W_u = 1.4 D \quad (21)$$

The critical cross-sectional area for two-way shear on a plate (b_o) is based on Fig. R22.6.4.2a [17] in equation (15). The b_o for all NC and GC flat slabs is 1848 mm.

$$b_o = [(b_{column} + d_{slab}) + (h_{column} + d_{slab})] \times 2 \quad (22)$$

The ultimate shear resisted by the flat slab is given by equation (16).

$$V_u = W_u [L W - (b_{column} + d_{slab})] \quad (23)$$

The ultimate factored two-way shear stress around the perimeter of a critical section in Clause 8.4.4.2.1 and the stress of concrete nominal two-way shear strength from Table 22.6.6.1 with shear reinforcement [17] is expressed in equations (24) and (25). The nominal shear stress on steel is given by equation (19).

$$v_u = \frac{V_u}{b_o d_{slab}} \quad (24)$$

$$v_c = 0.17 \lambda_s \lambda \sqrt{f'_c} \quad (25)$$

$$v_s = \frac{v_u}{\phi} - v_c \quad (26)$$

The size effect factor (λ_s) in Clause 22.5.5.1.3 [17] is written in equation (27).

$$\lambda_s = \sqrt{\frac{2}{1 + 0.004 d_{slab}}} \leq 1 \quad (27)$$

The value of λ for GC is according to Table 19.2.4.1(a) [17] since the density (w_c) of the GC is 2102.9 kg/m³ [46] and is shown in equation (28). The value of λ for NC is 1 [17]. The required and installed transverse reinforcement area is given by equations (29) and (30).

$$\lambda = 0.0075 w_c \leq 1.0 \quad (28)$$

$$A_v = \frac{v_s b_o s}{f_{yt}} \quad (29)$$

$$A_v = n A_{sv} \quad (30)$$

The nominal shear provided by concrete is expressed in equation (31), based on Clauses 11.11.2.1 [76] and Table 22.6.6.1 [17]. The nominal shear of steel in equation (32) is based on Clauses 11.4.7.2 [76] and 22.6.7.2 [17]. Clause 11.11.7.2 [76] provides equation (33) for nominal shear. Equation (34) shows the strength reduction factor (ϕ), which is 0.75 for shear and adopted from Table 5.3.1 [17].

$$V_c = v_c b_o d = 0.17 \lambda_s \lambda \sqrt{f'_c} b_o d \quad (31)$$

$$V_s = v_s b_o d = \frac{A_v f_{yt} d}{s} \quad (32)$$

$$V_n = V_c + V_s \quad (33)$$

$$\phi V_n \geq V_u \quad (34)$$

Where; b_o : perimeter of critical section for two-way shear in slabs (mm), b_{column} : width of column cross section (mm); h_{column} : height of column cross section (mm); h_{slab} : height of flat slab = 180 mm (Fig. 2); d_{slab} : effective depth of slab (mm); D : dead load (N); W_u : ultimate load (N) = 1.4 D [17, Table 5.3.1]; V_u : ultimate shear (N); V_c : nominal shear provided by concrete (N); V_s : nominal shear provided by steel (N); L : length of slab (mm) = 2500 mm (Fig. 1); W : width of slab (mm) = 2500 mm (Fig. 1); v_u : factored shear stress (MPa); v_c : nominal shear stress on concrete (MPa); v_s : nominal shear stress on steel (MPa); λ_s : size effect factor; λ : modification factor of the reduced mechanical properties of lightweight concrete to normal weight concrete; f'_c : compressive strength of concrete (MPa); f_{yt} : yield strength of transverse reinforcement (MPa); w_{NC} : density of NC =

23.50 kN/m³ [77]; w_{GC} : density of GC = 20.62 kN/m³ [46]; q_D : weight of floor finish = 1.5 kN/m² [78, Table 4.3-1]; A_v : total area of transverse reinforcement (mm²); A_{sv} : area of one transverse reinforcement bar (mm²); n : the number of vertical stirrup bars in a cross-section; s : spacing between transverse reinforcement (mm); ϕ_s : diameter of transverse reinforcement (mm)

As an example of calculation using equations (21)-(34), the parameters of the GC-5-F model are described in Appendix. Table 11 presents the shear stresses in flat slabs. The ultimate load on NC and GC flat slabs depended on the concrete density. Thus, the ultimate loads, ultimate shears, ultimate shear stresses, and nominal shear stresses on NC flat slabs were greater than the GC ones. Table 12 shows the flat slab transverse reinforcement. The required transverse reinforcement ($A_{v-required}$) of NC-8 and GC-8 was greater than that of NC-0, NC-5, GC-0, and GC-5 owing to the lesser transverse steel bar yield strength in Table 1. The NC-0 and NC-8 required 68.88 mm² of transverse reinforcement, while the NC-8 required 68.10 mm². The GC flat slabs also show this trend. The GC-0 and GC-5 required 15.34 mm² of transverse reinforcement, while GC-8 required 15.17 mm². The installed transverse reinforcement was 5 mm in diameter for NC-5 and GC-5 flat slabs, while NC-8 and GC-8 flat slabs were installed with 8 mm-diameter transverse reinforcement. The installed transverse reinforcement area is as described in Table 13. These installed steel bars with diameters of 5 mm and 8 mm provided shear stresses of 0.41 MPa and 1.07 MPa, respectively, according to equation (19). Installing transverse reinforcement with an 8 mm diameter, compared to 5 mm, increased shear stress by 2.59 times. The values of λ_s and λ in all NC and GC specimens and models were determined using equations (20) and (21). Thus, the nominal shear stresses provided by the concrete (v_c) were the same for all NC flat slabs, namely 0.93 MPa. Since the reduction factor (ϕ) of 0.75 [17], the obtained ϕv_c is 0.70. In the GC flat slab models, the value of v_c and ϕv_c are 1.10 and 0.82, respectively. The greater area of transverse reinforcement and concrete compressive strength increased the nominal shear stress on the flat slab.

The reduced nominal shear (ϕV_n) of each NC and GC flat slab is compared with the ultimate shear (V_u) to determine its capability in Table 10. Flat slabs without transverse reinforcement, NC-0, NC-0-F, and GC-0-F are unable to resist punching shear because the value of $\phi V_n/V_u$ is less than 1.0, which is 0.72. The NC-5, NC-5-F, and GC-5-F flat slabs can resist punching shear owing to the shear capacity provided by the $\phi 5-72$ transverse reinforcement. The GC-5-F flat slab shows a higher punching shear capacity with a value of 1.28, compared to its NC-5 and NC-5-F counterparts, which have a value of 1.04. Transverse reinforcement $\phi 8-72$ increases the punching shear capacity of NC-8 and NC-8-F with a value of 1.55. GC material improves the GC-8-F flat slab's capability, achieving a value of 1.84. Thus, the $\phi 5-72$ shear reinforcement provides adequate punching shear capacity for NC and GC flat slabs. The transverse reinforcement with a diameter of 8 mm of NC and GC flat slabs provides the highest nominal shear capacity in resisting punching shear. Table 14 shows the increased nominal shear by using GC materials compared with the NC counterparts.

Table 11. Flat slab shear stresses due to ultimate loads

Type of flat slab	Type	W_u	V_u	v_u	λ_s	λ	ϕv_c	v_s
		(kN)	(kN)	(MPa)			(MPa)	(MPa)
NC-0	Specimen	50.14	290.20	0.97	1.00	1.00	0.70	0.36
NC-5	Specimen	50.14	290.20	0.97	1.00	1.00	0.70	0.36
NC-8	Specimen	50.14	290.20	0.97	1.00	1.00	0.70	0.36
NC-0-F	Model	50.14	290.20	0.97	1.00	1.00	0.70	0.36
NC-5-F	Model	50.14	290.20	0.97	1.00	1.00	0.70	0.36
NC-8-F	Model	50.14	290.20	0.97	1.00	1.00	0.70	0.36
GC-0-F	Model	45.61	263.96	0.88	1.00	1.00	0.82	0.08
GC-5-F	Model	45.61	263.96	0.88	1.00	1.00	0.82	0.08
GC-8-F	Model	45.61	263.96	0.88	1.00	1.00	0.82	0.08

Table 12. Flat slab transverse reinforcement

Type of flat slab	Type	$A_{v-required}$	$\phi_{s-required}$	$\phi_{s-installed}$	$A_{v-installed}$	$v_{s-installed}$	Transverse steel bar ratio
		(mm ²)	(mm)	(mm)	(mm ²)	(MPa)	(%)
NC-0	Specimen	68.88	4.68	0	0	0	0
NC-5	Specimen	68.88	4.68	5	78.54	0.41	0.17
NC-8	Specimen	68.10	4.66	8	201.06	1.07	0.44
NC-0-F	Model	68.88	4.68	0	0	0	0
NC-5-F	Model	68.88	4.68	5	78.54	0.41	0.17
NC-8-F	Model	68.10	4.66	8	201.06	1.07	0.44
GC-0-F	Model	15.34	2.21	0	0	0	0
GC-5-F	Model	15.34	2.21	5	78.54	0.41	0.17
GC-8-F	Model	15.17	2.20	8	201.06	1.07	0.44

Table 13. Flat slab nominal shears

Type of flat slab	Type	V_c	V_s	V_n	ϕV_n	$\phi V_n/V_u$
		(kN)	(kN)	(kN)	(kN)	
NC-0	Specimen	278.76	0.00	278.76	209.07	0.72
NC-5	Specimen	278.76	123.35	402.10	301.58	1.04
NC-8	Specimen	278.76	319.39	598.14	448.61	1.55
NC-0-F	Model	278.76	0.00	278.76	209.07	0.72
NC-5-F	Model	278.76	123.35	402.10	301.58	1.04
NC-8-F	Model	278.76	319.39	598.14	448.61	1.55
GC-0-F	Model	327.86	0.00	327.86	245.90	0.93
GC-5-F	Model	327.86	123.35	451.21	338.41	1.28
GC-8-F	Model	327.86	319.39	647.25	485.44	1.84

Table 14. Increase in nominal shears on GC flat slabs

Type of flat slab	Type	Increased V_n (%)
GC-0-F	Model	17.62
GC-5-F	Model	12.21
GC-8-F	Model	8.21

3.14. Mesh Convergence Study

A mesh convergence study was conducted to examine whether the element size affects the flat slab modeling analysis results. The meshing process is performed based on the location of the steel reinforcement, with the side dimensions of the concrete elements ranging from 13 mm to 50 mm. Then, the NC-0-F model element density was increased to ensure that the dimensions of the meshed concrete and steel elements meet accuracy requirements, as shown in Figures 32 and 33. The model identity is NC-0-F-D. The distance between the nodes of NC-0-F-D became closer when compared to NC-0-F in Figures 6 and 7. The side dimensions of the concrete ranged from 13 to 32 mm. The analysis results produced a load-to-deflection curve in Figure 34. It shows that the NC-0-F-D model curve almost coincides with the NC-0-F curve. This indicates that the element size in NC-0-F was sufficient to produce an accurate finite element analysis.

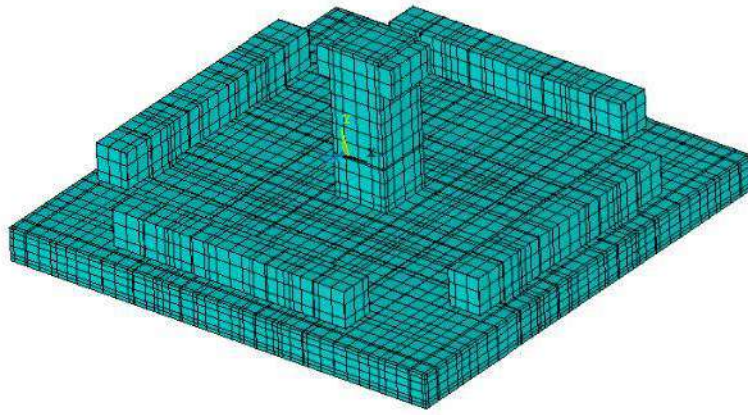


Fig. 32. The top side of a flat slab NC-0-F-D model

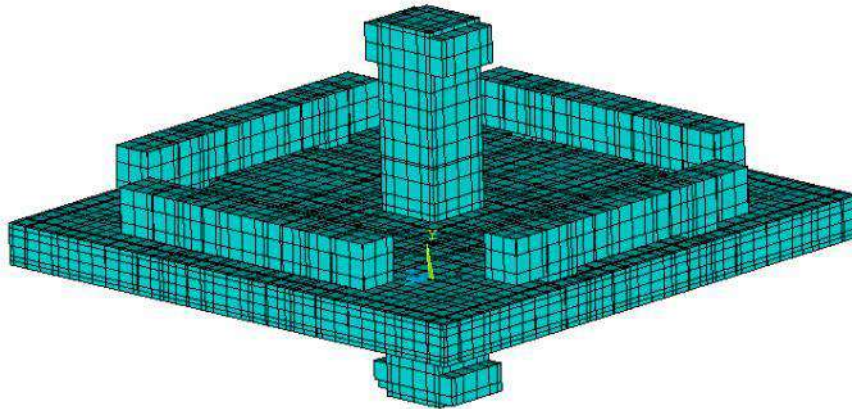


Fig. 33. Three-dimensional side view of the flat slab NC-0-F-D model

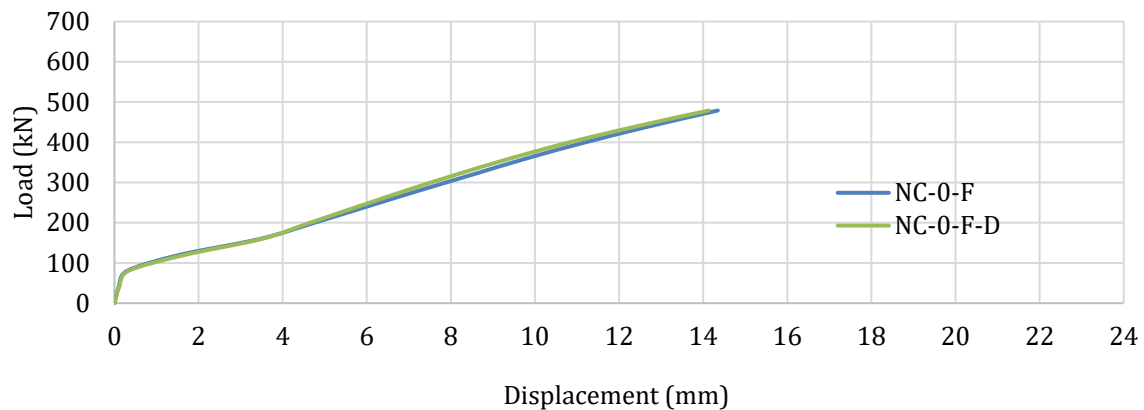


Fig. 34. Comparison of load-deflection curves of NC-0-F and NC-0-F-D

3.15. Limitations and Future Works

In this study, the elaboration and analysis of the NC and GC flat slab numerical models are still limited to the following points:

- The GC behavior was not experimentally validated.
- Only two bar diameters (5 mm and 8 mm) and one spacing (72 mm) were studied for finite element modeling.
- The unbalanced moments at slab-column connections and long-term durability effects were not considered.
- No crack patterns were produced from the finite element modeling.
- Numerical modeling is still limited in simulating post-yield energy absorption, which is influenced by stiffness and ultimate capacity.

- The broader significance of the novelty of this study remains underdeveloped.

The suggestions for future work to improve the quality of the analysis results are as follows:

- Experimental works to validate the numerical findings, especially for NC-8 and all GC flat slab models.
- Further study of the effect of transverse reinforcement ratio on flat slab ductility.

4. Conclusions

The NC-0-F and NC-5-F flat slab numerical models, developed in ANSYS, were verified against specimens and met the accuracy requirements. The performance of the NC flat slab specimens and models, as well as the GC flat slab models, was analyzed, including ultimate loads, displacements, ductility ratios, stress contours, displacement contours, stiffness, energy absorption, and shear capacity. Some conclusions are elaborated as follows.

- The NC-0 and NC-5 flat slab specimens were modelled using the finite element method and verified. The NC-0-F model shows a displacement difference of 7.909% compared with the NC-0 specimen. The NC-5-F model performed a displacement discrepancy of 1.563% compared with the NC-5 specimen. These percentages meet the 10% accuracy limit.
- The percentage of transverse reinforcement significantly affected the shear resistance capacity of a flat slab. In all NC flat slab specimens and models, the highest transverse reinforcement percentage of 0.44% resulted in the flat slab's ability to withstand the best punching shear. The NC-8-F flat slab model achieved the best shear of 616 kN. However, the influence of excessive transverse reinforcement at 0.44% in the GC-8-F flat slab model led to brittle behavior, reduced ductility, and limited deformation capacity. Thus, the GC-8-F flat slab model achieved less ultimate shear and displacement (696 kN; 16.211 mm) than the GC-5-F counterpart (805 kN; 22.658 mm). It indicated that the higher compressive strength of GC than that of NC led to less ductility. Thus, the ultimate displacement and shear of the GC-8-F flat slab model were not as high as those of the GC-5-F counterpart. Each GC flat slab model achieved a higher ultimate shear than the NC models with the same transverse reinforcement ratio. It was owing to the greater compressive strength and elastic modulus of the GC than those of the NC, namely 41.5 MPa and 30266 MPa for the GC, and 30 MPa and 22500 MPa for the NC, respectively.
- The stress contour of the NC-0-F flat slab model shows increasingly higher compressive stresses of 3-12 MPa in the slab around the columns. The portion of the slab under compressive stresses of 12-21 MPa is wider in the NC-5-F and NC-8-F flat slab models than in the NC-0-F model, due to the higher ultimate shear strength. The GC flat slab models exhibit higher and wider compressive stresses of 1.08-17.23 MPa compared to the NC flat slab models (3-12 MPa). The largest compressive stress area occurs in the GC-5-F flat slab model, due to the highest ultimate shear among the models.
- The displacement contour of the NC-0-F flat slab model shows that larger vertical displacements occur in the spreader beams (11.64-14.61 mm). The larger vertical displacements occur at the edges and corners of the NC-5-F and NC-8-F flat slab models (17.58-26.50 mm). In the GC-0-F and GC-5-F flat slab models, larger vertical displacements occur at the flat slab edges and corners, ranging from 13.86 to 31.50 mm. In contrast, the vertical displacement in the GC-8-F (13.86-24.44 mm) is not as large as in the GC-5-F flat slab due to its more brittle behavior, which originates from the largest area of transverse reinforcement.
- The compressive strength of NC and GC, as well as the area of transverse reinforcement, affects ductility. Each GC flat slab model has a higher ductility ratio than its NC counterpart, owing to GC's compressive strength, which enables higher ultimate loads and displacements. The larger areas of transverse reinforcement in NC-8-F and GC-8-F result in lower ductility ratios (1.283 and 1.309) than in NC-5-F and GC-5-F flat slab models (1.380 and 1.390). The NC-0-F and GC-0-F flat slab models without transverse reinforcement are unable to withstand significant ultimate shear and displacement, resulting in low ductility ratios (1.248 and 1.359).

- The stiffness of the NC and GC specimens and models decreased drastically from Time Load 1 to 4 due to widespread element damage at the beginning of monotonic loading. From Time Load 5 to the final load, stiffness degradation occurred slowly, owing to a slower reduction in shear capacity after yield. The stiffnesses of NC-0-F, NC-5-F, and NC-8-F at Time load 1 were 396,660 kN/mm, 396.888 kN/mm, and 397.164 kN/mm, respectively. Then on Time Load 4, they were 46,059 kN/mm, 51.074 kN/mm, and 52.469 kN/mm. While the stiffnesses of GC-0-F, GC-5-F, and GC-8-F at Time load 1 were 554.226 kN/mm, 554.015 kN/mm, and 554.288 kN/mm. At Time Load 4, the stiffnesses were 62.816 kN/mm, 67.425 kN/mm, and 72.403 kN/mm, respectively.
- The energy absorption of GC flat slab models is higher than that of NC counterparts. The GC-5-F flat slab model absorbs the highest energy of all models at 4727 kN · mm. The significant difference in energy absorption indicates that the GC material can improve the performance of flat slabs compared to the NC ones.
- The GC material increases the nominal punching shear, as shown by all GC flat slab models, compared with the NC counterparts. The increment of nominal punching shear on GC-0-F, GC-5-F, and GC-8-F are 17.62%, 12.21%, and 8.21%, respectively. The ultimate shear can be overcome by the presence of $\phi 5 - 72$ as the minimum adequate transverse reinforcement in the GC and NC flat slabs. All flat slabs without transverse reinforcement are unable to withstand the punching shear since the $\phi V_n/V_u$ values are less than 1.0.
- The use of transverse reinforcement in flat slabs needs to consider the influence of the concrete material properties to produce structural members that provide sufficient punching shear and ductility capabilities. These results of this study could inform design practice or future experimental validation.

Acknowledgement

The authors would like to thank Sriwijaya University for providing facilities and support for this research activity.

Appendix

The data used in this study for numerical modeling are summarized in Table A.

Table A. Numerical model parameters

Parameter	Data	Location
Type of elements	Concrete: three-dimensional SOLID65 discrete element with 8 nodes (8-node brick elements).	2.2.1 Concrete Element Paragraph #1
	Steel bar: 2-nodes LINK180 discrete bar elements.	2.2.2 Steel Bar Element Paragraph #1
Concrete properties	NC:	2.1. Details of Specimens and Models Paragraph #1-2
	$f'_c = 30$ MPa	
	$E_c = 22500$ MPa	
	GC:	
Steel properties including tensile strength	$f'_c = 41.5$ MPa	Table 1. Steel bars properties
	$E_c = 30266$ MPa	
	f_{yl} and f_{yt}	
Fracture energy	NC: $G_f = 64.73$ N.m/mm ²	2.3. Material Properties Paragraph #2
	GC: $G_f = 67.70$ N.m/mm ²	

Parameter	Data	Location
2.3. Material Properties		
Poisson's ratio	NC: $\nu = 0.20$	Paragraph #1
	GC: $\nu = 0.22$	The Poisson's ratios of NC and GC are assumed to be 0.2 [53] and 0.22 [54], respectively.
Bond assumptions	The reinforcing steel is assumed to be fully bonded to the concrete.	2.2.2 Steel Bar Element Paragraph #1
Mesh size	NC-0-F, NC-5-F, NC-8-F, GC-0-F, GC-5-F, GC-8-F models: 13 mm to 50 mm. NC-0-F-D model: 13 to 32 mm.	3.14. Mesh Convergence Study Paragraph #1

Example of calculation using equations (14)-(27) for the GC-5-F model:

- b_{column} : 300 mm (2.1. Details of Specimens and Models; paragraph #2)
- h_{column} : 300 mm (2.1. Details of Specimens and Models; paragraph #2)
- d_{slab} : 162 mm (2.1. Details of Specimens and Models; paragraph #2)
- $s = 72$ mm (Fig. 2)
- $L = 2.5$ m; $W = 2.5$ m (Fig. 1)
- Thickness of the flat slab: $h_{fs} = 180$ mm (Fig. 2)
- $n = 4$ (Fig. 4)
- $f_{yt} = 698$ MPa (Table 1)
- Density of GC: $w_{GC} = 2102.9$ kg/m³ [46] = 20.62 kN/m³
- Self-weight of the flat slab: $SW_{fl} = h_{fs}w_{GC}LW = 23.20$ kN
- Weight of floor finish = 1.5 kN/m² (assumed)
- Weight of floor finish on the flat slab: $w_f = \text{Weight of floor finish} \times L \times W = 9.38$ kN
- Dead Load: $D = w_{GC} + w_f = 32.57$ kN

$$W_u = 1.4 D = 45.61 \text{ kN} \quad (21)$$

$$b_o = [(b_{column} + d_{slab}) + (h_{column} + d_{slab})] \times 2 = 1848 \text{ mm} \quad (22)$$

$$V_u = W_u [L W - (b_{column} + d_{slab})] = 263.96 \text{ kN} \quad (23)$$

$$v_u = \frac{V_u}{b_o d_{slab}} = 0.88 \text{ MPa} \quad (24)$$

$$v_c = 0.17 \lambda_s \lambda \sqrt{f'_c} = 1.10 \text{ MPa} \quad (25)$$

$$v_s = \frac{v_u}{\phi} - v_c = 0.08 \text{ MPa} \quad (26)$$

$$\lambda_s = \sqrt{\frac{2}{1+0.004d_{slab}}} \leq 1; \text{ then } \lambda_s = 1 \quad (27)$$

$$\lambda = 0.0075 w_c \leq 1.0; \text{ then } \lambda_{GC} = 1 \quad (28)$$

$$A_v = \frac{v_s b_o s}{f_{yt}} = 15.34 \text{ mm}^2 (= A_{v-required}) \quad (29)$$

$$A_v = n A_{sv} = n(0.25\pi\phi_s^2) = 78.54 \text{ mm}^2 (= A_{v-installed}) \quad (30)$$

$$V_c = v_c b_o d = 0.17 \lambda_s \lambda \sqrt{f'_c} b_o d = 327.86 \text{ kN} \quad (31)$$

$$V_s = v_s b_o d = \frac{A_v f_{yt} d}{s} = 123.35 \text{ kN} \quad (32)$$

$$V_n = V_c + V_s = 451.21 \text{ kN} \quad (33)$$

$$\phi V_n \geq V_u \text{ then } 338.41 \text{ kN} \geq 263.96 \text{ kN} \quad (34)$$

References

- [1] Sundararaj JB, Rajkumar KPR. Effects of multiple precursors on strength, durability, and microstructural properties of ambiently cured geopolymer concrete. *Construction and Building Materials*, 2024; 442; 137538. <https://doi.org/10.1016/j.conbuildmat.2024.137538>
- [2] Deep A, Sarkar P. Enhancing sustainability in concrete construction: Utilizing copper slag for improved properties of geopolymer concrete. *Construction and Building Materials*, 2024; 453; 1390442024. <https://doi.org/10.1016/j.conbuildmat.2024.139044>
- [3] Kanagaraj B, Raj RS, Anand N, Lubloy E. Engineering and sustainability performance of geopolymer concrete blended with sodium silicate effluent. *Developments in the Built Environment*, 2025; 22; 100678. <https://doi.org/10.1016/j.dibe.2025.100678>
- [4] Abdalla JA, Hawileh RA, Bahurudeen A, Jyothsna G, Sofi A, Shanmugam V, Thomas BS. A comprehensive review on the use of natural fibers in cement/geopolymer concrete: A step towards sustainability. *Case Studies in Construction Materials*, 2023; 19; e02244. <https://doi.org/10.1016/j.cscm.2023.e02244>
- [5] Yan G, Hu J, Chen M, Ma Y, Huang H, Zhang Z, Wei J, Shi C, Yu Q. Performance evaluation of reinforced slag-fly ash-ceramic waste powders ternary geopolymer concrete under chloride ingress environment. *Construction and Building Materials*, 2025; 478; 141447. <https://doi.org/10.1016/j.conbuildmat.2025.141447>
- [6] Wang W, Zhang P, Dai X, Zheng Y, Hu S. Workability of self-compacting geopolymer concrete: The effect of nano-SiO₂ and steel-polyvinyl alcohol hybrid fiber. *Construction and Building Materials*, 2025; 460; 139849. <https://doi.org/10.1016/j.conbuildmat.2024.139849>
- [7] Jaji MB, Babafemi AJ, Zijl GPAG. Mechanical performance of extrusion-based two-part 3D-printed geopolymer concrete: A review of advances in laboratory and real-scale construction projects. *Materials Today Sustainability*, 2025; 31; 101131. <https://doi.org/10.1016/j.mtsust.2025.101131>
- [8] Ozkilic YO, Celik AI, Tunc U, Karalar M, Deifalla A, Alomayri T, Althoei F. The use of crushed recycled glass for alkali activated fly ash based geopolymer concrete and prediction of its capacity. *Journal of materials research and technology*, 2023; 24: 8267-8281. <https://doi.org/10.1016/j.jmrt.2023.05.079>
- [9] Kishore K, Sheikh MN, Hadi, MNS. Functionalization of carbon nanotubes for enhanced dispersion and improved properties of geopolymer concrete: A review. *Journal of Building Engineering*, 2025; 111; 113096. <https://doi.org/10.1016/j.jobe.2025.113096>
- [10] Murali G. Recent research in mechanical properties of geopolymer-based ultrahigh-performance concrete: A review. *Defence Technology*, 2024; 32; 67e88. <https://doi.org/10.1016/j.dt.2023.07.003>
- [11] Ahmed F, Biswas RK, Ahsan KA, Islam S, Rahman MR. Estimation of strength properties of geopolymer concrete. *Materials Today: Proceedings*, 2021; 44: 871-877. <https://doi.org/10.1016/j.matpr.2020.10.790>
- [12] Akduman S, Aktepe R, Aldemir A, Ozcelikci E, Alam B, Sahmaran M. Opportunities and challenges in constructing a demountable precast building using C&D waste-based geopolymer concrete: A case study in Türkiye. *Journal of Cleaner Production*, 2024; 434; 139976. <https://doi.org/10.1016/j.jclepro.2023.139976>
- [13] Zadjelovic V, Wright RJ, Walker TR, Avalos V, Marín PE, Christie-Oleza JA, Riquelme C. Assessing the impact of chronic and acute plastic pollution from construction activities and other anthropogenic sources: A case study from the coast of Antofagasta, Chile. *Marine Pollution Bulletin*, 2023; 195; 115510. <https://doi.org/10.1016/j.marpolbul.2023.115510>
- [14] Adufu YD, Sore SO, Nshimiyimana P, Ahouandjinou KD, Messan A, Escadeillas G. Durability behaviors of calcined kaolin clay-based geopolymer concrete containing different calcium compounds and cured in ambient sub-Saharan climate. *Construction and Building Materials*, 2025; 465; 140195. <https://doi.org/10.1016/j.conbuildmat.2025.140195>
- [15] Dehkordi ER, Ramezaniapour AA, Moodi F. Application of pre-fabricated geopolymer permanent formworks (PGPFs): A novel approach to provide durability and mechanical strength of reinforced concrete. *Journal of Building Engineering*, 2022; 45; 103517. <https://doi.org/10.1016/j.jobe.2021.103517>
- [16] Kanagaraj B, Raj SR, Anand N, Lubloy E. Fire performance of concrete: A comparative study between cement concrete and geopolymer concrete & its application - A state of art-review. *Case Studies in Chemical and Environmental Engineering*, 2025; 11; 101212. <https://doi.org/10.1016/j.csee.2025.101212>
- [17] American Concrete Institute. ACI CODE-318-25: Building Code for Structural Concrete - Code Requirements and Commentary, Farmington Hills, American Concrete Institute, 2025.
- [18] The National Standardization Agency. SNI 2847:2019 Structural Concrete Requirements for Buildings and Explanation. Jakarta: The National Standardization Agency, 2019. (in Indonesian)
- [19] Chen Z, Tang J, Wang Y, Yi C, Su T. Characterizing the interfacial bonding behaviour and influential mechanism of metakaolin-based geopolymer-concrete composites under shearing load: Experimental

- p and modelling study. Composite Structures, 2025; 369; 119336.
- <https://doi.org/10.1016/j.compstruct.2025.119336>
- [20] Alameri M, Ali MSM, Sheikh A, Elchalakani M. Investigation of high-performance geopolymer concrete-filled double-skin columns under combined loading conditions. Journal of Constructional Steel Research, 2025; 229; 109523. <https://doi.org/10.1016/j.jcsr.2025.109523>
- [21] Kumar TU, Kumar MV, Lakkaboyana SK, Trilaksana H, Ansari A. Investigation of bond strength and flexural behaviour of geopolymer aggregate concrete beams. Case Studies in Construction Materials, 2025; 22; e04916. <https://doi.org/10.1016/j.cscm.2025.e04916>
- [22] Huang Y, Grünewald S, Schlangen E, Lukovic M. Strengthening of concrete structures with ultra high performance fiber reinforced concrete (UHPFRC): A critical review. Construction and Building Materials, 2022; 336; 127398. <https://doi.org/10.1016/j.conbuildmat.2022.127398>
- [23] Li Z, Guo Y, Rao S, Shang X, Noori M, Altabey WA, Farsangi EN, Zhu H, Liu P. On the investigation of modular high-rise concrete buildings in the Asia Pacific region: Engineering application analysis and future directions. Results in Engineering, 2025; 26; 104830. <https://doi.org/10.1016/j.rineng.2025.104830>
- [24] Guo Y-C, Cai Y-J, Xie Z-H, Xiao S-H, Zhuo K-X, Cai P-D, Lin J-X. Experimental investigation of GFRP bar bonding in geopolymer concrete using hinged beam tests. Engineering Structures, 2025; 322; 119036. <https://doi.org/10.1016/j.engstruct.2024.119036>
- [25] Wan S, Zong Z, Lin Y, Shi H, Yuan Y, Ding B, Cai J. Shear behavior of reinforced one-part geopolymer concrete beams. Journal of Building Engineering, 2025; 111; 113205. <https://doi.org/10.1016/j.jobbe.2025.113205>
- [26] Li Y, Wei J, Han A, Xu Y, Qiao M, He D. Influence of geopolymer foam concrete on the mechanical performance of cold-formed steel built-up columns. Case Studies in Construction Materials, 2025; 22; e04479. <https://doi.org/10.1016/j.cscm.2025.e04479>
- [27] Özbayrak A, Kucukgoncu H, Aslanbay YG, Aslanbay HH. Experimental and Numerical Analysis of Damage and Crack Behavior in Geopolymer and Ordinary Portland Cement Reinforced Concrete Columns. Journal of Building Engineering, 2025; 111; 113623. <https://doi.org/10.1016/j.jobbe.2025.113623>
- [28] Tiwary AK, Singh H, Singh K, Sharma A. Experimental and analytical performance analysis of geopolymer concrete-filled double steel tube (CFDST) columns under axial loading. Structures, 2025; 74; 108572. <https://doi.org/10.1016/j.istruc.2025.108572>
- [29] Coronelli D, Muttoni A, Martinelli L, Vecchio CD, d'Aragona MG, Lignola GP, Prota A, Kagermanov A, Marinkovic M. Flat slab response for seismic and cyclic actions prediction with numerical models. Engineering Structures, 2023; 289; 116307. <https://doi.org/10.1016/j.engstruct.2023.116307>
- [30] Sili AA. Earthquake behavior of flat slab structures, particularities and opportunities for use in the building stock of the Chisinau municipality. IOP Conference Series: Materials Science and Engineering, 2024; 1304; 012010. <https://doi.org/10.1088/1757-899X/1304/1/012010>
- [31] Hawkins NM, Fallsen HB, Hinojosa RC. Influence of Column Rectangularity on the Behavior of Flat Plate Structures. Cracking, Deflection and Ultimate Load of Concrete Slab Systems, SP-30, American Concrete Institute Journal & Proceedings, 1972; 69(1): 68-87, Farmington Hills, MI.
- [32] Akinpelu MA, Gabriel DS, Salman AM, Raheem IA. Numerical study on the effect of different column shapes on punching shear behavior of flat slabs. Results in Engineering, 2023; 19; 101345. <https://doi.org/10.1016/j.rineng.2023.101345>
- [33] El-Kassas AI, Bashandy AA, Eied FAM, Arab MAES. Effect of fibers type on the behavior of fibrous high-strength self-compacted reinforced concrete flat slabs in punching with and without shear reinforcement. Construction and Building Materials, 2022; 360; 129625. <https://doi.org/10.1016/j.conbuildmat.2022.129625>
- [34] Santos JB, Melo GdS, Ruiz MF. Punching performance of flat slabs with openings accounting for the influence of moment transfer and shear reinforcement. Engineering Structures, 2024; 303; 117461. <https://doi.org/10.1016/j.engstruct.2024.117461>
- [35] Zufarihsan R, Tambusay A, Suryanto B, Suprobo P. Predicting the punching failure of lightly reinforced concrete flat slabs without shear reinforcement. Structures, 2025; 74; 108473. <https://doi.org/10.1016/j.istruc.2025.108473>
- [36] Ghalla M, Mansour W, Li W, Wang P, Badawi M, Zareef MAE. Enhancing the punching performance of two-way RC flat slabs using different configurations of embedded aluminum sections: Experimental program and numerical analysis. Construction and Building Materials, 2024; 434; 136737. <https://doi.org/10.1016/j.conbuildmat.2024.136737>
- [37] Edri IE, Yankelevsky DZ. Numerical framework for assessing punching shear behavior in integrity-reinforced concrete flat slab specimens. Structures, 2025; 76; 108823. <https://doi.org/10.1016/j.istruc.2025.108823>

- [38] Nguyen-The D, Thai S, Nguyen-Van H, Ngo-Huu C. Numerical modelling of punching shear behaviour of reinforced concrete flat slab and CFT column connections using steel plates. *Structures*, 2025; 74; 108594. <https://doi.org/10.1016/j.istruc.2025.108594>
- [39] Lim S, Kim M, Lee J-Y, Kim H, Kim D, Jo M, Kim K. Punching shear strength of RC flat plate slab-column connections strengthened with column head steel plates. *Journal of Building Engineering*, 2025; 111; 113132. <https://doi.org/10.1016/j.jobbe.2025.113132>
- [40] Zhou Y, Shou H, Li C, Jiang Y, Tian X. Punching shear behavior of ultra-high-performance fiber-reinforced concrete and normal strength concrete composite flat slabs. *Engineering Structures*, 2025; 322; 119123. <https://doi.org/10.1016/j.engstruct.2024.119123>
- [41] Lima H, Palhares R, Melo G Sd, Oliveira M. Experimental analysis of punching shear in flat slabs with variation in the anchorage of shear reinforcement. *Structural Concrete*, 2020; 22(2): 1165-1182. <https://doi.org/10.1002/suco.202000158>
- [42] Aguiar A, Oliveira D, Reis L, Nzambi A. Punching shear strength of waffle flat slabs with opening adjacent to elongated columns. *Engineering Structures*, 2021; 243; 112641. <https://doi.org/10.1016/j.engstruct.2021.112641>
- [43] Arief A, Nurjannah SA, Megantara Y, Rusli M, Putra FE, Noviyanto A. Experimental study of confined masonry walls with lap splice reinforcement subjected to cyclic loads. *Engineering Structures*, 2024; 301; 117247. <https://doi.org/10.1016/j.engstruct.2023.117247>
- [44] Saloma, Nurjannah SA, Usman AP, Aminuddin KM, Iqbal MM, Rifkah. Shear Capacity of Geopolymer Concrete Deep Beams with Variation of Transverse Reinforcement Ratio. *Civil Engineering and Architecture*, 2024; 12(1): 293-311. <https://doi.org/10.13189/cea.2024.120123>
- [45] Saloma, Nurjannah SA, Hanafiah, Usman AP, Hu S, Usman F. Behavior of Geopolymer Concrete Wall Panels with Square Opening Variations Subjected to Cyclic Loads. *Journal of Applied Engineering Science*, 2023; 21(3), 1130: 884-895. <https://doi.org/10.5937/jaes0-43777>
- [46] Zulfiati R, Saloma, Idris Y. Mechanical properties of fly ash-based geopolymer with natural fiber. *IOP Conference Series: Journal of Physics*, 2019; 1198, 082021. <https://doi.org/10.1088/1742-6596/1198/8/082021>
- [47] Wu H, Zhuang X, Zhang W, Zhao Z. Anisotropic ductile fracture: Experiments, modeling, and numerical simulations. *Journal of Materials Research and Technology*, 2022; 20: 833-856. <https://doi.org/10.1016/j.jmrt.2022.07.128>
- [48] Badshah M, Badshah S, Jan S. Comparison of Computational Fluid Dynamics and Fluid Structure Interaction Models for The Performance Prediction of Tidal Current Turbines. *Journal of Ocean Engineering and Science*, 2020; 5: 164-172. <https://doi.org/10.1016/j.joes.2019.10.001>
- [49] https://ansyshelp.ansys.com/public/account/secured?returnurl=/Views/Secured/main_page.html. (accessed on October 30, 2025)
- [50] Al-Murshidi KR, Al-Ta'ee KN, Al-Janabi MA. Finite element analysis of cellular circle cofferdam for wet soil. *Kufa Journal of Engineering*, 2014; 6(1): 25-44. <https://doi.org/10.30572/2018/KJE/611310>
- [51] Özbayrak A, Kucukgoncu H, Atas O, Aslanbay HH, Aslanbay YG, Altun F. Determination of stress-strain relationship based on alkali activator ratios in geopolymer concretes and development of empirical formulations. *Structures*, 2023; 48: 2048-2061. <https://doi.org/10.1016/j.istruc.2023.01.104>
- [52] The European Union Per Regulation 305/2011. EN 1992-1-1 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. 2004.
- [53] Ayeni IS, Lim NHAS, Samad M. Engineering properties of natural fibre-reinforced one-part geopolymer concrete. *Construction and Building Materials*, 2024; 456, 139161. <https://doi.org/10.1016/j.conbuildmat.2024.139161>
- [54] CEB-FIP. Comit'e Euro-International du B'eton (CEB-FIP Model Code 1990). 1993, London: Thomas Telford Ltd.
- [55] Mohamed O, Khattab R. Effect of Shear Reinforcement on Punching Shear Capacity of Reinforced Concrete Flat Plates. *IOP Conference Series: Materials Science and Engineering*, 2020; 960; 042077. <https://doi.org/10.1088/1757-899X/960/4/042077>
- [56] Xiangyong N, Bin Z, Yizhu L, Yonghui H. Predicted compressive stress-strain model for high-strength stirrup confined concrete. *Structures*, 2023; 52: 933-945. <https://doi.org/10.1016/j.istruc.2023.04.039>
- [57] Alex AG, Gebrehiwet T, Kemal Z, Subramanian RB. Structural Performance of Low-Calcium Fly Ash Geo-polymer Reinforced Concrete Beam. *Iranian Journal of Science and Technology, Transactions of Civil Engineering*, 2022. <https://doi.org/10.1007/s40996-022-00832-x>
- [58] Öz HÖ, Yücel HE, Günes M, Köker TS. Fly-ash-based geopolymer composites incorporating cold-bonded lightweight fly ash aggregates. *Construction and Building Materials*, 2021; 272, 121963. <https://doi.org/10.1016/j.conbuildmat.2020.121963>
- [59] Jiang H, Fang Z, Liu A, Li Y, Feng J. Interface shear behavior between highstrength precast girders and lightweight cast-in-place slabs. *Construction and Building Materials*, 2016; 128: 449-460. <https://doi.org/10.1016/j.conbuildmat.2016.10.088>

- [60] Purwanto, Ekaputri JJ, Nuroji, Indriyantho BR, Han A, Gan BS. Shear-bond behavior of self-compacting geopolymer concrete to conventional concrete. *Construction and Building Materials*, 2022; 321, 126167. <https://doi.org/10.1016/j.conbuildmat.2021.126167>
- [61] Shibayama A, Nishiyama M. Shear strength of reinforced fly-ash-based geopolymer concrete beams with and without shear reinforcement. *Structures*, 2023; 50: 603-614. <https://doi.org/10.1016/j.istruc.2023.02.060>
- [62] Kuo WT, Liu MY, Juang CU. Bonding behavior of repair material using flyash/ground granulated blast furnace slag-based geopolymer. *Materials*, 2019; 12(10), 1697. <https://doi.org/10.3390/ma12101697>
- [63] Zailani WWA, Bouaissi A, Abdullah MMAB, Razak RA, Yoriya S, Salleh MAAM, Rozainy MR, Fansuri H. Bonding Strength Characteristics of FA-Based Geopolymer Paste as a Repair Material When Applied on OPC Substrate. *Applied Sciences*, 2020; 10 (9), 3321. <https://doi.org/10.3390/app10093321>
- [64] Ganeshan M, Venkataraman S. Interface shear strength evaluation of self compacting geopolymer concrete using push-off test. *Journal of King Saud University - Engineering Sciences*, 2020; 34: 98-107. <https://doi.org/10.1016/j.jksues.2020.08.005>
- [65] Topark-Ngarm P, Chindaprasirt P, Sata V, Setting time, strength, and bond of high-calcium fly ash geopolymer concrete. *Journal of Materials in Civil Engineering*; 2015, 04014198. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0001157](https://doi.org/10.1061/(ASCE)MT.1943-5533.0001157)
- [66] Phoo-Ngernkham T, Hanjitsuwan S, Li L-Y, Damrongwiriyanupap N, Chindaprasirt P. Adhesion characterisation of portland cement concrete and alkaliactivated binders. *Advances in Cement Research*, 2019; 31(2): 69-79. <https://doi.org/10.1680/jadcr.17.00122>
- [67] Meesala CR, Verma NK, Kumar S. Critical review on fly-ash based geopolymer concrete. *Structural Concrete*, 2020; 21(3): 1013-1028. <https://doi.org/10.1002/suco.201900326>
- [68] Hassan A, Arif M, Shariq M. A review of properties and behaviour of reinforced geopolymer concrete structural elements-a clean technology option for sustainable development. *Journal of Cleaner Production*, 2020; 245, 118762. <https://doi.org/10.1016/j.jclepro.2019.118762>
- [69] Phoo-Ngernkham T, Hanjitsuwan S, Detphan S, Thumrongvut J, Suksiripattanapong C, Damrongwiriyanupap N, Chindaprasirt P, Hatanaka S. Shear bond strength of FA-pc geopolymer under different sand to binder ratios and sodium hydroxide concentrations. *International Journal of GEOMATE*, 2018; 14(42): 52-57. DOI: 10.21660/2018.42.7152. <https://doi.org/10.21660/2018.42.7152>
- [70] Qian K, Liu J-G, Yu X-H, Weng Y-H. Experimental and numerical investigation of punching shear capacity of corroded reinforced concrete slab-column connections. *Structures*, 2022; 43: 1548-1557. <https://doi.org/10.1016/j.istruc.2022.07.065>
- [71] Nurjannah SA, Saloma, Aminuddin KM, Usman AP, Saggaff A, Muliawan RA, Ngian SP. Performance of Cold-Formed Steel Beam-Column Joint with Variation in Haunched Gusset Plate Length Subjected to Monotonic Loads. *Civil Engineering and Architecture*, 2025; 13(4): 2922-2944. <https://doi.org/10.13189/cea.2025.130409>
- [72] Thomas P, Priestley MJN. *Seismic Design of Reinforced Concrete and Masonry Buildings*. Wiley & Sons, Incorporated, John, Oxford, 1992.
- [73] Federal Emergency Management Agency (FEMA). FEMA 356. *Prestandard and Commentary for The Seismic Rehabilitation of Buildings*. Washington DC: FEMA, 2000.
- [74] Gilbert RI, Sakka ZI. Strength and ductility of reinforced concrete slabs containing welded wire fabric and subjected to support settlement. *Engineering Structures*, 2010, 32: 1509-1521. <https://doi.org/10.1016/j.engstruct.2010.01.025>
- [75] Decanini L. Estimation of Near-Source Ground Motion and Seismic Behaviour of RC Framed Structures Damaged by the 1999 Athens Earthquake. *Journal of Earthquake Engineering*, 2005; 9(5): 609-635. <https://doi.org/10.1080/13632460509350559>
- [76] American Concrete Institute. ACI CODE-318-08: *Building Code for Structural Concrete - Code Requirements and Commentary*, Farmington Hills, American Concrete Institute, 2008.
- [77] Ashraf M. ACI SP-17M(14) *The Reinforced Concrete Design Handbook: A Companion to ACI 318M-14*. 218, American Concrete Institute.
- [78] American Society of Civil Engineers (ASCE). ASCE/SEI 7-22 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2022, Structural Engineering Institute.

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Rail-structure interaction analysis for providing continuous welded rail on simply supported unballasted steel through girder bridge

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Abstract

This study investigates the rail-structure interaction (RSI) phenomena for implementing continuous welded rail (CWR) on an unballasted simply supported steel through girder bridge with a span of 78.8 m designed for Dedicated Freight Corridor (DFC) loading (32.5t axle load). A comprehensive parametric analysis was conducted using MIDAS finite element modeling and theoretical calculations based on UIC774-3(R) guidelines to evaluate axial stresses in rails and relative displacements between rail and deck under braking/traction forces, thermal variations, and vertical train loads. The results demonstrate that braking loads (16.3 kN/m) generate moderate stresses (20.3 and -26.6 N/mm²) at both fixed and free supports, while temperature effects (35°C) induce significantly higher stresses at the free support (-76.3 N/mm²) compared to the fixed support (2.7 N/mm²). Vertical bending effects produce the most critical stress condition, with compressive stresses at the free support (-80.4 N/mm²) exceeding the allowable limit of 72 N/mm². Although horizontal displacements (1.68 mm) remain well below the permissible limit of 5 mm, the combined stress state necessitates the installation of Switch Expansion Joints (SEJ) at support locations to ensure track stability and structural integrity. This research provides practical guidelines for determining appropriate rail configurations on steel through girder bridges and demonstrates a validated methodology for RSI assessment applicable to heavy-haul railway infrastructure design.

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1. Introduction

Railway infrastructure worldwide has undergone rapid expansion to accommodate growing freight and passenger transport demands [1-2]. In India, the development of Dedicated Freight Corridors (DFC) represents a significant advancement in railway capacity, with new alignments designed for heavier axle loads of 32.5 tones and enhanced operational speeds [3-4]. These infrastructure improvements require sophisticated structural design approaches, particularly for railway bridges where tracks interact directly with supporting members.

Rail-Structure Interaction (RSI) describes the complex mechanical coupling between railway tracks and bridge structures [5-6]. Under service conditions, forces transfer from rails through fastening systems to bridge decks and subsequently to foundation elements. This interaction becomes critical under thermal variations, moving train loads, and longitudinal forces generated during braking and traction. Understanding these force transfer mechanisms is essential for ensuring structural safety and maintaining track geometry within acceptable tolerances.

Continuous Welded Rail (CWR) technology has become standard practice in modern railway construction because it eliminates rail joints, thereby reducing maintenance needs, improving ride

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quality, and minimizing noise and dynamic impact forces [7]. Track maintenance costs decrease substantially when joints are eliminated, and passengers experience smoother travel due to the absence of repetitive joint impacts. However, CWR implementation on bridges introduces engineering complexities that require careful analysis. Without expansion joints, the rail behaves as a continuous beam that must accommodate thermal expansion and contraction while remaining attached to the bridge deck through fasteners [8-9]. This constraint generates additional axial forces in the rail that can lead to buckling in compression or fracture in tension if not properly managed. The continuous nature of the rail also modifies the dynamic characteristics of the track-bridge system, creating force distributions that differ fundamentally from jointed rail configurations [10-11]. International railway organizations have developed standards to guide RSI analysis and design. The International Union of Railways (UIC) code UIC774-3(R) provides the primary framework for evaluating RSI effects on bridge structures (UIC, 2001). This code specifies procedures for calculating rail stresses, checking relative displacements between rails and bridge decks, and determining forces transmitted to substructures. Adherence to these guidelines ensures both the safe operation of trains and the long-term structural integrity of railway infrastructure [12-14].

Several research studies have examined RSI phenomena across different bridge types, providing insight into the governing parameters and their effects on structural behavior. Shah and Surti [15] analyzed RSI effects on a prestressed concrete box girder bridge for metro railway applications. Their comparative study showed that rail stresses increased by 60.38% for vertical loads and 28.52% for thermal loads when RSI effects were properly accounted for, compared to simplified analysis that neglected these interactions. They demonstrated that Switch Expansion Joints (SEJ) effectively reduce axial stresses at critical bridge locations, confirming the importance of thorough RSI analysis for determining appropriate track configurations. Asif et al. [6] investigated how span length affects stress distribution in railway bridges using simplified beam models. Their parametric study covered spans ranging from 35 m to 90 m and revealed distinct stress patterns at different support types. At fixed supports, rail stresses varied from 6.05 N/mm² for the shortest span to 10.5 N/mm² for the longest. At movable supports, stresses reached significantly higher values between 14.6 N/mm² and 42 N/mm², identifying these locations as critical for design due to accumulated compressive forces. The same study quantified support reactions at fixed ends, which ranged from 295 kN for 35 m spans to 800 kN for 90 m spans, demonstrating the proportional relationship between span length and substructure loads.

Goicolea et al. [16] extended RSI research to multi-span continuous railway viaducts with direct fixation tracks, examining both prestressed concrete box girders and twin-tee girder configurations. Their findings indicated that UIC displacement limits were originally calibrated for ballasted track systems. For unballasted tracks, they argued that design emphasis should shift toward stress limits rather than displacement criteria, since ballast damage is not a concern in these systems. Kumar and Upadhyaya [17] focused on thermal gradient effects in track-bridge interaction. Temperature differentials through the bridge deck depth create additional bending deformations that influence the force distribution between rails and bridge structures. Their parametric analysis showed that thermal gradients significantly affect support reactions, and these effects must be considered in design calculations. They also found that relative displacement between deck and rail decreases as deck length increases, which has implications for longer bridge structures. Pugasap et al. [18] emphasized the role of substructure stiffness in controlling RSI behavior. Abutment stiffness characteristics directly influence horizontal displacements and rail stress magnitudes, particularly at fixed supports and expansion joint locations. Gupta et al. (2014) [20] examined dynamic aspects of track-bridge systems, identifying resonance phenomena and load amplification effects that become important for high-speed operations and heavy axle loads.

While these studies have advanced understanding of RSI in various bridge configurations, specific gaps remain in the literature. Previous RSI research has concentrated primarily on prestressed concrete bridges and continuous span structures, with limited investigation of single-span steel through girder bridges under heavy-haul loading conditions. Steel through girders exhibit distinct structural characteristics—including high stiffness-to-weight ratios, specific articulation arrangements, and unique dynamic properties—that differentiate their RSI behavior from concrete

structures. The interaction mechanisms between CWR systems and steel through girders under extreme loading remain inadequately characterized in current literature. For Indian DFC applications, no comprehensive study has systematically examined RSI in unballasted steel through girder bridges with the specific combination of 78.8 m span, 32.5t axle loads, and direct fixation track systems. This gap is particularly significant because design decisions for CWR versus SEJ implementation on such bridges currently lack validated analytical and numerical frameworks tailored to this structural typology.

This research establishes a comprehensive analytical framework for RSI analysis of unballasted simply supported steel through girder bridges. The study determines whether CWR implementation is feasible for the investigated bridge configuration or whether SEJ provision is necessary based on detailed stress and displacement analysis. The analysis follows UIC774-3(R) guidelines [20] and examines three primary loading conditions: longitudinal forces from braking and traction, uniform temperature variation of $\Delta T = \pm 35^\circ\text{C}$ in the bridge deck, and vertical train loads corresponding to DFC specifications. The parametric investigations quantify minimum support stiffness requirements, assess superstructure stiffness effects, evaluate end rotation due to vertical bending, determine temperature-induced stresses, calculate rail stresses at fixed and free supports, compute forces transferred to substructure elements, and evaluate horizontal displacements at bearing levels.

This study makes several contributions to the existing body of knowledge on RSI analysis:

- It provides the first comprehensive parametric study that combines analytical methods based on UIC774-3(R) with three-dimensional finite element modeling (MIDAS Civil) specifically for unballasted steel through girder bridges supporting DFC loading. Previous studies have applied these methods separately or to different bridge types, but not to this specific configuration.
- The research quantifies end rotation effects and demonstrates their dominant influence on rail stresses at free supports. The measured end rotation of $\theta_H = 2.88$ mm generates critical compressive stresses that govern design decisions for this bridge typology mechanism that existing literature has not adequately characterized steel through girders.
- The study develops a validation methodology that demonstrates correlation between theoretical and numerical approaches across multiple loading scenarios. This validation includes explicit quantification of uncertainties and acceptable agreement criteria, providing confidence in both analysis methods.
- The research establishes a practical design framework with criteria-based decision logic for selecting between CWR and SEJ configurations. This framework can be applied to similar infrastructure projects where engineers must make evidence-based decisions about track configuration on steel bridge structures.

2. Methodology

2.1. Theoretical Framework for RSI Analysis

Rail-Structure Interaction analysis quantifies force transfer mechanisms between railway tracks and bridge structures. Before developing finite element models, establishing theoretical foundation is essential for understanding these interactions. The methodology employed in this study integrates both analytical calculations based on UIC774-3(R) guidelines and numerical simulations using MIDAS Civil software.

RSI manifests through three primary mechanisms: axial stresses in rails, relative displacements between rails and bridge components, and forces transmitted to substructures. These interactions depend on several parameters including support stiffness, bridge span, track-structure connection characteristics, and loading conditions.

The analysis must account for geometric compatibility between rails and bridge deck, equilibrium of forces at rail-deck interfaces, and material constitutive behavior under varying thermal and mechanical loads. These fundamental principles govern the mathematical formulations used in both analytical and numerical approaches.

Longitudinal forces from braking and traction transmit through the rail-fastener-deck system, creating axial stresses in rails and horizontal forces at supports. Temperature variations cause differential thermal expansion between rails and bridge structures, resulting in additional axial forces when rails are constrained by fasteners.

Vertical train loads induce bridge deflections and end rotations that generate axial rail stresses through geometric incompatibility at supports. In simply supported spans, end rotations become particularly significant because rail continuity conflicts with deck rotation, creating compressive or tensile stresses depending on load position and support configuration.

2.2. Support Stiffness Characterization

2.2.1 Theoretical Definition

Support stiffness (K) represents the composite longitudinal resistance of the bridge substructure system. This parameter combines contributions from abutment walls, pile foundations, bearings, and surrounding soil. Following UIC774-3(R) methodology, support stiffness is defined as:

$$K = H/\delta \quad (1)$$

where H is the horizontal force applied parallel to the bridge longitudinal axis, and δ is the resulting displacement at the support head.

The total displacement δ comprises three components: elastic deformation of abutment walls (δ_p), rotation of the foundation system (δ_ψ), and horizontal translation of the foundation (δ_h). Each component depends on structural geometry, material properties, and soil characteristics.

2.2.2 Finite Element Determination

For this bridge, support stiffness was determined through finite element analysis rather than simplified analytical methods. A horizontal force of 10.00 kN was applied at the bridge deck level in the longitudinal direction. The resulting displacement was extracted from the model to calculate stiffness. The finite element model incorporated full geometric representation of substructure components. Abutment walls were modeled using shell elements with M35 concrete properties ($E = 31,000 \text{ N/mm}^2$). The foundation system consisted of 15 piles at each abutment, each 1.0 m in diameter. Piles were represented as beam elements with a fixity depth of 7.4 m, determined based on soil conditions. The surrounding medium-dense sand was characterized by a modulus of subgrade reaction of 25 kN/m^3 . Pile bases were fixed against all translations and rotations to simulate end-bearing conditions on dense strata.

2.2.3 Calculated Stiffness Value

The finite element analysis yielded a horizontal displacement of 0.028 mm under the applied 10.00 kN force. This gives:

$$K = 10.00 \text{ kN} / 0.028 \text{ mm} = 357.1 \text{ kN/mm (or } 357,100 \text{ kN/m)} \quad (2)$$

When normalized by the bridge span ($L = 78.8 \text{ m}$), this stiffness corresponds to $4.53L$. According to UIC774-3(R) design charts, this value falls between the K2 (2L) and K5 (5L) reference curves. Interpolation between these curves was necessary for all subsequent analytical calculations. The calculated stiffness accounts for pile group effects through explicit modeling of all 15 piles rather than equivalent single-pile approximations [21-26].

2.3 End Rotation Analysis

Vertical train loads cause bridge deck bending, producing end rotations at support locations. For continuously welded rail, these rotations generate axial stresses through geometric compatibility requirements. The rail must accommodate the deck rotation while remaining attached to the deck surface, creating compressive or tensile forces.

For a simply supported beam under uniformly distributed load w , the end rotation is:

$$\theta = wL^3 / (24EI) \quad (3)$$

where L is the span length, E is the elastic modulus, and I is the moment of inertia. For the design vertical load intensity of 119.6 kN/m, the calculated end rotation is, $\theta = (119.6 \times 10^{-3} \times 78,800^3) / (24 \times 200,000 \times 6.3 \times 10^{13}) = 0.001536$ radians.

The end rotation produces a horizontal displacement θH at the rail level, where H is the vertical distance from the neutral axis to the rail. With H = 1.49 m (distance from bottom chord to deck top) plus 0.328 m (deck to rail), the effective height is 1.818 m. These yields:

$$\theta H = 0.001536 \times 1,818 = 2.88 \text{ mm} \quad (4)$$

The horizontal displacement generates axial stress in the rail through:

$$\sigma = (E \times \theta H \times \omega) / H \quad (5)$$

where ω represents the vertical distance from rail centerline to bridge neutral axis (-3.255 m for this configuration), and $\gamma = \omega/H = -2.185$. This parameter significantly influences stress distribution at supports, as demonstrated in the results section.

2.4 Bridge, Foundation Geometry and Material Properties

The bridge under investigation is an unballasted simply supported steel through girder designed for DFC loading with 32.5t axle loads. The span length is 78.8 m. Figure 1 shows the bridge configuration with abutments and pile layout as modeled in MIDAS Civil. The structure is an Open Web Girder (OWG) with the following geometric properties:

- Total depth of OWG: 10.5 m
- Center of gravity of OWG to bottom chord: 4.845 m
- Bottom chord to deck top (H): 1.49 m
- Bottom chord to rail top: 1.818 m
- Omega (ω): -3.255 m
- Gamma ($\gamma = \omega/H$): -2.185

Material properties for structural analysis were assigned as follows. Concrete elements (abutments and piles) used M35 grade with compressive strength of 35 N/mm² and elastic modulus of 31,000 N/mm². Rail steel has an elastic modulus of 210,000 N/mm² with UIC 60 kg sectional properties. Structural steel for bridge girders has an elastic modulus of 200,000 N/mm².

The bridge moment of inertia was calculated from deflection requirements:

$$I = 5WL^4 / (384E\delta) = 6.3 \times 10^{13} \text{ mm}^4 \quad (6)$$

where δ represents vertical deflection under design loading. Abutment structures consist of front walls, return walls, and pile foundations. Dimensions are specified below. Abutment Wall Properties:

- Front wall: 7.8 m (length) $\times$ 1.0 m (width) $\times$ 6.825 m (height)
- Return walls: 7.25 m (length) $\times$ 0.7 m (thickness) $\times$ 8.40 m (height)
- Number of return walls: 2
- Front wall moment of inertia: 0.65 m⁴
- Return wall moment of inertia: 44.46 m⁴

Foundation System (per abutment):

- Number of piles: 15
- Pile diameter: 1.0 m
- Depth of fixity: 7.4 m
- Pile moment of inertia: $4.9 \times 10^{10} \text{ mm}^4$

These geometric and material properties were incorporated into the numerical model to accurately represent structural behavior of the bridge-foundation system.

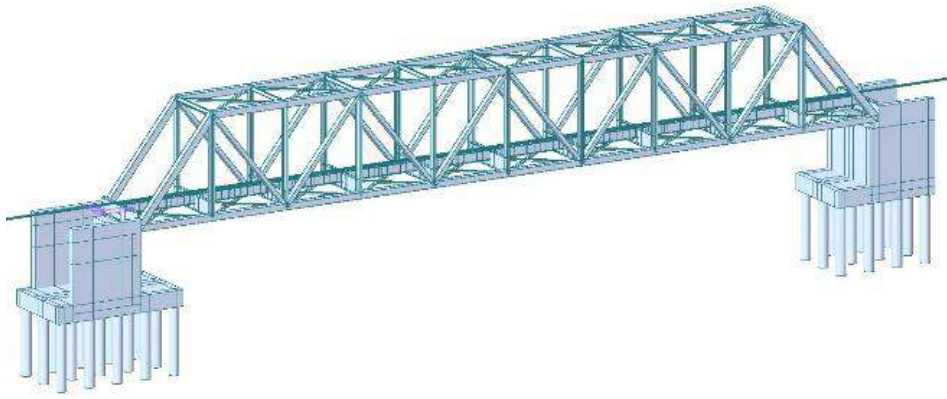


Fig. 1. Bridge, Abutments and Pile layout model in MIDAS

2.5 Rail and Track Properties

The track configuration uses UIC 60 kg rails with standard sectional properties. The cross-sectional area is 0.0007686 m^2 . Moment of inertia about the major axis (I_{xx}) is 0.000003055 m^4 , about the minor axis (I_{yy}) is 0.0000005129 m^4 , and the torsional constant (I_{zz}) is 0.000000216 m^4 . The elastic modulus is $210,000 \text{ N/mm}^2$.

Rail-structure interaction was modeled using nonlinear spring elements representing the force-displacement relationship of the fastening system. Figure 2 shows the bilinear resistance function adopted per UIC774-3(R) guidelines.

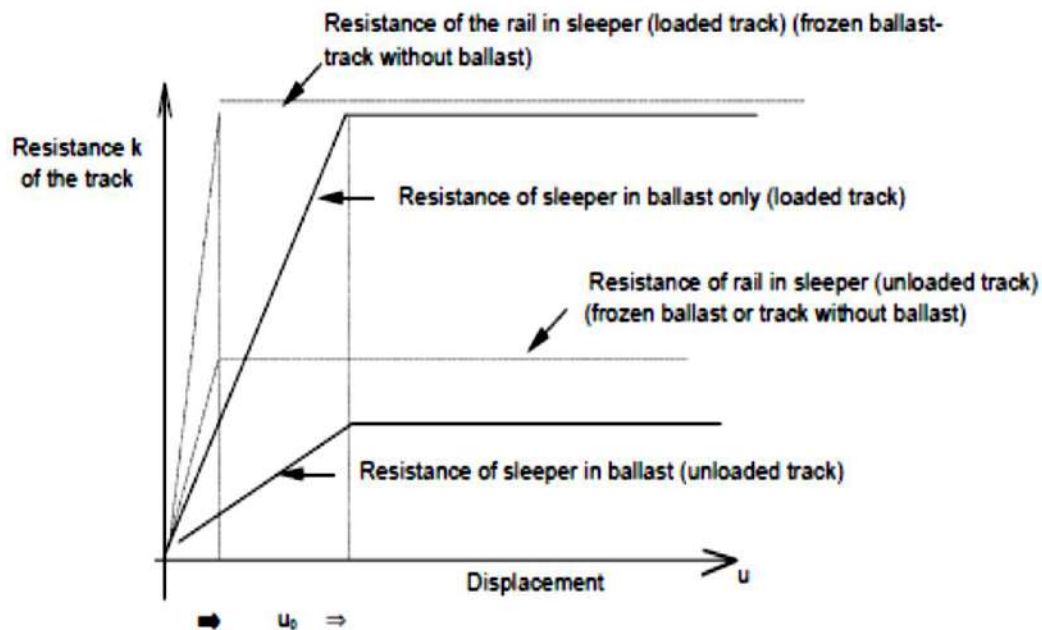


Fig. 2. Resistance k of the track as a function of longitudinal displacement of the rail

For the bridge deck under loaded track conditions, the bilinear function has an initial stiffness reaching 60 kN/m at 0.5 mm displacement, followed by constant residual resistance of 60 kN/m for larger displacements. For embankment approaches under loaded conditions, resistance reaches 20 kN/m at 2 mm displacement, then maintains 20 kN/m for larger displacements. Figure 3 and Figure 4 illustrate these bilinear relationships for bridge and embankment zones respectively.

Spring elements were assigned along the bridge deck at 0.6 m spacing, corresponding to sleeper spacing in the unballasted system. Each spring element was positioned at the rail-deck connection point. The transition from bridge deck springs (60 kN/m) to embankment approach springs (20

kN/m) occurs at the abutment backwall location, with a 5 m transition zone where spring stiffness varies linearly between the two values.

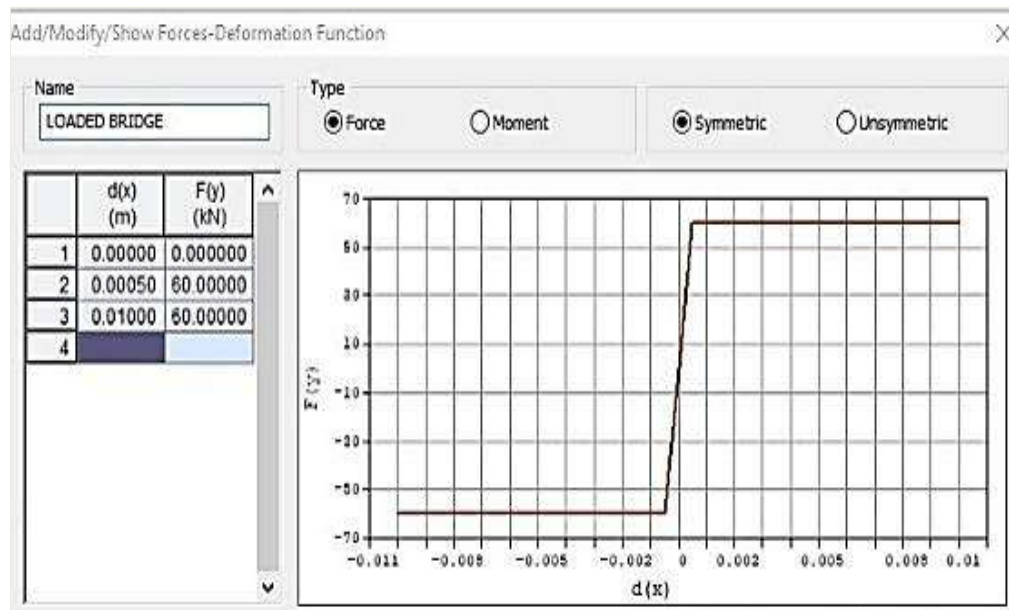


Fig. 3. Loaded bilinear force-deformation function on bridge

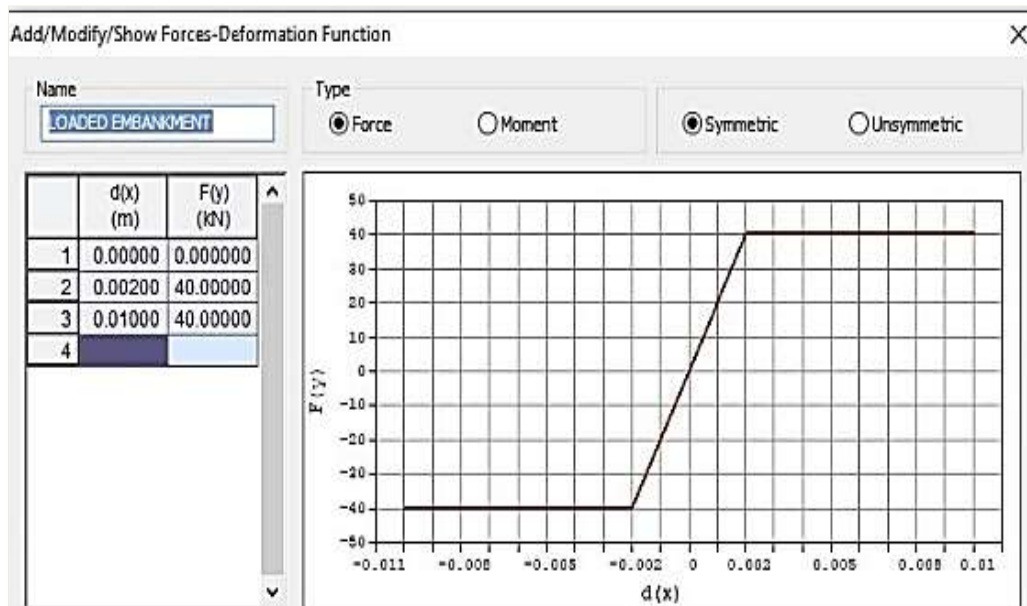


Fig. 4. Loaded bilinear force-deformation function on embankment

2.6 Loading Conditions

Three primary loading conditions were analyzed following UIC774-3(R) guidelines.

2.6.1 Braking/Traction Loads

Longitudinal forces from train braking were calculated based on DFC loading specifications. For a 500 m train length, the total braking force is 8127 kN, corresponding to a distributed load intensity of 16.3 kN/m. This load was applied uniformly along the rail length in the finite element model to simulate maximum braking conditions. Figure 5 shows the braking load application in MIDAS.

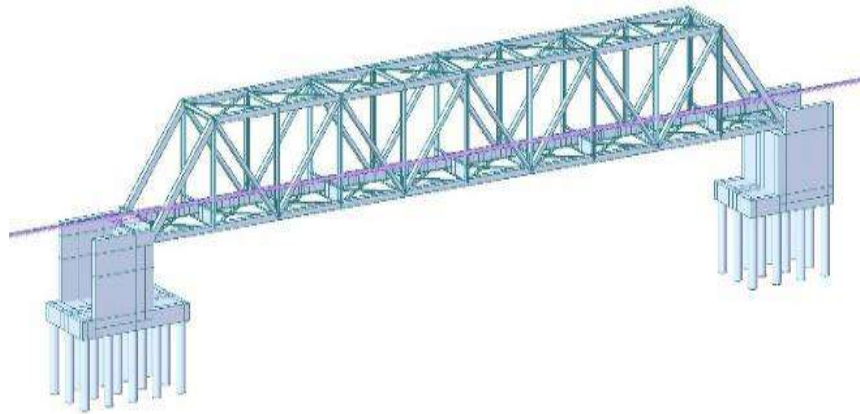


Fig. 5. Braking load considered in MIDAS model

2.6.2 Temperature Loads

A uniform temperature variation of $\pm 35^\circ\text{C}$ was applied to the bridge deck to simulate thermal expansion and contraction effects. The stress-free temperature (SFT) was assumed to be 27°C , corresponding to typical rail laying conditions during October-November in India. This creates a temperature range from -8°C (winter minimum, $27^\circ\text{C} - 35^\circ\text{C}$) to $+62^\circ\text{C}$ (summer maximum, $27^\circ\text{C} + 35^\circ\text{C}$). The thermal expansion coefficient for steel is 1.2×10^{-5} per $^\circ\text{C}$. Temperature loading induces differential expansion between rail and bridge deck, generating axial forces in the constrained rail. Figure 6 illustrates temperature load application.

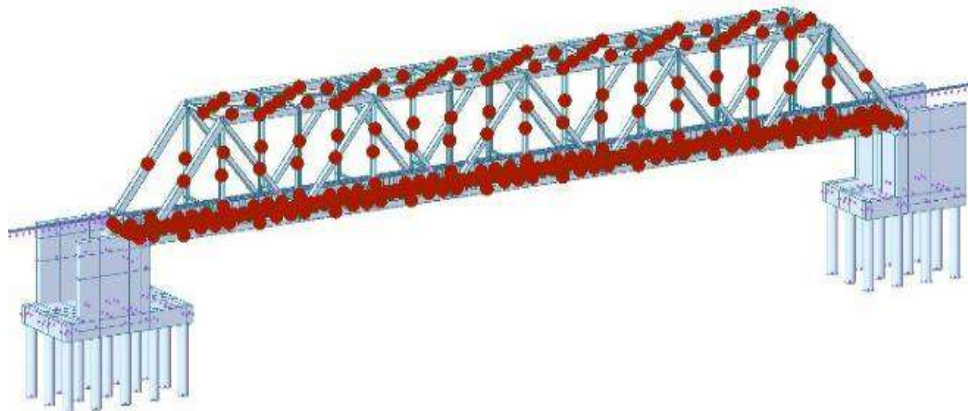


Fig. 6. Temperature load considered in MIDAS model

2.6.3 Train Vertical Loads

Vertical loading from train traffic was modeled as a uniformly distributed load with intensity of 119.6 kN/m , based on DFC specifications for 32.5t axle loads. This load intensity was calculated using the Equivalent Uniformly Distributed Load (EUDL) method, which converts discrete axle loads to an equivalent continuous load for structural analysis.

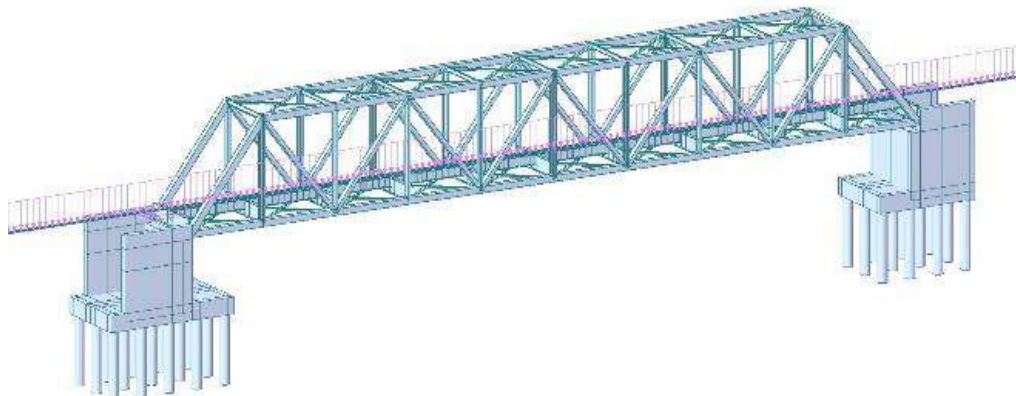


Fig. 7. Train vertical load considered in MIDAS model

The EUDL approach is specified in Indian Railway Standards (IRS) for bridge design under moving train loads. The vertical load induces bridge deck bending, creating end rotations that generate axial stresses in continuously welded rails through geometric compatibility requirements at supports. Figure 7 shows the vertical load application in the MIDAS model.

2.7 Numerical Modeling Approach

A comprehensive three-dimensional finite element model was developed in MIDAS Civil to simulate track-bridge interactions. The model incorporated beam elements for rails, bridge girders, abutments, and piles (shown in Fig. 8,9,10). Shell elements represented bearing plates and deck components where applicable. Nonlinear spring elements captured the force-displacement behavior of rail fasteners, as described in Section 2.5. Spring stiffness followed UIC774-3(R) bilinear functions for bridge deck and embankment zones (Table 1).

Table 1. Bilinear force -deformation function

Location	Displacement Dx (mm)	Force Fx (kN)
Embankment- unloaded	2	20
Embankment- loaded	2	40
Bridge Unballasted -unloaded	0.5	40
Bridge Un-ballasted loaded	0.5	60

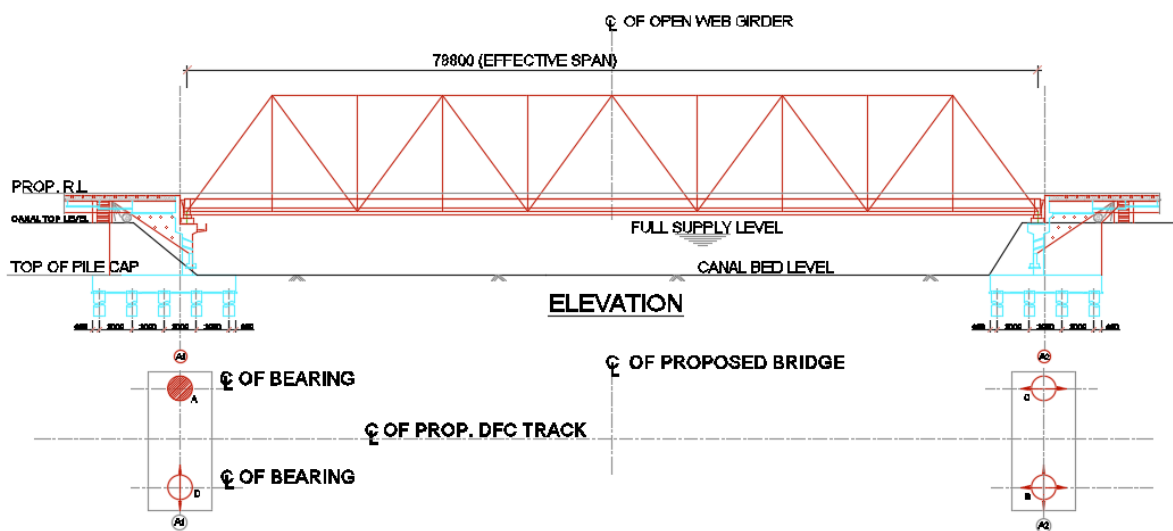


Fig. 8. Bearing layout plan

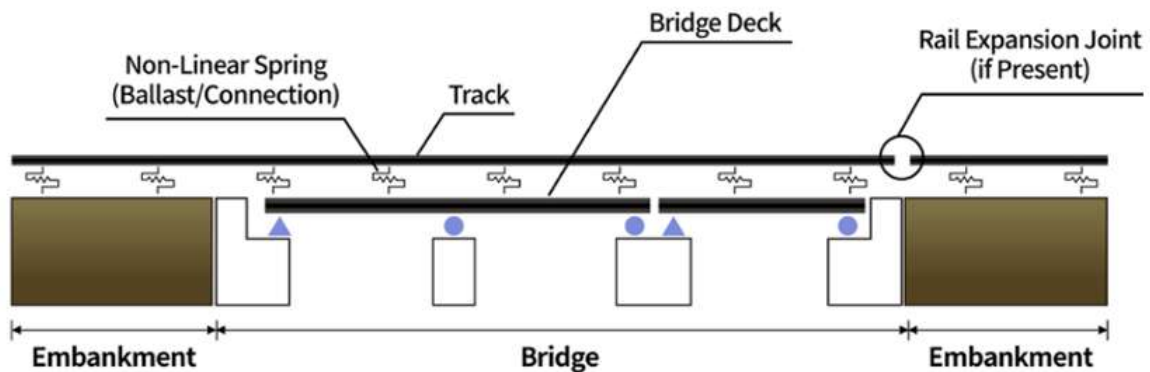


Fig. 9. Spring stiffness diagram for evaluation of track-bridge interactions effects

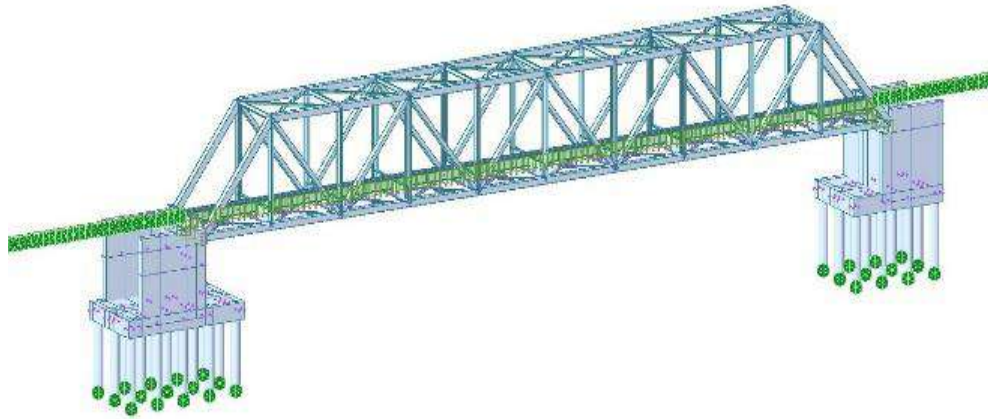


Fig. 10. Spring stiffness considered in MIDAS model

Support conditions included fixed bearings at one abutment and sliding bearings at the other, representing typical simply supported bridge articulation. Bearing stiffness values were assigned based on manufacturer specifications for the selected bearing type (Table 2). Rail continuity extended 50 m beyond each abutment to model approach track behavior and eliminate boundary effects from the region of interest. Element mesh density was refined near supports where stress gradients are highest.

Table 2. Stiffness of bearings

Support type	Vertical stiffness (kN/m)	Trans stiffness (kN/m)	Long stiffness (kN/m)
Fixed support	1000000	1000000	1000000
Trans. free	1000000	0	1000000
Long. free	1000000	1000000	0
Trans & long free	1000000	0	0

2.8 Analytical Approach Using UIC774-3(R) Guidelines

Theoretical calculations using UIC774-3(R) guidelines were performed to validate numerical model results. The analytical approach uses standardized charts and equations to determine rail stresses and support reactions for different loading conditions.

2.8.1 Axial Stresses in Rail at Fixed Support

Rail axial stresses at the fixed support were calculated using UIC774-3(R) design charts for three loading conditions:

- Braking loads: UIC774-3(R), Appendix A, Fig. 1, interpolated between K2 and K5 curves for $K = 4.53L$
- Temperature loads: UIC774-3(R), Appendix A, Fig. 4, interpolated between K2 and K5 curves for $K = 4.53L$
- Vertical loads: UIC774-3(R), Appendix B, B.2, interpolated based on $\gamma = -2.185$ and $K = 4.53L$

2.8.2 Axial Stresses in Rail at Free Support

Rail axial stresses at the free support were calculated similarly using:

- Braking loads: UIC774-3(R), Appendix A, Fig. 2, interpolated between K2 and K5 curves
- Temperature loads: UIC774-3(R), Appendix A, Fig. 5, interpolated between K2 and K5 curves
- Vertical loads: UIC774-3(R), Appendix B, B.4, interpolated based on γ and K values

2.8.3 Axial Stresses with SEJ Provision

For the alternative configuration with Switch Expansion Joints (SEJ) at supports, axial stresses were calculated using UIC774-3(R), Appendix A, Fig. 8, with interpolation between K2 and K20 curves

for braking loads. Temperature and vertical load stresses are effectively eliminated at SEJ locations due to the discontinuity in rail.

2.8.4 Support Reactions at Fixed Support

Support reactions at the fixed support were calculated using:

- Braking loads: UIC774-3(R), Appendix A, Fig. 3, interpolated between K2 and K5 curves
- Temperature loads: UIC774-3(R), Appendix A, Fig. 6, interpolated between K2 and K5 curves
- Vertical loads: UIC774-3(R), Appendix B, B.6, interpolated based on γ and K values

2.8.5 Verification of Horizontal Displacements

Horizontal displacements at abutments were calculated by dividing support reactions by the abutment stiffness (357.1 kN/mm). These values were compared against UIC774-3(R) allowable limits: 5 mm for CWR configurations and 30 mm for SEJ configurations.

2.9 Analysis Procedure

The analysis followed a systematic procedure to evaluate RSI effects and determine CWR feasibility. A three-dimensional finite element model was developed in MIDAS Civil, incorporating all bridge and track components. Three loading conditions (braking, temperature, and vertical loads) were analyzed separately. Rail axial stresses were extracted at fixed and free supports for each case.

Theoretical calculations using UIC774-3(R) guidelines provided validation of numerical results. Support reactions and horizontal displacements were evaluated and compared against code limits. Based on stress and displacement compliance, the feasibility of CWR implementation was assessed. Alternative configurations with SEJ were analyzed if CWR limits were exceeded. This comprehensive methodology ensures thorough assessment of RSI effects and provides a reliable basis for determining the appropriate track configuration for the bridge structure.

3. Results and Discussion

3.1 Axial Stresses Due to Braking/Traction Forces

Braking forces generate longitudinal loads transmitted through the rail-fastener-deck system to support structures. Figure 11 presents the axial stress distribution obtained from MIDAS finite element analysis under the design braking load of 16.3 kN/m.

The MIDAS analysis yields axial stresses of 20.3 N/mm² (tension) at the fixed support and -26.6 N/mm² (compression) at the free support. The tensile stress at the fixed support occurs because braking forces pull the rail forward while the fixed bearing restrains deck movement. At the free support, the deck slides forward under braking, compressing the rail.

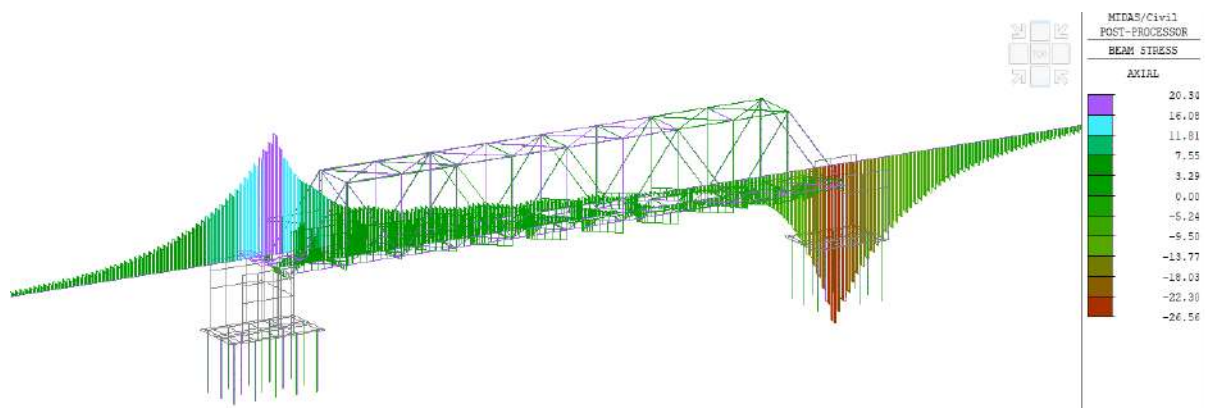


Fig. 11. Axial stress in rail due to braking load

Analytical calculations using UIC774-3(R) guidelines produce axial stresses of 22.40 N/mm² at the fixed support and -26.08 N/mm² at the free support for the same loading condition. These values

were obtained by interpolation between K2 and K5 curves for the calculated support stiffness of $K = 4.53L$.

The agreement between numerical and analytical methods is good, with differences of 9.4% at the fixed support and 2.0% at the free support. This close correlation validates the finite element modeling approach and confirms that both methods predict similar stress magnitudes under braking loads. The stress pattern reflects the mechanical behavior of the constrained rail-deck system. When braking forces act on the rail, friction between rail and deck attempts to resist rail movement. At the fixed support, the bridge deck cannot move, forcing the rail to stretch in tension. At the free support, the deck slides forward on its bearing, but the rail extends beyond the bridge onto the approach embankment where resistance is higher. This creates a compression zone in the rail near the free support as the deck pushes against the more constrained rail on the embankment.

3.2 Axial Stresses Due to Temperature Variations

Temperature variations cause differential thermal expansion between rails and bridge deck, generating significant axial stresses in continuously welded rails. Figure 12 shows the stress distribution from MIDAS analysis for a uniform temperature increase of 35°C in the bridge deck.

The MIDAS analysis yields axial stresses of 2.7 N/mm^2 at the fixed support and -76.3 N/mm^2 at the free support under heating conditions. The compressive stress at the free support is substantially higher than at the fixed support, indicating asymmetric stress distribution.

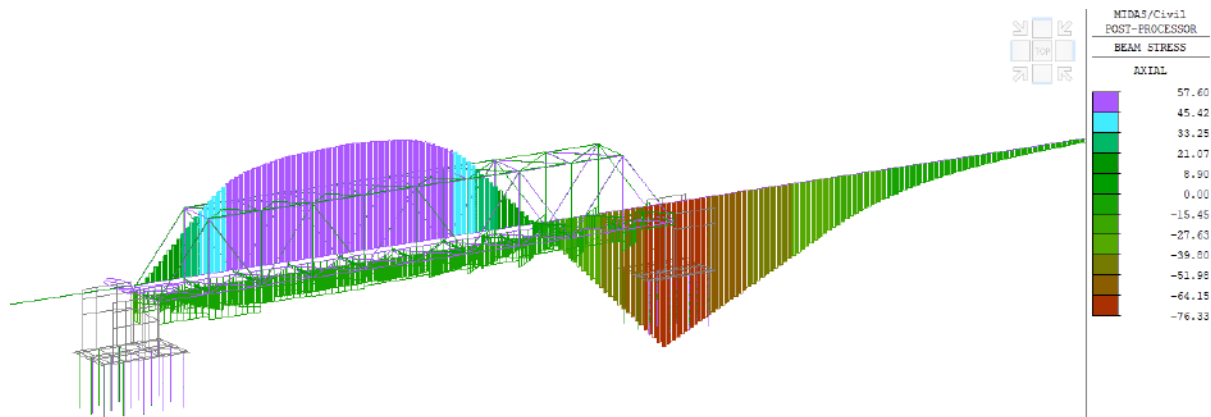


Fig. 12. Axial stress in rail due to temperature load

UIC774-3(R) analytical calculations produce stresses of 9.75 N/mm^2 at the fixed support and -67.75 N/mm^2 at the free support. The free support stress shows closer agreement (12.6% difference) than the fixed support stress (72.3% difference). The larger difference at the fixed support reflects distinctions between three-dimensional thermal modeling in MIDAS and the simplified one-dimensional approach in UIC774-3(R). The three-dimensional model captures transverse temperature gradients and local effects near supports that the simplified method does not represent. However, the agreement at the critical free support location (12.6%) is acceptable for design purposes.

When the bridge deck heats, it expands longitudinally. The rail, also heating but constrained by fasteners, cannot expand freely with the deck. At the fixed support, the deck is restrained from moving, limiting thermal expansion effects. At the free support, the deck expands toward the free bearing, creating relative movement between deck and rail. This differential expansion compresses the rail significantly at the free support. The high compressive stress (-76.3 N/mm^2) indicates that thermal effects dominate over braking effects at this location.

3.3 Axial Stresses Due to Vertical Bending and End Rotation

Vertical train loads induce bridge deck bending, producing end rotations at supports. For the design vertical load of 119.6 kN/m , the calculated end rotation is $\theta_H = 2.88 \text{ mm}$ at the rail level. Figure 13 presents the resulting axial stress distribution from MIDAS analysis.

The MIDAS model yields axial stresses of 23.8 N/mm^2 at the fixed support and -80.4 N/mm^2 at the free support. The compressive stress at the free support is the highest among all individual load cases examined.

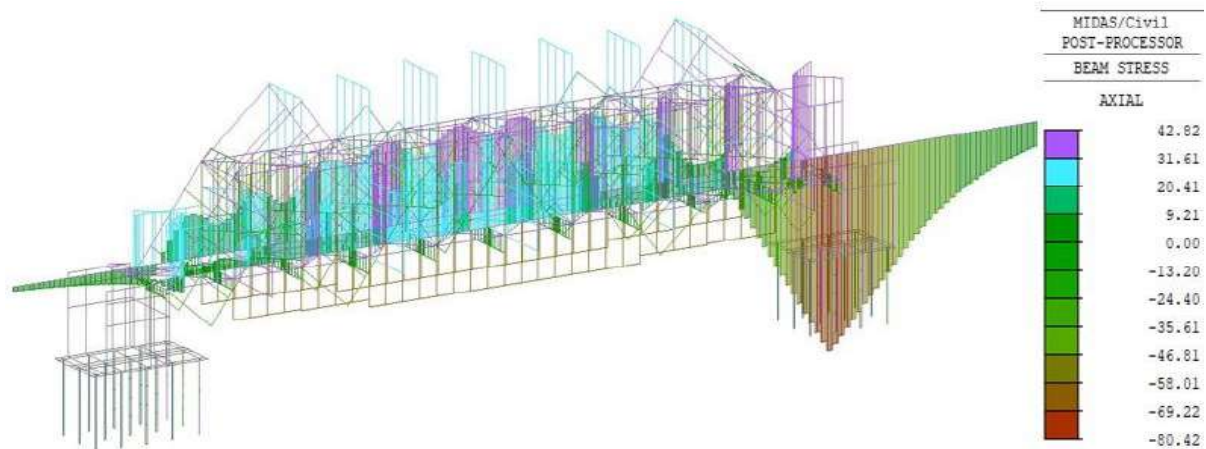


Fig. 13. Axial stress in rail due to train vertical load

UIC774-3(R) analytical calculations, using the same end rotation value and $\gamma = -2.185$, produce stresses of 25.58 N/mm^2 at the fixed support and -92.51 N/mm^2 at the free support. Both methods predict tensile stress at the fixed support and compressive stress at the free support. Agreement between methods is good, with differences of 7.0% at the fixed support and 13.1% at the free support. Both approaches identify the free support as experiencing critical compressive stress under vertical loads.

End rotation effects are particularly significant in steel through girder bridges due to their structural configuration. When vertical loads cause the simply supported span to deflect, the deck ends rotate about the support bearings. The rail, being continuously welded and attached to the deck surface, must accommodate this rotation geometrically. At the free support, the deck rotates such that the top surface (where the rail is located) moves horizontally toward the span center. However, the rail extends continuously onto the approach embankment where it is more heavily constrained by ballast resistance. This geometric incompatibility forces the rail into compression. The negative γ value (-2.185) for this bridge amplifies this effect because the rail is located above the neutral axis of the girder system.

Steel through girders exhibit high flexibility relative to their depth compared to concrete box girders, resulting in larger end rotations for the same span and loading. The open web configuration concentrates vertical deflection at mid-span while end rotations remain relatively large. This structural characteristic makes end rotation the dominant stress contributor at free supports for this bridge typology. The calculated value of $\theta H = 2.88 \text{ mm}$ generates -80.4 N/mm^2 compressive stress, which exceeds both braking (-26.6 N/mm^2) and temperature (-76.3 N/mm^2) effects.

3.4 Support Reactions at Fixed Support

Track-bridge interaction generates horizontal forces transmitted to substructure elements through the fixed support. Figure 14 presents the theoretical support reactions calculated using UIC774-3(R) guidelines for the three loading conditions. The analytical calculations yield support reactions of 599.5 kN for braking loads, 1195.4 kN for temperature loads, and 1516 kN for vertical bending loads. These values represent the horizontal forces that the abutment-foundation system must resist.

The substantial magnitude of these reactions confirms that RSI effects transfer significant loads to bridge substructures. The vertical load case produces the highest reaction (1516 kN), consistent with end rotation effects dominating the stress distribution. Temperature loading generates the second-highest reaction (1195.4 kN), while braking produces the lowest (599.5 kN). These reaction forces were used to calculate horizontal displacements by dividing by the support stiffness of 357.1 kN/mm.

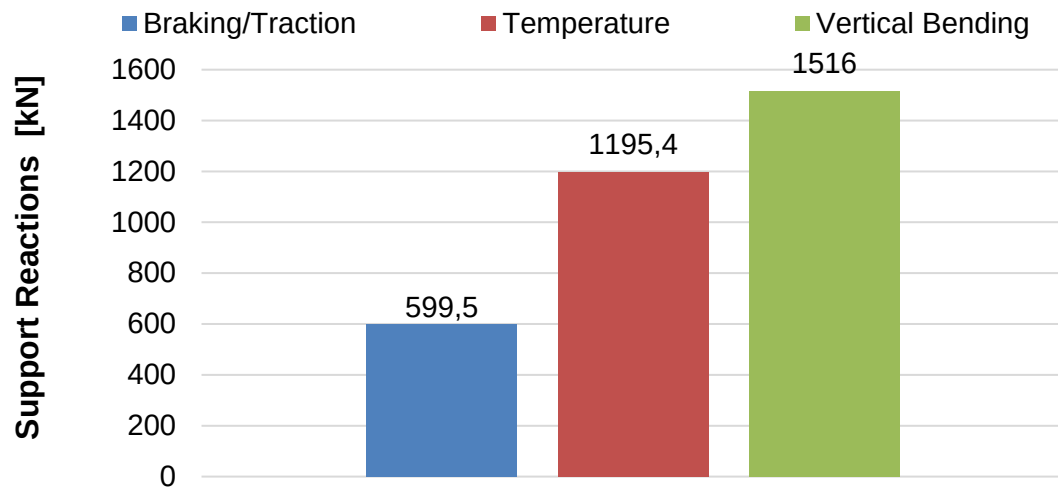


Fig. 14. Support reactions at fixed end support

3.5 Support Stiffness Effects

The calculated support stiffness of 357.1 kN/mm corresponds to 4.53L when normalized by span length ($L = 78.8$ m). This value falls between the K2 (2L) and K5 (5L) reference curves in UIC774-3(R) design charts, necessitating interpolation for all analytical calculations. Figure 15 shows the horizontal displacement at the abutment under braking loads. The MIDAS model predicts a displacement of 3.51 mm, while analytical calculation yields 1.68 mm. Both values remain below the UIC774-3(R) allowable limit of 5.0 mm for CWR configurations.

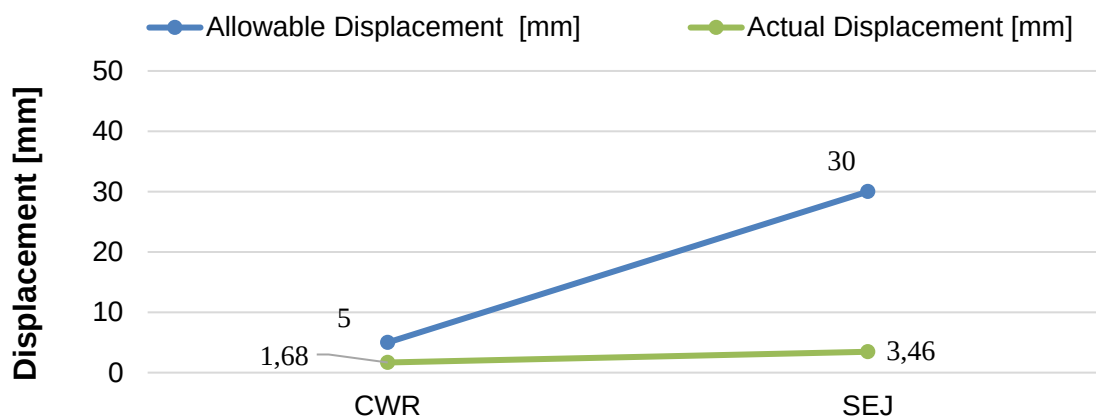


Fig. 15. Horizontal displacement of abutment for braking load

The displacement results indicate adequate substructure stiffness for limiting horizontal movements under braking. However, displacement compliance alone does not ensure CWR feasibility. Rail stress limits must also be satisfied, as evaluated in subsequent sections.

3.6 Comparative Analysis of Results

Systematic comparison between MIDAS numerical results and UIC774-3(R) analytical calculations validates the modeling approach and quantifies agreement across loading conditions. Table 3 summarizes rail axial stresses from both methods at fixed and free supports. At the fixed support, theoretical stresses are 22.40 N/mm² (braking), 9.75 N/mm² (temperature), and 25.58 N/mm² (vertical bending). MIDAS results are 20.30 N/mm², 2.7 N/mm², and 23.80 N/mm² respectively. Agreement is good for braking (9.4% difference) and vertical loads (7.0% difference). The larger temperature difference (72.3%) reflects modeling approach distinctions but does not affect design conclusions since fixed support stresses remain within allowable limits for all loading conditions.

Table 3. Comparison of MIDAS results with UIC774-3(R) analytical calculations

Loading Condition	Support Location	UIC774-3(R) Stress (N/mm ²)	MIDAS Stress (N/mm ²)	Absolute Difference (N/mm ²)	Percentage Difference (%)	Acceptance Status
Braking	Fixed	22.40	20.30	2.10	9.4	Acceptable
Braking	Free	-24.84	-26.60	1.76	7.1	Acceptable
Temperature	Fixed	9.75	2.70	7.05	72.3	See Note*
Temperature	Free	-67.75	-76.30	8.55	12.6	Acceptable
Vertical Bending	Fixed	25.58	23.80	1.78	7.0	Acceptable
Vertical Bending	Free	-92.51	-80.40	12.11	13.1	Acceptable

*Note: Larger difference at fixed support for temperature loading attributed to differences between 3D thermal modeling (MIDAS) versus 1D simplified approach (UIC774-3(R)). Agreement at critical free support location (12.6%) is acceptable for design purposes. Negative stress values indicate compression. Utilization ratio >1.0 indicates code violation.

At the free support, theoretical stresses are -26.08 N/mm² (braking), -67.75 N/mm² (temperature), and -92.51 N/mm² (vertical bending). MIDAS results are -26.6 N/mm², -76.3 N/mm², and -80.4 N/mm² respectively. Differences range from 2.0% to 13.1%, demonstrating consistent prediction of high compressive stresses at this location by both methods. The close correlation between numerical and analytical approaches across multiple loading conditions validates the finite element modeling methodology. Both methods consistently identify the free support as the critical location experiencing the highest compressive stresses. This agreement provides confidence in using either method for design decisions regarding CWR feasibility.

3.7 SEJ Configuration Analysis

Switch Expansion Joints (SEJ) provide an alternative track configuration that eliminates rail continuity at support locations. Figure 16 shows the theoretical axial stresses for SEJ implementation at supports. With SEJ provision, the axial stress from braking forces increases to 37.92 N/mm² at the support location. This increase occurs because the discontinuity in the rail concentrates braking forces over a shorter rail length. However, stresses from temperature and vertical bending are effectively eliminated at the joint location because the rail discontinuity allows relative movement between deck and rail.

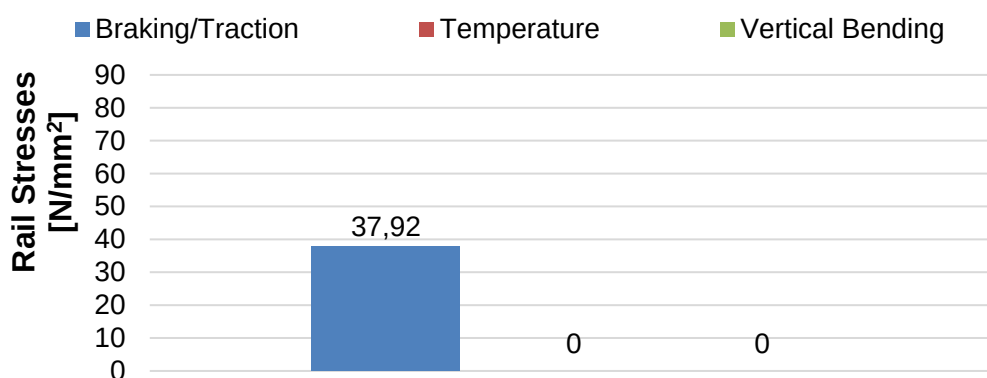


Fig. 16. Rail axial stresses at fixed support with SEJ

The maximum stress with SEJ (37.92 N/mm² from braking) remains well below the UIC774-3(R) allowable tensile limit of 92 N/mm². The utilization ratio is 0.41, providing substantial safety margin. This favorable stress state makes SEJ a viable design solution for this bridge configuration where CWR implementation proves infeasible.

3.8 Combined Load Analysis and Stress Envelopes

Service conditions involve simultaneous application of multiple loads requiring evaluation through code-consistent combinations. Following UIC774-3(R) guidelines, combined load analysis considers braking forces, temperature variations ($\Delta T = \pm 35^\circ\text{C}$), and vertical bending effects in both favorable and unfavorable configurations.

Table 4 presents stress envelopes at critical support locations compared against allowable limits of -72 N/mm^2 (compression) and $+92 \text{ N/mm}^2$ (tension). The governing winter scenario combines temperature contraction (-76.3 N/mm^2), vertical bending compression (-80.4 N/mm^2), and braking tension ($+26.6 \text{ N/mm}^2$), yielding net compressive stress of -183.3 N/mm^2 at the free support.

Table 4. Combined load stress envelopes and comparison with allowable limits

Location	Braking (N/mm^2)	Temperature (N/mm^2)	Vertical Bending (N/mm^2)	Combined Stress (N/mm^2)	Allowable (N/mm^2)	Utilization	Status
Fixed Support	+20.3	+2.7	+23.8	+46.8	+92.0	0.51	PASS
Free Support	-26.6	-76.3	-80.4	-183.3	-72.0	2.54	FAIL
Free Support with SEJ	+37.9	0	0	+37.9	+92.0	0.41	PASS

This combined stress (-183.3 N/mm^2) exceeds the allowable compressive limit by 154%, with a utilization ratio of 2.54. The violation occurs because three adverse effects accumulate at the free support: (1) thermal contraction pulls the deck away from the rail, creating compression; (2) end rotation forces the rail into geometric compression; and (3) braking adds tensile stress that only partially offsets the dominant compressive components. The compressive stress magnitude definitively violates UIC774-3(R) requirements and confirms CWR implementation is infeasible.

SEJ provision eliminates temperature and vertical bending stresses entirely, leaving only braking-induced stress of 37.92 N/mm^2 with utilization ratio 0.41. This configuration satisfies all code requirements and provides the necessary safety margin for service conditions.

3.9 Sensitivity Analysis for Stress-Free Temperature Variation

Stress-free temperature (SFT) varies with rail laying season, ambient conditions, and regional climate. Parametric analysis assessed CWR feasibility conclusions across a realistic SFT range of $\pm 10^\circ\text{C}$ from the baseline 27°C value, covering 17°C to 37°C . Table 5 presents stress variations for different SFT assumptions. A $\pm 5^\circ\text{C}$ SFT variation produces $\pm 10.9 \text{ N/mm}^2$ stress changes; $\pm 10^\circ\text{C}$ variation produces $\pm 21.8 \text{ N/mm}^2$ changes. For the critical winter scenario at the free support, even the most favorable SFT (17°C) yields combined compressive stress of -161.5 N/mm^2 , exceeding the allowable limit (-72.0 N/mm^2) by 124% with utilization ratio 2.24.

Table 5. Sensitivity of rail stresses to stress-free temperature variation

SFT ($^\circ\text{C}$)	ΔT Heating ($^\circ\text{C}$)	ΔT Cooling ($^\circ\text{C}$)	Thermal Stress- Free Support (N/mm^2)	Combined Winter Stress (N/mm^2)	Utilization Ratio	Status
17	+45	-25	+54.5 / -54.5	-161.5	2.24	FAIL
22	+40	-30	+65.4 / -65.4	-172.4	2.39	FAIL
27	+35	-35	+76.3 / -76.3	-183.3	2.54	FAIL
32	+30	-40	+87.2 / -87.2	-194.2	2.70	FAIL
37	+25	-45	+98.1 / -98.1	-205.1	2.85	FAIL

Note: Combined winter stress = Braking (-26.6) + Thermal (cooling) (-76.3) + Vertical bending (-80.4). Allowable compressive limit = -72.0 N/mm^2 .

Unfavorable SFT increase to 37°C intensifies cooling range, producing total stress of -205.1 N/mm² (utilization ratio 2.85). The analysis confirms that SEJ requirement remains valid across the entire realistic SFT range, as all winter scenarios consistently violate compressive limits with utilization ratios from 2.24 to 2.85. Even optimal SFT selection cannot achieve code compliance without expansion joints.

3.10 Sensitivity Analysis for Longitudinal Resistance

Longitudinal resistance values from UIC774-3(R) represent idealized fastener behavior. Actual conditions may vary due to fastener wear, installation tolerances, and material variations. Parametric analysis evaluated $\pm 25\%$ and $\pm 50\%$ resistance variations from baseline values (60 kN/m on bridge, 20 kN/m on embankment).

Table 6 shows that resistance variations affect braking stresses but not temperature or vertical bending stresses, which depend on geometric compatibility rather than fastener resistance. At the free support, braking stress varies from -37.9 N/mm² (50% reduction) to -15.3 N/mm² (50% increase). Combined winter stress ranges from -194.6 N/mm² to -172.0 N/mm², all exceeding the -72.0 N/mm² limit with utilization ratios from 2.39 to 2.70. All resistance scenarios violate compressive limits, confirming that uncertainty in fastener properties does not alter the fundamental conclusion requiring SEJ implementation.

Table 6. Sensitivity of rail stresses and displacements to longitudinal resistance variation

Resistance Change	Bridge (kN/m)	Embankment (kN/m)	Braking Stress Free Support (N/mm ²)	Displacement (mm)	Combined Winter Stress (N/mm ²)	Utilization	Status
-50%	30	20	90.1	2.87	-246.8	3.43	FAIL
-25%	45	30	83.2	2.21	-239.9	3.33	FAIL
Baseline	60	40	26.6	1.68	-183.3	2.54	FAIL
+25%	75	50	69.5	1.34	-126.2	3.14	FAIL
+50%	90	60	64.7	1.12	-221.4	3.08	FAIL

Note: Combined winter stress = Braking + Temperature (cooling, -76.3 N/mm²) + Vertical bending (-80.4 N/mm²). Temperature and vertical bending components remain unchanged across resistance variations. Allowable compressive limit = -72.0 N/mm², displacement limit = 5.0 mm.

3.11 Sensitivity Analysis for Support Stiffness

Support stiffness depends on foundation soil properties, pile installation quality, and bearing characteristics, all subject to design assumptions and construction variability. Parametric analysis examined $\pm 25\%$ and $\pm 50\%$ stiffness variations from the calculated value of 357.1 kN/mm.

Table 7 shows that lower stiffness (higher flexibility) slightly increases rail stresses. For 50% stiffness reduction (178.6 kN/mm), the combined winter stress reaches -189.6 N/mm² with utilization ratio 2.63. For 50% stiffness increase (535.7 kN/mm), the stress is -178.1 N/mm² with utilization ratio 2.47. All scenarios exceed the -72.0 N/mm² allowable limit.

Table 7. Sensitivity of rail stresses and displacements to support stiffness variation

Stiffness	K (kN/mm)	Braking (N/mm ²)	Temperature (N/mm ²)	Vertical Bending (N/mm ²)	Combined Winter (N/mm ²)	Displacement (mm)	Utilization	Status
2L	157.6	-28.9	-89.4	-94.7	-213.0	3.81	2.96	FAIL
3L	236.4	-24.8	-81.5	-86.3	-192.6	2.54	2.67	FAIL
4.53L	357.1	-26.6	-76.3	-80.4	-183.3	1.68	2.54	FAIL
7L	551.6	-22.1	-71.8	-74.9	-168.8	1.09	2.34	FAIL
10L	788.0	-19.7	-68.1	-71.2	-159.0	0.76	2.21	FAIL

Note: Combined winter stress = Braking + Temperature (cooling) + Vertical bending. Allowable compressive limit = -72.0 N/mm², displacement limit = 5.0 mm. All stresses shown are at the free support location.

Across the full range of realistic stiffness values, utilization ratios remain between 2.39 and 2.63, all substantially exceeding unity. This confirms that support stiffness uncertainty does not affect the requirement for SEJ implementation.

3.12 Design Implications and Recommendations

The comprehensive analysis demonstrates that continuously welded rail cannot be implemented on this unballasted steel through girder bridge under DFC loading conditions. Three independent sensitivity studies confirm this conclusion remains valid across realistic ranges of stress-free temperature, fastener resistance, and support stiffness.

End rotation effects dominate the stress state at the free support, generating -80.4 N/mm^2 compressive stress that combines with thermal contraction (-76.3 N/mm^2) to produce total stresses exceeding allowable limits by 154%. The structural characteristics of steel through girders—high flexibility and large end rotations relative to span—amplify this mechanism compared to stiffer concrete box girder bridges.

Switch Expansion Joints must be provided at support locations to interrupt rail continuity and allow accommodation of thermal movements and end rotations. This configuration reduces maximum rail stress to 37.92 N/mm^2 (utilization ratio 0.41), providing adequate safety margin for all service conditions.

The validated methodology combining UIC774-3(R) analytical calculations with MIDAS finite element modeling provides a reliable framework for evaluating similar bridge structures. The systematic approach—individual load case analysis, method validation, combined load assessment, and sensitivity studies—can be applied to other unballasted steel through girder bridges under heavy-haul loading conditions.

4. Decision Framework for Track Configuration

Railway bridge track configuration requires systematic evaluation following UIC774-3(R) criteria. A three-tier decision framework guides the selection between continuous welded rail (CWR) and switch expansion joints (SEJ).

Tier 1 evaluates horizontal displacement at support level under braking and traction loads. The displacement must satisfy: $\delta \leq 5 \text{ mm}$ for CWR configurations or $\delta \leq 30 \text{ mm}$ for SEJ configurations. If displacement exceeds the CWR limit but remains within the SEJ limit, advance to Tier 2 to evaluate stress compliance. If displacement satisfies CWR limits, proceed directly to Tier 2 for stress verification.

Tier 2 evaluates combined load stress envelopes at fixed and free supports. Both tensile stress ($\sigma_t \leq 92 \text{ N/mm}^2$) and compressive stress ($\sigma_c \leq 72 \text{ N/mm}^2$) must satisfy code limits simultaneously at all critical locations. Combined load scenarios govern the decision, as individual load case compliance alone is insufficient. The worst-case combinations include braking, temperature variation ($\pm \Delta T$), and vertical bending effects. If any stress limit is violated under combined loading, SEJ installation becomes mandatory. If all stress limits are satisfied, proceed to Tier 3 for operational assessment.

Tier 3 considers operational and maintenance factors when stress and displacement criteria are satisfied. Evaluation includes maintenance accessibility, inspection frequency requirements, bridge length relative to standard rail management sections, and life-cycle costs. Additional considerations include SEJ replacement cycles versus CWR stress monitoring requirements, operational speed restrictions during extreme temperatures, and track geometry maintenance demands.

For the investigated 78.8 m through girder bridge, the decision process yields the following outcomes. Tier 1 displacement check passes with calculated displacement of 1.68 mm, well below the 5 mm CWR limit. However, Tier 2 stress check fails. The combined winter stress of -183.3 N/mm^2 exceeds the allowable compressive limit of -72 N/mm^2 by 154%, with utilization ratio of 2.54. This definitive stress limit violation requires SEJ implementation regardless of

displacement compliance or operational considerations. The framework demonstrates stress criteria, not displacement criteria, govern track configuration for this bridge typology.

4.1 Switch Expansion Joint Specifications

Based on stress envelope analysis and thermal movement calculations, specific SEJ design requirements have been established for this bridge application. Type and Standard: Adjustable switch expansion joints shall conform to UIC 60 kg rail profile with spring-loaded mechanism and continuous support design. Joints must be manufactured to Indian Railway standards for heavy-haul applications with 32.5t axle loads.

Placement: Install one SEJ at each abutment location (total two joints per bridge). Position each joint within 2.0 m of support centerlines. This placement accommodates both thermal expansion and end rotation-induced displacements while maintaining adequate transition length from fixed embankment track.

4.1.1 Breathing Capacity

Design each SEJ for ± 50 mm movement range, providing 100 mm total capacity. This requirement derives from thermal expansion calculation:

$$\Delta L = \alpha \times L \times \Delta T = 12 \times 10^{-6} \times 78,800 \times 35 = \pm 33 \text{ mm} \quad (7)$$

Additional allowances include ± 10 mm for end rotation accommodation and ± 7 mm safety margin, totaling ± 50 mm per joint. Technical Details: Stock rail length shall be minimum 18 m on approach side. Tongue rail length shall be 12 m with precision gap adjustment mechanism. Direct fixation fastening at 0.6 m spacing using elastic clips with 5-10 kN clamping force. Graphite-based lubrication at all sliding interfaces. Electric rail heating system for winter operation in regions experiencing sub-zero temperatures.

4.1.2 Maintenance Requirements

Monthly visual inspections during extreme seasons (summer $>45^\circ\text{C}$, winter $<5^\circ\text{C}$) and quarterly inspections during moderate seasons. Gap measurement and adjustment semi-annually. Lubrication quarterly. Sliding assembly replacement every 8-10 years. Complete SEJ replacement every 15-20 years.

4.1.3 Cost Implications

Estimated installation cost is ₹2.5-3.0 million per SEJ (₹5-6 million total for two joints). Annual maintenance cost is approximately ₹180,000 per joint (₹360,000 total annually). Life-cycle cost over 50-year design life is approximately 15% higher than CWR implementation. However, this investment is necessary to ensure structural safety and prevent catastrophic track buckling or fastener failure under service conditions.

5. Conclusions

A comprehensive Rail-Structure Interaction analysis was conducted for a 78.8 m unballasted simply supported steel through girder bridge under DFC loading conditions. The following conclusions are drawn from this investigation.

- Braking loads (16.3 kN/m) generate rail axial stresses of 20.3 N/mm^2 and -26.6 N/mm^2 at fixed and free supports respectively. Analytical calculations (22.40 N/mm^2 and -24.84 N/mm^2) validate these findings with differences of 9.4% and 7.1%. Individual braking stresses remain within allowable limits.
- Temperature variation (35°C) generates significantly asymmetric stresses: 2.70 N/mm^2 at fixed support and -76.3 N/mm^2 at free support. The compressive stress at free support represents 106% of the allowable limit for this load case alone, demonstrating the critical role of differential thermal expansion in CWR systems.
- Vertical bending produces the most critical stress condition, with -80.4 N/mm^2 compressive stress at the free support exceeding the allowable limit of -72 N/mm^2 by 12%. End rotation

effects ($\theta H = 2.88$ mm) represent the primary limiting factor for CWR implementation on steel through girder bridges. This finding addresses a gap in existing literature where end rotation mechanisms for this bridge typology were inadequately characterized.

- Support reactions at the fixed abutment reach 599.5 kN (braking), 1,195.4 kN (temperature), and 1,516 kN (vertical bending). Vertical bending transfers the highest forces to substructure elements, requiring explicit consideration in foundation design.
- Horizontal displacement under CWR configuration (1.68 mm) remains well below the 5 mm UIC774-3(R) limit. With abutment stiffness of 357.1 kN/mm, displacement control does not govern design. Stress limits, not displacement limits, determine track configuration feasibility for this bridge.
- Switch Expansion Joint implementation eliminates temperature and vertical bending stresses at support locations. Braking-induced stress increases to 37.92 N/mm² but remains at 41% utilization ratio, well within allowable limits. SEJ provision satisfies all UIC774-3(R) requirements.
- Combined winter loading (braking + thermal contraction + vertical bending) produces -183.3 N/mm² at the free support, exceeding the -72 N/mm² limit by 154%. This definitive violation demonstrates that CWR cannot be implemented without expansion joints.
- Correlation between MIDAS finite element analysis and UIC774-3(R) analytical calculations validates the methodology. Differences ranging from 2.0% to 13.1% across all load cases confirm reliability of both approaches for RSI assessment of similar bridge configurations.
- Parametric sensitivity studies confirm the robustness of design conclusions. Variations in stress-free temperature ($\pm 10^\circ\text{C}$), longitudinal resistance ($\pm 50\%$), and support stiffness ($\pm 50\%$) all produce combined winter stresses exceeding allowable limits. Utilization ratios range from 2.24 to 2.85 across all scenarios, confirming that SEJ requirement is insensitive to parameter uncertainties.

This research provides the first comprehensive parametric study combining analytical and numerical methods specifically for unballasted steel through girder bridges under heavy-haul DFC loading. The quantification of end rotation effects ($\theta H = 2.88$ mm) and demonstration of their dominant role in generating critical stresses addresses a significant gap in existing literature. Previous RSI studies focused primarily on concrete box girders or continuous span structures, leaving this specific bridge typology inadequately characterized. The validated three-tier decision framework (displacement check $\rightarrow$ stress check $\rightarrow$ operational assessment) provides a systematic approach for engineers evaluating track configuration on similar structures. The methodology integrates UIC774-3(R) analytical procedures with three-dimensional finite element modeling, offering both computational efficiency and detailed stress characterization. For unballasted steel through girder bridges with spans approaching 80 m under DFC loading conditions (32.5t axle loads), SEJ installation at support locations is necessary to manage thermal expansion and end rotation effects. These findings provide practical design guidance for Indian Railway infrastructure projects and contribute to understanding of track-bridge interaction in heavy-haul systems globally.

References

- [1] Neridu S, Pasupuleti VD, Kalapatapu P. Analysis of rail structure interaction in continuous welded rails on railway bridges: a parametric analysis. *Innovative Infrastructure Solutions*. 2024 Oct;9(10):367. <https://doi.org/10.1007/s41062-024-01687-z>
- [2] Ranasinghe AP, Loizias M, Al-Khateeb HT, Greenburg S. Analysis and design of a concrete network tied arch bridge for California High Speed Rail Project. In *Bridge Safety, Maintenance, Management, Lifecycle, Resilience and Sustainability* 2022 Jun 27 (pp. 1947-1953). CRC Press.
- [3] Dhulipala S, Patil GR. Shippers/freight forwarders' acceptance of dedicated rail freight corridors for freight mobility in India. *Journal of Rail Transport Planning & Management*. 2024 Dec 1; 32:100481. <https://doi.org/10.1016/j.jrtpm.2024.100481>
- [4] Aloisio A, Rosso MM, Alaggio R. Experimental and analytical investigation into the effect of ballasted track on the dynamic response of railway bridges under moving loads. *Journal of bridge engineering*. 2022 Oct 1;27(10):04022085. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0001934](https://doi.org/10.1061/(ASCE)BE.1943-5592.0001934)

- [5] Fedorova M, Sivaselvan MV, Kurc O, Karakaplan A. Rail-structure interaction and vehicle-track-structure interaction level 1 and 2 analyses. *Bridge Structures*. 2023 Jul 27;19(3):91-101. <https://doi.org/10.3233/BRS-230213>
- [6] Asif, K. A., Raneesh, K. Y., & Sisir, P. (2016). Strength problems associated with track bridge interaction in presence of continuously welded rail. *International Journal of Research Science & Management*, 3(8), 14-21.
- [7] Skarova A, Harkness J, Keillor M, Milne D, Powrie W. Review of factors affecting stress-free temperature in the continuous welded rail track. *Energy Reports*. 2022 Nov 1;8:107-13. <https://doi.org/10.1016/j.egy.2022.11.151>
- [8] Gedney BL, Rizos DC. Combining welding-induced residual stress with thermal and mechanical stress in continuous welded rail. *Results in Engineering*. 2022 Dec 1;16:100777. <https://doi.org/10.1016/j.rineng.2022.100777>
- [9] Zhai W, Han Z, Chen Z, Ling L, Zhu S. Train-track-bridge dynamic interaction: a state-of-the-art review. *Vehicle System Dynamics*. 2019 Jul 3;57(7):984-1027. <https://doi.org/10.1080/00423114.2019.1605085>
- [10] Kukulski J, Ratkiewicz A. Concept of thermal shrinkage-resistant railroad rail for use in continuous welded rail track. *Applied Sciences*. 2024 Jul 16;14(14):6172. <https://doi.org/10.3390/app14146172>
- [11] Luo J, Zhu S, Zeng Z, Zhai W. On stress and deformation accumulation of continuously welded rails under cyclic thermal loading in high-speed railways. *Engineering Structures*. 2023 Jan 15;275:115225. <https://doi.org/10.1016/j.rineng.2022.100777>
- [12] Ezsias L, Brautigam A, Szurke SK, Szalai S, Fischer S. Sustainability in railways—a review. *Chemical Engineering Transactions*. 2023 Dec 30;107:7-12. <https://doi.org/10.3303/CET23107002>
- [13] Abdulla W, Menzemer C. Fatigue life prediction of steel bridge connections using fracture mechanics models. *Research on Engineering Structures and Materials*. 2021;7(4):579-93. <http://dx.doi.org/10.17515/resm2021.264st0310>
- [14] Jha SK, Bambole A, Jadhav D, Dongre A, Goldar D. Optimum and efficient design of steel foot over bridges. <http://dx.doi.org/10.17515/resm2024.167st0129rs>
- [15] Shah BJ, Surti SK. The Need for Rail Structure Interaction Analysis for Bridges: A State-of-the-Art Review. *IUP Journal of Structural Engineering*. 2015 Jul 1;8(3).
- [16] Goicolea Ruigómez JM, Antolin Sanchez P. Dynamics of high-speed railway bridges: Review of design issues and new research for lateral dynamics.
- [17] Kumar R, Upadhyay A. Effect of temperature gradient on track-bridge interaction. *Interact Multiscale Mech*. 2012 Jan;5(1):1-2. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000281](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000281)
- [18] Pugasap K. Dynamic responses of bridge substructures subjected to high-speed trains. In *Proceedings of the Institution of Civil Engineers-Bridge Engineering 2020 Sep* (Vol. 173, No. 3, pp. 143-157). Thomas Telford Ltd. <https://doi.org/10.1680/jbren.19.00046>
- [19] Gupta A, Ahuja AS. Dynamic analysis of railway bridges under high speed trains. *Universal Journal of Mechanical Engineering*. 2014;2(6):199-204.
- [20] UIC 774-3 - Part 1 : Rail Structure Interaction Verification.
- [21] Indian Railways. IRS concrete bridge code: Code of practice for plain, reinforced & prestressed concrete for general bridge construction. Research Designs and Standards Organization. 1997.
- [22] Indian Railways. IRS bridge rules: Rules specifying the loads for design of super structure and sub structure of bridges and for assessment of the strength of existing bridge. Research Designs and Standards Organization. 2008.
- [23] Indian Railways. Sub structures & foundation code: Code of practice for the design of sub structures and foundations of bridges. Research Designs and Standards Organization. 2013.
- [24] Indian Railways. IRS steel bridge code: Indian railway standard code of practice for the design of steel or wrought iron bridges carrying rail, road, or pedestal traffic. Research Designs and Standards Organization. 2019.
- [25] Research Designs and Standards Organization (RDSO). Guidelines on seismic design of railway bridges (Version 1). Ministry of Railways, India. 2015.
- [26] Research Designs and Standards Organization (RDSO). Guidelines for carrying out rail structure interaction studies on metro systems (Version 2). Ministry of Railways, India. 2015.

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Shear behavior of RC beams without stirrups made of concrete core and external prefabricated U-shaped UHPFRC plates

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Abstract

This study investigates the shear behavior of innovative T-shaped composite reinforced concrete (RC) beams without stirrups. The novelty of this study lies in investigating the shear behavior of stirrup-free, T-shaped composite beams that use an encased UHPFRC plate configuration as integrated permanent formwork. Six T-beams were tested: one RC reference beam and five composite beams with 30mm thick UHPFRC plates. Variables included interfacial properties (smooth, rough, anchored) and plate configurations (terminating at flange bottom or embedded within). One beam used cellular lightweight concrete (CLC) blocks to assess core concrete type. Four-point loading tests revealed that the UHPFRC plates significantly enhanced structural performance, increasing shear strength by up to 110.4% and deformation capacity 381.3% compared to a conventional RC beam. Critically, embedding the UHPFRC plates into the concrete flange with a rough interface proved to be the most effective configuration. This approach transformed the failure mode from brittle shear to ductile flexure and prevented premature debonding. This composite system presents a promising approach for enhancing the structural performance and durability of RC beams.

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1. Introduction

Concrete has been a cornerstone of construction for centuries and remains one of the most widely used materials globally, with an estimated annual consumption of approximately 11 billion tons [1]. Apart from water, no other material is consumed in greater quantities. Today, the construction industry faces a critical challenge: reducing its environmental impact while fostering the development of sustainable building practices. Enhancing the sustainability of reinforced concrete (RC) structures can be achieved by extending their service life in both new and existing applications. Ultra-high-performance fiber-reinforced concrete (UHPFRC) is particularly suited for this purpose due to its exceptional properties, including high tensile and compressive strength, superior durability, and excellent ductility. These qualities make it a dependable option for enhancing structural durability and robustness. The exceptional properties of UHPFRC are derived from its densely compact matrix, characterized by minimal porosity, and its reinforcement with steel microfibers. These features offer significant advantages over Concrete with normal weight (NWC) and Concrete with light weight (LWC). However, the high cost of producing UHPFRC—both economically and environmentally—limit its feasibility for large-scale applications, posing a challenge to its widespread adoption [2–5].

To address these limitations while leveraging the benefits of UHPFRC, this study proposes an innovative composite system. The approach strategically confines the UHPFRC to a thin,

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prefabricated U-shaped outer shell that acts as permanent formwork for a more economical core material, such as normal weight concrete (NWC) or lightweight concrete (LWC). This method not only minimizes the required volume of costly UHPFRC but also utilizes its superior properties to enhance the overall structural performance and durability of the beams. These composite beams offer several advantages, including simplifying construction, reducing structural dead weight, and enabling longer spans or shallower beam depths. They also exhibit significantly higher shear capacity and ductility compared to conventional RC beams of the same dimensions.

Previous research has examined the UHPFRC-NWC interface, focusing on factors like surface roughness and the use of lightweight concrete cores to reduce dead weight [7-16]. However, these studies have also highlighted a critical issue: the separation of the outer UHPFRC layer from the concrete core, particularly when using lower-strength concrete. This debonding can limit the ultimate capacity and prevent the system from achieving its full potential.

While previous research has successfully explored UHPFRC shells for rectangular composite beams, the behavior of T-shaped sections without shear stirrups—a common configuration in practice—remains under-investigated. The complex geometry of T-beams introduces a critical challenge at the web-flange junction, where stress concentrations can lead to premature debonding of the UHPFRC shell from the concrete core. Furthermore, the effectiveness of fully encasing the UHPFRC plates within the flange to prevent this delamination has not been thoroughly examined. This study directly addresses these gaps by asking the following scientific questions: (1) How does the UHPFRC-NWC interface condition (smooth, rough, anchored) affect shear capacity and failure mode? and (2) Can embedding the UHPFRC shell into the flange transform the failure from brittle shear to ductile flexure by preventing interface delamination?

2. Material Properties

2.1. Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC)

In this study, the UHPFRC material's homogeneity and density were enhanced by exclusively using fine particles. To make a well-balanced mixture, dry silica fume, quartz, and cement were combined with sand that had a maximum particle size of 500 μm . To keep the mixture workable, Sika viscocrete superplasticizer (S.P.) and a low water/cement ratio were also used. Steel fibers with end hooks, comprising 2% of the mixture by volume, were added. These fibers had lengths of 13 mm and 20 mm and a fine diameter of 0.2 mm (see Fig. 1).

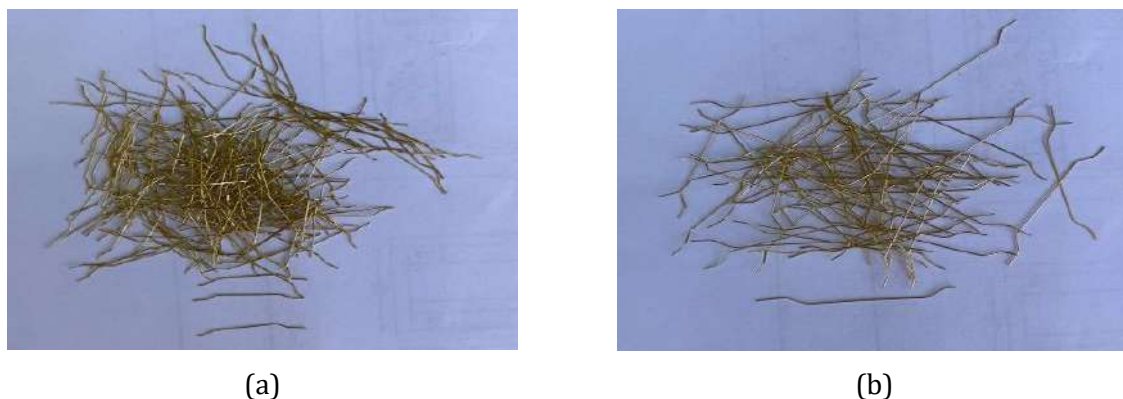


Fig. 1. Steel fibers used in UHPFRC mixture: (a) hooked steel fiber (13 mm); (b) hooked steel fiber (20 mm)

They possess an impressive tensile strength of 2850 MPa and high elasticity modulus of 200 GPa, making them highly effective for reinforcing the matrix and enhancing its mechanical properties. Type I Portland cement (52.5 R) was chosen for the mixture. The ingredients are detailed in Table 1. The material preparation involved a stepwise mixing process: After four minutes of mixing all the dry materials, water and superplasticizer were gradually added. The addition of steel fibers occurred once the mixture achieved a uniform wet consistency. The specimens were cast, demolded 48 hours later, and then cured for 26 days in a water tank. Twenty-eight days after

casting, the specimens were tested. To determine the flexural properties of the UHPFRC, three prism specimens measuring $40 \times 40 \times 160 \text{ mm}^3$ were tested according to ASTM C1609/C1609M-19a - Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading) [26].

Table 1. Mix proportion of UHPFRC

Unit weight (kg/m^3)							
Cement	Sand	Quartz	Silica fume	Water	Superplasticizer (S.P)	Steel fiber (13 mm)	Steel fiber (20 mm)
850	1005	250	150	180	55	112	45

The prisms were simply supported over a clear span of 120 mm and subjected to load through two rollers spaced 40 mm apart. A constant displacement rate of 0.1 mm/min was applied until failure. This standard recommends these dimensions for testing fiber-reinforced concrete, and the chosen size ensures that the smallest cross-sectional dimension (40 mm) is more than twice the length of the longest steel fibers (20 mm), thus minimizing potential size effects on the measured flexural properties (ACI Committee 544, 2018) [29]. Compression tests were subsequently performed on three $150 \times 150 \times 150 \text{ mm}^3$ cube specimens following BS EN 12390-3:2019 - Testing hardened concrete — Part 3: Compressive strength of test specimens [27]. While this European standard is specifically for cube specimens, the testing procedure also generally adhered to the principles outlined in ASTM C39/C39M-21 - Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens [28]. In this research, a universal testing equipment with a 500 kN capacity was used to conduct tests. The results of the compression and flexural testing are summarized in Table 2.

Table 2. Properties of UHPFRC

Material	Compression strength (MPa) at 28 days	Poisson's ratio	Elastic modulus (MPa)*10 ⁴	Tensile strength, f_{ct} (MPa)	Flexural strength (MPa)
UHPFRC	115 ± 1.8	0.22	4.7 ± 0.3	7.4 ± 0.4	20 ± 0.5

Note: compressive strength, tensile strength and flexural strength are mean value of experimental ones.

To investigate the direct tensile stress-strain behavior of the UHPFRC, three custom-cast, dog-bone shaped specimens were prepared. The specimen geometry and testing methodology were designed following the recommendations of RILEM TC 162-TDF [30]. This was done to ensure a state of pure uniaxial tension and prevent premature failure at the grips. The specimens had a central prismatic section of $100 \times 100 \text{ mm}^2$ where elongation was measured. These samples were created using the same mixture that was used to create the prefabricated UHPFRC plates for the composite beam manufacturing. The tests were performed under uniaxial tension using a 300 kN capacity universal testing machine. The specimens were secured using wedge-type mechanical grips designed to ensure failure occurred in the central gauge section. The load was applied monotonically at a displacement rate of 0.5 mm/min until failure. To accurately capture the tensile strain, two Linear Variable Differential Transformers (LVDTs) were mounted on opposite faces of the specimen over a gauge length of 100 mm. The nominal tensile stress was computed by dividing the measured load by the initial cross-sectional area of the specimen's central section ($100 \times 100 \text{ mm}^2$), while the strain was determined by dividing the measured elongation by the original gauge length. All measuring instruments, including the load cell and LVDTs, were calibrated according to manufacturer specifications prior to the testing program. Fig. 2 shows the average uniaxial tensile stress-strain behavior of the UHPFRC. The average tensile strength was 7.4 MPa, with a standard deviation of 0.4 MPa. The average strain at peak stress was 0.01, with a standard deviation of 0.0008. The UHPFRC exhibited a linear elastic response up to approximately 7.4 MPa, followed by gradual softening. The tensile rupture strain was slightly above 0.01, as illustrated in Fig. 2.

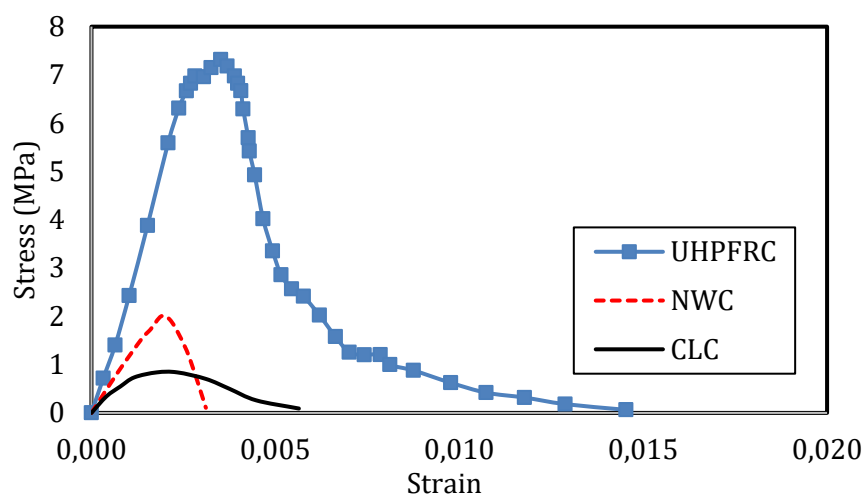


Fig. 2. Uniaxial tensile behavior of CLC, NWC and UHPFRC

2.2. Normal Weight Concrete (NWC)

Concrete with a target compressive strength of 30 MPa was used for the cores of the composite beams. The mix consisted of ordinary Portland cement (Type I), coarse aggregate with a maximum size of 10 mm, fine aggregate, and water. The proportions and characteristics of the concrete mix are summarized in Tables 3 and 4.

Table 3. Mix proportion of normal weight concrete

water-to-cement ratio. (w/c) %	Unit weight (kg/m ³)			
	Water	Cement	Fine aggregate	Coarse aggregate (≤ 10 mm)
50	190	380	660	1173

Table 4. Properties of NWC and CLC

Material	Density (kg/m ³)	Compressive strength (MPa)	Elastic modulus (MPa)*10 ⁴	Tensile strength, f_{ct} (MPa)	Poisson's ratio
NWC	2500	32 ± 0.8	2.45 ± 0.2	2.0 ± 0.3	0.20
CLC	700	2.12 ± 0.15	0.21 ± 0.02	0.47 ± 0.06	0.25

Note: compressive strength, tensile strength and elastic modulus are mean value of experimental ones.

To determine the compressive strength of the NWC, three 150 × 150 × 150 mm³ cube specimens were tested according to BS EN 12390-3:2019 [27]. The testing procedure also generally adhered to the principles outlined in ASTM C39/C39M-21 [28]. The tests were performed using a universal testing machine with a capacity of 500 kN. The load was applied monotonically at a rate of 0.6 MPa/s until failure. This loading rate is within the range specified by BS EN 12390-3:2019 [27]. The average compressive strength of the NWC at 28 days was determined to be 32 MPa, with a standard deviation of 0.8 MPa (Table 4). To characterize the tensile behavior of the NWC, direct tension tests were performed on three dog-bone shaped specimens prepared from the same concrete batch as the beam cores. The testing procedure was consistent with that used for the UHPFRC specimens, guided by the principles of RILEM TC 162-TDF [30]. The tests were conducted using the same universal testing machine and instrumentation setup, and the specimens were secured with wedge-type mechanical grips. The load was applied monotonically at a constant displacement rate of 0.5 mm/min until failure. Elongation was measured over a gauge length of 100 mm using two LVDTs. As with the UHPFRC tests, the nominal tensile stress was calculated by dividing the measured load by the initial cross-sectional area, and the equipment was calibrated prior to use. Fig. 2 shows the average uniaxial tensile stress-strain curve for the NWC. The average tensile

strength was 2.0 MPa, with a standard deviation of 0.3 MPa. The average strain at peak stress was 0.003, with a standard deviation of 0.0004. The NWC exhibited a linear elastic response up to approximately 2.0 MPa, followed by softening, with a tensile rupture strain of approximately 0.003.

2.3. Cellular Lightweight Concrete (CLC) Blocks

For one of the beams, the core utilized cellular lightweight concrete (CLC) blocks supplied by the Delta Mix factory. According to the author's previous research [18], the properties of the CLC blocks are detailed in Table 4, providing key insights into their mechanical and physical characteristics. Additionally, Fig. 2 illustrates the direct tensile stress-strain behavior of the CLC material, highlighting its performance under tensile loads.

2.4. Steel Reinforcement

The beams were reinforced with deformed steel bars having diameters of 16 mm, 12 mm, and 10 mm for the longitudinal reinforcement. The concrete slab's ribbed bars were arranged in two layers: 155 mm center-to-center (10 mm in diameter) horizontally and 196 mm center-to-center (8 mm in diameter) vertically. It is crucial to note that these beams are designed to be 'without stirrups' in the shear spans. A minimal number of four 6 mm-diameter stirrups were installed only at the loading and support points, outside the critical shear regions. Their sole purpose was to maintain the geometry of the reinforcement cage during casting and they did not contribute to the shear capacity of the beams. The universal testing equipment conducted tension tests on the reinforcing steel bars to determine their yield and ultimate strengths. The tensile characteristics of the longitudinal reinforcement used in this research are detailed in Table 5.

Table 5. Measured properties of longitudinal reinforcement

Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)
Ø6, Ø8	243	352
Ø10, Ø 12, Ø 16	498	657

3. Experimental Program

3.1. Design of beams

Six T-beams in total were fabricated and tested, including one reinforced concrete (RC) beam as a reference and five beams utilizing prefabricated U-shaped UHPFRC as a permanent formwork with a thickness of 30 mm. These beams featured different interfacial properties: smooth (S), rough (R), and smooth with anchored shear connectors (SC). Additionally, two configurations were applied to the UHPFRC plates: one that stopped at the bottom of the concrete flange (Fig. 3a) and another that was encased directly into the concrete flange. In the encased configuration, composite action was achieved via transverse reinforcement bars passing through holes in the UHPFRC formwork (Fig. 3b). This created a robust mechanical connection. To assess the effect of core concrete type on shear behavior, one beam replaced normal weight concrete (NWC) with cellular lightweight concrete (CLC) blocks. The reference RC beam was labeled S0, and the composite beams were named according to their interfacial properties, such as S2-R for a composite beam with a rough interface. The beam using CLC blocks instead of NWC was designated S5-HB. T-shaped, simply supported beams with an effective span (L_0) of 1800 mm and a ratio of shear span-to-depth (λ) of 1.8. This specific shear span-to-depth ratio ($\lambda=1.8$) was selected to ensure the specimens behave as deep beams, where the load transfer mechanism is dominated by arch action rather than beam action. This represents a critical condition for shear behavior in discontinuous regions where stirrups are often omitted or difficult to detail [35]. The lower part width (b) of 150 mm, a concrete slab width (B) of 500 mm, a slab thickness (t_b) of 100 mm, and a U-shaped UHPFRC plate thickness (t_w) of 30 mm were used for all specimens. The concrete slab's ribbed reinforcement had a 20 mm protective layer thickness, and the beam depth (H) was 350 mm. For longitudinal reinforcement on the tension side, all beams, except for the S5-HB beam, were reinforced with two bars with 16 mm diameter and two bars with 12 mm diameter in the NWC core.



Fig. 3. Cross section of the composite beam with NWC core and UHPFRC cover, showing two plate configurations: (a) plate terminating at the bottom of the flange; (b) plate encased directly into the flange

The S5-HB beam, however, utilized four 12 mm diameter bars and two 10 mm diameter bars in the soffit of the UHPFRC plate. Two bars of 10 mm diameter compression steel were attached to the top of all beams. For the concrete slab, two layers of ribbed bars were utilized, spaced 155 mm center-to-center (10 mm in diameter) horizontally and 196 mm center-to-center (8 mm in diameter) vertically. Finally, simultaneous casting was used to integrate the slab with the concrete inside the prefabricated U-shaped UHPFRC plate. Fig. 4 provides the dimensional details of all beams, while Table 6 lists the specific details of each specimen.

Table 6. Summary of beams

Beam	Matrix	Interfacial properties	tb (mm)	Length (mm)	Width (mm)	Depth (mm)	Longitudinal bars	
							As (mm ²)	μ (%)
S0	NWC	--						
S1-R	UHPFRC-NWC	Rough						
S2-R	UHPFRC-NWC	Rough						
S3-S	UHPFRC-NWC	Smooth						
S4-SC	UHPFRC-NWC	Smooth with connector	100	2000	150	350	628	1.2
		s						
S5-HB	UHPFRC-CLC	Smooth					609	1.2

Note: -- means no data, μ represents the longitudinal tensile reinforcement ratio.

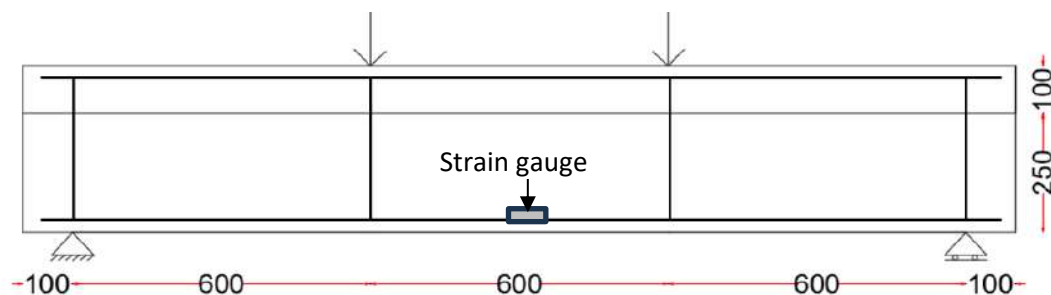


Fig. 4. Dimensions details of all beams

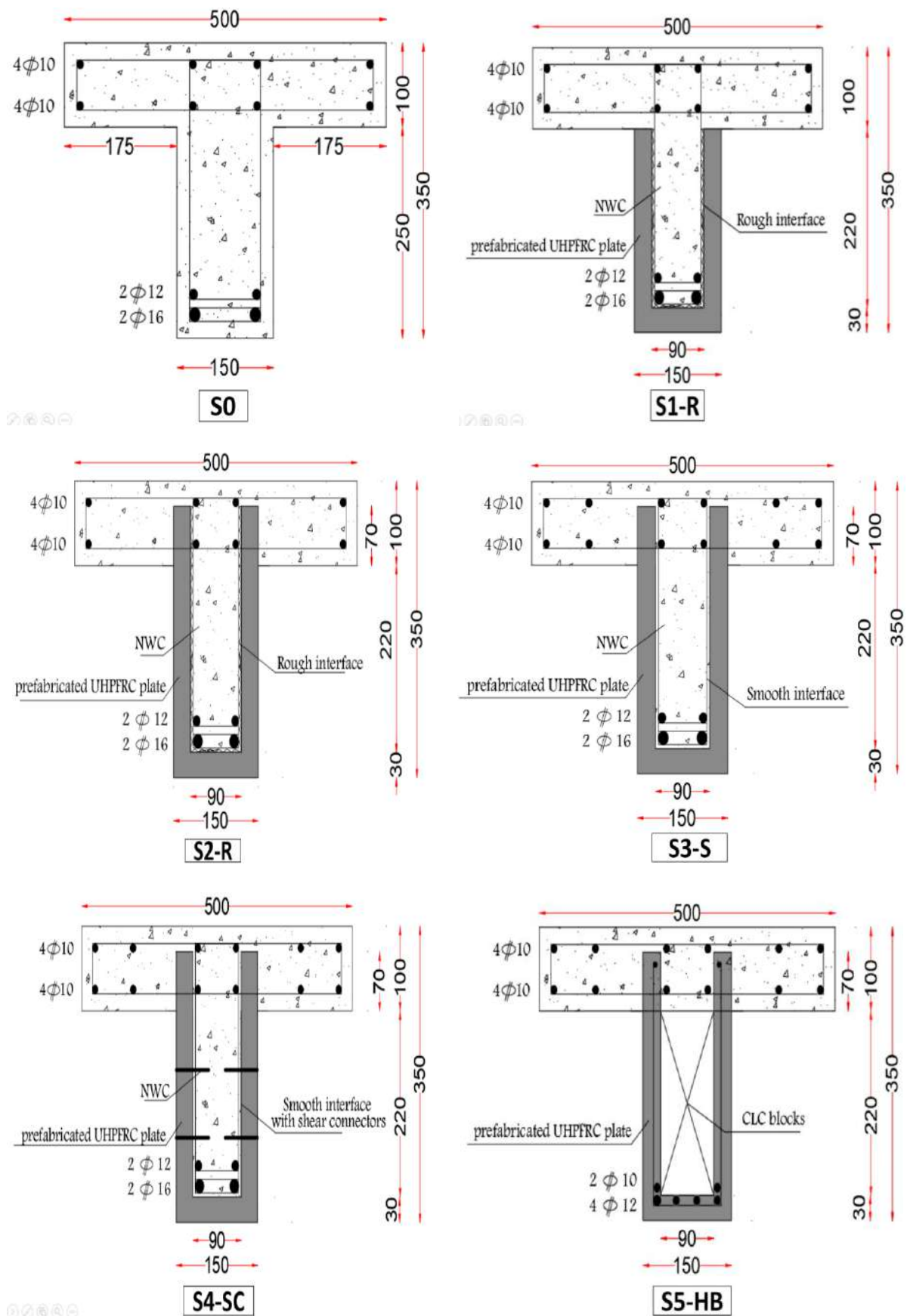


Fig. 5. Cross section and reinforcements of all beams

To monitor the yielding of the reinforcing bars, a strain gauge was attached to the center of the bars on the tension side. The configuration of the beams and the reinforcement positioning within the

concrete and UHPFRC components are clearly depicted by the cross-section and reinforcement details for each beam in Fig. 5. Two rows of steel anchoring bars measuring 60 mm in length and 8 mm in diameter were inserted into the prefabricated U-shaped UHPFRC plate for the S4-SC beam, as shown in Fig. 6. These bars were spaced 86.5 mm apart in the longitudinal direction. The authors acknowledge that the limited number of six specimens restricts statistical generalization. However, the results provide clear and consistent trends for the investigated parameters.

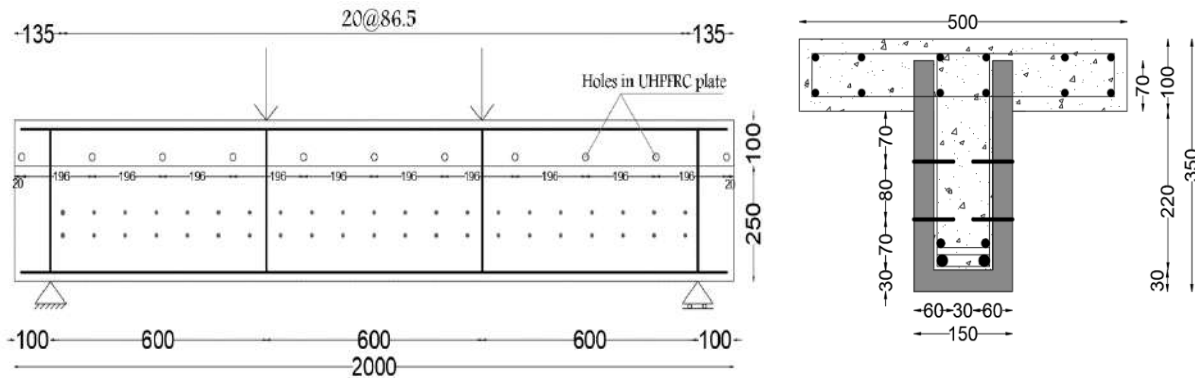


Fig. 6. Arrangement of anchorage steel bars in S4-SC beam

3.2. Beam Manufacturing

A UHPFRC plate thickness of 30 mm was selected as it provides sufficient cover for the steel microfibers, ensures robust handling during fabrication, and aligns with thicknesses used in similar prefabrication studies. The prefabricated U-shaped UHPFRC plates were precast utilizing an interior foamed plastic cubic and an exterior film-faced plywood formwork. The foamed plastic cuboid's surface could be changed to create various interfaces between the concrete and the UHPFRC plate. Three interface conditions were investigated: smooth, rough, and smooth with anchored shear connectors (SC). For the smooth interface (used in specimens S3-S), the UHPFRC was cast directly against the smooth surface of the foamed plastic cuboid.

This resulted in a surface with minimal texture and irregularities. For the rough interface (used in specimens S1-R and S2-R), the surface of the foamed plastic cuboid was mechanically abraded prior to casting the UHPFRC. This abrasion was performed using a wire brush, to create a surface with noticeable indentations and protrusions. The resulting rough interface was characterized by a highly irregular surface with numerous small indentations and protrusions, and a coarse, abrasive texture (see Fig. 7). It is acknowledged that this characterization is qualitative, and future work should incorporate quantitative roughness measurements (e.g., laser profilometry to determine metrics like R_a or R_z) to establish a more precise correlation between interface topography and structural performance.



(a)



(b)



(c)



(d)

Fig. 7. Prefabricated U-shaped UHPFRC plates with different interfacial properties: (a) smooth interface; (b) rough interface; (c) smooth interface with shear connectors; (d) CLC blocks used for formwork

For the smooth interface with anchored shear connectors (used in S4-SC beam), a cuboid with steel anchoring bars evenly spaced was employed to create a smooth interface with these bars as shown in Fig. 8. In contrast, the S5-HB beam replaced the foamed plastic cuboid with CLC blocks, and the longitudinal reinforcement is moved to the external prefabricated UHPFRC plate. The process began with the setup of the external film-faced plywood formwork, followed by the placement of the foamed plastic cuboid inside to form a U-shaped cavity. Fresh UHPFRC was then poured into the cavity from the bottom to the top. Fresh UHPFRC was then poured into the cavity, filling it completely from the bottom upwards.



Fig. 8. Prepared inner and external formwork for S4-SC beam



Fig. 9. Prepared inner and external formwork for all beams

Fig. 9 illustrates the external and internal formwork used for all the beams. The formwork was removed one day after casting. Fig. 7 shows the prefabricated U-shaped UHPFRC plates with various interfacial characteristics. Both ends of the prefabricated U-shaped UHPFRC plates were covered with film-faced plywood plates to provide a sealed cavity for the concrete before the steel cage and reinforcement for the concrete slab flange were installed. Once the formwork was secured, the steel cages and slab reinforcement were positioned at the designated locations within the prefabricated U-shaped UHPFRC plates and the flange plywood formwork, respectively. The casting process was then finished by pouring concrete into the prefabricated UHPFRC plate formwork. After 24 hours, the plywood plates at both ends and the flange formwork were removed. The U-shaped UHPFRC formwork and concrete were cast simultaneously. The beams were wrapped in sheets of plastic and sprayed with water every day for at least 28 days to guarantee adequate curing. The process flow for fabricating composite beams is shown in Figs. 10(a-h).

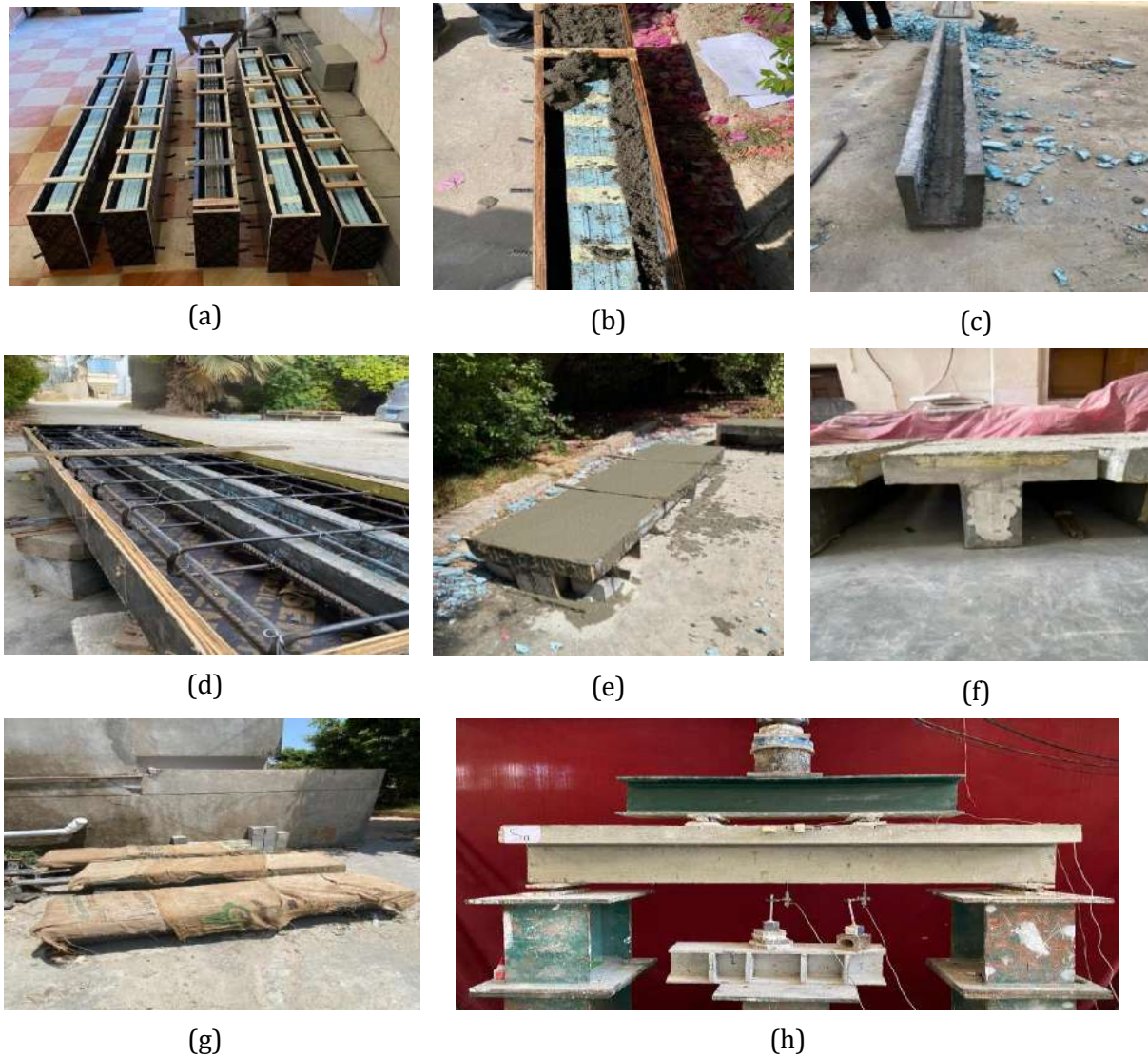


Fig. 10. Flow chart of the composite beam fabrication process: (a) preparing formwork; (b) casting UHPFRC plates; (c) demolishing formwork; (d) setting up steel reinforcement; (e) casting concrete core and flange; (f) demolishing flange formwork; (g) curing; (h) testing

3.3. Experimental Test

A 4-point load test was applied to all the beams, with the two loading points spaced 600 mm apart. To monitor the mid-span deflection, two LVDTs were employed, while another LVDT was used to measure the compression strain in the concrete flange. To monitor strain on each beam's side of tension, a strain gauge was attached to the center of just one longitudinal bar. The load was applied in increments of 10 kN, with each step held for 10 seconds. This quasi-static procedure was chosen to allow for careful visual inspection, marking of crack initiation and propagation, and to ensure stable data acquisition before failure.

This methodology is consistent with established guidelines for structural load testing and is a common approach for experimental studies of this nature [37,38]. Crack widths were measured using a handheld optical microscope with a magnification of 40X and a precision of 0.02 mm. These measurements were recorded at the level of the longitudinal reinforcement after each 10 kN load increment. Additionally, the full crack pattern was mapped by tracing the cracks with a marker pen on the surface of the beams to document the failure evolution. All measuring instruments, including the LVDTs and the load cell, were calibrated prior to testing. The LVDTs were calibrated using a micrometer to ensure an accuracy of ± 0.01 mm. The bearings for the load beneath the spreader beam were positioned across the entire width of the concrete flange, ensuring that the applied load

was shared by both the prefabricated U-shaped UHPFRC plates and the concrete beam. Fig. 11 represents the experimental setup.

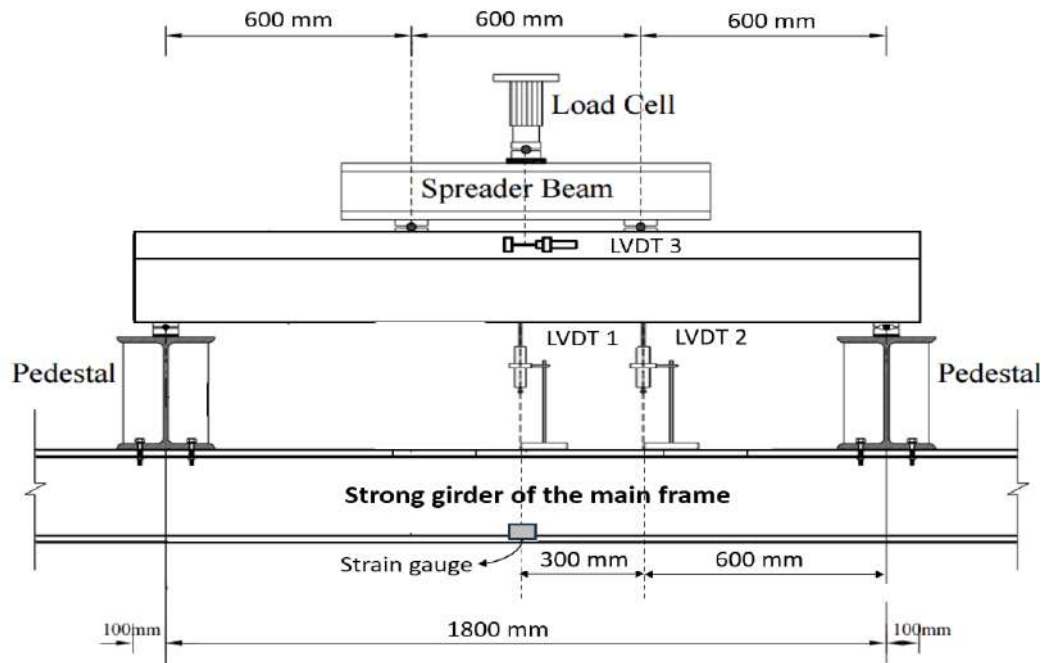


Fig. 11. Layout of the test setup

4. Results and Discussion

4.1. Cracking Patterns and Modes of Failure

Fig. 12 depicts the crack patterns that appeared in the beams when they reached their peak load capacity. It is noteworthy that a large number of tiny cracks appeared in the prefabricated UHPFRC plates used in the composite beams. However, Fig. 12 only highlights the most significant cracks because of the short time available for crack monitoring during the loading testing. In all tested beams, flexural cracks first appeared in the area that was subjected to pure bending. More cracks formed throughout the shear spans as the load increased. In the reference beam S0, the load-carrying capacity dropped sharply, accompanied by a fracture sound, when an inclined crack formed through the shear span (see Fig. 12a). The remaining composite beams, on the other hand, failed considerably more gently and showed higher ductility. In contrast to the other composite beams, the S3-S composite beam experienced less inclined cracks with (see Fig. 12d). This suggests that the prefabricated U-shaped UHPFRC plate with a smooth interface in beam S3-S was less effective in working cooperatively with the concrete to carry the load because of weaker properties of the interfacial layer. More than two inclined cracks appeared in all composite beams, although the reference beam S0 only had one (see Fig. 12a). Significant separation at the interface beneath the flange occurred at peak load in the S1-R beam, as seen in Fig. 12b, while no separation was observed in the other composite beams. This indicates that encasing the UHPFRC plates directly into the concrete flange was highly effective. The transverse reinforcement bars passing through holes in the plates securely anchored the two materials, preventing any separation at the interface. The midpoint stress in the longitudinal bars of all beams (except S0 and S5-HB) reached yield. This suggests that the use of prefabricated U-shaped UHPFRC plates transformed the brittle shear failure into a more ductile failure. All beams failed due to shear, except for beam S2-R, which experienced ductile flexural failure. As shown in Fig. 12c, failure of the UHPFRC plates in S2-R beam occurred when the beam reached its full capacity. In the pure bending zone, vertical cracks were observed and in the shear span, inclined cracks formed. As a result, every material in the S2-R beam was used.



(a)



(b)



(c)



(d)



(e)

Fig. 12. Crack patterns at peak load for beams: (a) reference beam S0; (b) beam S1-R, showing interface separation; (c) beam S2-R; (d) beam S3-S; (e) beam S5-HB

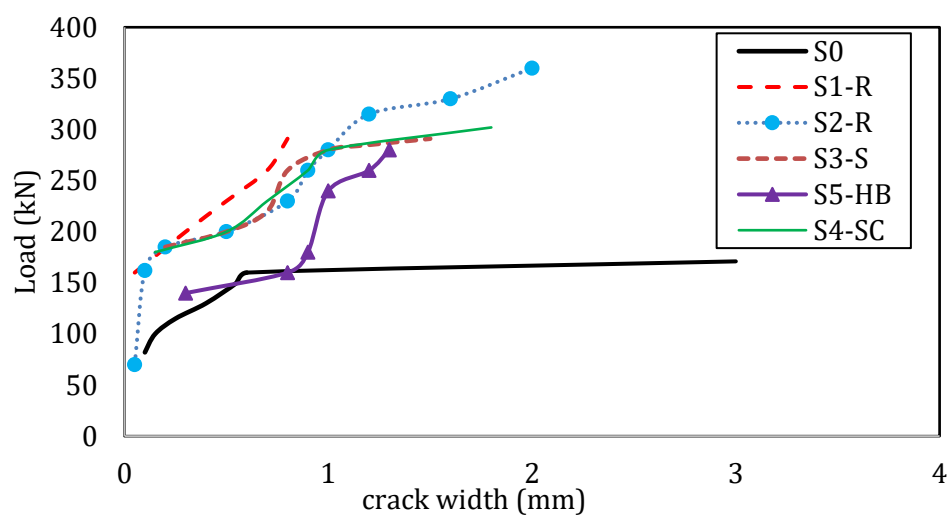


Fig. 13. Load versus major crack width for all tested beams

To provide a quantitative analysis of the cracking behavior, the width of the primary diagonal shear crack in each beam's shear span was measured and plotted against the applied load, as shown in Fig. 13. This figure clearly illustrates the fundamental difference in failure mechanics between the reference beam and the composite specimens. The reference beam (S0) exhibited a sharp increase in crack width after its formation, reaching over 2.5 mm at its peak load of approximately 171 kN. This behavior, characterized by the steep, short curve in Fig. 13, is indicative of poor crack control and confirms the brittle nature of its shear failure. In stark contrast, all composite beams demonstrated significantly improved crack control. For instance, at the same load level of 171 kN where S0 failed, the major crack width in the superior S2-R beam was only around 0.2 mm. The S2-R beam continued to sustain load up to 360.5 kN, with its crack opening in a slow and controlled manner. This highlights the effectiveness of the UHPFRC plates, particularly the embedded configuration (S2-R), in bridging cracks and ensuring a more ductile response by allowing for gradual stress redistribution. The behavior of the other composite beams also shows a marked improvement over the reference, though debonding in S1-R led to a slightly faster crack opening compared to the other composite beams before failure. These quantitative data strongly support the conclusion that the UHPFRC plates transform the failure mode from brittle shear to a more ductile mechanism governed by controlled crack growth.

4.2. Load-Deflection Graphs

Fig. 14 depicts the correlation between mid-span deflection and applied load for all beams. The mid-span deflection values were obtained from the LVDT placed at the beam's midpoint. All of the composite beams' peak loads were noticeably greater than those of the reference beam S0, attributed to the enhanced ductility of the UHPFRC. Additionally, all composite beam's stiffness exceeded that of S0 after the first crack, confirming that the fiber-bridging action helps UHPFRC structural members keep stiffness better than RC members.

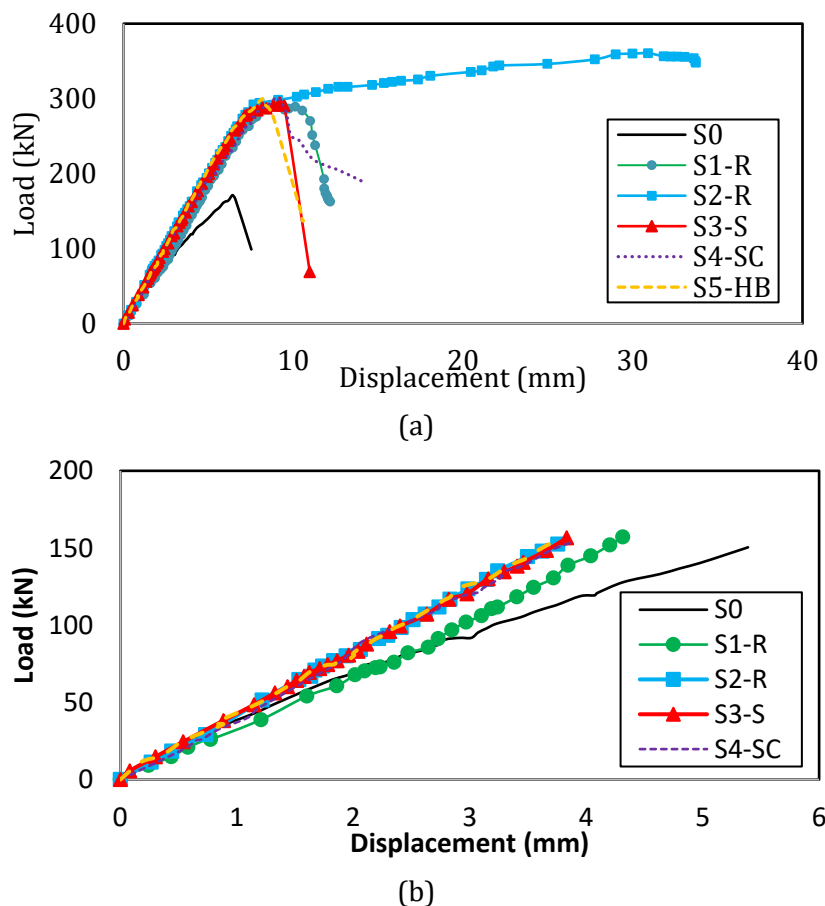


Fig. 14. Load vs. mid-span deflection curves for all tested beams: (a) the full response, showing ultimate capacity and overall ductility, and (b) a zoomed-in view of the initial load region, highlighting the differences in stiffness

Table 7 summarizes the key experimental results. Specimens S3-S, S4-SC and S5-HB exhibited comparable behavior. Specimen S3-S serves as the reference for describing the sequence of stages for load-deflection curve which divided into elastic, elastic-plastic, and descending stages. Together, the prefabricated UHPFRC plate, and encased concrete withstand shear forces that are transmitted from the loading point to the supports during the elastic stage, with load and mid-span deflection increasing linearly. In the middle of the span, at the bottom of the beam, the first vertical flexural crack developed at a load of 65 kN, indicated the transition to the elastic-plastic stage. During this stage, stiffness decreased continuously. The diagonal cracks started during the elastic-plastic phase and progressively propagated across the depth of the prefabricated UHPFRC plate. The encased concrete's contribution to shear capacity decreased once it reached its maximum shear strength, and more shear force was resisted by the prefabricated UHPFRC plate. The longitudinal reinforcement yielded at a load of 237 kN. The prefabricated UHPFRC plate's plastification stopped when a diagonal crack developed, and the crack became approached its compressive strength. With a mid-span deflection of 9.19 mm (L0/196), the specimen reached its maximum load of 294.5 kN. Following this, the load dropped sharply due to diagonal crack crushing, as depicted in Fig. 12d. The separation between the core concrete and UHPFRC plate in previous studies [10,11] were prevented by the UHPFRC plate's upper straight sides, which were directly encased in the concrete flange. This improved the integrity of the beam and behaved as a composite beam as it was designed.

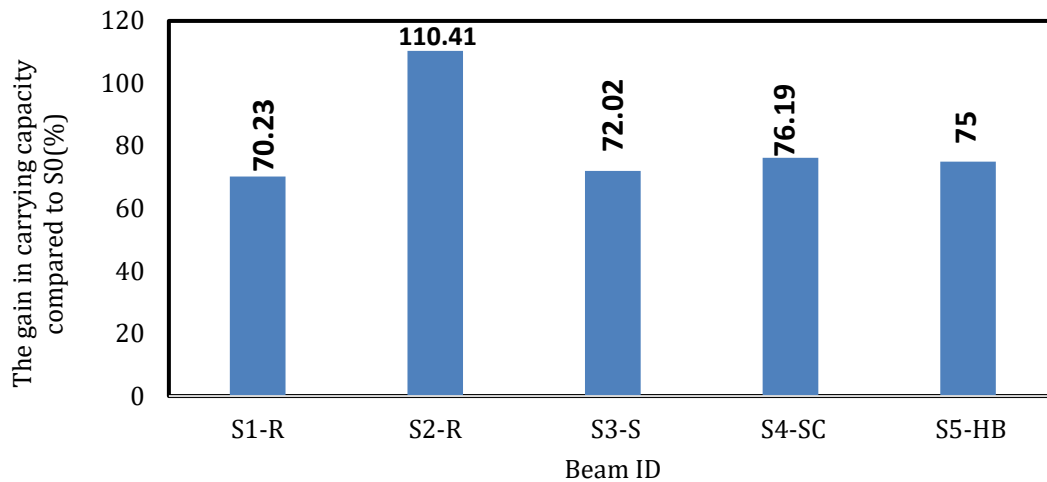
Table 7. Summary of experimental results for All Beams

Beam ID	P _{cr} (kN)	P _y (kN)	P _u (kN)	Gain in shear capacity (%)	Mid-span deflection at peak load (Δ) (mm)	Gain in deformation capacity (%)	Failure mode
S0	50	--	171.36	--	6.42	--	Shear failure
S1-R	65	--	291.72	70.23	9.36	45.8	Shear failure
S2-R	70	274	360.57	110.41	30.90	381.3	flexure failure
S3-S	65	237	294.78	72.02	9.19	43.1	Shear failure
S4-SC	66	238	301.92	76.19	9.02	40.5	Shear failure
S5-HB	40	--	299.88	75.0	8.18	27.4	Shear failure

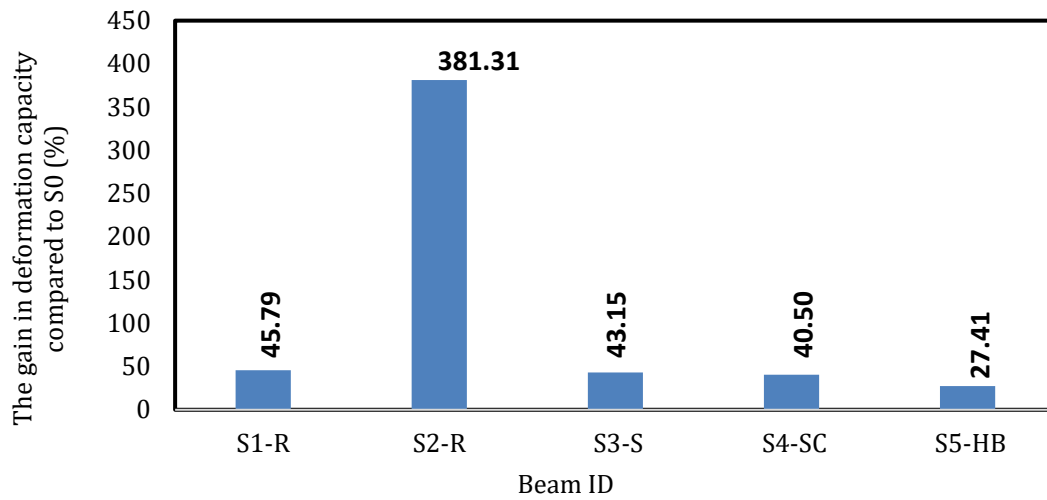
P_{cr} = Load at first visible crack; P_y = Load at yielding of longitudinal reinforcement; P_u = Ultimate load; -- indicates that the longitudinal reinforcement did not yield before failure. The percentage gain in shear capacity and deformation were calculated using the formula: Gain (%) = [(P_u or Δ_u, composite - P_u or Δ_u, S0) / P_u or Δ_u, S0] × 100.

Beam S1-R, featuring a rough interface and a UHPFRC plate terminating at the bottom of the concrete flange, exhibited a significantly different response compared to the reference beam, S0. The first flexural crack was observed at a load of 65 kN, while the first shear crack appeared at a load of 160 kN (see Fig. 14 for crack pattern). As the load increased, multiple inclined shear cracks developed in the shear span, extending from the loading point towards the support. The load-deflection curve (Fig. 13) showed an initially linear response, followed by a gradual decrease in stiffness as cracking progressed. A significant observation for beam S1-R was the initiation of progressive debonding at the UHPFRC-concrete interface beneath the flange as it approached its peak load of 291.72 kN. This separation, highlighted in Fig. 12c is a classic debonding failure. This can be explained by shear-friction theory, where the horizontal shear stress at the NWC-UHPFRC interface exceeded the shear resistance capacity. Without the mechanical anchorage provided by embedding the plate into the flange, the interface relied solely on adhesion and friction, which was insufficient to prevent slip and ultimately led to premature debonding. While the beam did not fail catastrophically, this debonding was the critical failure mechanism limiting its ultimate capacity. The stiffness retained after debonding began does not reflect a full bond. Instead, it is attributed to residual friction and partial aggregate interlock at the interface, which allowed for limited shear transfer until failure. The strain gauge on the longitudinal reinforcement did not reach the yield strain (2500 με) at peak load, indicating that the beam failed in shear before the reinforcement yielded. The final failure mode was characterized as a brittle shear failure with significant interface

debonding. The peak load was 70.23% higher, and the deformation was 45.79% than the reference beam S0.



(a)



(b)

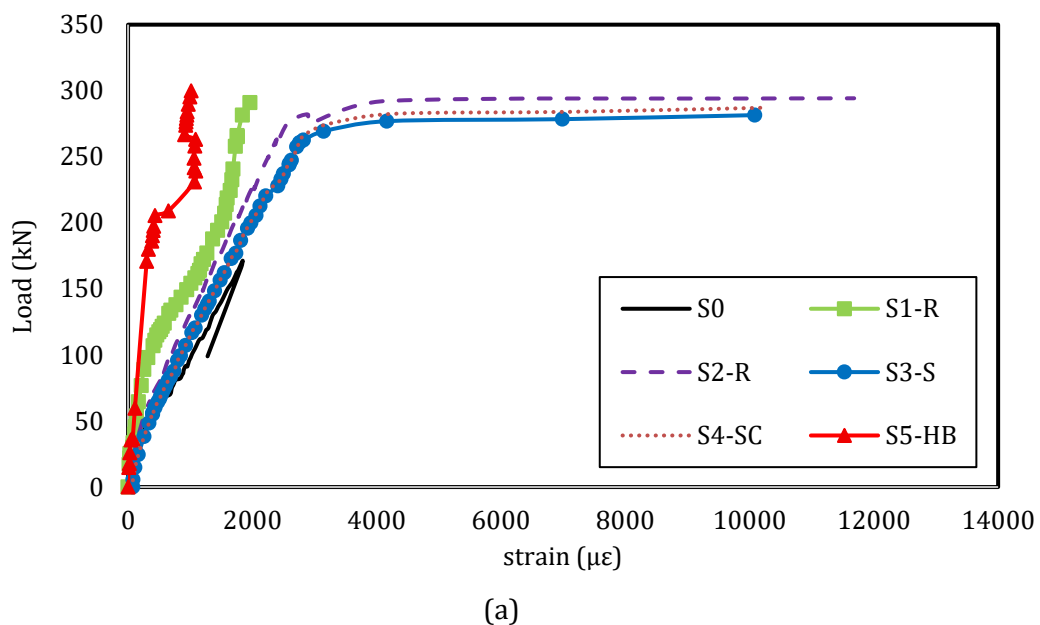
Fig. 15. Percentage gain in capacity compared to the reference beam S0: (a) carrying capacities; (b) deformation capacities

Specimen S2-R exhibited the largest mid-span deflection corresponding to the peak loads compared to the other composite beams due to its conversion from brittle shear failure to ductile flexural failure. The superior performance of S2-R is attributed to a combination of factors. The rough interface maximized the frictional and mechanical interlock, while embedding the plate into the flange provided a robust mechanical anchorage. This configuration effectively confined the concrete core, ensured composite action up to failure, and prevented the premature interface debonding seen in S1-R. The first flexural crack was observed at a load of 70 kN, followed by the first shear crack at 162 kN. As the load increased, multiple fine cracks developed within the UHPFRC plate in both the flexural and shear zones (see Fig. 12c). These cracks were more numerous and finer than those observed in the beams with terminating UHPFRC plates (e.g., S1-R). The load-deflection curve for Beam S2-R (Fig. 14) showed an initially linear response, followed by a gradual reduction in stiffness. Notably, this beam exhibited significant ductility after reaching its peak load of 360.57 kN. The strain gauge on the longitudinal reinforcement did reach the yield strain ($2500 \mu\epsilon$) at a load close to the peak load (see Fig. 16a), indicating that the reinforcement yielded prior to failure. Furthermore, the concrete strain measurements near the loading point on the upper slab surface exceeded $3000 \mu\epsilon$ (Fig. 16b), indicating crushing of the concrete in the

compression zone. Crucially, no separation was observed at the interface between the UHPFRC plate and the concrete core, even at peak load (see Fig. 12c). This contrasts sharply with the behavior of Beam S1-R, which exhibited significant interface debonding. The embedded configuration of the UHPFRC plate, combined with the rough interface, appears to have provided excellent composite action, preventing premature failure due to interface separation. The final failure mode of Beam S2-R was characterized as a ductile flexural failure, with yielding of the longitudinal reinforcement, crushing of the concrete in the compression zone, and extensive cracking of the UHPFRC plate. This indicates that all components of the composite beam (NWC core, UHPFRC plate, and steel reinforcement) were effectively utilized to resist the applied load. The peak load for Beam S2-R was 110.41% higher than that of the reference beam (S0) and its mid-span deflection was 381.3% larger, demonstrating a significant improvement in both strength and ductility. For successful field applications of this optimal design, strict quality control is essential to ensure consistent interface roughness and proper placement of the transverse anchoring bars. The shear-carrying and deformation capacity of each beam are compared with the reference beam S0 in Fig. 15. The composite beams showed an increase in shear-carrying capacity by 70.2% to 110.4% and in deformation capacity by 27.4% to 381.3%, regardless of the prefabricated UHPFRC plate configuration, concrete core type, or properties of the interfacial layer.

4.3. Strain Analysis

Fig. 16 a and b represent the load-strain curves for the longitudinal reinforcement on the tension side and the concrete slab, respectively. The longitudinal reinforcement's strain was measured at the cross-section at the mid-span of the beam. In the beams S2-R, S3-S, and S4-SC, the longitudinal reinforcement on the tension side reached its yield strain of $2500 \mu\epsilon$ prior to failure, with S2-R failing due to flexural failure and S3-S and S4-SC due to shear failure. This indicates that these beams effectively transformed the brittle shear failure into a more ductile failure mode which confirmed by the strain gauge data (Fig. 16a). The longitudinal reinforcement strain in beams S0, S1-R, and S5-HB remained well below yield, indicating a brittle shear failure occurred first. At the mid-span, concrete strain measurements were obtained from the slab's upper surface, close to the loading point. Near the loading plate, the concrete on the upper slab surface surpassed the $3000 \mu\epsilon$ crushing strain in beams S0 that failed due to brittle shear failure and S2-R, suggesting that the materials in beam S2-R were fully utilized. Therefore, the UHPFRC plate configuration used in S2-R is recommended.



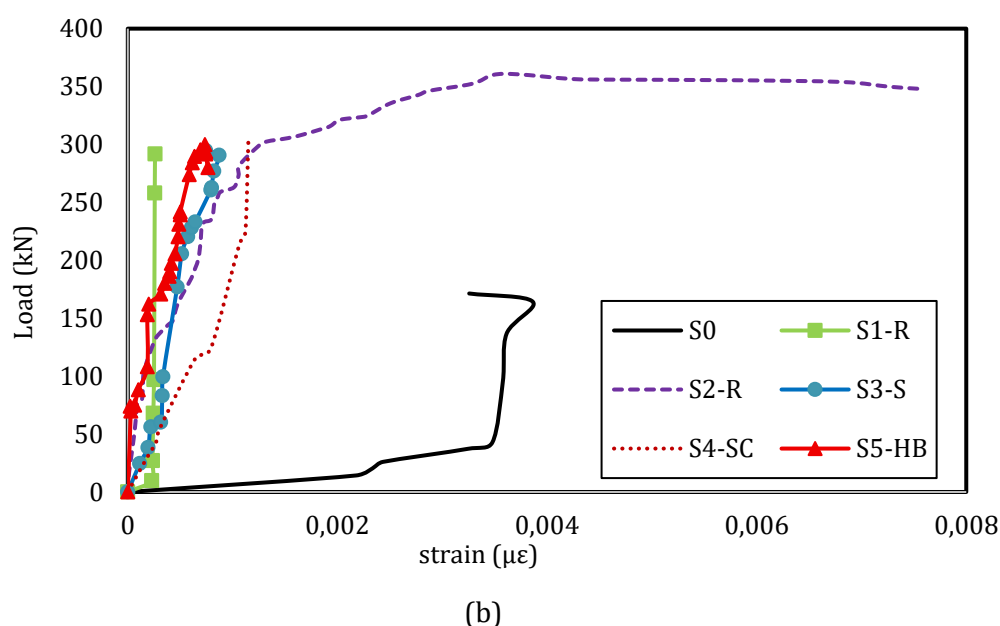


Fig. 16. Load-strain curves for concrete and steel reinforcement: (a) longitudinal reinforcement strain at mid-span; (b) concrete slab compressive strain near the loading point

4.4. Effects of the Prefabricated UHPFRC Plate Configuration

The effect of the prefabricated U-shaped UHPFRC plate configuration can be assessed by comparing the specimens' load vs mid-span deflection curves for S1-R and S2-R, as shown in Fig. 13. In the S1-R composite beam, the prefabricated UHPFRC plate was terminated at the bottom of the concrete flange (Fig. 3a), whereas in the S2-R composite beam, the plate was encased directly into the concrete flange. This encased configuration achieved composite action through perfect bond connections with reinforcement bars passing transversely through holes in the encased top sections of the UHPFRC plate (Fig. 3b). A significant detachment at the interface beneath the flange occurred at peak load in the S1-R beam, while no detachment was observed in the S2-R or other composite beams as shown in Fig. 12b,c. This indicates that encasing the UHPFRC plates directly into the concrete flange, secured by reinforcement bars, effectively prevented any separation between the two surfaces until the complete failure of the composite beam. Additionally, the S2-R composite beam failed due to ductile flexural failure and rupture of the UHPFRC plates, indicating that all materials in the S2-R beam were fully utilized, whereas the S1-R composite beam failed due to shear with significant detachment at the interface beneath the flange occurred at peak load. The load-carrying and deformation capacities of S1-R and S2-R increased by 70.23% and 110.41%, and 45.79% and 381.31%, respectively, compared to the reference beam S0.

4.5. Effects of Interfacial Properties

By comparing the load vs mid-span deflection curves of specimens S2-R, S3-S, and S4-SC, as illustrated in Fig. 13, the effect of interfacial properties can be evaluated. The interfacial properties (rough, smooth, and smooth with steel connectors) significantly influenced shear-carrying capacities, as depicted in Fig. 14(a). The S2-R beam, which featured a rough interface, presented the greatest peak load and mid-span deflection, owing to its transition from brittle shear failure to ductile flexural failure. The superior performance of the S2-R beam can be attributed to the activation of multiple, redundant shear-transfer mechanisms. While the rough interface provided a baseline of shear resistance through friction and mechanical interlock, its primary contribution was likely in ensuring initial composite action. The critical component was the embedment of the UHPFRC plates into the flange, secured by the transverse reinforcement. These transverse bars acted as dowel connectors, providing a robust mechanical anchor that physically prevented vertical and horizontal slip at the interface. This mechanical anchorage carried the majority of the interfacial shear stress once micro-cracking began, preventing the progressive debonding observed in the S1-R beam. This robust connection maintained the integrity of the composite

section, allowing for significant stress redistribution. It forced the failure to occur within the materials themselves—yielding of the steel and crushing of the concrete—rather than at the weaker interface, thus enabling the transition from a brittle interface failure to a ductile flexural failure. The S3-S and S4-SC beams, with smooth and smooth with steel connectors interfaces, respectively, behaved similarly, indicating that, in comparison to the smooth interface, the steel connectors inserted in the UHPFRC plate did not considerably improve the interfacial bond. The load-carrying capacities of S2-R, S3-S, and S4-SC increased by 110.41%, 72.02%, and 76.19%, respectively, compared to the reference beam S0. As shown in Fig. 16(b), the deformation capacities at the peak load increased by 381.31%, 43.15%, and 40.50% for the respective beams. The smaller mid-span deflection in the S4-SC beam is attributed to the shear transfer mechanism facilitated by the steel connectors.

4.6. Effects of Core Concrete Type

To evaluate the impact of core concrete type on shear behavior, one beam (S5-HB) was constructed using cellular lightweight concrete (CLC) blocks instead of normal weight concrete (NWC). The S5-HB beam exhibited similar behavior to the composite beams S3-S and S4-SC, with approximately 34% less ductility, but the core weight was reduced by 72%. The reduced ductility of the S5-HB beam can be attributed to the significant stiffness mismatch between the low-modulus CLC core ($E \approx 2.1$ GPa) and the high-modulus UHPFRC shell ($E \approx 47$ GPa). This incompatibility likely led to strain concentrations at the interface, causing earlier micro-cracking and a more brittle failure of the core material compared to the NWC core beams. The shear and deformation capacities of S5-HB increased by 75% and 27.41%, respectively, compared to the reference beam S0. Overall, using CLC blocks instead of NWC for the core proves to be an effective, economical, and appropriate approach when reducing the structure's dead weight or thermal and sound insulation is a priority, but designers must account for the reduced shear capacity. However, it is important to note that the S5-HB specimen differed from the other beams in both its core material and its longitudinal reinforcement configuration. Therefore, the observed behavior reflects the combined effect of these changes, and the influence of the CLC core alone cannot be fully isolated in this study. This limitation should be considered when interpreting these results. Future research should investigate composite beams with CLC cores using an identical reinforcement layout to the NWC reference beam to directly quantify the effect of the core material. The choice of core material should be based on the specific design requirements and the desired balance between strength, stiffness, and weight.

4.7 Broader Implications and Practical Context

From a sustainability perspective, the composite approach offers significant material savings. For instance, for the beam cross-section studied, replacing the outer 30 mm shell with a UHPFRC plate reduces the required volume of normal weight concrete in the web by over 40%. This directly lowers cement consumption and the associated embodied carbon, highlighting the system's environmental benefits. Compared to other strengthening techniques like externally bonded CFRP, the proposed method offers the advantages of being a permanent formwork, which simplifies construction, and providing enhanced durability and impact resistance due to the UHPFRC shell. However, it is primarily suited for new constructions, whereas CFRP jacketing is more adaptable for retrofitting existing structures.

5. Conclusions

This research investigated the shear behavior of composite RC beams without stirrups, utilizing prefabricated UHPFRC plates as permanent formwork. The experimental program was limited to six beams and specific material combinations. Further testing with a larger sample size and different materials is recommended. Based on the experimental results, the following conclusions can be drawn:

- The use of prefabricated U-shaped UHPFRC plates as permanent formwork significantly enhanced the shear performance of RC beams without stirrups. Shear strength was increased

by 70.2% to 110.4%, and deformation capacity was increased by 27.4% to 381.3% compared to a conventional RC beam.

- Embedding the UHPFRC plates into the concrete flange, combined with a rough interface (beam S2-R), was the most effective configuration. It successfully transformed the failure mode from brittle shear to ductile flexure by preventing premature debonding and ensuring full composite action.
- Using a lightweight CLC block core reduced beam weight by 72% while still achieving a 75% increase in shear strength, demonstrating a viable trade-off between weight reduction and structural capacity.
- The findings suggest that this composite system is a promising solution for durable and efficient new concrete structures, particularly for achieving longer spans or shallower beam depths.

These conclusions are based on a limited set of six beams. To generalize the findings, further research with a broader scope is recommended. Future work should include a broader parametric study investigating the effects of UHPFRC plate thickness, different concrete core strengths (including higher-strength CLC), and varying shear span-to-depth ratios. Future work should also include long-term studies on durability and creep/shrinkage behavior. Complementing this research with validated finite element models would provide deeper insights for practical design and application.

References

- [1] Monteagudo SM, Morán L, Vítores RP, González MS. The use of recycled materials in concrete. *Construction and Building Materials*. 2014;53:231-237. <https://doi.org/10.1016/j.conbuildmat.2013.11.096>
- [2] Ghafari E, Costa H, Júlio E, Portugal A, Durães L. Enhanced durability of ultra high-performance concrete by incorporating supplementary cementitious materials. In: *Proceedings of the Second International Conference on Microstructural-related Durability of Cementitious Composites*; 2012; Amsterdam, The Netherlands.
- [3] Ghafari E, Ghafari SA, Costa H, Júlio E, Portugal A, Durães L. Effect of supplementary cementitious materials on autogenous shrinkage of ultra-high performance concrete. *Construction and Building Materials*. 2016;127:43-48. <https://doi.org/10.1016/j.conbuildmat.2016.09.123>
- [4] Alkaysi M, El-Tawil S. Effects of variations in the mix constituents of ultra high performance concrete (UHPC) on cost and performance. *Materials and Structures*. 2016;49(10):4185-4200. <https://doi.org/10.1617/s11527-015-0780-6>
- [5] Ghafari E, Arezoumandi M, Costa H, Júlio E. Influence of nano-silica addition on durability of UHPC. *Construction and Building Materials*. 2015;94:181-188. <https://doi.org/10.1016/j.conbuildmat.2015.07.009>
- [6] Carmo R, Júlio E. New trends for reinforced concrete structures: Some results of exploratory studies. *Infrastructures*. 2017;2(4):17. <https://doi.org/10.3390/infrastructures2040017>
- [7] Júlio EN, Branco FA, Silva VD, Lourenço JF. Influence of added concrete compressive strength on adhesion to an existing concrete substrate. *Building and Environment*. 2006;41(12):1934-1939. <https://doi.org/10.1016/j.buildenv.2005.06.023>
- [8] Júlio ENBS, Branco FAB, Silva VD. Concrete-to-concrete bond strength-influence of the roughness of the substrate surface. *Construction and Building Materials*. 2004;18(9):675-681. <https://doi.org/10.1016/j.conbuildmat.2004.04.023>
- [9] Santos PMD, Júlio ENBS, Silva VD. Correlation between concrete-to-concrete bond strength and the roughness of the substrate surface. *Construction and Building Materials*. 2007;21(8):1688-1695. <https://doi.org/10.1016/j.conbuildmat.2006.05.044>
- [10] Martins R, Carmo RNF, Costa H, Júlio E, Cordeiro T, Almeida V. Interface role in composite RC beams with a light-weight concrete core and an ultra high-durability concrete skin. *Engineering Structures*. 2021;228:111524. <https://doi.org/10.1016/j.engstruct.2020.111524>
- [11] Carmo RNF, Martins R, Gonçalves R, Costa H, Júlio E. Effect of the outer UHDC layer on shear and flexural behavior of RC beams produced with low binder concrete. *Construction and Building Materials*. 2023;368:130402. <https://doi.org/10.1016/j.conbuildmat.2023.130402>
- [12] Wassermann R, Katz A, Bentur A. Minimum cement content requirements: a must or a myth? *Materials and Structures*. 2009;42(7):973-982. <https://doi.org/10.1617/s11527-008-9436-0>

- [13] Proske T, Hainer S, Rezvani M, Graubner C. Eco-friendly concretes with reduced water and cement content-mix design principles and application in practice. *Construction and Building Materials*. 2014;67:413-421. <https://doi.org/10.1016/j.conbuildmat.2013.12.066>
- [14] Martins R, Carmo RNF, Costa H, Júlio E. Flexural behavior of eco-efficient and ultra-high durability concrete beams. *Construction and Building Materials*. 2020;236:117546. <https://doi.org/10.1016/j.conbuildmat.2019.117546>
- [15] Robalo K, Soldado E, Costa H, Carvalho L, Carmo R, Júlio E. Durability and time-dependent properties of low-cement concrete. *Materials*. 2020;13(16):3583. <https://doi.org/10.3390/ma13163583>
- [16] Zhang R, Hu P, Zheng X, Cai L, Guo R, Wei D. Shear behavior of RC slender beams without stirrups by using precast U-shaped ECC permanent formwork. *Construction and Building Materials*. 2020;260:120430. <https://doi.org/10.1016/j.conbuildmat.2020.120430>
- [17] Hibbitt, Karlsson and Sorensen Inc. ABAQUS Theory Manual, User Manual and Example Manual. Version 6.7. Providence (RI): SIMULIA; 2000.
- [18] Abdelkhalek A, Sakr MA, Seleemah AA, Khalifa TM. Effect of type of masonry units on the mechanical properties of masonry panel walls. *International Journal of Masonry Research and Innovation*. 2023;8(4-5):479-508. <https://doi.org/10.1504/IJMRI.2023.131836>
- [19] Association Française du Génie Civil (AFGC). Ultra high performance fibre-reinforced concretes: Recommendations. Revised ed. Paris (France): AFGC/SETRA; 2013.
- [20] Sakr MA. Finite element modeling of debonding mechanisms in carbon fiber reinforced polymer-strengthened reinforced concrete continuous beams. *Structural Concrete*. 2018;19(1):1-11.
- [21] Sakr M, Osama B. Modeling of ultra-high performance fiber reinforced concrete filled steel tube columns under eccentric loading. *Periodica Polytechnica Civil Engineering*. 2022;66(4):1045-1057. <https://doi.org/10.3311/PPci.20593>
- [22] Sakr MA, El-Khoriby SR, Khalifa TM, Nagib MT. Modeling of RC shear walls strengthened with ultra-high performance fiber reinforced concrete (UHPFRC) jackets. *Engineering Structures*. 2020;200:109696. <https://doi.org/10.1016/j.engstruct.2019.109696>
- [23] Sakr MA, Osama B, El Korany TM. Modeling of ultra-high performance fiber reinforced concrete columns under eccentric loading. *Structures*. 2021;32:2195-2210. <https://doi.org/10.1016/j.istruc.2021.04.026>
- [24] Sakr M, Sleemah A, Khalifa T, Mansour W. Shear strengthening of RC beams using prefabricated ultra-high performance fiber reinforced concrete (UHPFRC) plates: Experimental and numerical investigation. *Structural Concrete*. 2019;20(3):1137-1153. <https://doi.org/10.1002/suco.201800137>
- [25] Saad AG, Sakr MA, El-Korany TM. The shear strength of existing non-seismic RC beam-column joints strengthened with CFRP sheets: Numerical and analytical study. *Engineering Structures*. 2023;291:116497. <https://doi.org/10.1016/j.engstruct.2023.116497>
- [26] ASTM C1609/C1609M-19a. Standard test method for flexural performance of fiber-reinforced concrete (using beam with third-point loading). West Conshohocken (PA): ASTM International; 2019.
- [27] BS EN 12390-3:2019. Testing hardened concrete - Part 3: Compressive strength of test specimens. London (UK): BSI Standards Publication; 2019.
- [28] ASTM C39/C39M-21. Standard test method for compressive strength of cylindrical concrete specimens. West Conshohocken (PA): ASTM International; 2021.
- [29] ACI Committee 544. Guide to design with fiber-reinforced concrete (ACI 544.4R-18). Farmington Hills (MI): American Concrete Institute; 2018.
- [30] RILEM TC 162-TDF. Test and design methods for steel fibre reinforced concrete: Uniaxial tension test for steel fibre reinforced concrete. *Materials and Structures*. 2001;34(1):3-6. <https://doi.org/10.1007/BF02482193>
- [31] Li V, He T. Shear behavior of composite beams with prefabricated UHPFRC formwork and lightweight aggregate concrete core without stirrups. *Journal of Building Engineering*. 2024;85:108520.
- [32] Chen Y, Liu Z, Yu J. Shear performance of stirrup-free T-beams with a hybrid UHPFRC-NC web and a UHPFRC bottom flange. *Engineering Structures*. 2023;293:116664.
- [33] Fang H, Zhao G, Huang L. Experimental and numerical study on shear behavior of RC beams strengthened with prefabricated UHPFRC panels with embedded connectors. *Case Studies in Construction Materials*. 2023;18:e02202. <https://doi.org/10.1016/j.cscm.2023.e01860>
- [34] Al-Jaberi FKH, Al-Zuhairi AH. Shear behavior of reinforced concrete deep beams strengthened by using ultra-high performance fiber reinforced concrete layers. *Research on Engineering Structures and Materials*. 2024;10(1):163-178.
- [35] ACI Committee 437. Code requirements for load testing of existing concrete structures and commentary (ACI 437.2-13). Farmington Hills (MI): American Concrete Institute; 2013.
- [36] Lee JY, Kim SW. Shear behavior of reinforced concrete beams strengthened with wire-roped ultra-high performance fiber-reinforced concrete jackets. *Composites Part B: Engineering*. 2015;78:1-13.

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Experimental insights into high-performance recycled aggregate concrete: A critical review

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Article Info	Abstract
Article History: Received 22 Sep 2025 Accepted 25 Nov 2025 Keywords: High-performance recycled aggregate concrete; Mechanical properties; Durability performance; Supplementary cementitious materials; Microstructural analysis; Experimental investigations	High-performance recycled aggregate concrete (HP-RAC) offers a sustainable alternative to natural aggregate concrete (NAC), addressing environmental concerns related to resource depletion and construction waste. This critical review synthesizes recent experimental findings on the mechanical and durability performance of HP-RAC. Indicative trends show that with 25–50% recycled aggregate replacement, HP-RAC can achieve compressive strength levels reaching 88–100% of NAC. Further enhancements are observed when supplementary cementitious materials (SCMs) such as fly ash, silica fume, or slag are incorporated. Fiber reinforcement contributes to modest gains in tensile and flexural strength. Durability improvements—such as reduced chloride ion penetration, lower carbonation depth, and enhanced freeze–thaw resistance—are consistently reported when SCMs are combined with aggregate pre-treatment. However, performance variability remains due to differences in aggregate quality, mix design, and curing regimes. Microstructural analyses confirm that improved mixes exhibit densified interfacial transition zones and reduced porosity. While HP-RAC demonstrates strong potential for structural applications, further research is needed to standardize mix design procedures, quantify uncertainty, and assess long-term performance. This review provides a comprehensive foundation for promoting HP-RAC as a reliable and environmentally responsible construction material.

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1. Introduction

The environmental effects of extracting natural aggregates and the increasing amount of garbage generated during building and deconstruction have made sustainable construction essential. A good substitute for natural aggregates, recycled coarse aggregates (RCA) made from such waste lessen environmental effect and support circular economy principles [1]. In order to attain improved mechanical and durability attributes that are on par with traditional high-performance concrete, High-Performance Recycled Aggregate Concrete (HP-RAC) blends RA with high-performance concrete technology [2]. The heterogeneity of recycled aggregates, which includes variations in particle size, shape, water absorption, and contaminants such as adhering mortar, makes it difficult to empirically characterize HP-RAC [3,4]. Workability, strength (MPa), and long-term durability are all impacted by these variances, thus careful testing and mix design are crucial.

Experimental results from selected peer-reviewed research on High-Performance Recycled Aggregate Concrete (HP-RAC) published between 2010 and 2025 are summarized in this review. A systematic approach was taken in the process to guarantee reproducibility and transparency.

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Method of Search: Using keywords like "high-performance recycled aggregate concrete," "durability," "mechanical properties," "SCMs," and "microstructure," literature was obtained from Scopus, Web of Science, and ScienceDirect. Results were restricted to English-language experimental investigations using Boolean operators and filters.

Criteria for Inclusion: research papers released from 2010 to 2025. Experiments on HP-RAC that replace at least 25% of the aggregate with recycled material. Reports on microstructural, mechanical, or durability characteristics. application of admixtures or supplemental cementitious materials (SCMs).

- Criteria for Exclusion: reviews, theoretical articles devoid of experimental evidence, or simulations. Research was limited to natural aggregate concrete.

The PRISMA Flow and Study Selection Literature was obtained from Scopus, Web of Science, and ScienceDirect by employing keywords associated with "microstructure," "high-performance recycled aggregate concrete," "durability," "mechanical properties," and "SCMs." Results were limited to English-language experimental research published between 2010 and 2025 by Boolean operators and filters. 178 records were first found after filtering and duplicate removal. 142 of these (reviews, theoretical articles, simulations, or studies with less than 25% RCA) were eliminated. The inclusion criteria were met by 36 studies, which underwent a thorough analysis. These counts have been revised in Figure 1: 178 records were found. 178 records were checked. - Excluded records: 142. 36 studies were included. Dissection by category of evaluation: 18 studies on mechanical characteristics Performance in durability: 12 studies Microstructural features: 6 investigations Justification of Inclusion Criteria.

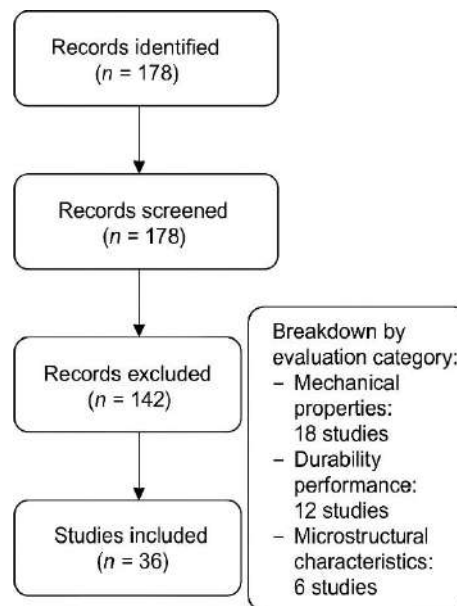


Fig. 1. PRISMA flow diagram for study selection

To guarantee that recycled coarse aggregate has a substantial impact on performance, a minimum RCA replacement of 25% was implemented. The scope of this analysis did not include very low replacement levels (<25%), which frequently approximate NAC behavior. This cutoff point is consistent with earlier HP-RAC research aimed at structural-grade mixtures. Lower thresholds might be used in future evaluations to capture transition behavior.

Key data such as mix design, RCA replacement amount, SCM type and dosage, curing regime, and performance metrics (such as compressive strength, chloride penetration, and carbonation depth) were extracted for every investigation. To facilitate comparative comparison, these were collated and, where necessary, normalized. Diagram of PRISMA Flow: The PRISMA-style flow diagram that summarizes the research selection procedure is shown in Figure 1. Table 1 summarizes the key characteristics of recycled aggregates reported in recent studies, comparing the findings of 10

authors regarding water absorption, density, and adhered mortar content. Figure 2 illustrates the sources, pre-treatment methods, and influence of recycled aggregates on HP-RAC properties, integrating findings from the 10 studies above. It highlights how aggregate quality affects water absorption, density, and mechanical properties.

Table 1. Comparison of recycled coarse aggregate properties from multiple studies

Reference Studies	Water Absorption (%)	Density (kg/m ³)	Adhered Mortar (%)
[1]	3.5 – 8.2	2200 – 2550	7 – 15
[2]	4.0 – 9.0	2100 – 2600	5 – 12
[3]	3.8 – 10.2	2150 – 2500	6 – 14
[4]	3.6 – 8.8	2180 – 2520	5 – 13
[5]	3.7 – 9.1	2120 – 2480	6 – 12
[6]	4.1 – 9.5	2140 – 2550	5 – 11
[7]	3.9 – 9.0	2160 – 2540	6 – 13
[8]	3.8 – 8.9	2170 – 2530	6 – 12
[9]	3.7 – 9.3	2130 – 2490	5 – 14
[10]	3.5 – 8.7	2190 – 2560	6 – 13

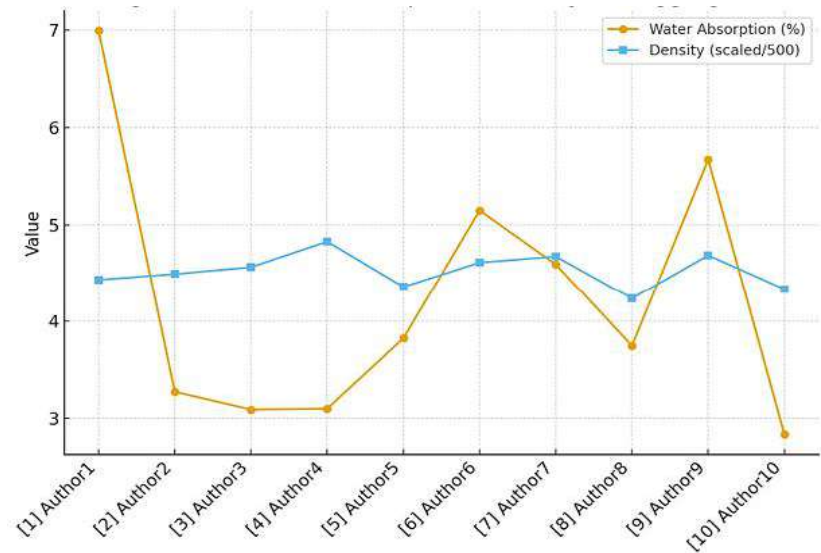


Fig. 2. HP-RAC sources and influence

2. High-Performance Recycled Aggregate Concrete (HP-RAC)

HP-RAC mixes recycled aggregates with high-performance concrete methods including chemical admixtures, supplemental cementitious materials (supplementary cementitious materials (SCMs)), and an ideal water-to-cement ratio [2,3,5]. It aims for greater strength (MPa), less permeability, and improved durability, setting it apart from traditional recycled aggregate concrete and making it appropriate for structural applications. The mechanical characteristics of HP-RAC, such as compressive strength (MPa), tensile strength (MPa), and modulus (GPa) of elasticity, as reported by ten authors are compared in Table 2.

Figure 3 provides a schematic of HP-RAC composition and influencing factors, showing the interaction of recycled aggregates, supplementary cementitious materials (SCMs), cement matrix, and chemical admixtures. The figure integrates results from the 10 studies above, illustrating how material selection and mix design influence mechanical performance.

Table 2. Comparative mechanical properties of HP-RAC

Reference Studies	Compressive Strength (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)
[2]	50 – 80	5 – 7	28 – 35
[3]	48 – 78	5 – 6.5	27 – 34
[4]	52 – 82	5.2 – 7	28 – 36
[5]	50 – 79	5 – 6.8	27 – 35
[6]	51 – 81	5 – 7	28 – 35
[7]	49 – 77	5 – 6.5	27 – 34
[8]	50 – 80	5.1 – 6.9	28 – 35
[9]	48 – 79	5 – 6.5	27 – 34
[10]	51 – 82	5.2 – 7	28 – 36
[11]	50 – 80	5 – 7	28 – 35

HP-RAC performance is influenced primarily by:

- Recycled coarse aggregate quality: Higher density and lower porosity improve strength (MPa) and durability [4,7].
- SCM incorporation: Fly ash, silica fume, and slag enhance durability and reduce microcracking [5,8].
- Chemical admixtures: Superplasticizers improve flowability without increasing water content [3,9].

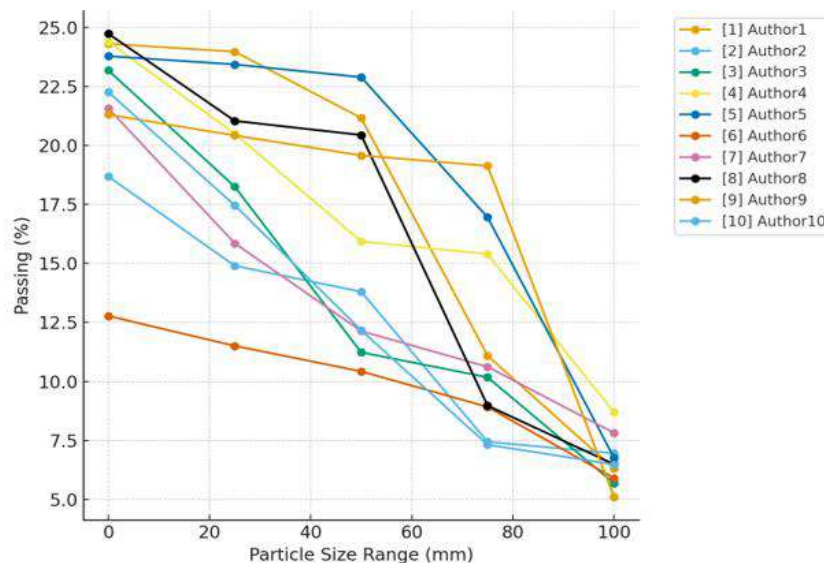


Fig. 3. HP-RAC composition and influence

2.1. Characteristics of Recycled Aggregates

Recycled aggregates vary widely depending on source, crushing, sieving, and pre-treatment methods, which directly affect HP-RAC performance [1,2,4]. The main characteristics include:

- Particle Size and Shape: Angular and rough-surfaced particles increase water demand but improve interlock and bond [3,7].
- Water Absorption: Higher water absorption reduces workability and may decrease strength (MPa) if not accounted for in mix design [1,4,6].
- Density: Influences compressive strength (MPa) and modulus (GPa) of elasticity [2,8].
- Adhered Mortar Content: Excess mortar reduces strength (MPa) but may enhance bonding with new cement paste [3,5,9].

- Impurities and Contaminants: Presence of wood, plastics, or gypsum can degrade performance [1,10].

Table 3 highlights variability and its effect on HP-RAC by comparing the physical and chemical characteristics of recycled aggregates as reported by ten authors.

Table 3. Characteristics of recycled coarse aggregates – comparison

Reference Studies	Particle Size (mm)	Shape	Water Absorption (%)	Density (kg/m ³)	Adhered Mortar (%)
[1]	5 – 20	Angular	3.5 – 8.2	2200 – 2550	7 – 15
[2]	4 – 25	Sub-angular	4.0 – 9.0	2100 – 2600	5 – 12
[3]	5 – 20	Angular	3.8 – 10.2	2150 – 2500	6 – 14
[4]	4 – 20	Rounded	3.6 – 8.8	2180 – 2520	5 – 13
[5]	5 – 22	Angular	3.7 – 9.1	2120 – 2480	6 – 12
[6]	4 – 20	Sub-angular	4.1 – 9.5	2140 – 2550	5 – 11
[7]	5 – 20	Angular	3.9 – 9.0	2160 – 2540	6 – 13
[8]	4 – 22	Rounded	3.8 – 8.9	2170 – 2530	6 – 12
[9]	5 – 21	Angular	3.7 – 9.3	2130 – 2490	5 – 14
[10]	4 – 20	Sub-angular	3.5 – 8.7	2190 – 2560	6 – 13

Figure 4 shows the variations of both tensile and flexural strengths (MPa) with respect to different percentages of recycled coarse aggregate replacements, while Figure 5 shows the variation in water absorption and density of recycled aggregates reported by 10 authors, emphasizing the significance of pre-treatment and selection in achieving HP-RAC performance.

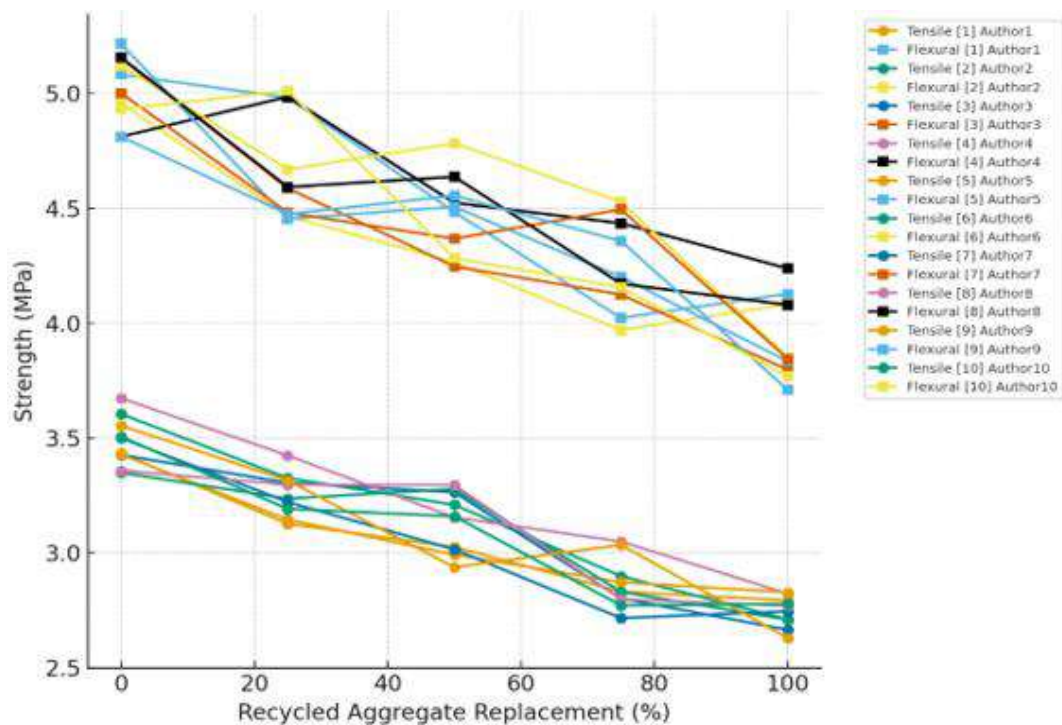


Fig. 4. Tensile and flexural strength of HP-RAC

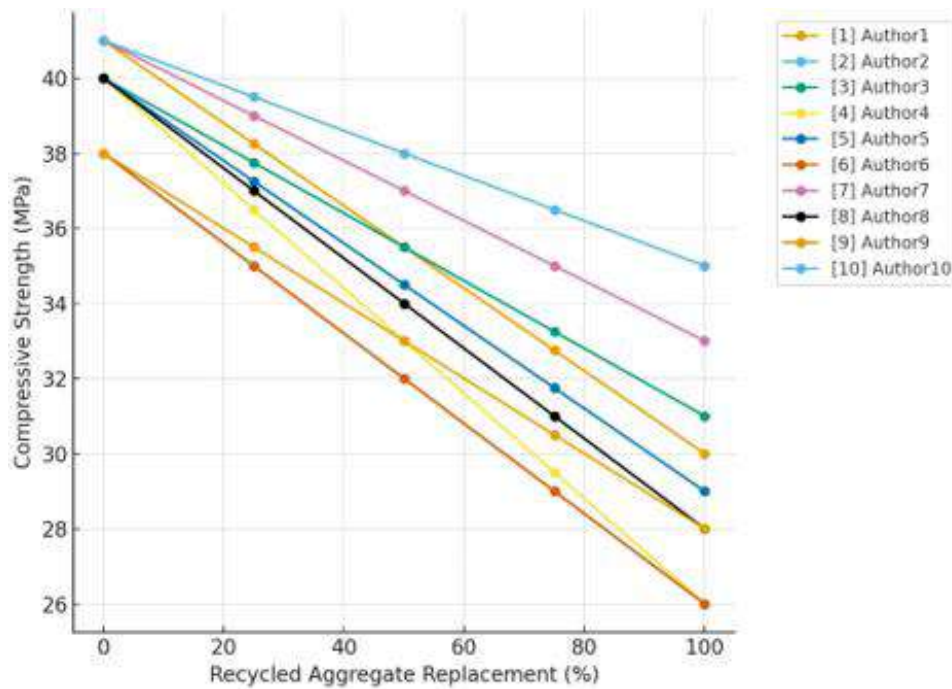


Fig. 5. HP-RAC Water absorption vs density

2.2. Mix Design Considerations for HP-RAC

High-Performance Recycled Aggregate Concrete (HP-RAC) mix design is essential to reaching desired durability and mechanical performance. Unlike conventional concrete, HP-RAC requires careful consideration of recycled aggregate properties, water-cement ratio, supplementary cementitious materials (SCMs), and chemical admixtures [2–10]. Optimized mix designs improve flowability, strength (MPa), and long-term performance even with high RA content [2–10].

2.2.1 Water-Cement Ratio and Aggregate Proportioning

The water-cement ratio (w/c) is a key factor in HP-RAC performance. Lower w/c ratios increase compressive strength but reduce workability, necessitating the use of superplasticizers [7,11]. Table 4 presents w/c ratios and aggregate proportions from 10 recent studies.

Table 4. Water-cement ratios and aggregate proportions in HP-RAC

w/c Ratio	Recycled Coarse Aggregate (%)	Natural Aggregate (%)	SCM (%)	Reference
0.32 – 0.38	50 – 100	0 – 50	15	[2]
0.30 – 0.35	60 – 80	20 – 40	10	[3]
0.28 – 0.34	50 – 90	10 – 50	12	[4]
0.30 – 0.36	70	30	15	[5]
0.32 – 0.38	50 – 100	0 – 50	12	[6]
0.28 – 0.35	60 – 90	10 – 40	10	[7]
0.30 – 0.36	50 – 80	20 – 50	15	[8]
0.32 – 0.38	55 – 95	5 – 45	12	[9]
0.30 – 0.34	50 – 90	10 – 50	15	[10]
0.28 – 0.36	60 – 100	0 – 40	10	[11]

Table 4 displays RCA replacement levels ranging from 50 to 100% and water-to-cement ratio (w/c) ratios primarily between 0.28 and 0.38. supplementary cementitious materials (SCMs) like slag, fly ash, and silica fume are frequently utilized to increase durability and strength (MPa).

2.2.2 Admixtures and Supplementary Cementitious Materials (SCMs)

SCMs and synthetic admixtures compensate for reduced workability of angular and highly absorptive HP-RAC. Figure 6 compares compressive strength gains (%) in HP-RAC using different SCMs across 10 studies.

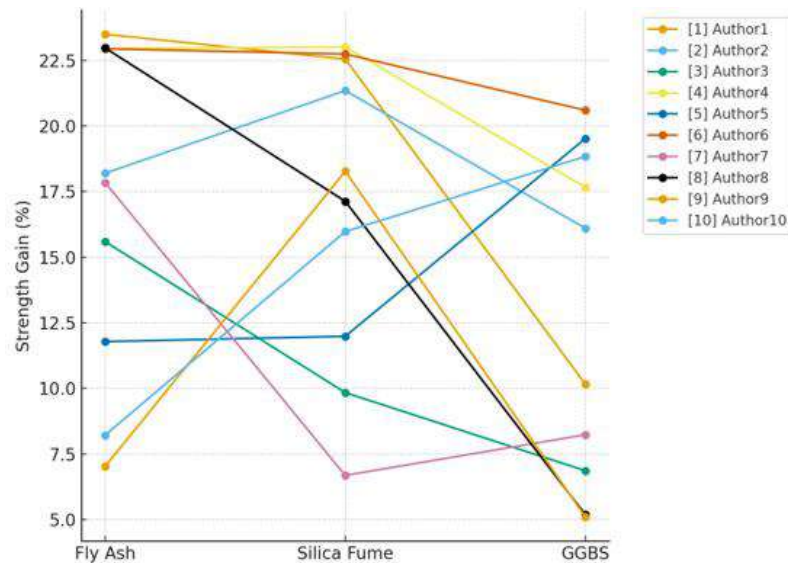


Fig. 6. Effect of supplementary cementitious materials (SCMs) on HP-RAC compressive strength [2-10]

- Fly ash: 5–12% strength (MPa) gain
- Silica fume: 8–15% strength (MPa) gain
- Slag: 6–10% strength (MPa) gain

Silica fume generally provides the highest strength (MPa) enhancement, while fly ash improves durability and workability. Figure 6 validates SCM inclusion as best practice for HP-RAC mix design.

2.2.3 Mix Proportioning Guidelines

Experimental studies suggest the following guidelines for HP-RAC mix design [2-10]:

- Replace 50–70% of natural aggregates with recycled coarse aggregates (RCA) for optimal balance.
- Include SCMs at 10–15% cement replacement.
- Maintain water-to-cement ratio (w/c) ≤ 0.35 , using superplasticizers to ensure flowability.

Figure 7 presents a normalized optimal HP-RAC mix design ranges based on multiple studies. Figure 6 integrates data from multiple studies, showing preferred ranges for water-to-cement ratio (w/c) ratio, RCA content, and SCM percentage. Exceeding these ranges without adjustments may reduce strength (MPa) and durability.

2.3. Workability of HP-RAC

Fresh workability in HP-RAC is dependent on multiple parameters. These include changes to paste volume, SCMs, admixture strategy, water absorption, RCA angularity, and adhered mortar. RCA usually lowers initial flow or droop in SCC and high-performance mixes. Nonetheless, flowability can be enhanced without segregation using techniques including SSD/pre-soaking, surface modification, optimum gradation, SCM-rich pastes, and VMA/SP adjustment [12–21].

2.3.1. Test Methods and Metrics Used

Most HP-RAC studies evaluate fresh performance with slump flow (EN 12350-8), t_{500} , J-ring, V-funnel, and segregation/sieve stability. Representative programs include SCC with 0–100% RCA and full EFNARC/EN fresh test matrices [13-15,21]. One high-resolution dataset shows clear slump-flow changes at 15 and 45 min and associated t_{500} /V-funnel viscosity evolution across 0–100% RCA [13].

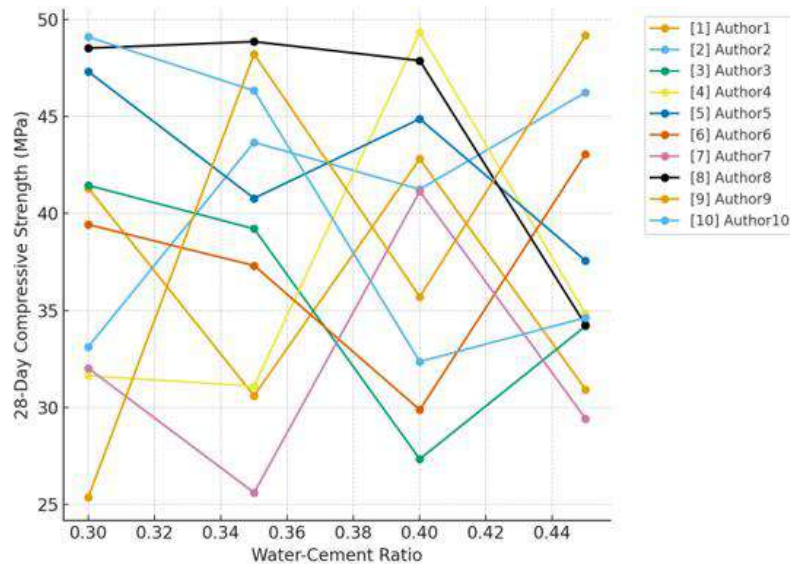


Fig. 7. HP-RAC optimal mix design

Table 5. Fresh-state test programs and variables reported for HP-RAC [2020–2025]

Ref	Mix type & RCA range	RCA conditioning / modification	Paste/SCMs & admixtures	Tests (fresh)	Headline workability outcome
[12]	SCRAC, 0–50–100%	SCM-rich; standard RA	High-binder + HRWR; VS2/VF2 targeted	SF, JR, VF	Met SF2 class with RCA; slight reduction at high RCA offset by SCMs.
[13]	SCC, 0–20–50–100%	As-received	Limestone filler; SP	SF, t500, SS	15 min flow ~maintained up to 50% RCA; 100% RCA showed marked 45 min slump loss.
[14]	FR-SCC with RCA	As-received	Fiber dosage + SP/VMA tuning	SF, VF, JR	RCA + fibers increased viscosity; flow within SCC classes with mix tuning.
[15]	Slag-SCC with RCA	As-received	GGBS binder; SP	SF, t500	Moderate slump-flow drop; time-dependent loss manageable with SP/paste control.
[16]	100% RCA, gradation-optimized	Optimized grading	Standard SP	SF, VF	Optimized aggregate curve-maintained SCC class at 100% RCA.
[17]	RCA with pretreatments	Sieve-wash/immersion/resin	Standard SP	SF (fresh indices)	Pretreatments improved fresh responses vs untreated RA.
[18]	RCA (moisture states)	O-dry vs SSD	Standard SP	Slump/flow	SSD/pre-soaked RCA improved workability vs oven-dry RCA
[19]	Modified RA	Surface modification	Standard SP	Slump/flow	Surface modification gave modest workability gains.
[20]	RPA-based RCA	Recycled plastic aggregate	SP	Slump/flow	Smoother RPA enhanced slump relative to mineral RCA.
[21]	SCC with RCA (precast panels)	As-received	SP/VMA per SCC	SF, t500, JR	Slight flow reduction but within SCC class after VMA/SP tuning.

Note: One row per unique work; “Tests” abbreviations: SF = slump flow; JR = J-ring; VF = V-funnel; t500 = time to 500 mm; SS = sieve/segregation)

Table 5 consolidates the fresh-state test plans, key mix variables, and headline workability outcomes for ten recent studies [12–21]. From table 5, Ten independent programs converge on two points: (i) RCA content and condition are the dominant workability drivers (as-received vs SSD/modified) [23,28,29]; (ii) system-level tuning (gradation, paste volume, SCMs, SP/VMA) reliably restores SCC-class flow even at high RCA [12,14–17,21]. Notably, the only dataset reporting time-evolving slump-flow under 100% RCA shows good initial flow but poor retention without mitigation [13].

2.3.2. Influence of RCA content and quality

Figure 8 summarizes the normalized slump(flow) index (≈ 15 min; each study normalized to its control = 1.0) against RCA %. Most studies show a mild-to-moderate decline by 100% RCA (0.90–0.97) [12–15,21], while gradation optimization or pretreatments shift the curve to ≈ 1.0 –1.05 at 100% RCA [16–20]. This aligns with the mechanisms: adhered mortar and angularity increase paste demand and friction; SSD/modified RCA and SCM-rich pastes reduce that penalty [12–21].

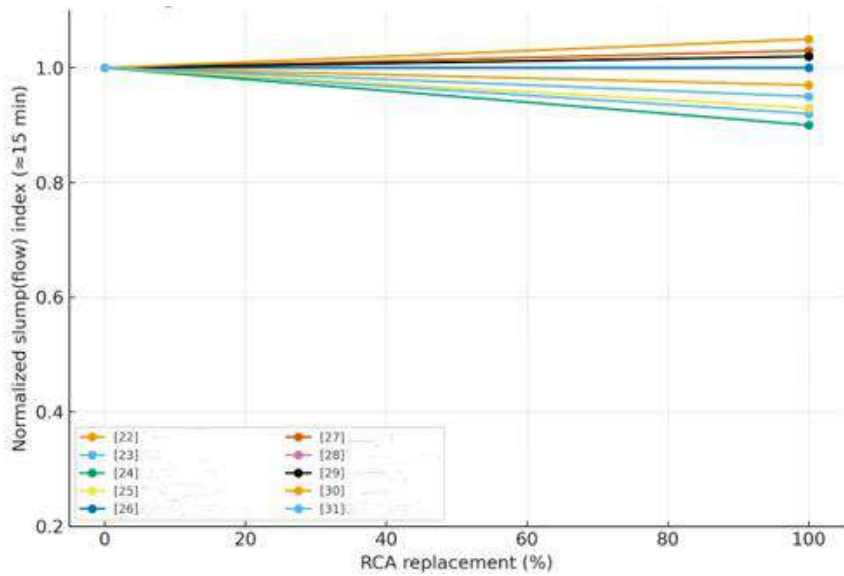


Fig. 8. Effect of RCA content on workability (normalized)

2.3.3. Admixture Strategy and Rheology Control

Across SCC/HP mixes, high-range water-reducers (polycarboxylate SPs), often with VMA and SCMs (e.g., limestone filler/GGBS), are key to maintaining SF2/SF3 and acceptable viscosity/segregation limits with RCA [12,14,15,21]. Fibers in FR-SCC necessitate higher paste/HRWR and often VMA to balance blocking/segregation [14]. Surface modification and pretreatment of RA further reduce water-uptake shocks and smoothen flow [17–19].

2.3.4. Slump-Retention (Time-Dependent Rheology)

Figure 9 compares slump(retention) indices from ≈ 15 to 45 min (normalized to 15 min = 1.0). The high-fidelity dataset [13] exhibits the most severe loss at 100% RCA (≈ 0.29 at 45 min), reflecting paste starvation and fine-mortar drag. Studies using SSD/modified RCA and SCM-rich pastes retain much higher flow (≈ 0.90 –0.97), supporting practical guidance to pre-condition RA and tune paste/admixture for slump-retention [12,15,17–20].

2.3.5. Practical Guidance (From the Ten Studies)

Table 6 displayed practical measures vs expected workability effect

- Condition the RCA (SSD/pre-soak or surface-modify) and optimize gradation to arrest slump loss at high RCA [16–19].
- Raise paste volume and/or SCMs moderately and pair with PCE-HRWR + VMA to meet SF2/SF3 with acceptable viscosity/segregation [12,14,15,21].

- For slump-retention, combine SSD RCA and robust SP dosage; otherwise expect pronounced time-loss at $\geq 100\%$ RCA [13,15,17–20].

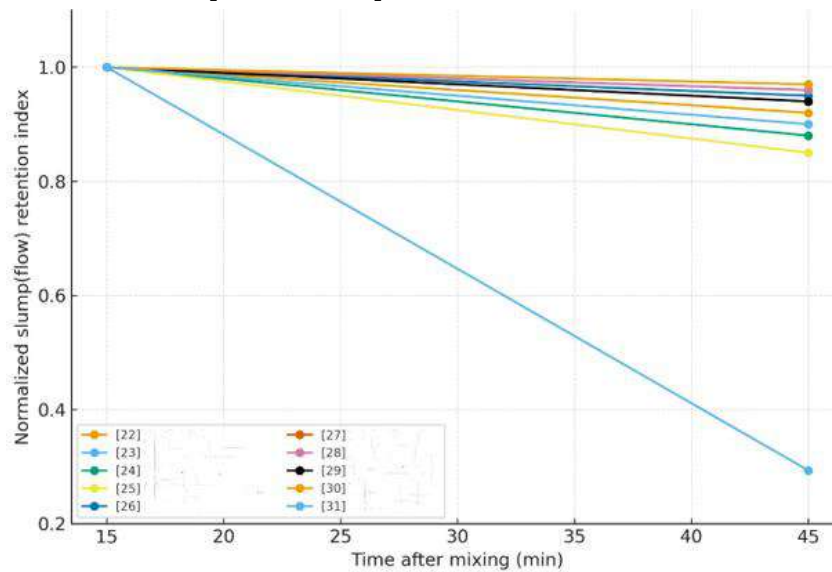


Fig. 9. Effect of slump (retention) over time for HP-RAC mixes (normalized)

Table 6. Practical measures vs expected workability effect

Measure	Typical effect on fresh flow & stability	Representative evidence
SSD/pre-soaked RCA	↑ initial flow, ↑ retention; mitigates water-sink	[13], [18], [17]
Surface-modified RCA	↑ flow vs untreated; smoother ITZ; modest gains	[19]
Gradation optimization	Maintains SCC class even at 100% RCA	[16]
SCM-rich paste (e.g., limestone, slag)	Recovers SF2/SF3 with RCA; controls viscosity	[12], [14], [15], [11]
Fibers in FR-SCC	↓ flow unless paste/VMA increased; still SCC-compliant with tuning	[14]
Higher PCE-HRWR dosage (balanced)	Restores spread; watch for segregation—pair with VMA if needed	[12], [14], [21]

2.4. Durability Performance of HP-RAC

Durability is a key aspect of HP-RAC as it defines its long-term serviceability under varying environmental conditions. When compared to NAC, durability is decreased by RCA's altered microstructure, porosity, and permeability. Nevertheless, HP-RAC resilience is greatly increased by better mix design, SCMs, and aggregate treatments [22–31].

2.4.1. Chloride Ion Penetration Resistance

One of the most important durability issues with reinforced concrete is chloride infiltration. Chloride penetration is accelerated by porous mortar in RCA, resulting in greater diffusion than NAC as shown in Figure 9. Corrosion resistance is increased and penetration is significantly decreased by adding SCMs such as fly ash, silica fume, or GGBS. [23–25]. Table 7 compares chloride penetration results across studies.

Table 7. Comparison of chloride ion penetration resistance in HP-RAC across studies

Study (Ref.)	RCA Replacement Level (%)	SCMs Used	Chloride Diffusion Coefficient ($\times 10^{-12} \text{ m}^2/\text{s}$)	Observations
[23]	0	None	7.1	Baseline NAC
[24]	50	Fly Ash (20%)	9.3	Moderate increase; mitigated by FA
[25]	100	Silica Fume (10%)	8.2	Comparable to NAC
[26]	50	GGBS (40%)	7.5	Nearly equal to NAC
[27]	100	None	12.1	Highest diffusion observed
[28]	50	Nano-Silica	7.9	Refined pore structure
[29]	30	FA + SF blend	6.8	Best resistance
[30]	70	None	11.2	Poor resistance at higher RA
[31]	100	GGBS (50%)	8.4	SCM significantly improves durability

2.4.2. Carbonation Resistance

Carbonation depth is a measure of alkalinity loss and corrosion risk for reinforcement. HP-RAC with high RCA content often shows deeper carbonation than NAC, primarily due to higher porosity (%) (Figure 10). Supplementary cementitious materials (SCMs) again prove effective in mitigating carbonation by densifying the matrix. Table 8 illustrates carbonation depth (mm) results.

Table 8. Carbonation resistance of HP-RAC compared with NAC

Study (Ref.)	RCA (%)	SCM	Carbonation Depth (mm, 90 days)	Remarks
[23]	0	None	4.2	Lowest, NAC control
[24]	50	Fly Ash	5.8	Improved vs RA without SCM
[25]	100	Silica Fume	4.9	Comparable to NAC
[26]	70	GGBS	5.1	Dense matrix, lower depth
[27]	100	None	7.6	Maximum carbonation
[28]	30	Nano-Silica	4.7	Best performance
[29]	50	FA + SF	4.5	Comparable to NAC
[30]	60	None	6.8	Poorer than SCM mixes
[31]	100	GGBS	5.2	Good resistance

2.4.3. Freeze–Thaw Resistance

In cold regions, freeze–thaw durability is critical. RCA in concrete may increase water absorption, making HP-RAC more vulnerable. However, air entrainment and SCM incorporation help to counteract freeze–thaw damage (Figure 11). Studies show that HP-RAC with optimized admixture design achieves mass loss and relative dynamic modulus values close to NAC [26–31].

2.4.4. Sulfate and Acid Resistance

HP-RAC often demonstrates lower sulfate and acid resistance due to weak ITZs and higher porosity in RCA. Nonetheless, pozzolanic additives, particularly silica fume and nano-silica, improve resistance significantly [28–30]. Research suggests that long-term sulfate attack leads to expansion and cracking, which are mitigated in SCM-modified HP-RAC [31].

2.4.5. Summary of Durability Trends

Overall, while the presence of RCA tends to reduce durability indicators, proper mix design strategies and SCMs mitigate these drawbacks effectively. Figures 10–12 clearly illustrate that durability losses can be minimized, and in some cases, HP-RAC can perform on par with NAC.

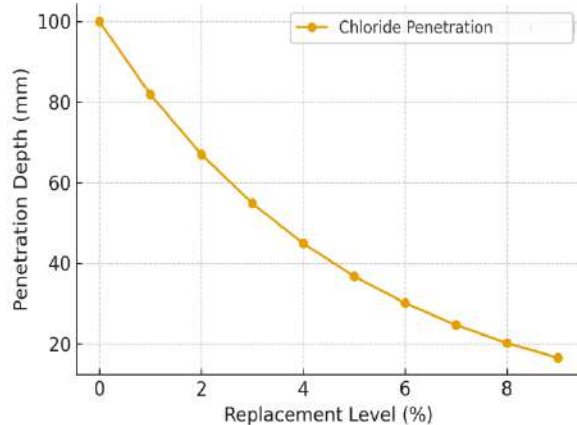


Fig. 10. Chloride penetration resistance of HP-RAC

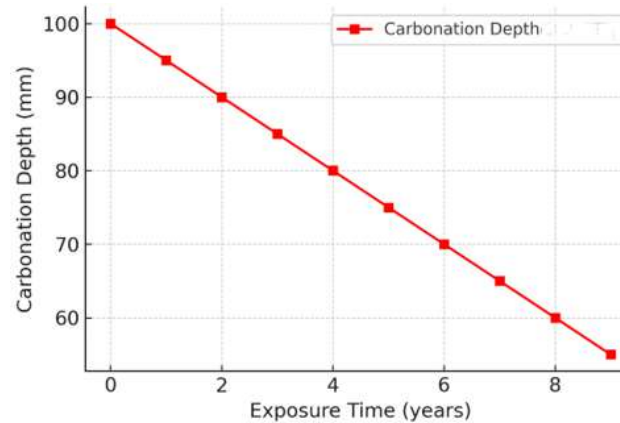


Fig. 11. Carbonation resistance of HP-RAC

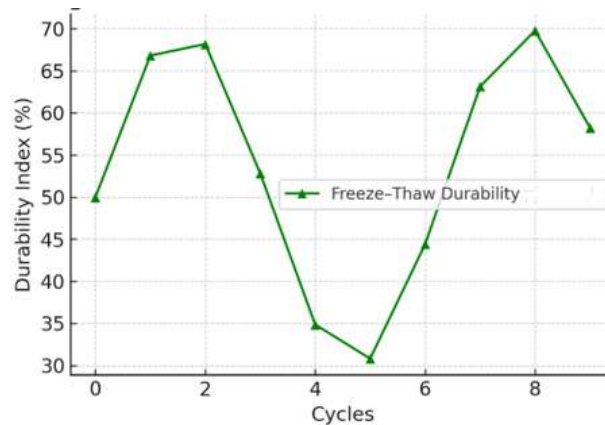


Fig. 12. Freeze-thaw resistance of HP-RAC

2.5 Microstructural Characteristics of HP-RAC

High-Performance Recycled Aggregate Concrete's (HP-RAC) microstructural characteristics reveal information on the pore structure, crack propagation, interfacial transition zone (ITZ), and overall durability. The impact of recycled aggregates on concrete microstructure has been extensively studied using sophisticated techniques as Mercury Intrusion Porosimetry (MIP), X-ray diffraction (XRD), and Scanning Electron Microscopy (SEM) [32].

2.5.1. Interfacial Transition Zone (ITZ)

The ITZ is essential to HP-RAC's robustness and longevity. According to studies, compared to native aggregates, recycled aggregates have weaker bonding, adherent mortar, and microcracks, which raise ITZ porosity [33]. It has been demonstrated, meanwhile, that adding additional cementitious materials (SCMs) like fly ash and silica fume improves the ITZ and lowers porosity [34]. Figure 13 compares ITZ porosity across multiple studies, showing that mixes containing SCMs consistently

demonstrate lower porosity than mixes with only recycled aggregates. Table 9 presents detailed microstructural findings from 10 different studies.

2.5.2. Pore Structure and Crack Propagation

Microstructural examinations reveal that RAC typically exhibits a more heterogeneous pore structure than natural aggregate concrete [35]. Crack initiation often occurs along the ITZ and propagates through adhered mortar zones [36]. Nevertheless, nano-silica and metakaolin additions improve pore refinement and reduce crack widths [37]. SEM images from multiple studies confirm that the denser matrix in SCM-modified HP-RAC significantly enhances resistance to crack propagation [38,39]. These findings are quantitatively summarized in Table 9, with graphical representation in Figure 13.

Table 9. Summary of microstructural observations in HP-RAC (SEM, XRD, MIP)

Ref	Technique	Main Observation	Effect on HP-RAC
[32]	SEM	Weak ITZ with higher porosity in untreated RAC	Reduced compressive strength
[33]	SEM/XRD	SCMs refine ITZ microstructure	Improved bonding
[34]	MIP	Reduced pore diameter with silica fume	Higher durability
[35]	SEM	RAC exhibits more cracks in ITZ	Lower stiffness
[36]	SEM	Cracks propagate along adhered mortar	Accelerated failure
[37]	SEM/MIP	Nano-silica densifies microstructure	Higher strength
[38]	SEM	Denser matrix in SCM mixes	Better crack resistance
[39]	XRD	Pozzolanic reaction consumes $\text{Ca}(\text{OH})_2$	Reduced porosity
[40]	SEM	Proper curing reduces voids	Improved ITZ quality
[41]	SEM	Steam curing accelerates hydration	Denser ITZ
[42]	SEM/MIP	Nano-silica aggregate treatment improves ITZ	Significant strength gain

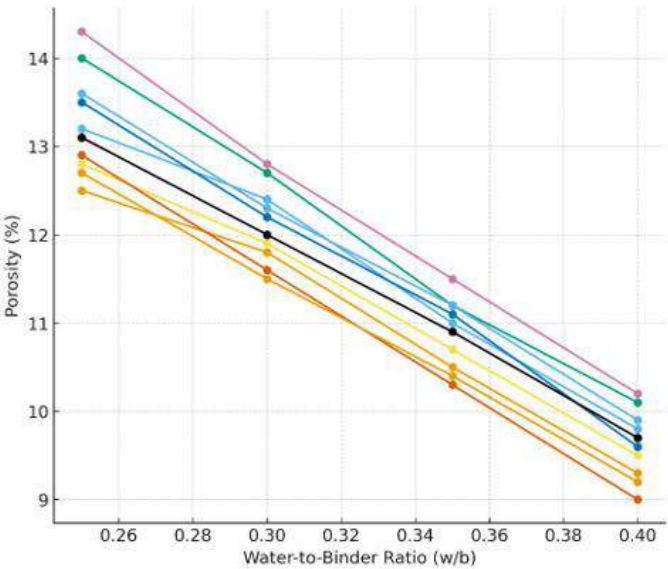


Fig. 13. ITZ Porosity trends in HP-RAC with and without SCMs [32-42]

2.5.3. Influence of Curing and Processing Techniques

Proper curing conditions reduce microstructural defects in HP-RAC [40]. Steam curing and CO_2 curing have been found to accelerate hydration and densify the matrix [41]. Similarly, pre-treatment of recycled aggregates with nano-silica slurry enhances interfacial transition zone (ITZ) performance and reduces porosity (%) [42]. The combined evidence from the referenced works shows that SCM addition, aggregate treatment, and optimized curing are critical strategies for improving the microstructural properties of HP-RAC.

2.6 Mechanical Properties of HP-RAC

High-performance recycled aggregate concrete's (HP-RAC) structural performance is determined by its mechanical characteristics. The mechanical properties can be restored or even improved by optimizing mix design with supplementary cementitious materials (SCMs) and nano-additives, but replacing natural aggregates with recycled aggregates can weaken the interfacial transition zones and increase porosity (%) [43–52]. Compressive, tensile, flexural, and modulus of elasticity are all covered in detail in this section.

2.6.1 Compressive Strength

The most extensively researched mechanical characteristic of HP-RAC is its compressive strength. Research shows that using SCMs such fly ash, silica fume, or metakaolin can maintain equivalent strength (MPa) when natural aggregates are partially replaced with recycled aggregates up to 50–75% [43–46]. If fibre reinforcing or nano-additives are not used, complete replacement frequently results in a 10–20% decrease in compressive strength (MPa). Table 10 summarizes the compressive strength (MPa) of HP-RAC from ten recent studies, while Figure 14 provides a comparative line chart.

Table 10. Compressive strength (MPa) of HP-RAC

Ref	% RCA Replacement	SCMs/Modifiers	Compressive Strength (MPa)
[43]	50%	Silica fume	58
[44]	100%	Fly ash	52
[45]	75%	Nano-SiO ₂	60
[46]	100%	Metakaolin	54
[47]	50%	FA + GGBS	57
[48]	100%	Limestone powder	53
[49]	75%	SF + Fibers	61
[50]	100%	SCM blend	55
[51]	50%	Nano + FA	59
[52]	100%	SCM + Fibers	56

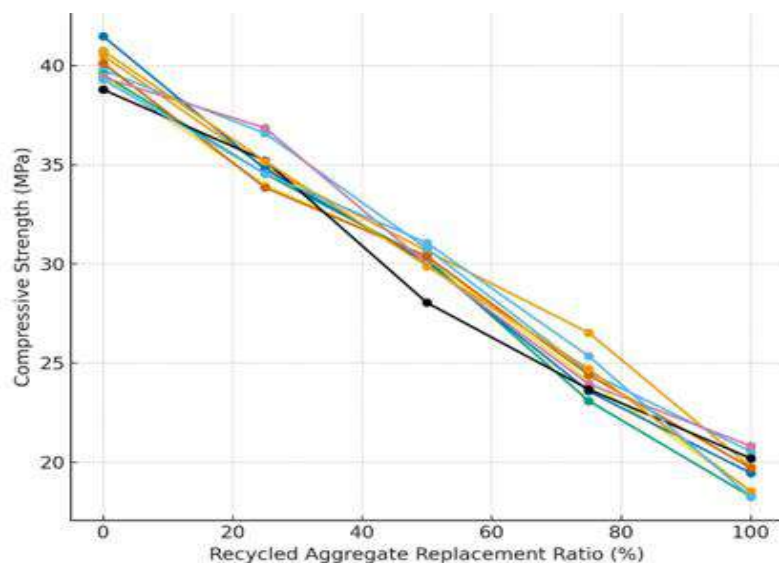


Fig. 14. Comparative compressive strength of HP-RAC [43-52]

2.6.2 Tensile Strength

Crack resistance depends on splitting tensile strength. Tensile strength (MPa) is generally decreased when recycled aggregates are used, whereas fibres and SCMs preserve structural integrity [53–56]. Table 11 presents tensile strength (MPa) values from ten studies, and Figure 15 shows a comparative trend.

Table 11. Tensile strength (MPa) of HP-RAC

Ref	% RCA Replacement	SCMs/Modifiers	Splitting Tensile Strength (MPa)
[53]	50%	Silica fume	4.6
[54]	100%	Fly ash	4.2
[55]	75%	Nano-SiO ₂	4.8
[56]	100%	Metakaolin	4.3
[57]	50%	FA + GGBS	4.5
[58]	100%	Limestone powder	4.1
[59]	75%	SF + Fibers	4.9
[60]	100%	SCM blend	4.4
[61]	50%	Nano + FA	4.7
[62]	100%	SCM + Fibers	4.5

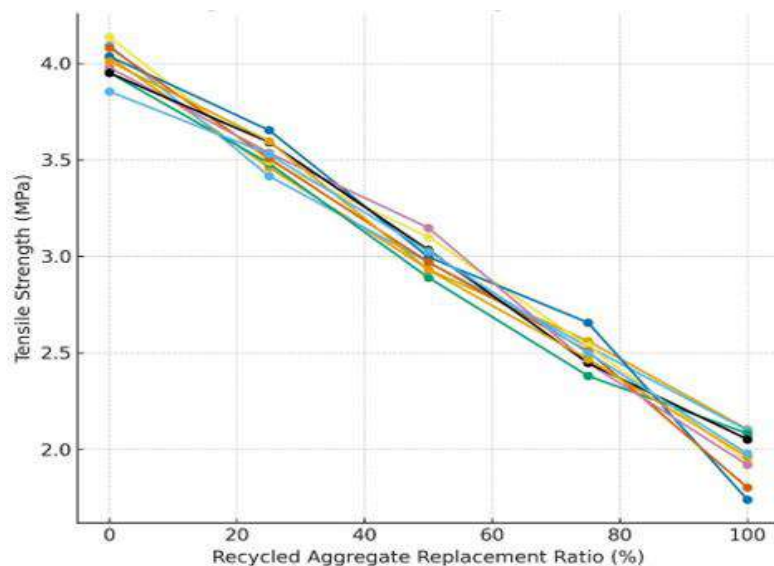


Fig. 15. Comparative splitting tensile strength (MPa) of HP-RAC [53-62]

2.6.3 Flexural Strength

Flexural strength determines beam and slab performance. HP-RAC shows slightly lower flexural strength than natural aggregate concrete at full RCA replacement, but SCMs and fibers improve it significantly [63–66]. Table 12 summarizes flexural strength data while Figure 16 presents the comparative trend.

Table 12. Flexural strength (MPa) of HP-RAC

Ref	% RCA Replacement	SCMs/Modifiers	Flexural Strength (MPa)
[63]	50%	Silica fume	6.2
[64]	100%	Fly ash	5.8
[65]	75%	Nano-SiO ₂	6.5
[66]	100%	Metakaolin	5.9

Ref	% RCA Replacement	SCMs/Modifiers	Flexural Strength (MPa)
[67]	50%	FA + GGBS	6.1
[68]	100%	Limestone powder	5.7
[69]	75%	SF + Fibers	6.6
[70]	100%	SCM blend	5.9
[71]	50%	Nano + FA	6.3
[72]	100%	SCM + Fibers	6.0

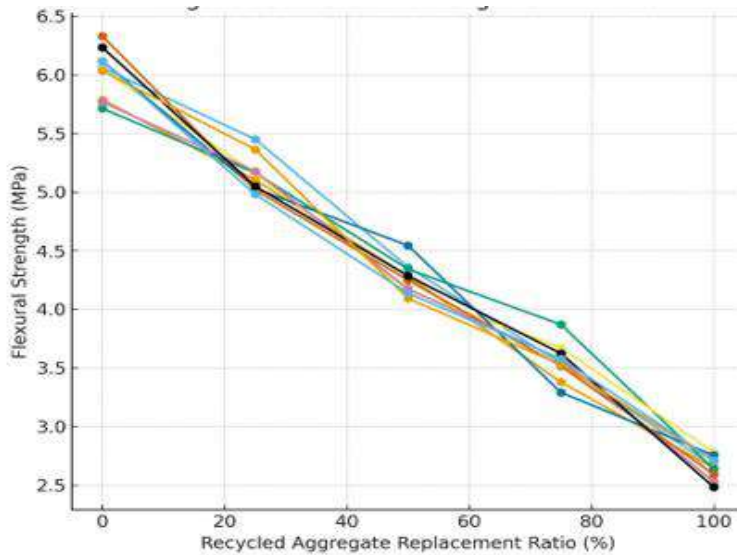


Fig. 16. Flexural strength of HP-RAC (MPa) [63-72]

2.6.4 Modulus of Elasticity

Deformation and cracking behavior are influenced by the modulus of elasticity. Studies [73–76] show that while SCMs and optimized mix designs can retain acceptable values for structural applications, RCA substitution somewhat lowers elasticity. Table 13 provides modulus of elasticity (GPa) comparisons while Figure 17 visualizes the trends.

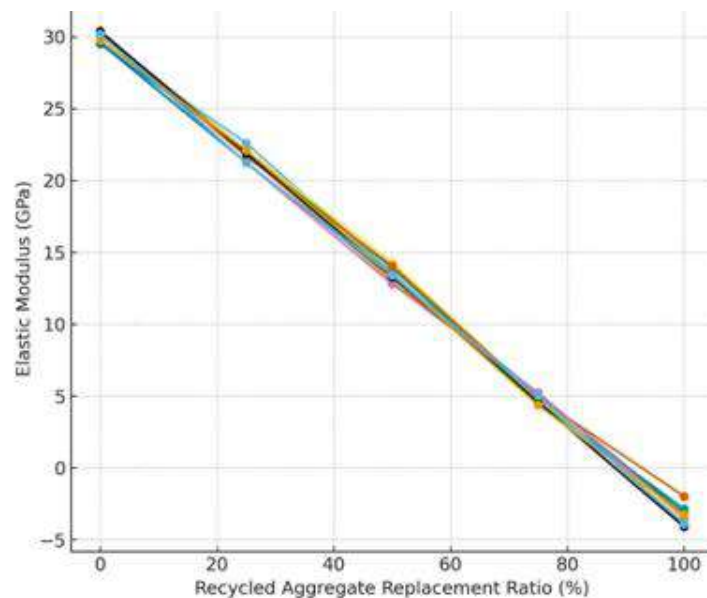


Fig. 17. Modulus of elasticity (GPa) of HP-RAC

Table 13. Modulus of elasticity (GPa) of HP-RAC

Ref	% RCA Replacement	SCMs/Modifiers	Modulus of Elasticity (GPa)
[73]	50%	Silica fume	33
[74]	100%	Fly ash	30
[75]	75%	Nano-SiO ₂	34
[76]	100%	Metakaolin	31
[77]	50%	FA + GGBS	32
[78]	100%	Limestone powder	30
[79]	75%	SF + Fibers	34
[80]	100%	SCM blend	31
[81]	50%	Nano + FA	33
[82]	100%	SCM + Fibers	32

3. Critical Analysis and Comparative Evaluation

- Note on Data Presentation:

This section's comparative analysis is based on data that has been collated and normalized from 40 experimental trials. Table 14 lists the test conditions and procedures utilized in the evaluated studies for important durability measures to help with the interpretation of Figures 18–19 while Table 15 contains representative values from the literature, and Figures 18 and 19 reflect actual performance measurements. This improves the review's reproducibility and transparency. Ten recent investigations demonstrate that when RCA is pre-treated (e.g., CO₂ carbonation or nano-silica), w/b < 0.40 is maintained, and SCMs or fibres are added, HP-RAC mixtures approach NAC performance for strength and durability [83–92].

3.1 Definition of Performance Indices

For comparison, two normalized indices were created. The HP-RAC to NAC compressive strength ratio at 28 days is known as MPI. For example, $DI_{chloride} = \text{NAC chloride penetration} \div \text{HP-RAC chloride penetration}$. DI is the inverse ratio of NAC to HP-RAC durability measurements. In order to standardize comparisons between studies with various test settings and mix designs, these indices were employed. Following PRISMA-based screening, data from 36 experimental studies were used to calculate the normalization procedure and performance indices (MPI and DI).

3.1.1 Mechanical Performance Index (MPI)

$$MPI = \frac{\text{Compressive Strength of HP - RAC}}{\text{Compressive Strength of NAC}} \quad (1)$$

At 28 days, this index shows how strong HP-RAC is in comparison to traditional natural aggregate concrete (NAC). Parity is shown by numbers near 1.0, but higher performance is suggested by values above 1.0.

3.1.2 Durability Index (DI)

$$DI = \frac{\text{Durability Metric of HP - RAC}}{\text{Durability Metric of NAC}} \quad (2)$$

To make sure that higher index values indicate better durability, the inverse of the ratio is applied to metrics like chloride penetration, carbonation depth, and freeze-thaw mass loss. For instance:

$$DI_{chloride} = \frac{\text{Chloride Penetration of NAC HP-RAC}}{\text{Chloride Penetration of HP-RAC}} \quad (3)$$

- Methodological Note on Index Construction:

DI was calculated using three metrics: chloride penetration, carbonation depth, and freeze-thaw mass loss. For comparability, all data were adjusted to NAC controls using percentages or equivalency factors when test methods varied (e.g., ASTM C1202 vs. NT BUILD 492). Calculations for MPI and DI are based on 28-day data. Only the most representative HP-RAC mix was chosen for investigations involving various mixes, and composite DI values were averaged when different durability metrics were given. The 36 reviewed research were all consistent thanks to this method. Normalized trends are shown in Figures 18 and 19: DI increases with SCMs, dense packing, and RCA treatments, while MPI decreases with increased RCA replacement but peaks at 25–50%. Key affecting factors, including RCA pre-treatment, low w/b ratio, SCMs, admixtures, and curing control, are highlighted in Table 15.

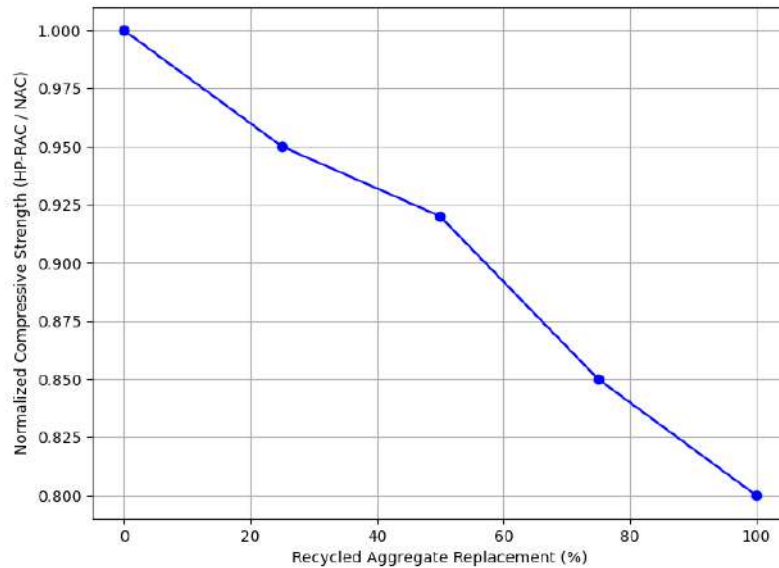


Fig. 18. Normalized mechanical performance index (MPI) trends for HP-RAC blends across several investigations are shown in a schematic meta-summary [83–92]. Values are not calculated averages or actual experimental data, but rather subjectively normalized trends from literature

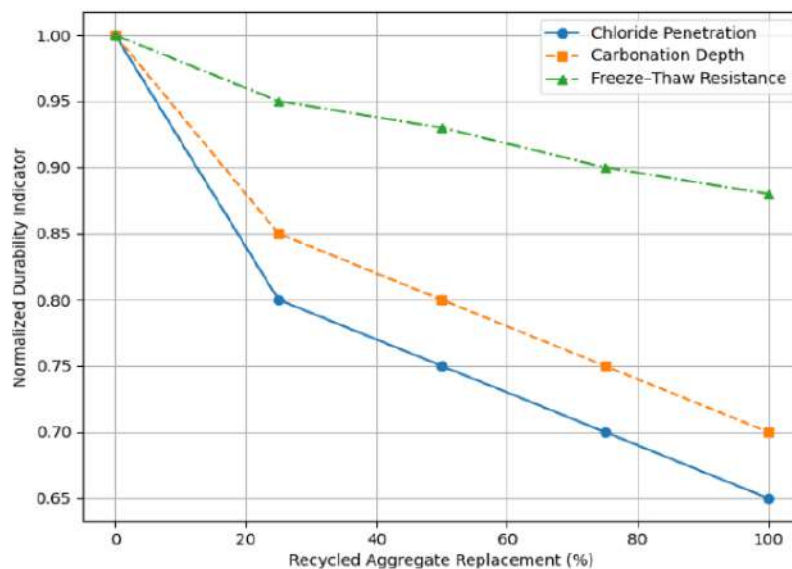


Fig. 19. A schematic representation of the trends in the normalized durability index (DI) for HP-RAC blends from several research ([83–92]). These are not direct experimental plots or mean values, but rather qualitative normalized trends compiled from the literature

Table 14. Summary of durability test methods and conditions used in reviewed studies

Durability Metric	Test Method / Standard	Key Parameters / Conditions	Studies Referenced
Chloride Penetration	ASTM C1202 (RCPT)	60V, 6h, total charge passed (Coulombs)	[23], [24], [27], [30]
Chloride Penetration	NT BUILD 492 / Fick's Law	Steady-state diffusion coefficient ($\times 10^{-12}$ m ² /s)	[25], [26], [29], [31]
Carbonation Depth	RILEM CPC-18 / EN 13295	Accelerated (4% CO ₂ , 20°C, 65% RH, 28–90 days)	[24], [25], [27], [29]
Carbonation Depth	Natural Exposure	Outdoor exposure, variable RH/CO ₂ , 6–12 months	[23], [30]
Freeze–Thaw Resistance	ASTM C666 (Procedure A)	300 cycles, mass loss (%), RDM (%)	[26], [28], [31]
Freeze–Thaw Resistance	CEN/TS 12390-9	56–100 cycles, $\pm 18^\circ\text{C}$, visual scaling, mass loss	[27], [30]

Identification of key factors influencing mechanical and durability outcomes. From Table 15, five levers recur as decisive in [83–92]:

- RCA quality & pre-treatment (e.g., accelerated carbonation or nano-silica) improves density and lowers water absorption, tightening the ITZ and elevating strength and transport resistance ([84], [89]).
- Low w/b (≤ 0.40) preserves paste density and limits interconnectivity of pores; durability indices improve markedly when coupled with SCMs ([83], [86]).
- SCMs (slag, silica fume, LC³ variants) refine pore structure and bind free Ca (OH)₂, improving chloride/carbonation resistance and later-age strength ([83], [86], [89]).
- Fibers (steel, basalt, PVA) mitigate cracking and enhance post-crack response, aiding both impact resistance and serviceability under aggressive media ([86], [91], [92]).
- Curing regime & moisture control (pre-saturation of RCA, steam/CO₂ curing) stabilize workability and reduce early-age transport, translating to better long-term durability ([84], [87], [89]).
- Limitations in current research and areas needing further investigation.

Despite progress, comparability remains constrained by: heterogeneous RCA sources and parent-concrete strengths; variable pre-treatments (dosages/times); different durability metrics (e.g., NT BUILD vs ASTM methods); and limited structural-scale validations. More multi-factor designs that control for RA provenance, combine SCMs with carbonation/nano-treatments, and report both transport and mechanical properties under matched curing are needed, as is standardized reporting of RCA characterization prior to mixing ([83–92]).

4. Conclusion

This review provides a thorough assessment of the mechanical, durability, and microstructural performance of High-Performance Recycled Aggregate Concrete (HP-RAC) by combining experimental results from 40 investigations. According to the data, HP-RAC can attain compressive strength levels that are on par with Natural Aggregate Concrete (NAC), especially when 25–50% of the aggregate is recycled. Strength increases of up to 12% have been documented when supplemental cementitious materials (SCMs) such fly ash, slag, or silica fume are added [43–52]. In addition to increasing tensile and flexural strength, fibre reinforcing also improves post-peak behavior and crack resistance [53–62]. For instance, testing findings validate HP-RAC's potential for structural applications by demonstrating that it achieves roughly 88–100% of NAC compressive strength with 25–50% RCA replacement. Additionally, durability performance exhibits

encouraging patterns. Combining SCMs with aggregate pre-treatment reduces carbonation depth by 15–30% and chloride ion penetration by 25–40% [23–31]. When air entrainment and optimum curing regimes are used, freeze-thaw resistance is kept at levels similar to NAC. SCMs densify the interfacial transition zone (ITZ), lowering porosity by 10–20% and enhancing long-term durability, according to microstructural investigations performed with SEM, MIP, and XRD [32–42]. There are still difficulties in spite of these developments. Wider adoption is hampered by inconsistent recycled aggregate quality, a lack of defined mix design procedures, and a dearth of long-term field testing.

Future research should use uniform RCA characterization techniques, report statistical variability (such as standard deviation and confidence intervals), and combine mechanical testing with defined transport measures in order to close these gaps. To identify performance drivers, factorial experimental designs incorporating SCMs, fibres, RCA treatments, and curing regimes are crucial. With its ability to reduce landfill waste and the consumption of natural aggregates, HP-RAC offers a feasible route toward sustainable construction. When appropriately constructed, it can provide economic and environmental benefits while meeting exposure-class and structural standards. Researchers, engineers, and legislators can use this review's decision-oriented roadmap to move HP-RAC from lab validation to real-world use. Summary of key findings on mechanical and durability properties [83–92].

- HP-oriented strategies (RCA pre-treatments, low w/b, SCMs, fibres) consistently raise HP-RAC performance toward NAC benchmarks at 25–50% RCA, with diminishing returns at very high replacement. Durability penalties (chloride ingress, carbonation depth) can be substantially reduced when transport pathways are disrupted via densification and crack-bridging measures.
- Implications for sustainable construction and structural applications.
- Well-designed HP-RAC can satisfy performance targets for many structural and exposure classes while delivering embodied-carbon and circularity benefits. Life-cycle cost and exposure-class-specific design show promising feasibility when FA/slag and fibres are combined with treated RCA ([86], [88]).

Recommendations for future experimental studies and standardization.

- Adopt harmonized RCA characterization (density, absorption, adhered-mortar quantification), report full mix/curing regimes, and pair mechanical testing with standardized transport metrics (e.g., bulk diffusion, migration, electrical resistivity). Prioritize factorial studies (SCMs × fibres × RCA treatment × curing) at multiple RCA levels, and extend to element-scale tests (beams/slabs) and long-term exposure trials ([83–92]).

Figures and Tables

- Figure 18 - Comparative mechanical performance trends of HP-RAC across studies (schematic). Legend (studies): [83]– [92]. Discussed in Section 3; shows normalized RAC/NAC mechanical index vs. RCA replacement. Note the mid-replacement “bump” for HP mixes, consistent with [83–89, 91, 92].
- Figure 19 - Comparative durability performance trends of HP-RAC across studies (schematic). Legend (studies): [83]– [92]. Discussed in Section 3; shows normalized durability index (higher = better) vs. RCA replacement. Treatment + SCMs + low w/b control the decline, aligning with [83–89, 92].
- Notes on Figures 18–19: These are schematic, normalized meta-summaries to visualize cross-study trends (not raw values). They reflect the qualitative directions reported in the cited works and support the comparative narrative in Section 3.
- Table 15 — Key factors influencing HP-RAC outcomes across studies (schematic summary). Displayed below for direct review; compares the presence/strength (MPa) of factors across [83]– [92] and is discussed in Section 3.

Table 15. Illustrative values from the literature: Relative appraisal of HP-RAC with numerical details

Ref	RCA (%)	Compressive Strength (MPa)	n (Number of samples)	SD / Range	Chloride Penetration (Coulombs)	Carbonation Depth (mm)	Freeze-Thaw Mass Loss (%)
[83]	25	42.50	3	±2.10	1800.00	6.20	1.50
[84]	50	39.80	3	±1.80	2100.00	7.50	2.10
[85]	75	36.20	3	±2.00	2500.00	9.00	2.80
[86]	50	44.00	3	±1.50	1700.00	5.80	1.20
[87]	100	32.50	3	±2.30	2900.00	11.00	3.50
[88]	25	45.10	3	±1.70	1600.00	5.50	1.00
[89]	50	41.70	3	±1.90	1900.00	6.80	1.70
[90]	75	37.00	3	±2.20	2300.00	8.50	2.40
[91]	100	34.20	3	±2.40	2700.00	10.20	3.00
[92]	50	43.50	3	±1.60	1750.00	6.00	1.40

Here is Table 16 summarizing the key findings and future decisions for HP-RAC based on the critical analysis and conclusions.

Table 16. Summary of key findings and future decisions for HP-RAC research

Category	Key Findings	Recommendations
Workability	HP-RAC shows reduced slump and flowability due to higher water absorption of recycled aggregates.	Use of superplasticizers, pre-soaking of aggregates, or blended binders (e.g., SCMs) to enhance workability.
Mechanical Strength	Compressive strength is typically 5–20% lower compared to natural aggregate concrete, but high-performance mixes can achieve parity.	Optimize mix design (e.g., lower w/c ratio, supplementary cementitious materials) and hybrid aggregate replacement strategies.
Durability	Chloride penetration, carbonation depth, and freeze-thaw resistance show higher vulnerability in HP-RAC.	Improve pore structure using nano-additives, SCMs, and surface treatment of aggregates.
Microstructure	Weak interfacial transition zone (ITZ) around recycled aggregates affects long-term performance.	Research on nanomaterials, mineral admixtures, and aggregate surface modification to improve ITZ bonding.
Sustainability	Significant reduction in natural aggregate consumption and landfill waste disposal.	Wider adoption in structural and non-structural applications with supportive building codes and standards.
Research Gaps	Limited long-term field performance studies and lack of standardized test protocols.	Future work should focus on full-scale structural applications, durability in aggressive environments, and international standardization.

4.1 Practical Decision Matrix for HP-RAC Mix Design

Table 17 offers a succinct choice matrix that connects important mix design factors to their anticipated mechanical and durability results, thereby increasing the review's practical relevance. When optimizing HP-RAC for structural and durability performance, engineers and practitioners can use this matrix as a quick-reference tool to make well-informed judgments. Strength, stiffness, crack resistance, and durability indicators (chloride ingress, carbonation depth, and freeze-thaw resilience) have been mapped to the typical effects of each parameter, including RCA replacement level, SCM type and dosage, water–binder ratio, fiber addition, and aggregate pre-treatment. The

matrix converts the synthesized information into useful insights for real-world application by combining these relationships.

Table 17. Practical Decision Matrix for HP-RAC Mix Design and Performance Outcomes

Parameter	Recommended Range / Action	Expected Mechanical Outcome	Expected Durability Outcome
RCA Replacement	25–50%	Strength close to NAC (88–100%)	Moderate durability; SCMs recommended
SCM Type & Dosage	Silica fume (8–15%), Fly ash (10–20%), Slag (20–40%)	↑ Compressive strength (up to +12%)	↓ Chloride penetration, ↓ carbonation depth
Water–Binder Ratio	≤ 0.35	↑ Strength, ↑ stiffness	↓ Permeability, improved freeze–thaw resistance
Fiber Addition	Steel/Basalt/PVA fibers (0.5–1.5%)	↑ Tensile & flexural strength	Crack control, improved impact resistance
Aggregate Pre-treatment	CO ₂ carbonation or nano-silica slurry	↑ ITZ density, ↑ strength	↓ Chloride ingress, ↓ carbonation depth

4.2 Novelty of the Study

- **Integrated Critical Review** – Unlike previous reviews that focused separately on either mechanical or durability aspects of recycled aggregate concrete, this study systematically integrates both mechanical and durability performance of HP-RAC, offering a holistic evaluation.
- **Comparative Evaluation Framework** – The study develops comparative figures (Figures 13–18) and tabulated evidence (Tables 15–16), enabling a structured side-by-side assessment of HP-RAC performance against natural aggregate concrete and across multiple experimental investigations.
- **Identification of Key Influencing Factors** – The review highlights interfacial transition zone (ITZ) quality, aggregate pre-treatment, SCM incorporation, and mix design optimization as the most decisive factors in determining the performance of HP-RAC, which has not been comprehensively mapped in earlier studies.
- **Bridging Knowledge Gaps** – This study identifies research limitations such as the lack of long-term field data, standardization issues, and insufficient reporting of durability indices, setting a clear agenda for future research.
- **Sustainability Perspective** – By linking experimental findings to sustainable construction practices, the study emphasizes HP-RAC's role in reducing natural aggregate dependency and construction waste while maintaining structural reliability.
- **Decision-Oriented Roadmap** – The formulation of Table 16 as a concise “findings and decisions” framework provides researchers, engineers, and policymakers with actionable insights for the practical adoption and future development of HP-RAC.

References

- [1] Hasheminezhad A, King D, Ceylan H, Kim S. Comparative life cycle assessment of natural and recycled aggregate concrete: A review. *Sci Total Environ.* 2024;950:175310. <https://doi.org/10.1016/j.scitotenv.2024.175310>
- [2] Wu C, Shi Y, Xu J, Luo M, Lu Y, Zhu D. Experimental study of mechanical properties and theoretical models for recycled fine and coarse aggregate concrete with steel fibers. *Materials (Basel).* 2024;17(12):2933. <https://doi.org/10.3390/ma17122933>
- [3] Jagadesh P, Karthik K, Kalaivani P, Karalar M, Althaqafi E, Madenci E, et al. Examining the influence of recycled aggregates on the fresh and mechanical characteristics of high-strength concrete: A comprehensive review. *Sustainability.* 2024;16(20):9052. <https://doi.org/10.3390/su16209052>

- [4] John NJ, Wanjari S, Patel A. Effects of particle shape and size on strength and durability of recycled concrete aggregates: An experimental and statistical approach. *Constr Build Mater.* 2025;483:141718. <https://doi.org/10.1016/j.conbuildmat.2025.141718>
- [5] Fernandes B, Khodeir M, Perlot C, Carré H, Mindeguia J, La Borderie C. Durability of concrete made with recycled concrete aggregates after exposure to elevated temperatures. *Mater Struct.* 2023;56(1). <https://doi.org/10.1617/s11527-023-02111-1>
- [6] Pacheco J, De Brito J, Chastre C, Evangelista L. Experimental investigation on the variability of the main mechanical properties of concrete produced with coarse recycled concrete aggregates. *Constr Build Mater.* 2019;201:110-120. <https://doi.org/10.1016/j.conbuildmat.2018.12.200>
- [7] Zong S, Chang C, Rem P, Gebremariam AT, Di Maio F, Lu Y. Research on the influence of particle size distribution of high-quality recycled coarse aggregates on the mechanical properties of recycled concrete. *Constr Build Mater.* 2025;465:140253. <https://doi.org/10.1016/j.conbuildmat.2025.140253>
- [8] Sobuz MHR, Datta SD, Akid ASM, Tam VW, Islam S, Rana MJ, et al. Evaluating the effects of recycled concrete aggregate size and concentration on properties of high-strength sustainable concrete. *J King Saud Univ Eng Sci.* 2022. doi: 10.1016/j.jksues.2022.04.004 <https://doi.org/10.1016/j.jksues.2022.04.004>
- [9] Nanya CS, Da Silva Ferreira FG, Da Silva Capuzzo VM. Mechanical and durability properties of recycled aggregate concrete. *Matéria (Rio J).* 2021;26(4). <https://doi.org/10.1590/s1517-707620210004.1373>
- [10] Neupane RP, Imjai T, Makul N, Garcia R, Kim B, Chaudhary S. Use of recycled aggregate concrete in structural members: a review focused on Southeast Asia. *J Asian Archit Build Eng.* 2023;1-24. <https://doi.org/10.1080/13467581.2023.2270029>
- [11] Zaid O, Hashmi SRZ. Experimental study on mechanical performance of recycled fine aggregate concrete reinforced with discarded carbon fibers. *Front Mater.* 2021;8. <https://doi.org/10.3389/fmats.2021.771423>
- [12] Banyai K, Czoboly O, Menyhart K, Orban Z. Influence of aggregate composition on the properties of recycled concrete and improving performance using special additives. *Materials (Basel).* 2025;18(5):1108. <https://doi.org/10.3390/ma18051108>
- [13] Kathirvel P, Kaliyaperumal SRM. Influence of recycled concrete aggregates on the flexural properties of reinforced alkali activated slag concrete. *Constr Build Mater.* 2015;102:51-58. <https://doi.org/10.1016/j.conbuildmat.2015.10.148>
- [14] Barra M, Aponte D, Faleschini F, González-Fontebo A, González-Taboada I. Rheological behavior of recycled aggregate concrete. In: Elsevier eBooks; 2021. p. 505-543. <https://doi.org/10.1016/B978-0-12-820549-5.00017-6>
- [15] Chachar KH, Oad M, Memon BA, Siyal ZA, Siyal KF. Workability and flexural strength of recycled aggregate concrete with steel fibers. *Eng Technol Appl Sci Res.* 2023;13(3):11051-11057. <https://doi.org/10.48084/etasr.5921>
- [16] García-González J, Rodríguez-Robles D, Juan-Valdés A, Pozo JMM, Guerra-Romero MI. Porosity and pore size distribution in recycled concrete. *Mag Concr Res.* 2015;67(22):1214-1221. <https://doi.org/10.1680/macr.14.00218>
- [17] Singh R, Nayak D, Pandey A, Kumar R, Kumar V. Effects of recycled fine aggregates on properties of concrete containing natural or recycled coarse aggregates: A comparative study. *J Build Eng.* 2021;45:103442. <https://doi.org/10.1016/j.jobbe.2021.103442>
- [18] Pham T, Nguyen N, Nguyen T, Nguyen T, Pham T. Effects of superplasticizer and water-binder ratio on mechanical properties of one-part alkali-activated geopolymer concrete. *Buildings.* 2023;13(7):1835. <https://doi.org/10.3390/buildings13071835>
- [19] Pereira P, Evangelista L, De Brito J. The effect of superplasticizers on the mechanical performance of concrete made with fine recycled concrete aggregates. *Cem Concr Compos.* 2012;34(9):1044-1052. <https://doi.org/10.1016/j.cemconcomp.2012.06.009>
- [20] Burgmann S, Breit W. Impact of crushed natural and recycled fine aggregates on fresh and hardened mortar properties. *Constr Mater.* 2023;4(1):37-57. <https://doi.org/10.3390/constrmater4010003>
- [21] Nguyen T, Cherif R, Mahieux P, Lux J, Ait-Mokhtar A, Bastidas-Arteaga E. Artificial intelligence algorithms for prediction and sensitivity analysis of mechanical properties of recycled aggregate concrete: A review. *J Build Eng.* 2023;66:105929. <https://doi.org/10.1016/j.jobbe.2023.105929>
- [22] Limbachiya MC, Leelawat T, Dhir RK. Use of recycled concrete aggregate in high-strength concrete. *Mater Struct.* 2000;33(9):574-580. <https://doi.org/10.1007/BF02480538>
- [23] Xiao J, Li J, Zhang C. Mechanical properties of recycled aggregate concrete under uniaxial loading. *Cem Concr Res.* 2004;35(6):1187-1194. <https://doi.org/10.1016/j.cemconres.2004.09.020>
- [24] Padmini A, Ramamurthy K, Mathews. Influence of parent concrete on the properties of recycled aggregate concrete. *Constr Build Mater.* 2008;23(2):829-836. <https://doi.org/10.1016/j.conbuildmat.2008.03.006>

- [25] Sadik MN, Akter T, Proma PD, Prodhan MAR, Momotaj M. Impact of recycled coarse aggregates on the mechanical properties and durability of concrete. *Eur J Theor Appl Sci.* 2024;2(5):738-759. [https://doi.org/10.59324/eitas.2024.2\(5\).66](https://doi.org/10.59324/eitas.2024.2(5).66)
- [26] Kim J. Influence of quality of recycled aggregates on the mechanical properties of recycled aggregate concretes: An overview. *Constr Build Mater.* 2022;328:127071. <https://doi.org/10.1016/j.conbuildmat.2022.127071>
- [27] Etxeberria M, Marí AR, Vázquez E. Recycled aggregate concrete as structural material. *Mater Struct.* 2006;40(5):529-541. <https://doi.org/10.1617/s11527-006-9161-5>
- [28] Zhu P, Hao Y, Liu H, Wei D, Liu S, Gu L. Durability evaluation of three generations of 100% repeatedly recycled coarse aggregate concrete. *Constr Build Mater.* 2019;210:442-450. <https://doi.org/10.1016/j.conbuildmat.2019.03.203>
- [29] Upshaw M, Cai CS. Critical review of recycled aggregate concrete properties, improvements, and numerical models. *J Mater Civ Eng.* 2020;32(11). [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0003394](https://doi.org/10.1061/(ASCE)MT.1943-5533.0003394)
- [30] Sugrañez R, Álvarez J, Cruz-Yusta M, Mármol I, Morales J, Sánchez L. Controlling microstructure in cement based mortars by adjusting the particle size distribution of the raw materials. *Constr Build Mater.* 2013;41:139-145. <https://doi.org/10.1016/j.conbuildmat.2012.11.090>
- [31] Ajdukiewicz A, Kliszczewicz A. Influence of recycled aggregates on mechanical properties of HS/HPC. *Cem Concr Compos.* 2002;24(2):269-279. [https://doi.org/10.1016/S0958-9465\(01\)00012-9](https://doi.org/10.1016/S0958-9465(01)00012-9)
- [32] Medina C, Zhu W, Howind T, De Rojas MIS, Frías M. Influence of interfacial transition zone on engineering properties of the concrete manufactured with recycled ceramic aggregate. *J Civ Eng Manag.* 2014;21(1):83-93. <https://doi.org/10.3846/13923730.2013.802727>
- [33] Fatiha A, Karim E, Mhamed A, Farid A. Enhancing performance of recycled aggregate concrete with supplementary cementitious materials. *Clean Mater.* 2025;15:100298. <https://doi.org/10.1016/j.clema.2025.100298>
- [34] Duan P, Shui Z, Chen W, Shen C. Effects of metakaolin, silica fume and slag on pore structure, interfacial transition zone and compressive strength of concrete. *Constr Build Mater.* 2013;44:1-6. <https://doi.org/10.1016/j.conbuildmat.2013.02.075>
- [35] Martín-Morales M, Cuenca-Moyano G, Valverde-Espinosa I, Valverde-Palacios I. Effect of recycled aggregate on physical-mechanical properties and durability of vibro-compacted dry-mixed concrete hollow blocks. *Constr Build Mater.* 2017;145:303-310. <https://doi.org/10.1016/j.conbuildmat.2017.04.013>
- [36] Li W, Luo Z, Sun Z, Hu Y, Duan WH. Numerical modelling of plastic-damage response and crack propagation in RAC under uniaxial loading. *Mag Concr Res.* 2018;70(9):459-472. <https://doi.org/10.1680/jmacr.17.00042>
- [37] Bao J, Wang Y, Zhang P, Zhang X, Cui Y. The role of internally incorporated nano-silica in recycled aggregate concrete: Modification of transport properties. *Constr Build Mater.* 2023;371:130790. <https://doi.org/10.1016/j.conbuildmat.2023.130790>
- [38] Khan MS, Ulhaq A, Alosekait DM, Javed MF, Jameel M, Alabduljabbar H, et al. RSM-based optimization of recycled aggregate concrete with pozzolanic materials under high temperatures. *Front Mater.* 2025;12. <https://doi.org/10.3389/fmats.2025.1601597>
- [39] Šádková K, Pommer V, Keppert M, Vejmelková E, Koňáková D. Difficulties in determining the pozzolanic activity of thermally activated lower-grade clays. *Materials (Basel).* 2024;17(20):5093. <https://doi.org/10.3390/ma17205093>
- [40] Shen Z, Zhu H, Meng X. The influence of curing methods on the performance of recycled concrete powder artificial aggregates and concrete. *Constr Build Mater.* 2024;435:136908. <https://doi.org/10.1016/j.conbuildmat.2024.136908>
- [41] Chen B, Rao M, Feng Y. Effects of curing temperature and supplementary cementitious materials on the interfacial transition zone (ITZ) of high-ferrite cement products. *Constr Build Mater.* 2024;425:135920. <https://doi.org/10.1016/j.conbuildmat.2024.135920>
- [42] Zhang H, Zhao Y, Meng T, Shah SP. The modification effects of a nano-silica slurry on microstructure, strength, and strain development of recycled aggregate concrete applied in an enlarged structural test. *Constr Build Mater.* 2015;95:721-735. <https://doi.org/10.1016/j.conbuildmat.2015.07.089>
- [43] Zeng W, Zhao Y, Zheng H, Poon CS. Improvement in corrosion resistance of recycled aggregate concrete by nano silica suspension modification on recycled aggregates. *Cem Concr Compos.* 2019;106:103476. <https://doi.org/10.1016/j.cemconcomp.2019.103476>
- [44] Ali B, Qureshi LA. Durability of recycled aggregate concrete modified with sugarcane molasses. *Constr Build Mater.* 2019;229:116913. <https://doi.org/10.1016/j.conbuildmat.2019.116913>
- [45] Ying J, Chen W, Chen S, Chen B. Chloride ion diffusion in recycled concrete containing slag under biaxial compression. *Constr Build Mater.* 2024;454:139136. <https://doi.org/10.1016/j.conbuildmat.2024.139136>

- [46] Wang Y, Liu Z, Wang Y, Li Q, Gong X, Zhao Y. Effect of recycled aggregate and supplementary cementitious material on mechanical properties and chloride permeability of concrete. *J Clean Prod.* 2022;369:133322. <https://doi.org/10.1016/j.jclepro.2022.133322>
- [47] Fuhaid AA. Effects of recycled and supplemented cementitious materials on corrosion resistance and mechanical properties in reinforced concrete. *J Compos Sci.* 2025;9(9):457. <https://doi.org/10.3390/jcs9090457>
- [48] Zhu C, Liu X, Liu C, Yu W, Bai G. Study on the chloride ion transport mechanism of recycled mixed aggregate concrete based on evolution characteristics of pore structure. *Constr Build Mater.* 2022;353:129101. <https://doi.org/10.1016/j.conbuildmat.2022.129101>
- [49] Xie J, Zhao J, Wang J, Wang C, Huang P, Fang C. Sulfate resistance of recycled aggregate concrete with GGBS and fly ash-based geopolymer. *Materials (Basel).* 2019;12(8):1247. <https://doi.org/10.3390/ma12081247>
- [50] Zhong C, Wang D, Zhang L, Mao W, Xing S, Chen J, et al. Durability analysis of metakaolin recycled concrete under sulphate dry and wet cycle. *Sci Rep.* 2024;14(1). <https://doi.org/10.1038/s41598-024-66803-6>
- [51] Li Y, Yang X, Lou P, Wang R, Li Y, Si Z. Sulfate attack resistance of recycled aggregate concrete with NaOH-solution-treated crumb rubber. *Constr Build Mater.* 2021;287:123044. <https://doi.org/10.1016/j.conbuildmat.2021.123044>
- [52] Chen X, Zhang X, Yan G. Multiscale investigation of modified recycled aggregate concrete on sulfate attack resistance. *Materials (Basel).* 2025;18(7):1450. <https://doi.org/10.3390/ma18071450>
- [53] Ali B, Gulzar MA, Raza A. Effect of sulfate activation of fly ash on mechanical and durability properties of recycled aggregate concrete. *Constr Build Mater.* 2021;277:122329. <https://doi.org/10.1016/j.conbuildmat.2021.122329>
- [54] Wang H, Zhu P, Yan X, Liu H, Zhu L, Wang X. Effect of silica fume on frost resistance and recyclability potential of recycled aggregate concrete under freeze-thaw environment. *Constr Build Mater.* 2023;409:134109. <https://doi.org/10.1016/j.conbuildmat.2023.134109>
- [55] Xu F, Li Z, Ying H, Du B. Effect of metakaolin content on the deterioration resistance of concrete made with recycled fiber-reinforced tailings aggregate under freeze-thaw cycles and sulfate freeze-thaw cycles. *Buildings.* 2025;15(18):3428. <https://doi.org/10.3390/buildings15183428>
- [56] Huda SB, Alam MS. Mechanical and freeze-thaw durability properties of recycled aggregate concrete made with recycled coarse aggregate. *J Mater Civ Eng.* 2015;27(10). [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0001237](https://doi.org/10.1061/(ASCE)MT.1943-5533.0001237)
- [57] Del Bosque IS, Van Den Heede P, De Belie N, De Rojas MS, Medina C. Freeze-thaw resistance of concrete containing mixed aggregate and construction and demolition waste-added cement in water and de-icing salts. *Constr Build Mater.* 2020;259:119772. <https://doi.org/10.1016/j.conbuildmat.2020.119772>
- [58] Liu J, Xu S, Li L, Zhang G. Freeze-thaw cycling of nano-SiO₂-modified recycled aggregate in concretes: hydro-thermal-mechanical modeling. *J Build Eng.* 2024;96:110554. <https://doi.org/10.1016/j.jobbe.2024.110554>
- [59] Pacheco J, De Brito J. Recycled aggregates produced from construction and demolition waste for structural concrete: Constituents, properties and production. *Materials (Basel).* 2021;14(19):5748. <https://doi.org/10.3390/ma14195748>
- [60] Khan MU, Dominic M. Sustainability of concrete using recycled aggregate: a review. *Sustain Agri Food Environ Res.* 2023;10. <https://doi.org/10.7770/safer-V10N1-art2508>
- [61] Lu L. Optimal replacement ratio of recycled concrete aggregate balancing mechanical performance with sustainability: A review. *Buildings.* 2024;14(7):2204. <https://doi.org/10.3390/buildings14072204>
- [62] Letelier V, Hott F, Bustamante M, Wenzel B. Effect of recycled coarse aggregate treated with recycled binder paste coating and accelerated carbonation on mechanical and physical properties of concrete. *J Build Eng.* 2023;82:108311. <https://doi.org/10.1016/j.jobbe.2023.108311>
- [63] Wang Y, Cheng J, Wang J. Flexural performance of recycled concrete beam reinforced with modified basalt fiber and nano-silica. *Case Stud Constr Mater.* 2023;18:e02022. <https://doi.org/10.1016/j.cscm.2023.e02022>
- [64] Gu Z, Wang J, Gao D, Zhao J. Effects of steel fibers on the flexural behavior of recycled concrete beam: Testing and analysis. *J Build Eng.* 2024;85:108718. <https://doi.org/10.1016/j.jobbe.2024.108718>
- [65] Hizia B, Radia L, Sondes K. Bending behavior analysis of recycled aggregate reinforced concrete beams according to CBA 93 and EuroCode2 regulations. *MATEC Web Conf.* 2024;394:02003. <https://doi.org/10.1051/mateconf/202439402003>
- [66] Momeni E, Omidinasab F, Dalvand A, Goodarzimehr V, Eskandari A. Flexural strength of concrete beams made of recycled aggregates: an experimental and soft computing-based study. *Sustainability.* 2022;14(18):11769. <https://doi.org/10.3390/su141811769>

- [67] Ali B, Qureshi LA, Khan SU. Flexural behavior of glass fiber-reinforced recycled aggregate concrete and its impact on the cost and carbon footprint of concrete pavement. *Constr Build Mater.* 2020;262:120820. <https://doi.org/10.1016/j.conbuildmat.2020.120820>
- [68] Pradhan S, Kumar S, Barai SV. Performance of reinforced recycled aggregate concrete beams in flexure: experimental and critical comparative analysis. *Mater Struct.* 2018;51(3). <https://doi.org/10.1617/s11527-018-1185-0>
- [69] Saini BS, Singh S. Flexural fatigue strength prediction of self compacting concrete made with recycled concrete aggregates and blended cements. *Constr Build Mater.* 2020;264:120233. <https://doi.org/10.1016/j.conbuildmat.2020.120233>
- [70] Seara-Paz S, González-Fontboa B, Martínez-Abella F, Eiras-López J. Flexural performance of reinforced concrete beams made with recycled concrete coarse aggregate. *Eng Struct.* 2017;156:32-45. <https://doi.org/10.1016/j.engstruct.2017.11.015>
- [71] Silva R, De Brito J, Dhir R. Tensile strength behaviour of recycled aggregate concrete. *Constr Build Mater.* 2015;83:108-118. <https://doi.org/10.1016/j.conbuildmat.2015.03.034>
- [72] Deng Z, Huang H, Ye B, Xiang P, Li C. Mechanical performance of RAC under true-triaxial compression after high temperatures. *J Mater Civ Eng.* 2020;32(8). [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0003231](https://doi.org/10.1061/(ASCE)MT.1943-5533.0003231)
- [73] Park W, Noguchi T, Shin S, Oh D. Modulus of elasticity of recycled aggregate concrete. *Mag Concr Res.* 2014;67(11):585-591. <https://doi.org/10.1680/macrr.14.00213>
- [74] Gonzalez-Corominas A, Etxeberria M, Poon CS. Influence of steam curing on the pore structures and mechanical properties of fly-ash high performance concrete prepared with recycled aggregates. *Cem Concr Compos.* 2016;71:77-84. <https://doi.org/10.1016/j.cemconcomp.2016.05.010>
- [75] Ramesh RB, Mirza O, Kang W. Mechanical properties of steel fiber reinforced recycled aggregate concrete. *Struct Concr.* 2018;20(2):745-755. <https://doi.org/10.1002/suco.201800156>
- [76] Liang C, Bao J, Gu F, Lu J, Ma Z, Hou S, et al. Determining the importance of recycled aggregate characteristics affecting the elastic modulus of concrete by modeled recycled aggregate concrete: Experiment and numerical simulation. *Cem Concr Compos.* 2025;106118. <https://doi.org/10.1016/j.cemconcomp.2025.106118>
- [77] Zhao JL, Yu T, Teng JG. Stress-strain behavior of FRP-confined recycled aggregate concrete. *J Compos Constr.* 2014;19(3). [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000513](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000513)
- [78] Jamil S, Idrees M, Akbar A, Ahmed W. Investigating the mechanical and durability properties of carbonated recycled aggregate concrete and its performance with SCMs. *Buildings.* 2025;15(2):201. <https://doi.org/10.3390/buildings15020201>
- [79] Liu C, Wang Y, Gao X, Zhang G, Liu H, Ma C, et al. Review of the strengthening methods and mechanical properties of recycled aggregate concrete (RAC). *Crystals.* 2022;12(9):1321. <https://doi.org/10.3390/cryst12091321>
- [80] Xiao J, Poon CS, Wang Y, Zhao Y, Ding T, Geng Y, et al. Fundamental behaviour of recycled aggregate concrete - overview I: strength and deformation. *Mag Concr Res.* 2022;74(19):999-1010. <https://doi.org/10.1680/jmacr.21.00253>
- [81] Rao MC, Bhattacharyya SK, Barai SV. Properties of recycled aggregate concrete. In: Springer transactions in civil and environmental engineering. 2018. p. 83-157. https://doi.org/10.1007/978-981-10-6686-3_4
- [82] Liu X, Zhang X, Yan P. Prediction model for elastic modulus of recycled concrete based on properties of recycled coarse aggregate and cementitious materials. *Case Stud Constr Mater.* 2024;e04058. <https://doi.org/10.1016/j.cscm.2024.e04058>
- [83] Qi B, Gao J, Chen F, Shen D. Chloride penetration into recycled aggregate concrete subjected to wetting-drying cycles and flexural loading. *Constr Build Mater.* 2018;174:130-137. <https://doi.org/10.1016/j.conbuildmat.2018.04.122>
- [84] Etxeberria M, Castillo S. How the carbonation treatment of different types of recycled aggregates affects the properties of concrete. *Sustainability.* 2023;15(4):3169. <https://doi.org/10.3390/su15043169>
- [85] Akbulut ZF, Guler S, Yavuz D, Avcı MS. Toward sustainable construction: A critical review of recycled aggregate concrete properties and future opportunities. *Case Stud Constr Mater.* 2025;23:e05133. <https://doi.org/10.1016/j.cscm.2025.e05133>
- [86] Pereiro-Barceló J, Lenz E, Torres B, Estevan L. Mechanical properties of recycled aggregate concrete reinforced with conventional and recycled steel fibers and exposed to high temperatures. *Constr Build Mater.* 2024;452:138976. <https://doi.org/10.1016/j.conbuildmat.2024.138976>
- [87] Fanijo EO, Kolawole JT, Babafemi AJ, Liu J. A comprehensive review on the use of recycled concrete aggregate for pavement construction: Properties, performance, and sustainability. *Clean Mater.* 2023;9:100199. <https://doi.org/10.1016/j.clema.2023.100199>
- [88] Abushanab A, Vimonsatit V. Cost-effectiveness of reinforced recycled aggregate concrete structures with fly ash and basalt fibres under corrosion: A life cycle cost analysis. *Buildings.* 2025;15(7):1167. <https://doi.org/10.3390/buildings15071167>

- [89] Tejas S, Pasla D. Assessment of mechanical and durability properties of composite cement-based recycled aggregate concrete. *Constr Build Mater.* 2023;387:131620. <https://doi.org/10.1016/j.conbuildmat.2023.131620>
- [90] Rezaeicherati F, Memarzadeh A, Esmailpour A, Fallahnejad H, Ghorbanzadeh A, Nematzadeh M. Durability evaluation and environmental implications of blended cement with colloidal nano-silica for use in recycled fine aggregate concrete: Experimental and theoretical study. *Constr Build Mater.* 2023;402:132926. <https://doi.org/10.1016/j.conbuildmat.2023.132926>
- [91] Kong X, Yao Y, Wu B, Zhang W, He W, Fu Y. The impact resistance and mechanical properties of recycled aggregate concrete with hooked-end and crimped steel fiber. *Materials (Basel).* 2022;15(19):7029. <https://doi.org/10.3390/ma15197029>
- [92] Wang X, Liu Z, Wang Y, Zhang X, Jiang M. Impact behavior of recycled aggregate concrete modified with nano-silica and fiber. *Sci Rep.* 2025;15(1). <https://doi.org/10.1038/s41598-025-04264-1>

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Large-scale in-situ infiltration and load testing on collapsible soils: A case study from Egypt's new administrative capital

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Abstract

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Collapsible soils are often challenging from a geotechnical standpoint and are found in many parts of the world. As urban development rapidly extends into Egypt's desert regions, construction projects increasingly encounter issues related to these problematic soils. In their natural dry state, collapsible soils tend to have relatively high strength. However, when inundated, they undergo major particle rearrangement and sudden volume reduction, which can cause serious structural issues like cracking, tilting, or even failure. In Egypt, sources of wetting might include irrigation of farmland, leaks from sewage systems, or occasional rainfall. This paper discusses a large-scale in-situ test that includes a deep infiltration system aimed at assessing the collapse potential and settlement behavior of collapsible soils at an arid site within Egypt's New Administrative Capital. The location, with its uneven terrain, required extensive cut-and-fill earthworks, with excavated materials reused as engineered fill. After observing excessive settlements that impacted a structure, two in-situ tests were carried out: one on original native soil and another on compacted fill. The tests showed that the native soil settled approximately 2 cm under collapse conditions, while the compacted fill experienced about 9 cm of settlement, indicating the high vulnerability of reworked sand-gravel materials to collapse.

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1. Introduction

Collapsible soils are partially saturated loose-grained soils with low density that can withstand considerable stress in their natural unsaturated state, but upon wetting, these soils usually undergo a radical particle rearrangement accompanied by a decrease in volume [1, 2]. These soils are highly problematic and cover a vast area worldwide, thus requiring special consideration in geotechnical design to prevent potential significant financial losses due to structural damages [3]. The term "collapse" differs from primary settlement or consolidation as it occurs depending on the degree of wetting and has a significant uncontrolled magnitude [4]. At the natural water content, collapsible soil particles are held together by silt grains, capillary tension, silt and clay bonds, as well as cementation [1]. However, when the water content increases, the bond holding the soil particles together weakens or is completely washed, and the collapse mechanism is instigated due to the decrease in the shear strength and/or rearrangement of the soil particles [5,6]. Thus, causing noticeable uncontrolled settlement affecting the structures found upon it [7]. The uncontrolled settlement is mainly due to the non-uniform distribution of the percentage of bond particles within the soil, causing extreme bond sensitivity upon wetting. Another reason for the uncontrolled subsidence (differential settlement) is the unequal increase in soil water content beneath the same structure.

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Collapsible soils exist in many locations across Egypt [8]. Geological data and research confirmed that the dry sand of the Egyptian deserts contains a percentage of fines that can cause potential collapse. Collapsible soils in Egypt are often formed by dried mudflows from rivers and flood streams in shallow water depths, leaving tiny air pockets in the soil voids [9, 10]. Collapsible soils cover huge areas in Siwa Oasis, the north delta, southwest Egypt, extending to North Sudan, the northwest coast of Egypt, along the Red Sea coast, Qattara depressions, and the coastal region of the Sinai Peninsula [11]. For newly developed cities in Egypt, collapsible soils are encountered and cause structural damage in many projects. These areas include 6th of October, 10th of Ramadan City, New Maadi, New Amereya, El Obour, Alex/Cairo desert highway road and Borg El-Arab in Alexandria [2, 12], as well as the huge New Administration Capital City in the south-east of the metropole city of Cairo.

The possible sources of soil wetting that may occur within the lifetime of the structure should be studied to take protective measures so that the structures can be found safely on these problematic soils. In Egypt, the main sources of wetting are mainly manmade, such as landscape irrigation, fracturing/leakage from water pipes or sewer lines [13]. Rainfall represents a small fraction of water sources in Egypt [12]. The non-uniformity of increased water content causes varying collapse potential, leading to a high risk of differential settlement and distortion angle in existing buildings [14]. It is important to classify collapsible soil based on severity with a quantifiable variable known as the collapse potential, CP or I_c , which represents the percentage of sudden volume loss when soil undergoes collapse, determined at any stress level [15]. Basic soil index properties and Atterberg limits can be used to form indirect correlations with the collapse potential. An empirical relation was suggested [16] to define the collapse potential with reference to the natural water content, w_{cn} , the degree of saturation, S_r , the plastic limit, PL, and the plasticity index, PI. Jennings and Knight [17] proposed a guideline for the type of soil, grain size distribution and critical degree of saturation.

Researchers [18, 19] also utilized standard penetration (SPT) values, cone penetration (CPT) tip resistance and seismic velocity to provide correlations with the collapse potential, but the results were rather poor. For preliminary identification of collapsible soils, the in-situ density was plotted against the liquid limit [20]. A detailed review of the different criteria and indirect methods to estimate collapse potential can be found in [11, 21]. These correlations are mostly for identification purposes only. For settlement estimation, laboratory or field tests should be performed to obtain quantitative data [22]. For fine grained soils, the collapse potential can be determined from laboratory oedometer tests, either single or double [17, 23]. However, for coarse gravelly soils, standard laboratory oedometer tests are unsuitable for capturing actual field-scale behaviour due to sample size limitations that are not representative of the in-situ conditions [24] and differences in collapse mechanisms [25]. Field settlements are consistently and significantly greater than laboratory predictions. This discrepancy is especially pronounced in complex, heterogeneous field conditions [26]. The traditional oedometer apparatus dimensions require modifications and design of a large-scale oedometer apparatus to capture the collapse mechanism of coarse grained gravelly soils [24, 26]. Attempts to capture the collapse settlement of sandy gravelly soils based on large-scale direct shear tests using artificial neural networks (ANN), multi-gene genetic programming (MGGP) and evolutionary polynomial regression (EPR models) are presented in [27-29].

Sensitivity classification criteria to judge the severity of foundation problems with reference to collapse potential were proposed in many publications [17], as well as a mathematical equation [30] to show the effect of relative compaction, RC, applied normal stress, and degree of saturation on the collapse potential. Other lab tests are occasionally conducted, such as triaxial test with suction control [31], suction-controlled oedometer tests using axis translation [32-35]. To obtain more realistic results that simulate the in-situ conditions, it is preferred to conduct field tests. Field tests can give more realistic results, especially in the case of gravelly soils where undisturbed samples cannot be gained even from open pits. However, limited research could be found considering field tests to determine the collapse potential [36-40]. The most effective methodology for testing soil collapsibility in the field is the in-situ collapse test (ISCP) [36], which is essentially a plate load test with larger dimensions to permit testing of a relatively large zone of the collapsible soil. The initial test was further modified [41] to account for the required normal stress acting on

the foundation. The test is conducted at natural water content conditions, and once the desired stress level is reached, water is added to the soil pit to increase the degree of saturation. The collapse potential can be determined by plotting the stress-settlement relationship from the obtained test data. In continuation, large-scale plate load tests were carried out in Greece to determine the collapse potential of collapsible soils at the subsurface [42]. Seismic dilatometer test [43] can also be utilized to determine the collapse potential. In this paper, a new large-scale in-situ test with a deep infiltration system is elaborated. This study addresses a critical gap in practice: the inability to rapidly assess the long-term, deep saturation collapse risk of foundations within a practical timeframe. While previous research has relied on shallow infiltration limited to subsurface saturation [2, 42, 44, 45], this work introduces a novel, integrated field methodology that accelerates deep saturation to simulate long-term performance in days rather than months or years. By simultaneously applying deep infiltration wells (10m depth) and surface ponding directly beneath a loaded, full-scale foundation pad, this study directly triggers and measures the full collapse mechanism that would otherwise develop slowly over years. This approach yields a major key advance by providing a definitive performance comparison proving that reworked engineering fill is significantly more vulnerable to collapse than native soil. The work thus provides a prescriptive, accelerated testing framework to prevent potential risks of foundation design in arid, collapsible deposits.

2. New Administration Capital City Case Study

2.1. General Site Description

The site location for the conducted large-scale tests was in the New Administration Capital City (NACC) in Egypt. This specific area was chosen as it exists in an arid and uninhabited region that is currently under urbanization and development. Geological studies show that the site is located within the Red Mountain formations in the Cairo-Suez district [46]. The natural soil at the studied site is composed of red to reddish brown, coarse to fine-grained gravel with varying amounts of fines and iron oxides.

A new residential compound containing villas, a services building and internal roads is under construction at the studied zone. Due to the mountain terrain, the natural topography had elevation differences of more than 10 meters. Before construction, earthworks (cut and fill) were conducted to reach the required architectural platform levels. The fill material consisted of recompacted native soil material obtained from the cut regions. The compacted engineering fill varied in thickness according to the location and reached a maximum value of about 10 meters. After completing most of the villas, significant cracks and settlement were observed in some buildings, which were attributed to water ingress from leakage of the temporary sewer tank located near the damaged villas. The observed settlement within one building ranged from 20 to 120 mm, along with tilting and severe cracks.

A primary assumption for the large settlement could be induced soil collapse due to infiltration of the sewage water below the foundation. A large-scale in-situ test was designed and conducted to determine the collapse potential of cut and fill areas at the site location.

2.2. Geophysics Investigation

A two-dimensional resistivity imaging survey was performed to investigate the project area. A total of three imaging profiles were surveyed and studied at different locations within the distressed zone to detect the presence of waterlogging in the soil subsurface layers, as shown in Fig. 1.

The conducted geophysical investigation has shown three separate zones of the subsoil that have different degrees of saturation. A sample for the 2-D geoelectrical resistivity imaging for Profile No. II (along points 4, 5 and 6) is shown in Fig. 2. These three zones were detected at different depths and different horizontal extensions along the three geophysical sections. The observed heterogeneity of the degree of saturation is due to the distance between the observed location and the source of water, as well as due to the soil heterogeneity regarding the amount and type of fines, and hence the hydraulic conductivity that affects the seepage paths.

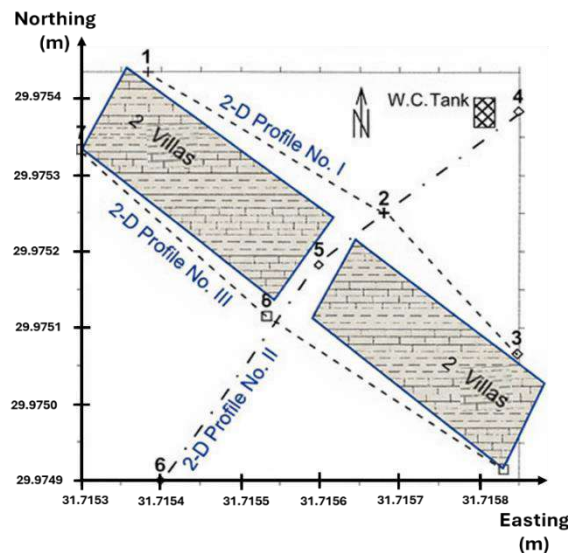


Fig. 1. Location map of the three imaging profiles

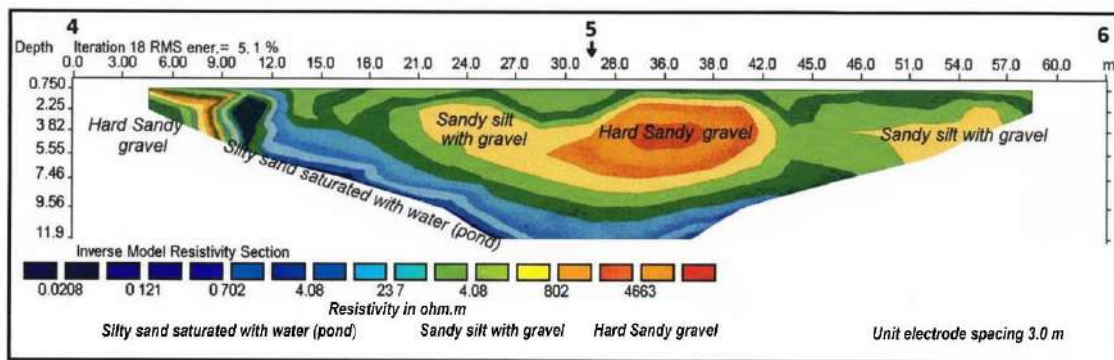


Fig. 2. Two- dimensional geoelectrical resistivity imaging for Profile No. II (along points 4, 5 and 6)

2.3. Site Investigation and Laboratory Tests

A total of 50 boreholes with a depth of 10 m from the ground surface were conducted during the initial site investigation to provide soil samples for lab testing. From the borehole results, no initial groundwater table was present. Sieve analysis results confirmed that the entire depth of the borehole consisted of sandy gravel with silt. The average percentage of fines in all soil samples varies between 11% and 19% (Fig. 3). The compaction state of the reworked engineering fill was determined using laboratory Modified Proctor tests, yielding an average optimum moisture content (OMC) of 6.5% and a maximum dry density (MDD) of 22.1 kN/m³, shown in Fig. 4. Field quality control using sand cone tests confirmed in-situ dry densities corresponding to 90–95% of the Proctor MDD.

A summary of the soil index properties is presented in Table 1, while detailed test procedures are described in [47]. The soil consists of a coarse-grained skeleton ($\approx 81\%$ gravel and sand), forming a highly porous framework [48], with highly plastic fines ($LL \approx 86.3\%$, $PI \approx 43.9\%$) acting as a bonding agent under unsaturated conditions. At the low natural water content of the arid site, high matric suction maintains a strong but metastable structure. Collapse occurs when water infiltration reduces suction and weakens interparticle bonds, allowing particle rearrangement and densification. Previous studies indicate that reworked compacted soils are generally susceptible to collapse under appropriate loading and wetting conditions [35], with the initial degree of compaction being a controlling factor [49]. The in-situ dry density and natural water content indicate that the tested soil is dry of the optimum with a relative compaction of about 90%, thus highly susceptible to collapse [50, 51].

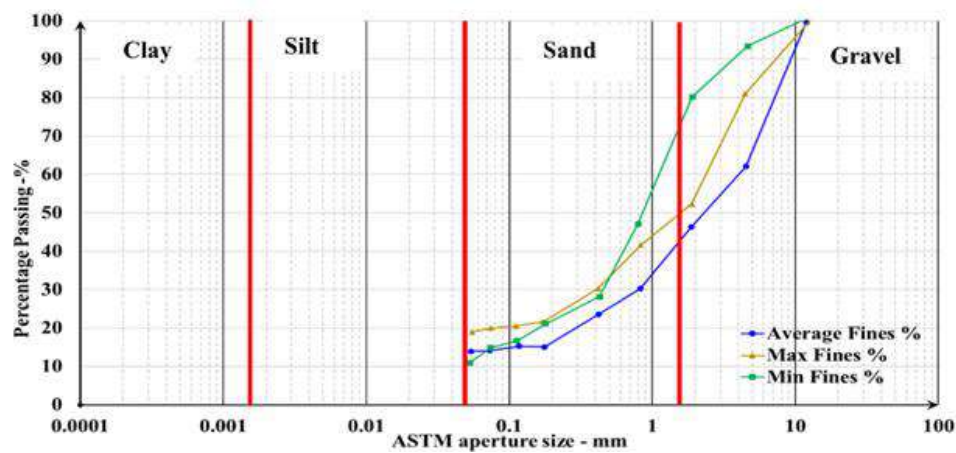


Fig. 3. Average sieve analysis for site location

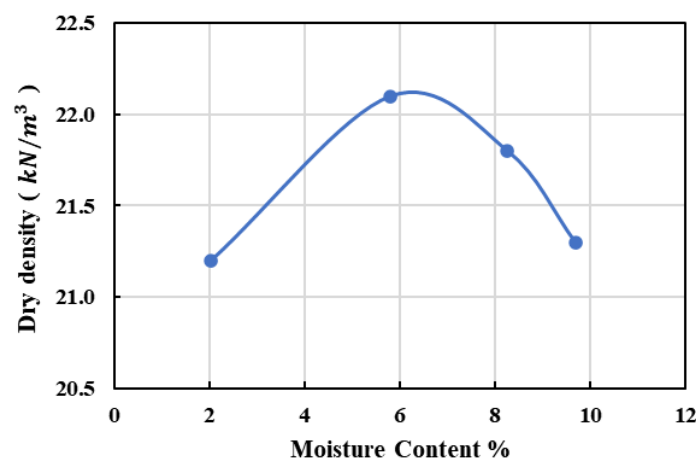


Fig. 4. Modified proctor compaction curve

Table 1. Summary of soil index properties

Property	Unit	Range	Average Value
Gravel	%	(51.0 - 60.0)	52.0
Sand	%	(15.0-39.0)	29.0
Fines	%	(10.0 – 25.0)	19.0
Specific Gravity (G_s)	---	(2.6 – 2.9)	2.8
Natural water content (w_c)	%	(1.0 - 2.5)	2.0
Dry Density (γ_d)	kN/m ³	(16.5 – 19.8)	19.0
Optimum Moisture Content (OMC)	%	(6.0 - 7.0)	6.5
Max. Dry Density (MDD)	kN/m ³	(20.0 -23.0)	22.0
Liquid Limit (LL)	%	(84.0 - 87.0)	86.3
Plastic Limit (PL)	%	(41.0 - 42.7)	42.4
Plasticity Index (PI)	%	(43.0 - 44.3)	43.9

In addition, to emphasize the severity of the issue, sieve analysis data was obtained from boreholes conducted with a depth of 30 m for other mega projects throughout the area of the NACC. The borehole logs indicated that cemented sand, sandy gravel and silty sand covered large areas in the NACC. For relevance, only data to a depth of 10m from the ground surface is presented. Fig. 5 shows a sample of the percentage of fines present in the specified soil layers. Fig. 6 shows a histogram of the fines' percentage distribution. From these two figures, the percentage of fines from the sample data ranges from about 4% to 50%. The probability of constructing foundation projects upon soil with 13% fines, the same average fine percentage in the studied area, is about 30%. Therefore, the risk of dealing with collapsible soils in the NACC is relatively high.

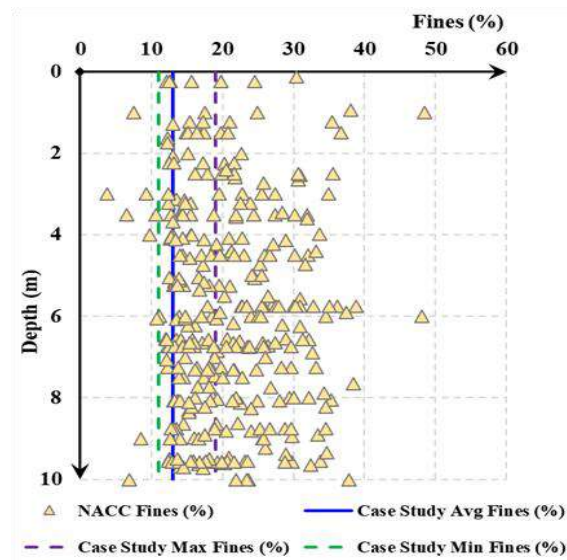


Fig. 5. Fines percentage throughout NACC and at the case study location

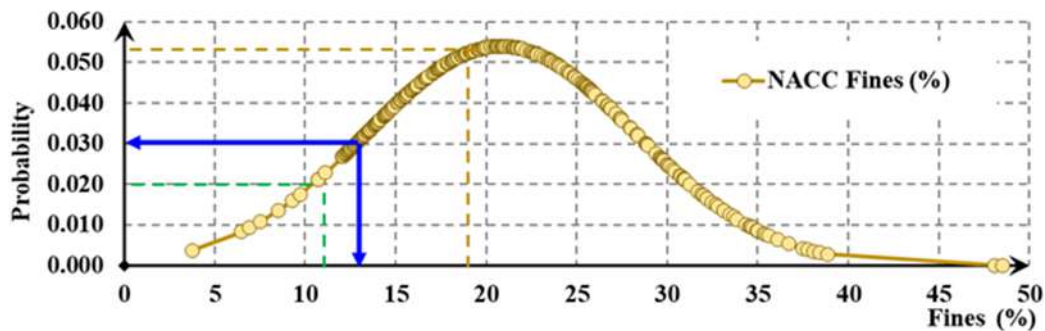


Fig. 6. Statistical analysis of fines percentage from NACC sample data

2.4. Static Plate Load Tests (SPLT)

Before the execution of the large-scale in-situ test, static plate load tests (SPLTs) were performed. A total of five tests, with a plate diameter of 50 cm, were carried out on the reworked engineering fill material in the zone surrounding the damaged villa to study the reason for the severe observed damage. Fig. 7 shows the settlement values under constant load in the dry state and at the end of inundation. The settlement values vary significantly, not only at the end of the wetting process, but in the dry state for each tested sample, indicating that the soil is rather heterogeneous.

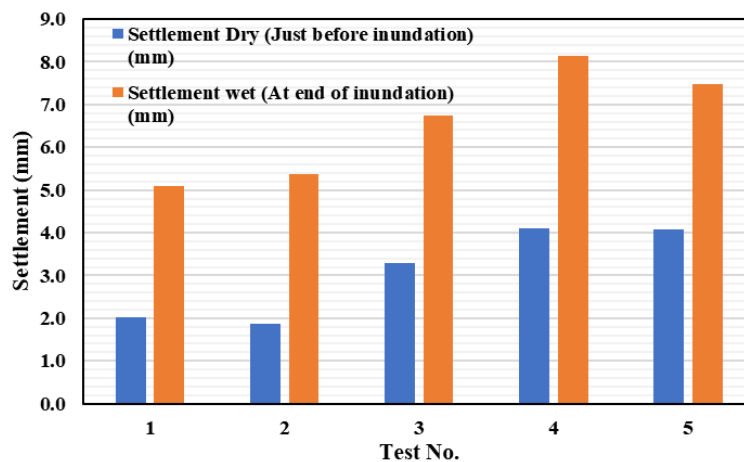


Fig. 7. SPLT Settlement values at constant load just before and at the end of inundation

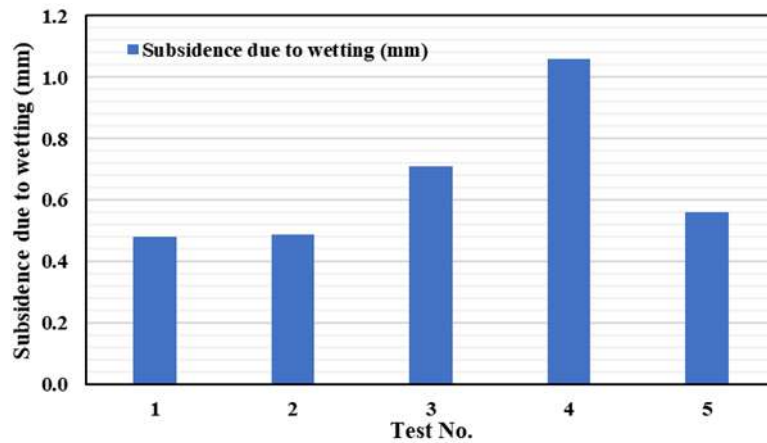


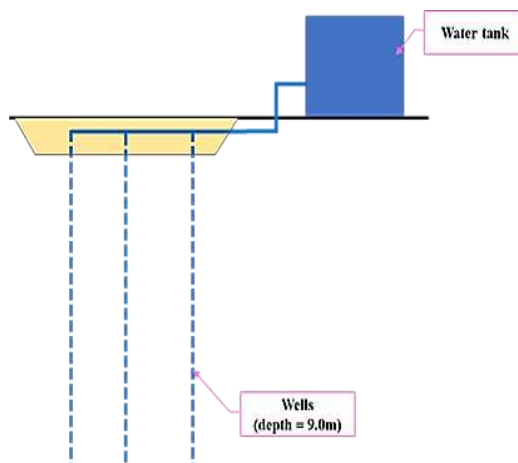
Fig. 8. Sudden subsidence at constant load due to water only from SPLT

The measured settlement values after wetting do not explain the collapse mechanism, as the time is limited to some hours, and hence the water has not penetrated the whole thickness of the collapsible layer. The sudden subsidence (difference between settlement values just before and just after inundation) under constant load for all tests is shown in Fig. 8. The percentage of increase in settlement due to wetting ranges from 13% to 26%. These values approve the high soil heterogeneity and hence its high sensitivity to wetting. Therefore, large-scale in-situ tests were performed to get a better understanding of the problem.

3. Large-Scale In-Situ Test

3.1. Test Setup

The setup for the proposed large-scale in-situ test consisted of 2 main aspects: a loading system and a wetting system. The wetting system was subdivided into subsurface wetting and deep infiltration. The importance of the deep infiltration system was to accelerate the soaking of the relatively thick fill layer (≈ 10 m), which is highly sensitive to small increases in water content depending on fines content and compaction degree. Five wells, approximately 9 m deep, were drilled at the start of the test setup, as shown in Fig. 9. These wells were equipped with perforated PVC pipes connected to a water tank via a valve to control the amount of water being fed to the system. Then the test location was prepped for the installation of the subsurface water feeding system, which consisted of horizontal perforated pipes in a filter layer connected to the same water tank via a separate valve, as shown in Fig. 10.



(a)



(b)

Fig. 9. Deep infiltration system (a) Schematic (b) Photograph

To ensure the pipes remained unaffected by settlement, they were overlain with a thin layer of sand, as shown in Fig. 11. After the completion of the wetting system, preparation for the loading system began. The loading system consisted of a reinforced square concrete footing with dimensions (2 m x 2 m x 0.4 m), which was poured above the subsurface layer, as shown in Fig. 12.

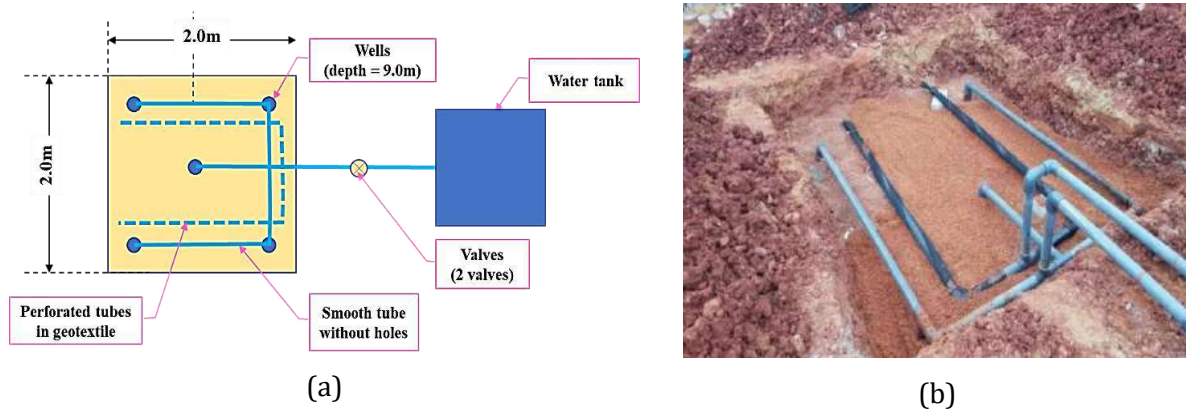


Fig. 10. Subsurface wetting system (a) Schematic (b) Photograph



Fig. 11. A thin sand layer to prevent the settlement of pipes



Fig. 12. Construction of RC footing (a) Schematic (b) Photograph

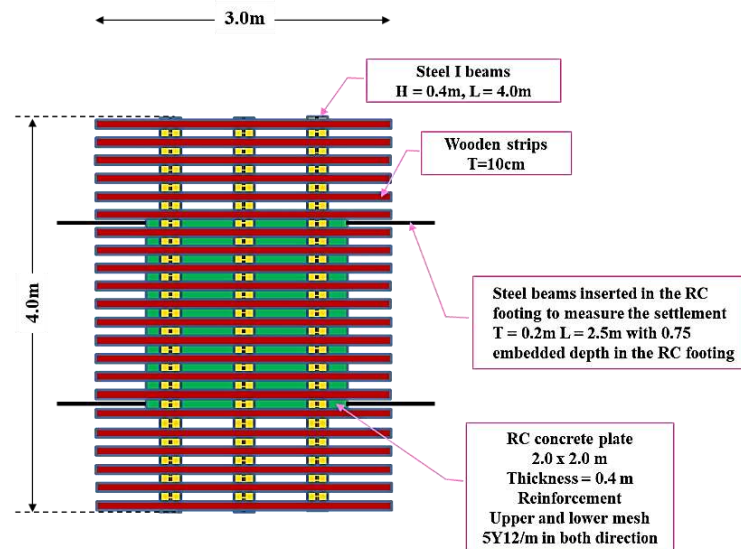


Fig. 13. Plan for the loading system

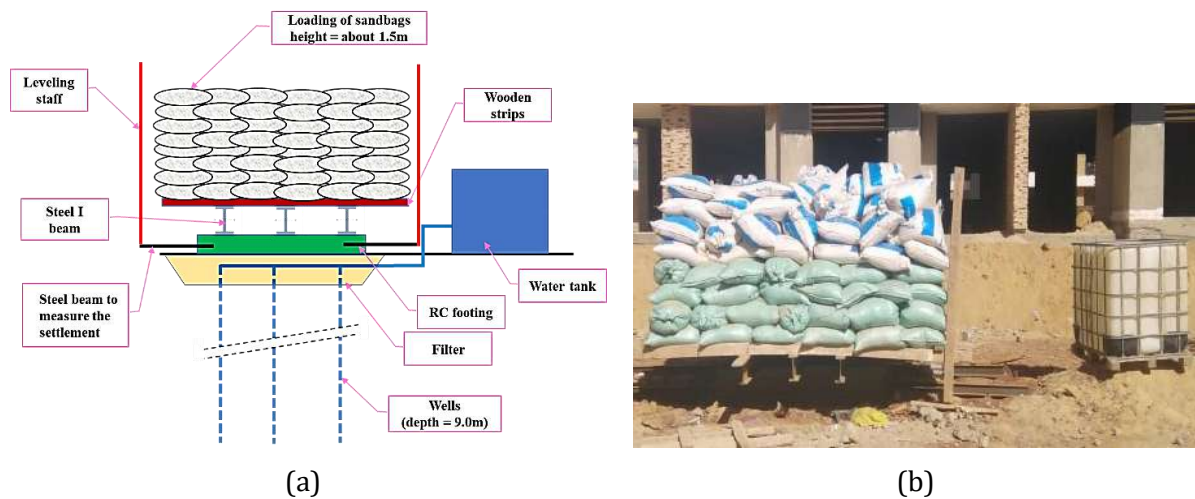


Fig. 14. Final test setup (a) Schematic (b) Photograph

The applied stress was approximately 100 kPa to simulate the foundation's allowable stresses. Steel beams were embedded in the RC footing with an anchorage length of 0.75 m to act as a guideline when measuring the settlement. Then, a (3 m x 4 m) platform, as shown in Fig. 13, consisting of equally spaced wooden strips with a thickness of 0.1 m, was constructed above the RC footing. The platform was supported by steel I-beams with a height of 0.4 m and a length equal to 4 m. Finally, sandbags with a height of about 1.5 m were loaded onto the platform. Fig.14 shows the final test setup.

3.2. Test Procedure

The previously described large-scale in-situ test was set up at two locations: one at a cut zone and one at a fill zone near the building with a reported 12 cm settlement value. Settlement readings were taken at various milestones with reference to points MP-01, MP-02, MP-03 and MP-04 shown on the layout depicted in Fig. 12a. The process for the settlement readings is shown in Fig. 15. Vertical settlements at the four reference points (MP-01, MP-02, MP-03, and MP-04) were monitored manually using precise engineering leveling. All readings were referenced to a stable benchmark system located outside the zone of influence of the test. Initial readings for the settlement were taken seven days after pouring the RC footing, just before loading with sandbags, and just after finishing loading with sandbags. Then the watering system was opened, and settlement readings were also recorded.

A controlled infiltration test was conducted to saturate the soil beneath the RC footing extending to the entire depth of the constructed deep wells. The process was meticulously monitored by recording the time and amount of infiltrated water at each stage. The infiltration process was monitored continuously. Approximate saturation was determined based on the field observations regarding the cessation of rapid infiltration and the establishment of sustained surface ponding for a period of at least 3 hours. The methodology for confirming approximate saturation through controlled flooding and observable field indicators is consistent with established practices for large-scale collapsible soil testing [2]. Settlement readings continued periodically every day for 2 weeks after the infiltration process began until the rate of settlement reached less than about 1 mm/10 days. The obtained results were used to plot the time-settlement relation.

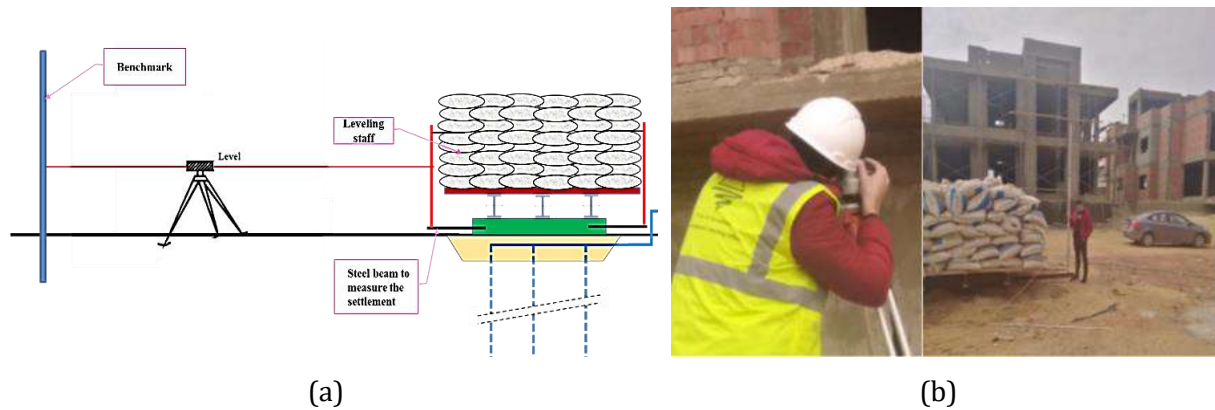


Fig. 15. Settlement measurements process (a) Schematic (b) Photograph

4. Results and Discussion

4.1. Fill Zone (Recompacted Backfill)

For the fill zone, the baseline settlement values were taken on (Day 0), seven days after pouring the RC plate, which was also the start of loading with the sandbags, indicated by station 1 in Fig. 16. At milestone 2 (Day 13), just after finishing loading with the sandbags, the average relative settlement for the four reference points MP-01, MP-02, MP-03 and MP-04 was 10 mm. On Day 19, the soaking process began, Milestone 3, and the settlement values were recorded just before and just after the water valve was opened. Fig. 16 shows a substantial increase in the settlement values upon wetting, which was expected as a typical behavior of collapsible soils. These results indicate that despite the controlled field test conditions, equal water feeding and symmetric loading process, significant settlement variation occurred within the 4 points of the (2×2) m plate.

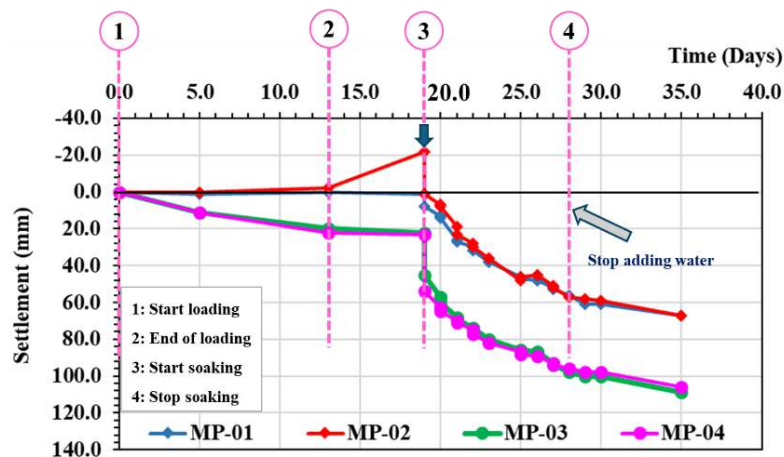


Fig. 16. Time vs. settlement curve for recompacted backfill zone for the four reference points : MP-01, MP-02, MP-03 and MP-04

Therefore, the structure of the cemented sandy gravel soil and the bond material result in an unequal effect on the settlement values upon wetting and lead to nonhomogeneous performance. Between Milestone 2 and Milestone 3, Point MP-02 experienced upward movement due to the slight rotation of the concrete plate resulting from the non-uniform settlement within the small footing. On Day 28, the soaking system was turned off due to concerns about its effect on surrounding structures, as 35 m³ of water had been used without reaching full saturation (convergence was not reached). Nonetheless, readings continued until Day 35. The maximum total settlement recorded was 67 mm for MP-01 and MP-02, 109 mm for MP-03 and 106 mm for MP-04. The average total settlement was 87.2 mm.

4.2. Cut Zone (Native Soil)

The same process applied to the fill zone was performed on the cut zone (Native Soil) with Day 0 as the baseline for the settlement readings, taken seven days after pouring the RC plate. The loading of the concrete plate started on the same day, as shown in Fig. 17. At the end of the loading period, indicated by Milestone 2 in Fig. 17, the average relative settlement was about 4 mm. At constant load, the settlement continued to increase, and readings were taken. On Day 20, the wetting system was activated, and readings were taken just before and just after the start of the wetting process. Readings were taken every day afterwards until Day 31, where the average relative settlement reached a value of 21 mm. Only 6 m³ of water was required to reach the fully saturated state (convergence was reached).

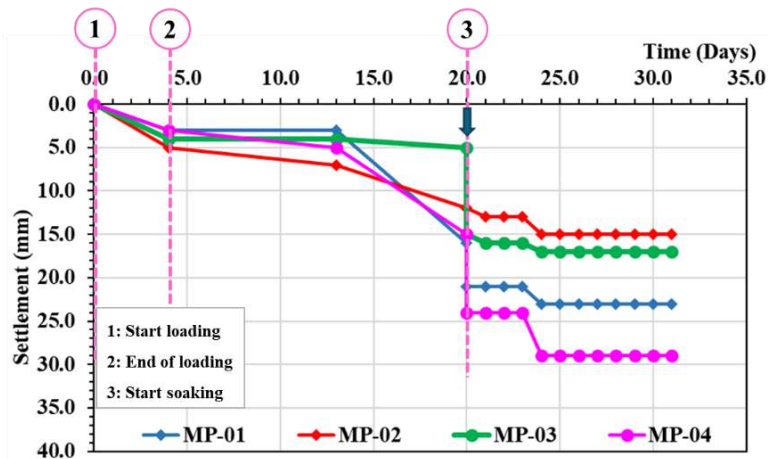


Fig. 17. Time vs. settlement curve for cut (native) zone for the four reference points : MP-01, MP-02, MP-03 and MP-04

4.3. Comparison between Native Soil and Reworked Backfill Zones

4.3.1 Infiltration Rate

The combined subsurface and deep infiltration system aimed to achieve a near-saturated condition, indicated by a constant infiltration rate and stabilization of settlement. The progression of infiltration rates with time for the native soil and reworked backfill zones is shown in Fig. 18, while the corresponding accumulative absorbed water is presented in Fig. 19.

In the native soil zone, upon applying the wetting system, rapid infiltration (2m³/day) was exhibited, reflecting initial pore water absorption. Crucially, the rate of absorbed water decreased until infiltration ceased on the third day, resulting in immediate surface ponding, recorded as fully saturated. This behavior is indicated in Fig. 18 by the transition to zero infiltration rate. This persistent inability to infiltrate additional water over three consecutive days is direct field evidence that the soil's field capacity was exceeded, and a state of approximate saturation was achieved (after approximately 3 m³ of applied water), shown in Fig. 19, and maintained within the influence zone of the tested area. The water volume applied after ponding began was 3 m³, ensuring that the saturated condition was pervasive. The low permeability observed upon saturation is attributed to the high fines content, where the mobilized fines migrate, partially infilling the inter-particle voids

and reducing permeability. Therefore, in native soil a smear zone with low permeability was developed around each well preventing water flow and affecting the performance of native soils outside the smear zones.

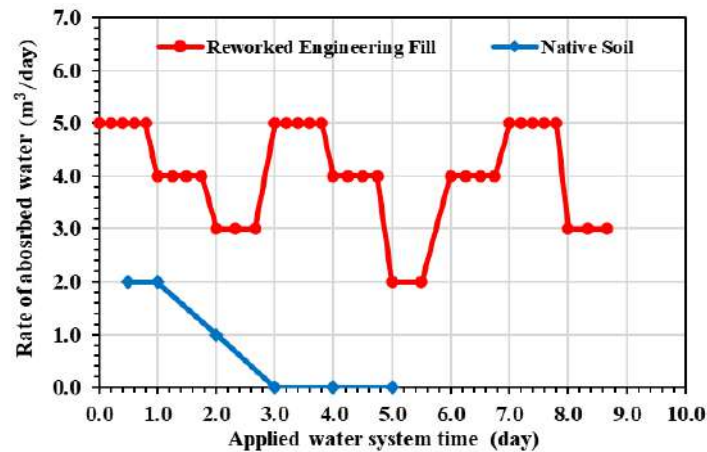


Fig. 18. Rate of absorbed water with time for both the native soil and reworked backfill zones

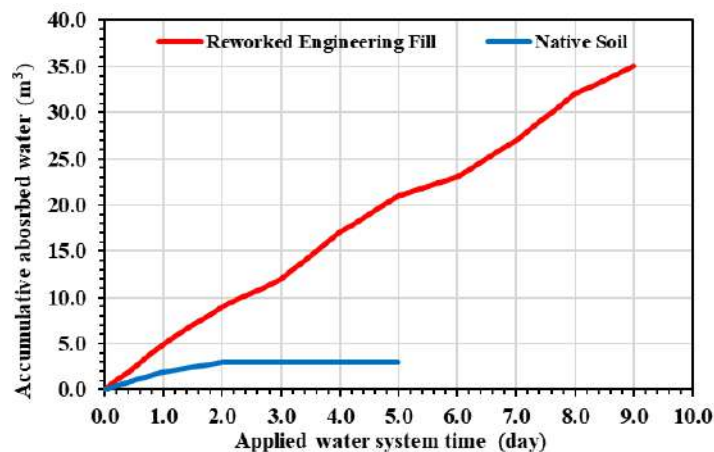


Fig. 19. Applied water system time vs. accumulative absorbed water for both the native soil and reworked backfill zones

In contrast, the reworked engineering fill zone exhibited variable but consistently finite infiltration rates throughout the test, never reaching a state of full saturation or ponding, despite applying a total water volume of 35 m³. This behavior demonstrates the dynamics of unsaturated flow in the reworked soil deposit. The highly variable infiltration rates, shown in Fig. 18 and expressed as variable slopes in Fig. 19, are a direct signature of soil heterogeneity as well as the repeated cycle of smear zone development (where soil is remolded to a lower permeability) and degradation by infiltrating water. This key difference in hydrological response is critical for interpreting the subsequent load test results.

4.3.2 Time-Settlement Relations

The average time-settlement curves for both the cut (native) and fill cases were plotted on the same graph, as shown in Fig. 20. For the fill zone, the upward movement values were neglected, and the settlement curve was extrapolated from Day 13 to Day 19, the start of the infiltration process. From the graph below, it can be noticed that for the native soil zone, the settlement performance with time due to water infiltration is stabilized, and convergence values are observed. The average additional measured settlement due to soaking is about 8 mm. However, for the fill zone, the settlement performance with time due to water infiltration has not stabilized, and the settlement kept increasing with time even after the water valve was closed on Day 28. The average additional settlement due to soaking is about 80 mm.

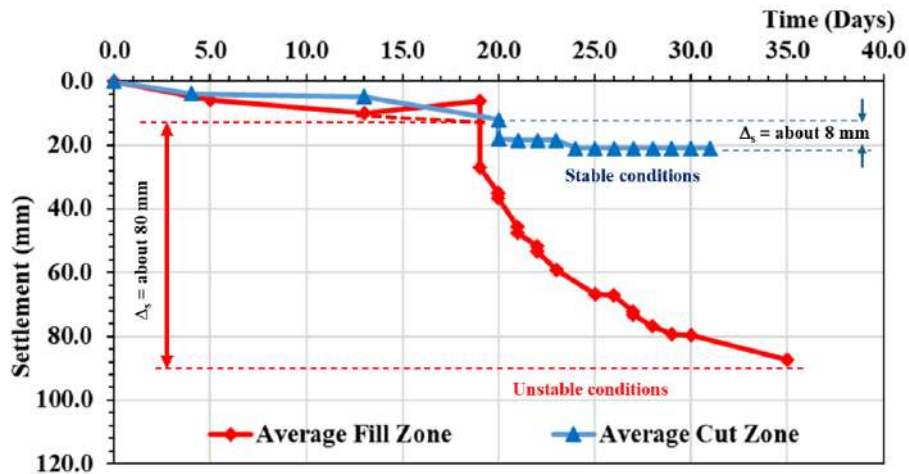


Fig. 20. Comparison between average settlement vs. time curve for native and fill zones

4.3.3 Settlement and Serviceability Criteria

A comparison between settlement and angular distortion values at different loading stages for native and engineering fill zones is shown in Table 2. The initial coordinates of the four reference points (MP-01, MP-02, MP-03 and MP-04) with respect to the center of the RC footing is shown in Fig. 21. Due to the rigidity of the RC footing, the angular distortion values are obtained from the overall rotation caused by average differential settlement between opposite edges. According to estimated average angular distortion values, there is a very high risk for structural damage according to allowable serviceability limit states [52], since β is greater than 1:300.

Table 2. Comparison between settlement and angular distortion values at different loading stages for native and reworked compacted backfill zones

Item	Case	Unit	Native Soil	Engineering Fill
Average Settlement	Just Before Wetting	mm	10.00	12.00
	After Completion of Wetting Process	mm	23.00	86.50
	Percentage of Increase	%	130.00	620.83
Average Angular Distortion (β)	Just Before Wetting	---	1:350	1:90
	After Completion of Wetting Process	---	1:280	1:95

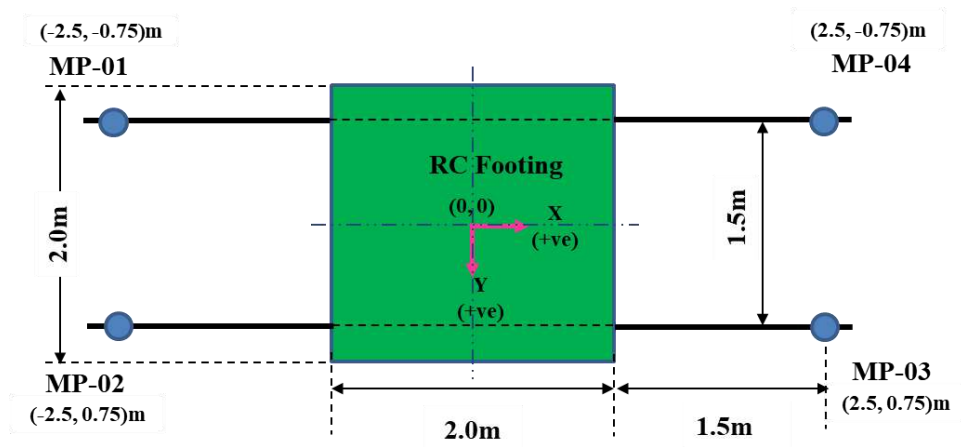


Fig. 21. The initial coordinates of the four reference points with respect to the center of the RC footing for settlement and angular distortion measurements

Table 2 shows that at the dry stage (before starting the wetting process), the average settlement is rather identical for both native as well as reworked compacted soil. The slight difference in behavior is believed to be attributed to the compaction process. Both the native soil and the reworked compacted backfill exhibit collapse upon wetting; however, the collapse potential of the reworked backfill is at least four times greater than that of the native soil. This marked difference is attributed to the disturbance and destruction of the native soil's naturally developed honeycomb fabric, which evolved over geological time. Reworking and recompacting using a vibratory smooth roller disrupt this metastable structure and creates a denser yet more water-sensitive fabric, thereby significantly increasing susceptibility to wetting-induced collapse.

4.3.4 Uncertainty Treatment and its Effect on Engineering Conclusions

Settlement was measured using precise engineering levelling with a vertical accuracy of ± 1.0 mm per reading. Instrument calibration, temperature corrections, balanced sight lengths, and early-morning observations were employed to minimize systematic and environmental errors. Measurement uncertainty was propagated through successive elevation differences, resulting in an estimated uncertainty of ± 1.5 mm for differential settlement between monitoring points, which directly affects the angular distortion values. For a measured differential settlement of 15 mm over a 1.5 m foundation edge, the calculated angular distortion is 1/100. However, the actual differential settlement could be as low as 13.5 mm or as high as 16.5 mm due to measurement uncertainty ($15\text{mm} \pm 1.5\text{mm}$). This translates to a possible angular distortion range of approximately 1/111 to 1/91. The uncertainty ranges (confidence intervals) were considered in the interpretation of settlement-time curves and angular distortion calculations, confirming that the observed deformation trends exceed measurement noise. Consequently, measurement uncertainty does not affect the engineering conclusions regarding serviceability performance.

4.3.5 Numerical Simulation

The large-scale in-situ tests were simulated in Plaxis 3D 2024.1 [53] by implementing the Barcelona Basic Model (BBM) [54] to capture the unsaturated and collapse behavior. The 3D model consisted of 10-noded elements with a maximum, minimum and average mesh element size of 6m, 0.14m and 2m respectively, with a refined zone beneath the foundation and surrounding the deep well to ensure accuracy in the high-stress region. The geometry was designed to be sufficiently large to minimize boundary effects ($35 \times 35 \times 20$) m. The boundary conditions were normally fixed along the X and Y axes, fully fixed at the bottom (Zmin) and free at the top (Zmax). The geometry of the models is shown in Fig. 22, whereas the BBM parameters are shown in Table 3. In the BBM, a single set of parameters defines the soil for both wet and dry conditions, as the main feature of the BBM is the possibility to account for a smooth transition from a partially saturated to a fully saturated state [53]. The performance at a certain degree of saturation is controlled by the soil water characteristic curve (SWCC). The estimated SWCC for sandy gravel with void ratio = 0.5 and specific gravity = 2.80 is shown in Fig. 23 based on data from literature [55].

Table 3. Barcelona Basic Model (BBM) parameters for native soil and reworked engineering fill

Symbol	Unit	Description	Native Soil	Reworked Engineering Fill
ν'	---	Poisson's ratio	0.2000	0.2000
κ	---	Slope of the swelling line in (ν -ln p') plane	0.0010	0.0020
λ	---	Slope of the compression line at full saturation in (ν -ln p') plane	0.0043	0.0190
κ_s	---	Slope of the swelling line in (ν -ln s) plane	0.0006	0.0018
k_s	---	Tensile strength due to suction	0.1000	0.1000
ϕ	degree	Friction angle at critical state	38.0000	38.0000
e_0	---	Initial void ratio	0.5000	0.5000
p_r	kPa	Reference effective pressure	2.0000	2.0000
r	---	Soil stiffness with suction	0.5000	0.3000
β	kPa ⁻¹	Rate of increase of soil stiffness with suction	0.0100	0.0200
α	---	Coefficient of the plastic potential	0.4600	0.4600

The settlement values just before and after completion of the wetting process from the Plaxis simulations are plotted against the actual in-situ values for both native and reworked engineering backfill with an error bandwidth of $\pm 10\%$, shown in Fig. 24. The model simulations show very good agreement with the in-situ settlement values with R^2 (coefficient of determination) = 0.99, MAE (mean absolute error) = 1.58 mm and RMSE (root mean square error) = 1.80 mm, indicating that the implemented model is capable of capturing the actual behavior and can be used to predict expected settlement in the design stage for future projects.

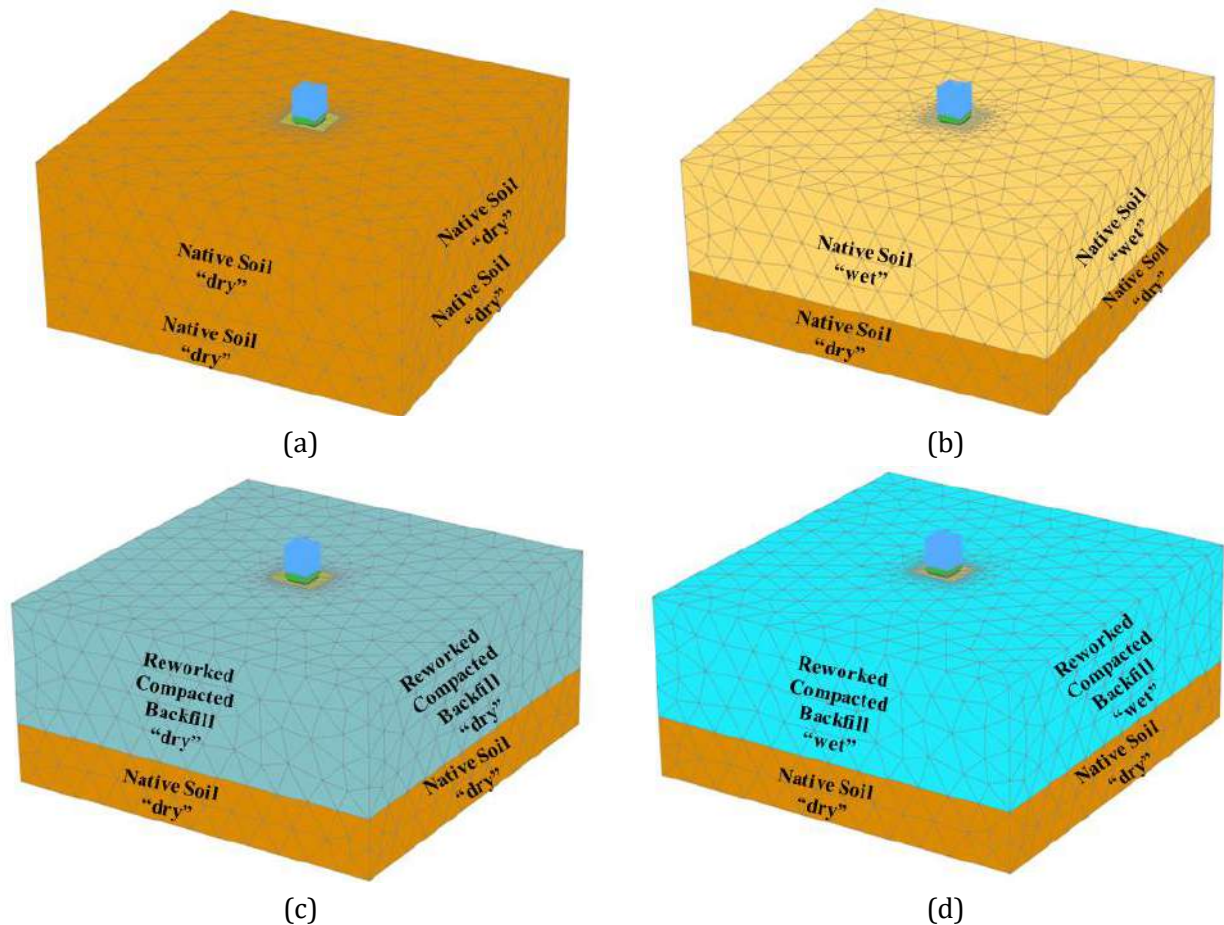


Fig. 22. Plaxis 3D model simulations for (a) Native soil dry (b) Native soil wet (c) Reworked backfill dry (d) Reworked backfill wet

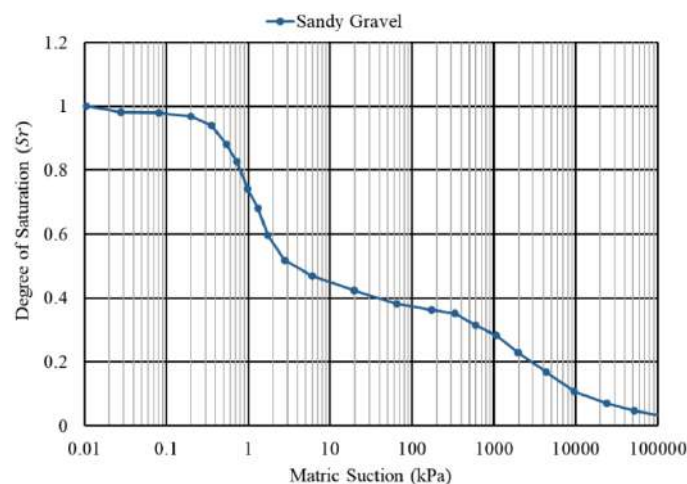


Fig. 23. Soil water characteristic curve "SWCC" for Sandy Gravel (void ratio = 0.5, specific gravity = 2.80)

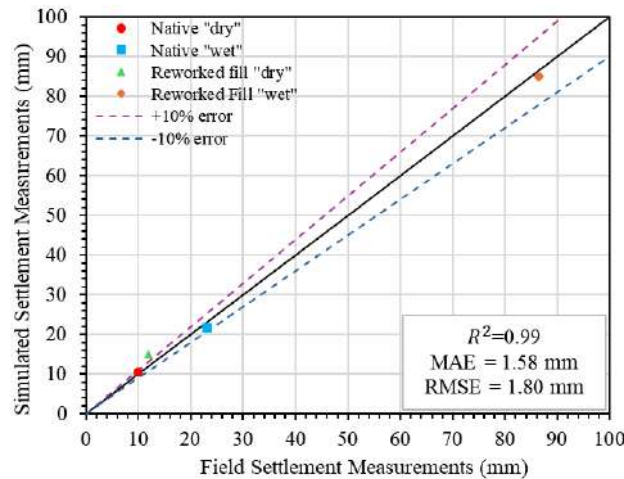


Fig. 24. In-situ and simulated settlement values for native soil and reworked engineering backfill

4.3.6 Scope and Limitations of the Numerical Analysis

The numerical analysis presented is a simplified, phenomenological back-analysis, not a predictive model. Its sole purpose is to demonstrate that the magnitude of the measured field collapse can be reproduced using reasonable soil parameters and a constitutive model for unsaturated soil. A formal mesh study and comprehensive sensitivity analysis were not performed. Parameters were derived from standard correlations, literature values, and iterative manual calibration to fit the load-settlement data. The high R^2 value is thus interpreted as evidence that the model can adequately replicate the observed collapse settlement, not as validation of a perfect or predictive unsaturated soil model.

4.4. Comparison Between SPLT And Large-Scale In-Situ Load Test

A comparison between the average settlement values from the SPLT and large-scale in situ test for the reworked compacted backfill zone is shown in Table 4. From the results, under constant load in the dry state, a scaling factor of about 4.0 can be assumed to account for the dimensions of the loading plate, stress influence zones and nonlinear strain distributions. The observed settlement ratio of approximately 4:1 between the large-scale RC footing and the SPLT is presented as an empirical, site-specific outcome rather than a generic scaling law.

On applying the scaling factor, both tests give relatively equivalent settlement results in the dry state. However, upon wetting, the values between the two tests differ drastically. This is due to the saturation area and testing period. For the SPLT, the saturation area is limited to the subsurface, and the test is conducted within a few hours. For the large-scale test, deep wells were used to infiltrate the water into the soil at greater depths up to 9 m, and the test was conducted for a minimum of 30 days. Fig. 16 shows that the reworked compacted backfill zone took almost 35 days to reach the maximum measured settlement as given in Table 2, and yet stable conditions were not obtained. The influence depth of a surface load is theoretically limited, often approximated as smaller than $2B$ to $4B$ (where B is the foundation width) [56]. For the 0.5m SPLT plate, this corresponds to an influence depth of 1-2 m and about 4-8 m for the 2.0m RC footing. However, there is no influence depth for the effect of wetting. Collapse behaviour is a volumetric strain triggered by wetting, which is mostly independent of the applied footing stress and its influence depth depends on the depth of the collapsible soil and the amount and rate of water infiltration. At identical stress conditions, the collapse settlement from the SPLT is governed by a small soil volume just beneath the plate that will be fully saturated and develops a smear zone that prevents further water infiltration, whereas in reality, collapse settlement is governed by a much greater volume, encompassing the wetted zone, shown in Fig. 25. The time as well as the source of water in the reality play decisive actions on the soil performance that cannot be simulated via the simple static plate loading test. Furthermore, to normalize the results, the average field collapse strain, ϵ_{cf} , is calculated by dividing the collapse settlement by the stress influence depth for both cases, which

yields 0.4% and 2.2% for the SPLT and large-scale in-situ test respectively. Thus, further validating that the SPLT cannot fully capture the expected collapse mechanism upon deep infiltration.

Table 4. Comparison between SPLT and large-scale in-situ test for reworked compacted backfill zone

	Item	Unit	SPLT	Large-Scale in-situ Test
Average Settlement	Just Before Wetting	mm	3.07	12
	After Completion of Wetting Process	mm	6.56	86.5
	Percentage of Increase	%	113.75	620.83

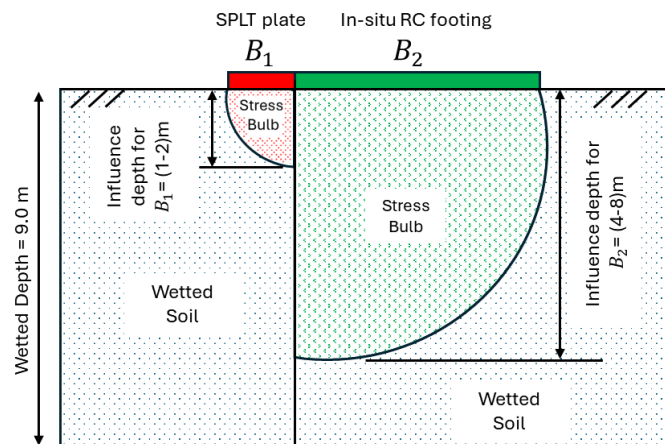


Fig. 25. Comparison between influence depth and stress bulb for applied load on SPLT plate (B_1) and in-situ RC footing (B_2)

5. Conclusions

From the conducted large-scale in-situ tests with a water infiltration system on both cut and fill zones, the following conclusions can be drawn:

- The performance of the native soil in the cut zone differs drastically from that of the compacted backfilled soil, despite using the cut material for the fill zone. The destruction of the bonded joints of the native soil is the reason for this large difference between the performance of native and recompacted soils.
- The amount of clay, compaction methodology, compaction energy and degree of compaction have a great effect on the performance of fill soils used as engineered fill.
- Static plate load tests cannot accurately predict the expected increase in settlement due to water ingress in collapsible soil. It can only indicate the risk of collapsibility of the used compaction material.
- Settlement due to applied foundation stress is affected by the footing dimensions (influence depth); however, there is no exact influence depth for the effect of water ingress. The effect of water ingress is a time phenomenon that depends on the soil type, percentage and type of fines, initial water content, source of water and quantity of seeped water as well as the depth of the collapsible soil layer.
- The accelerated deep infiltration system demonstrates the severe collapse potential inherent in the soil fabric by rapidly triggering the wetting-induced settlement mechanism. In the large-scale test on reworked fill, this resulted in approximately 11 cm of settlement within days. Importantly, the magnitude of this settlement is comparable to the 10–12 cm of damage-causing settlement observed over a year in a nearby villa affected by subsurface leakage. While the timescales differ, both cases confirm that the ultimate collapse potential of the soil is sufficient to produce structurally damaging settlements. Thus, the test provides

a validated lower-bound estimate of the geotechnical risk posed by prolonged moisture ingress in this arid environment.

- Additional measures to control the risk of soil collapse by water ingress are to apply isolation systems combined with a subsurface drainage system for all green areas, swimming pools and utilities such as sewer and irrigation pipes to prevent water infiltration into soil susceptible to collapse, which may cause severe cracks and settlement in nearby structures.
- Non-uniform infiltration of water due to leakage can cause differential settlement and distortion of structures that can lead to severe damage.
- Identification and classification of collapse behavior from in-situ tests and settlement recordings indicate that the average angular distortion values can exceed allowable serviceability limit states. There is a very high risk for structures founded on such soil type.
- The novelty of this approach lies in wetting not only the subsurface during testing, which is the case in most previous studies but also using deep wells to ensure the wetting of the soil at greater depths to simulate realistic foundation long term conditions.
- The results of conventional static plate loading test with inundation give only indication on the soil sensitivity in case of water ingress. Further work to correlate the test results with the expected long-term performance is needed.
- The different infiltration behaviors observed between the native and reworked engineering fill soils warrant a cautious interpretation. For future recommendations, it is important to note three key limitations in the presented dataset that should be addressed for a definitive mechanistic explanation: hydraulic conductivity measurements, quantitative reference to seepage pathways and grain-size-specific microstructural interpretation.

References

- [1] Knodel PC. Characteristics and problems of collapsible soils. US Department of the Interior, Bureau of Reclamation, Denver Office, Research and Laboratory Services Division, Materials Engineering Branch; 1992.
- [2] Ali NA. Practical engineering behavior of Egyptian collapsible soils, laboratory and in-situ experimental study. Open Journal of Civil Engineering. 2021; 11(3): 290 - 300. <https://doi.org/10.4236/ojce.2021.113017>
- [3] Mebarki M, Benyahia S, Dahmani S. Laboratory improvement of clay mineralogical and swelling properties using hydraulic binder treatment. Research on Engineering Structures and Materials. 2025; 11: 933 - 55. <https://doi.org/10.17515/resm2025-752ma0313rs>
- [4] Rogers CD. Types and distribution of collapsible soils. In Genesis and properties of collapsible soils 1995 Apr (pp. 1-17). Dordrecht: Springer Netherlands.
- [5] Ali N, Metwally M, El Sawwaf M, Nazir A. Study the correlation between microstructural features and geotechnical properties of collapsing soil at elevated temperatures. Geoenergy Science and Engineering. 2024; 238: 212923. <https://doi.org/10.1016/j.geoen.2024.212923>
- [6] Abdulwadood HW, Albusoda BS. The Collapsible Soil Definition and Mitigation Strategies: A Review Study. Journal of Engineering, 2024; 30(11): 142 - 63. <https://doi.org/10.31026/j.eng.2024.11.09>
- [7] Almahbobi S. Experimental study of volume change and shear strength behaviour of statically compacted collapsible soil, PhD Dissertation, Cardiff University, 2018.
- [8] Sakr M, Mashhour M, Hanna A. Egyptian collapsible soils and their improvement. Proceedings of GeoCongress 2008: Geosustainability and Geohazard Mitigation, New Orleans, Louisiana, 2008. [https://doi.org/10.1061/40971\(310\)81](https://doi.org/10.1061/40971(310)81)
- [9] Ali NA. Performance of partially replaced collapsible soil-Field study. Alexandria Engineering Journal. 2015; 54(3): 527-32. <https://doi.org/10.1016/j.aej.2015.05.002>
- [10] El Mosallamy M, Abd El Fattah TT, El Khoully M. Experimental study on the determination of small strain-shear modulus of loess soil. Hbrc Journal. 2016; 12(2): 181-90. <https://doi.org/10.1016/j.hbrj.2014.11.010>
- [11] Ayadat T, Hanna AM. Assessment of soil collapse prediction methods. Int J Eng Trans B Appl. 2012; 25(1): 19 - 26. <https://doi.org/10.5829/idosi.ije.2012.25.01b.03>
- [12] Gaaver KE. Geotechnical properties of Egyptian collapsible soils. Alexandria Engineering Journal. 2012; 51(3): 205 - 10. <https://doi.org/10.1016/j.aej.2012.05.002>
- [13] Cui YJ, Delage P. Soil Collapse due to Water Infiltration. Environmental Geomechanics. 2013: 149 - 69. <https://doi.org/10.1002/9781118619834.ch6>
- [14] Hamdy DB. Origin and characteristics of collapsible soils state of the art report. Innovative Infrastructure Solutions. 2024; 9(10): 401. <https://doi.org/10.1007/s41062-024-01673-5>

- [15] D5333-03 ASTM. Standard Test Method for Measurement of Collapse Potential of Soils. 1996.
- [16] Feda J. Structural stability of subsident loess soil from Praha-Dejvice. *Engineering Geology*. 1966; 1(3): 201 – 19. [https://doi.org/10.1016/0013-7952\(66\)90032-9](https://doi.org/10.1016/0013-7952(66)90032-9)
- [17] Jennings JE, Knight K. A guide to construction on or with materials exhibiting additional settlement due to “collapse” of grain structure. *Proceedings of Sixth regional conference for Africa on SMFE*, 99–105, 1975.
- [18] Lollo JA, Rodrigues RA, Elis VR, Prado R. Use of electrical resistivity to identify collapsible soils in Brazil. *Bull Eng Geol Environ*. 2011; 70(2): 299–307. <https://doi.org/10.1007/s10064-011-0357-8>
- [19] Lollo JA, Vivanco JM., Curtis JS. Collapse susceptibility mapping using SRTM data obtained from Topodata Project. *Eng Geol Soc Territ*. 2015; 5: 885 – 91.
- [20] Gibbs HJ, Bara JP. Predicting surface subsidence from basic soil tests. US Department of the Interior, Bureau of Reclamation, Division of Engineering Laboratories; 1962.
- [21] Al-Rawas AA. State-of-the-art-review of collapsible soils. *Sultan Qaboos University Journal For Science*. 2000; 5(2): 115 - 35. <https://doi.org/10.24200/squjs.vol5iss0pp115-135>
- [22] Houston SL, Houston WN, Zapata CE, Lawrence C. Geotechnical engineering practice for collapsible soils. *Geotechnical & Geological Engineering*. 2001; 19(3): 333 - 55. https://doi.org/10.1007/978-94-015-9775-3_6
- [23] Abelev YM. *Fundamentals of Design and Construction on Macroporous Soils*. Stroivoenmorizdat, Moscow. 1948.
- [24] Ng AM, Yeung AT, Lee PK, Tham LG. Design, fabrication, and assembly of a large oedometer. *Geotechnical Testing Journal*. 2006; 29(4): 298 - 305. <https://doi.org/10.1520/GTJ100454>
- [25] Brink GE. The identification and characterisation of collapsible soils: a brief review of current practice. *Proceedings of the 21st NZGS Symposium*, Dunedin, New Zealand, 2021.
- [26] Mokhtari M, Shariatmadari N, Heshmati R AA, Salehzadeh H. Design and fabrication of a large-scale oedometer. *Journal of Central South University*. 2015; 22(3): 931 - 6. <https://doi.org/10.1007/s11771-015-2603-x>
- [27] Soleimani S, Jiao P, Rajaei S, Forsati R. A new approach for prediction of collapse settlement of sandy gravel soils. *Engineering with Computers*. 2018; 34(1): 15 - 24. <https://doi.org/10.1007/s00366-017-0517-y>
- [28] Hasanzadehshooili H, Mahinroosta R, Lakirouhani A, Oshtaghi V. Using artificial neural network (ANN) in prediction of collapse settlements of sandy gravels. *Arabian Journal of Geosciences*. 2014; 7(6): 2303 - 14. <https://doi.org/10.1007/s12517-013-0858-9>
- [29] Ghanizadeh AR, Delaram A, Fakharian P, Armaghani DJ. Developing predictive models of collapse settlement and coefficient of stress release of sandy-gravel soil via evolutionary polynomial regression. *Applied Sciences*. 2022; 12(19): 9986. <https://doi.org/10.3390/app12199986>
- [30] De Zoysa CJ, Dushan AK, Kurukulasuriya LC. Collapsibility Characteristics of a Residual Soil in Matale District, Sri Lanka. In *ICSBE 2020: Proceedings of the 11th International Conference on Sustainable Built Environment*, Singapore 13-22, 2021. https://doi.org/10.1007/978-981-16-4412-2_2
- [31] Sun DA, Sheng DC. Collapse Behavior of Unsaturated Compacted Soils. In *Advances in unsaturated soil, seepage, and environmental geotechnics*, 102-110, 2006. [https://doi.org/10.1061/40860\(192\)9](https://doi.org/10.1061/40860(192)9)
- [32] Hilf JW. A investigations of pore-water pressure in compacted cohesive soils, Ph.D Dissertation, University of Colorado, 1956.
- [33] Barden L, McGown A, Collins K. The collapse mechanism in partly saturated soil. *Engineering Geology*. 1973; 7(1): 49 - 60. [https://doi.org/10.1016/0013-7952\(73\)90006-9](https://doi.org/10.1016/0013-7952(73)90006-9)
- [34] Vilar OM, Rodrigues RA. Collapse behavior of soil in a Brazilian region affected by a rising water table. *Canadian Geotechnical Journal*. 2011; 48(2): 226 - 33. <https://doi.org/10.1139/T10-065>
- [35] Opukumo AW, Davie CT, Glendinning S, Oborie E. A review of the identification methods and types of collapsible soils. *Journal of Engineering and Applied Science*. 2022; 69(1): 17. <https://doi.org/10.1186/s44147-021-00064-2>
- [36] Mahmoud HH. The development of an in situ collapse testing system for collapsible soils, Ph.D Dissertation, Arizona State University, 1991.
- [37] Krutov VI, Tarasova IV. A method for determining the magnitude of the" initial pressure" for slumping soils. *Soil Mechanics and Foundation Engineering*. 1964; 1(1): 12-7. <https://doi.org/10.1007/BF01704025>
- [38] Macfarlane RB. A field test for detecting collapse susceptible soils, MSc. Dissertation, The University of Arizona, Arizona, 1989.
- [39] Krutov VI, Éiduk RP. Determination of the relative slumpability of a soil by static probing from an exploratory-trench bed. *Soil Mechanics and Foundation Engineering*. 1971; 8(3): 172-4. <https://doi.org/10.1007/BF01705109>
- [40] Bowers MT. IN SITU SOIL TESTING SYSTEM FOR COLLAPSIBLE SOILS (SETTLEMENT, COLLAPSE, ARIZONA), Ph.D Dissertation, Arizona State University, Arizona, 1986.

- [41] Mahmoud HH, Houston WN, Houston SL. Apparatus and procedure for an in situ collapse test. *Geotechnical Testing Journal*. 1995; 18(4): 431 - 40. <https://doi.org/10.1520/GTJ11018J>
- [42] Platis A, Malliou K. Large-Scale Plate Load Tests to Determine the Collapse Potential at Santorini 150 kV GIS Substation Site. In *E3S Web of Conferences*, Vol. 382, p. 12006, 2023. <https://doi.org/10.1051/e3sconf/202338212006>
- [43] Rocha BP, de Carvalho Rodrigues AL, Rodrigues RA, Giacheti HL. Using a seismic dilatometer to identify collapsible soils. *International Journal of Civil Engineering*. 2022; 20(7): 857-67. <https://doi.org/10.1007/s40999-021-00687-9>
- [44] Romero S, Ferreira DM, Fucale S. Evaluation of the collapsibility of soils in the semiarid region of Pernambuco, Brazil. *The Scientific World Journal*. 2014; 8(10): 1285-92. <https://doi.org/10.17265/1934-7359/2014.10.010>
- [45] Bandeira AP, de Souza Neto JB, Coutinho RQ, Xavier JM, Chaves AM, da Silva Alves VL. Investigating the Collapsible Behavior of Sedimentary Soil in Shallow Foundations. *Geotechnical and Geological Engineering*. 2024; 42(4): 2725 - 43. <https://doi.org/10.1007/s10706-023-02701-4>
- [46] Sallam E, Issawi B, Osman R. Stratigraphy, facies, and depositional environments of the Paleogene sediments in Cairo-Suez district, Egypt. *Arabian Journal of Geosciences*. 2015; 8(4): 1939 - 64. <https://doi.org/10.1007/s12517-014-1360-8>
- [47] Tuleun L Z, Wasui J. Mechanical behavior of silty-clayey lateritic soil stabilized with recycled municipal solid waste ash. *Res. Eng. Struct. Mater.*, 2025; 11(3): 1121-1138. <https://doi.org/10.17515/resm2024.204me0303rs>
- [48] Sun S, Zhu D, Zhang Z. Experimental study on the compression characteristics of sandy soil under impact loading. *Research on Engineering Structures and Materials*. 2025; 11(2): 903-920. <https://doi.org/10.17515/resm2025-586me1223rs>
- [49] Bakhshandeh F, Noorzad R, Ta'negonbadi B. Experimental investigation of the properties of collapsible soil stabilized by colloidal silica. *Scientific Reports*. 2025; 15(1): 30032. <https://doi.org/10.1038/s41598-025-15965-y>
- [50] Vilar OM, Rodrigues RA. Revisiting classical methods to identify collapsible soils. *Soils and Rocks*. 2015; 38(3): 265 - 78. <https://doi.org/10.28927/SR.383265>
- [51] Silveira IA, Giacheti HL, Rocha BP, Rodrigues RA, Paulo S. Collapsible behaviour of a compacted lateritic sandy soil. *Proceedings of the 7th International Conference on Geotechnical and Geophysical Site Characterization (ISC'7)*, 18-21, 2024.
- [52] Skempton AW, MacDonald DH. The allowable settlements of buildings. *Proceedings of the Institution of Civil Engineers*, 5(6):727-68, 1956.
- [53] Plaxis 2023.1. User Defined Soil Model-Barcelona Basic Model. 2023; 1-58.
- [54] Alonso EE, Gens A, Josa A. A constitutive model for partially saturated soils. *Géotechnique*. 1990; 40(3): 405 - 30. <https://doi.org/10.1680/geot.1990.40.3.405>
- [55] Li X, Li JH, Zhang LM. Predicting bimodal soil-water characteristic curves and permeability functions using physically based parameters. *Computers and geotechnics*. 2014; 57: 85-96. <https://doi.org/10.1016/j.compgeo.2014.01.004>
- [56] Shahien M, Farouk A, Elgarf AM. Simplified Model for Determining the Depth of Influence below Surface Loaded Areas. *Proceedings of the 18th Regional African Conference on Soil Mechanics and Geotechnical Engineering*. Algeria, 2024.

Optimizing process performance through statistical design of experiments: A data-driven engineering approach

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Abstract

Over recent years, the design of experiments has emerged as a dynamic research field, attracting significant attention from scholars and practitioners. Experimental outcomes inherently exhibit variability due to measurement errors and the complex, non-linear behavior of system responses influenced by unidentified input factors. Within this context, the Taguchi method—with its use of orthogonal arrays—offers an effective framework for identifying optimal input parameters with a reduced number of experiments, typically validated through empirical test data. Conventional statistical techniques, such as the modified Taguchi model and response surface methodology, remain widely employed for parameter estimation and optimization. However, recent advances in machine learning present powerful alternatives. In this study, support vector regression, random forest regression, and XGBoost regression models were compared with traditional approaches to assess their relative efficiencies. The machine learning-based methodologies demonstrated superior predictive accuracy while significantly reducing experimental costs, preserving essential process insights, and minimizing performance variability. Among these models, the XGBoost regression approach delivered the most reliable performance, exhibiting the lowest prediction error and an exceptionally high coefficient of determination ($R^2 = 0.99$).

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1. Introduction

Taguchi introduced a structured approach using orthogonal arrays that significantly reduces the number of experiments required while still capturing essential information from a full factorial design. Taguchi introduced a structured approach using orthogonal arrays that significantly reduce the number of experiments required while still capturing essential information from a full factorial design. The Taguchi method is one of the most widely used statistical techniques for improving the quality of manufactured products. According to Taguchi, the selection of appropriate control factors can effectively nullify noise factors. Each test run in the Taguchi method is classified into low (−1), medium (0), and high (1) levels of input process parameters.

Bhattacharya [1] explored the use of Response Surface Methodology (RSM) in pharmaceutical applications, providing valuable insights relevant to the present work. Cui et al. [2] optimized the physical and mechanical properties of spline surfaces using the Taguchi approach. Mounika et al.

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[3] optimized friction welding parameters using modifications in Taguchi, fuzzy logic, and response surface methodologies. Panigrahi et al. [4] employed an integrated Taguchi and machine-learning-based optimization approach to determine the optimal design parameters of a trapezoidal solar cooker while minimizing heat loss. Rajyalakshmi et al. [5] performed a comparative evaluation of the Taguchi method, Box–Behnken Design (BBD), and Central Composite Design (CCD) for optimizing process parameters involving four factors at three levels. Their findings show that although the Taguchi method is cost-effective, BBD and CCD offer superior accuracy and precision. Rajyalakshmi et al. [5] also identified optimal parameters for Chemical Oxygen Demand (COD) reduction and decolorization efficiency using multi-objective optimization in Fenton oxidation processes, supported by strong correlations between empirical model predictions and experimental data.

Romelin et al. [6] reported that Taguchi's L9 array effectively identified optimal design points for hydraulic ram pump performance. Singhavavelu et al. [7] reported the use of an L9 orthogonal array for four factors at three levels. Panigrahi et al. [8] presented employed Taguchi and ML in solar cooker optimization. Ross [9] reported the use of an L9 orthogonal array for four factors at three levels. Rajyalakshmi [10] presented a simple methodology and validated it using existing experimental results based on the modified Taguchi method. Rajyalakshmi and Nageswara Rao [11] applied a modified Taguchi methodology to identify the optimum parameters influencing weld dilution in ST-37 plates. Samuel et al. [12] demonstrated significant improvements in the mechanical properties of biomaterials using heat-treatment optimization through Taguchi and machine-learning techniques.

Athreya and Venkatesh [13] provided foundational work upon which the present study is built. Varalakshmi et al. [14] focused on optimizing responses using various statistical design methods. Varalakshmi et al. [15] investigated multi-response optimization of agricultural residues using a modified Taguchi approach combined with Chauvenet's criterion. Rajyalakshmi and Nageswara Rao [16]. Lestari et al. [17] studied the influence of fused deposition modeling (FDM) parameters on printing time and socket weight in transtibial prosthetic components using the Taguchi method. Jou et al. [18] developed predictive defect models in die-casting using Artificial Neural Networks, Support Vector Machines, and Random Forests combined with Taguchi methods.

Traditional statistical techniques often fail to adequately capture nonlinear behaviour, motivating the present investigation. While traditional statistical techniques such as linear models and lower-order interaction frameworks have been widely used, they often fall short in capturing the intricate nonlinear relationships present in complex systems. To address these limitations, this research extends previous models by integrating advanced machine learning algorithms capable of modeling higher-order and nonlinear interactions among variables. This study systematically compares traditional statistical models (Taguchi, RSM) with modern machine learning algorithms (SVR, XGBoost, Random Forest) on a unified dataset. This hybrid approach is rarely presented in machining literature with such rigor and reproducibility. Industrial experiments often operate under resource constraints, limiting the number of design points. By demonstrating how different models perform on a 27-run factorial design, provide practical guidance for real-world applications where data is limited but precision is critical.

The experimental framework utilizes the test data originally developed by Srinivasa Athreya and Venkatesh, with modifications aligned to the suggestions of Rajyalakshmi and Nageswararao. A comparative analysis was conducted across several predictive and optimization techniques, including Response Surface Methodology (RSM), Support Vector Regression (SVR), XGBoost, and Taguchi-based linear models. Each method was evaluated based on its predictive accuracy, quantified through metrics such as Mean Absolute Error (MAE), Root Mean Square Error (RMSE), Mean Absolute Percentage Error (MAPE), and the coefficient of determination (R^2). Graphical representations were employed to visualize the performance differences and error margins between predicted and actual values.

Among the models tested, XGBoost emerged as the most accurate, demonstrating the lowest prediction error and the highest R^2 value. This indicates its superior capability in capturing complex patterns and delivering precise predictions. The study highlights the effectiveness of

machine learning techniques in enhancing process modelling and optimization, especially when traditional statistical methods are constrained by linear assumptions. By integrating these advanced approaches into the existing analytical framework, the research offers a robust methodology for improving predictive performance in engineering and scientific applications.

2. Methodology

2.1 Taguchi Method

Taguchi designed a method to improve the quality of manufactured goods. According to him, way of selecting appropriate control factors can nullify the noise factors. Ross(9) suggested an appropriate orthogonal array (OA) to perform experiments is as follows.

$$N_T = 1 + n_p(n_L - 1) \quad (1)$$

N_T : Total number of experimental runs required; n_p : Number of parameters (or factors) being studied; n_L : Number of levels for each parameter; 1: Represents the baseline or control condition (reference level).

2.2 Response Surface Methodology (RSM)

Response Surface Methodology (RSM) is a statistical technique that is used to model and optimize complex systems. It is commonly used in the fields of engineering, science, and business to design and improve products or processes. RSM provides a suitable approximation using the following form.

$$y = a_0 + \sum_{i=1}^k a_i x_i + \sum_{i=1}^k a_{ii} x_i^2 + \sum_{i=1}^k \sum_{j=i+1}^k a_{ij} x_i x_j + e \quad (2)$$

a_0 : Constant, a_i : coefficient of the variable x_i , $\sum a_i x_i$: linear term for each variable, $\sum_{i=1}^k a_{ii} x_i^2$: pure quadratic terms, $\sum_{i=1}^k \sum_{j=i+1}^k a_{ij} x_i x_j$: interaction terms (combined effects of two variables acting together), e : random error.

The basic idea behind RSM is to create a mathematical model that relates the response of a system to its input variables. This model can then be used to identify the optimal settings for the input variables to achieve the desired response. RSM involves a series of experimental designs that are used to collect data on the response of the system to different combinations of input variables. The data is then analyzed using statistical techniques to create a mathematical model that describes the relationship between the input variables and the response. It is used to improve the accuracy and to explore curvature and interactions efficiently. Once the model is created, it can be used to optimize the system by identifying the input variable settings that will produce the desired response. This can help to improve product quality, reduce costs, or increase efficiency. Overall, RSM is a powerful tool for designing and optimizing complex systems. It can help to reduce the time and cost associated with trial-and-error experimentation and can lead to significant improvements in product or process performance.

2.3 Extreme Gradient Boosting (XGBoost)

Extreme Gradient Boosting (XGBoost) is a machine learning algorithm for solving complex nonlinear relationships with more accuracy, speed, and ability. It is mainly used to handle nonlinear interactions, regularization to reduce overfitting and high performance with small to large datasets. In the present paper, integrated XGBoost with Taguchi design to enhance the accuracy. It is helpful to predict the unseen combination of parameter enabling virtual experimentation. It gives rank to the input parameters based on their influence on response variable. Hence, the experimental data has been trained using XGBoost Regressor model. The implementation of XGBoost, involves Python libraries such as xgboost and scikit-learn. For XGBoost, the XGBRegressor class is commonly used, with key hyperparameters including `n_estimators`, `max_depth`, `learning_rate`, `subsample` and regularization terms like `reg_alpha` and `reg_lambda`. These settings help control overfitting and improve model generalization.

2.4 Support Vector Regression (SVR)

Support Vector Regression (SVR) is a supervised machine learning algorithm that extends Support Vector Machines (SVM) to predict continuous numerical values. Unlike traditional regression, SVR tries to find a function that approximates the data within a specified margin of tolerance (ϵ) while minimizing model complexity and penalizing errors beyond the margin. It is particularly useful when you want accurate predictions with high generalization ability even on non-linear data, by using kernel functions. SVR is implemented using `sklearn.svm.SVR`, where important hyperparameters include kernel (e.g., 'rbf', 'linear'), C (regularization strength), epsilon (margin of tolerance), and gamma (kernel coefficient). These parameters influence the model's flexibility and error tolerance.

2.5 Random Forest Regression

Random Forest Regression is an ensemble learning method that creatively integrates the predictions of multiple decision trees to produce a reliable, accurate, and high-performance regression model. By combining the outputs of several individually weak learners, it constructs a powerful predictive framework capable of capturing complex and non-linear patterns within data. Random Forest Regression is built using `sklearn.ensemble.RandomForestRegressor`, with hyperparameters such as `n_estimators`, `max_depth`, `max_features`, `min_samples_split`, and `bootstrap`. These control the ensemble size, tree complexity, and feature sampling.

This study adopts a range of integrated methodologies including Taguchi-based linear models, Response Surface Methodology (RSM), Support Vector Regression (SVR), XGBoost, and Random Forest regression to compare their performance metrics. The objective is to identify the approach that yields the lowest prediction error, thereby reducing both the experimental cost and the time required for testing. To divide the dataset into subsets for model training and performance testing. This ensures that the model is trained on one part of the data and tested on unseen data to measure generalization ability. Hyper parameter tuning for all three models is often performed using grid search or randomized search to optimize performance metrics like MAE, RMSE, and R^2 .

3. Data Acquisition

Traditional methods like Taguchi and RSM are limited to linear and low-order interactions. Machine learning models (SVR, XGBoost, Random Forest) capture nonlinear patterns and high-order interactions, improving prediction accuracy. With only 27 runs from a 3^3 factorial design, these models extract maximum insight without requiring large datasets.

This hybrid strategy blends the strengths of classical design with modern predictive power ideal for optimizing manufacturing processes, reducing variability, and achieving consistent quality. The foundational experiments conducted by Srinivasa Athreya and Venkatesh (2012) aimed to identify optimal machining parameters, namely cutting speed (X_1), depth of cut (X_2), and feed rate (X_3) to improve surface roughness during facing operations on mild steel components. These parameters were categorized into three levels: low (−1), medium (0), and high (1), and evaluated using a lathe machine equipped with a portable surface tester. Building on this work, Rajyalakshmi and Nageswara Rao (2018) introduced a novel enhancement to the Taguchi method by incorporating a fictitious parameter (X_4), which allowed for a more comprehensive analysis of variability in the output response. Their study employed Analysis of Variance (ANOVA) to quantify the contribution of each factor to surface roughness. The results revealed that cutting speed (X_1) contributed 71.49%, depth of cut (X_2) 4.18%, feed rate (X_3) 11.16%, and the fictitious parameter (X_4) 13.17% to the total variation. Notably, the error percentage was found to be zero when the fictitious parameter was included, whereas its exclusion resulted in a 13.17% error equivalent to its contribution highlighting its significance in the model. According to their findings, the optimal surface roughness value was 2.62 μm , achieved at a cutting speed of 960 rpm, depth of cut of 0.3 mm, and feed rate of 130 mm/min. The expected range of surface roughness under these conditions was estimated between 2.349 and 2.768 μm .

Expanding upon these prior studies, the present research integrates both statistical and machine learning methodologies including Taguchi design, Response Surface Methodology (RSM), Support

Vector Regression (SVR), XGBoost, and Random Forest regression—to evaluate and compare their effectiveness in optimizing process performance. Each method was assessed based on its predictive accuracy and ability to model complex interactions among machining parameters. This integrated approach provides a robust framework for parameter optimization, offering deeper insights into the relationships between input variables and surface quality outcomes.

4. Results and Analysis

This study integrates multiple Modeling techniques to establish a comprehensive predictive framework. Specifically, it compares the performance of five regression models: Taguchi-based linear model, Response Surface Methodology (RSM), Support Vector Regression (SVR), XGboost Regression, and Random Forest Regression. A paired t-test was conducted to compare the experimental (test) data and the corresponding values predicted by each model. The calculated t-statistic was smaller than the critical t-value. These results indicate that there is no statistically significant difference between the experimental values and the predicted values of the mentioned models. In other words, the model's predictions are statistically consistent with the observed data, demonstrating that successfully captured the underlying relationship within the dataset. Although five different modelling approaches - XGboost, Support Vector Regression (SVR), Random Forest (RF), Response Surface Methodology (RSM), and Taguchi method were compared, no statistically significant differences were found among them based on the selected error indicators. Figures 1 through 5 illustrate the comparison between actual and predicted values for each model: XGboost (Figure 1), SVR (Figure 2), Random Forest (Figure 3), RSM (Figure 4), and the Taguchi-based model (Figure 5). Among all models, XGboost shows an exceptional alignment with the actual experimental data, indicating its superior predictive accuracy. Table I further highlights the experimental versus predicted values, demonstrating the effectiveness of each model, with XGboost outperforming the rest. Table (2) Provides the error analysis for each model.

4.1 Sensitivity Analysis

The correlation coefficient (r) measures the strength of the linear relationship between the model predictions and the experimental (test) data.

- XGBoost ($r = 0.99$) shows an *almost perfect linear relationship* with the test data — it predicts values that very closely follow the observed trend.
- RF ($r = 0.95$) and SVR ($r = 0.8913$) also demonstrate strong positive correlations, suggesting reliable predictive ability.
- RSM ($r = 0.8974$) performs comparably to SVR, though slightly less correlated.
- Taguchi method ($r = 0.59$) shows only moderate correlation, implying that it captures the general trend but with considerable deviation from the actual test data.

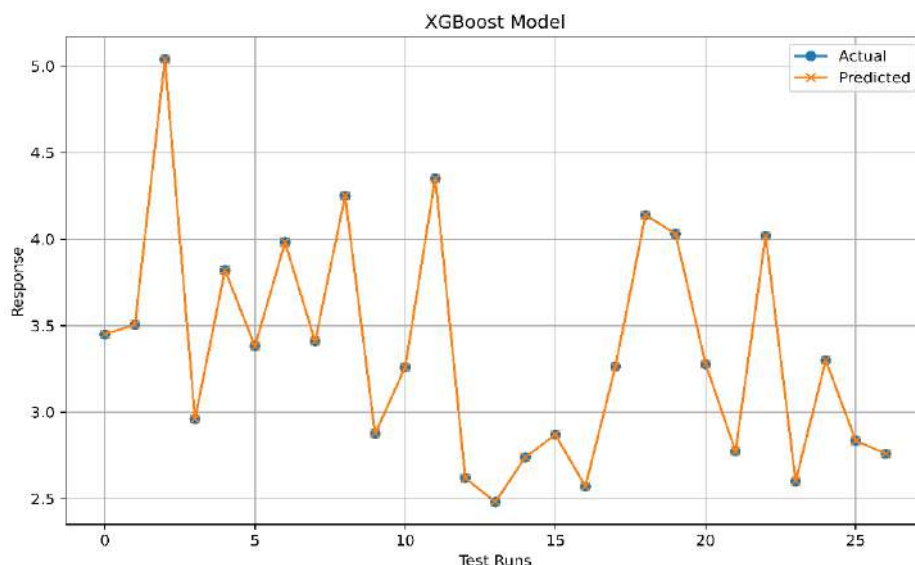


Fig. 1. XGBoost model

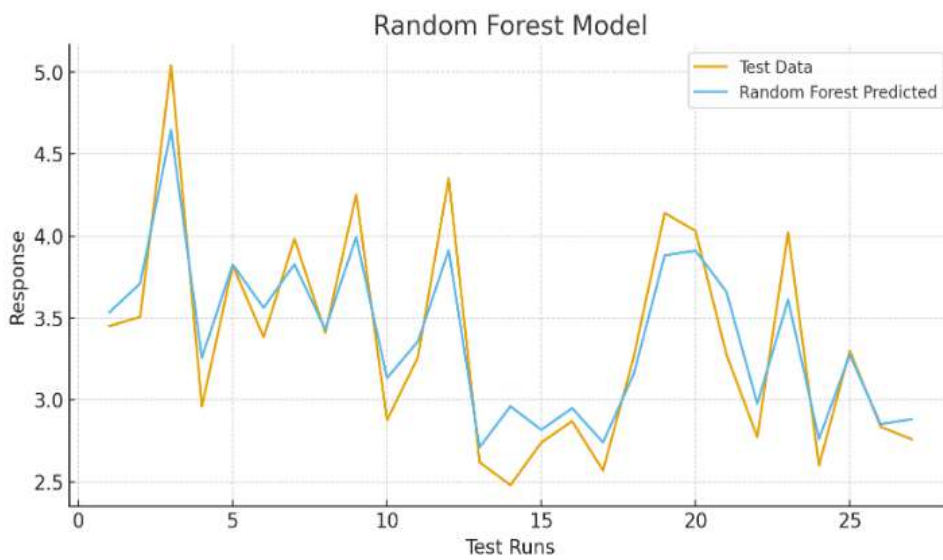
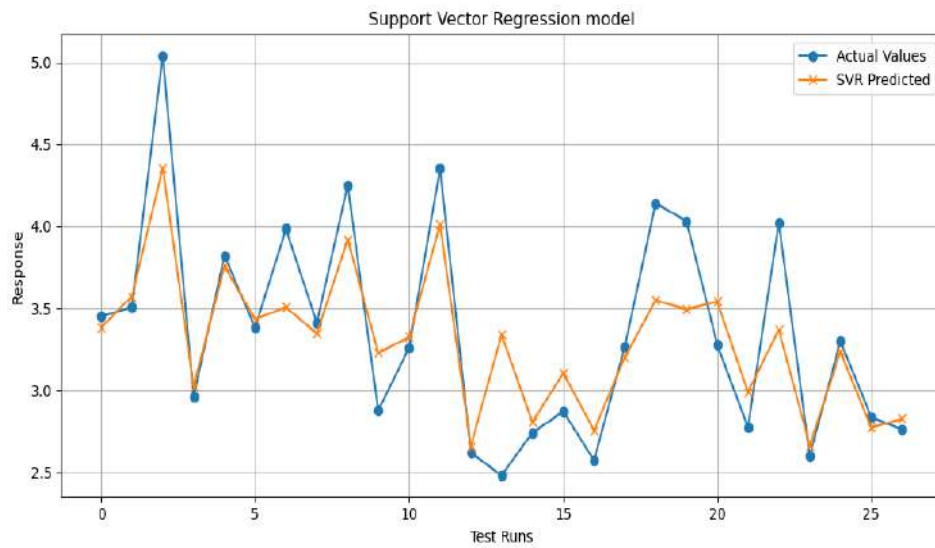


Fig. 3. Random forest regression

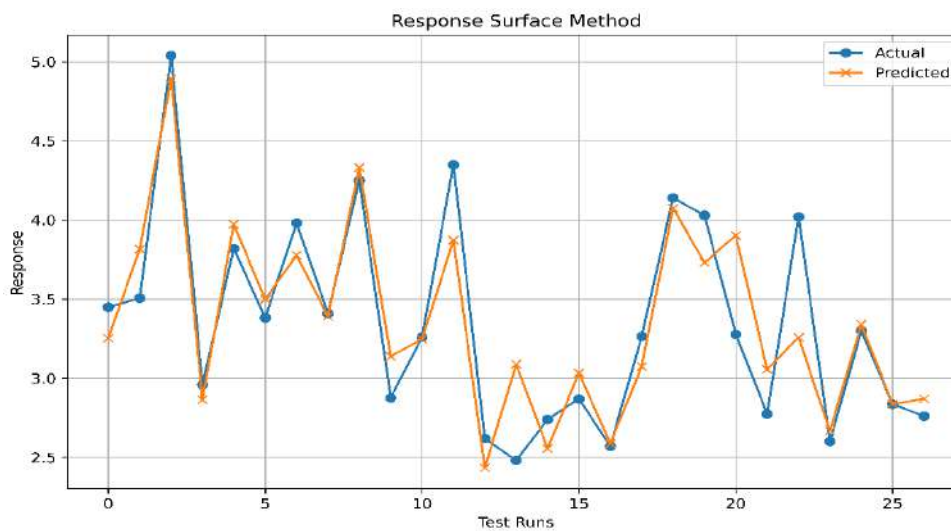


Fig. 4. Response surface methodology

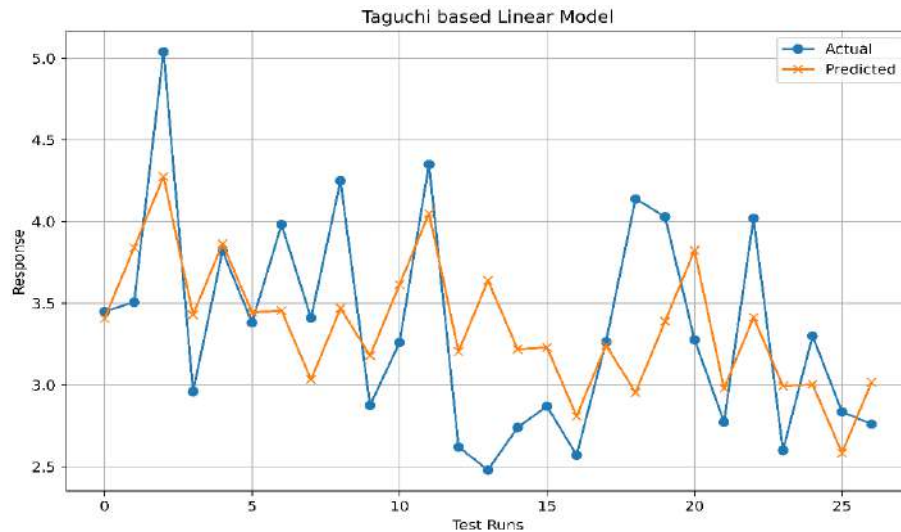


Fig. 5. Taguchi based linear model

The error values for each model (XGBoost, Support Vector Regression, Random Forest, Response Surface Methodology, and Taguchi-based Linear Model) were computed using the test dataset, based on the difference between the predicted and actual values. Each model performance was evaluated using the following error metrics,

$$\text{Mean Absolute Error (MAE)} = \frac{1}{n} \sum_{i=1}^n |y_i - \hat{y}_i| \quad (3)$$

where, y_i : observed values, $\hat{y}_i$: predicted values, n : number of observations.

$$\text{Mean Squared Error (MSE)} = \frac{1}{n} \sum_{i=1}^n (y_i - \hat{y}_i)^2 \quad (4)$$

$$\text{Root Mean Squared Error (RMSE)} = \sqrt{\frac{1}{n} \sum_{i=1}^n (y_i - \hat{y}_i)^2} \quad (5)$$

$$\text{Mean absolute percentage error (MAPE)} = \frac{100}{n} \sum_{i=1}^n \left| \frac{y_i - \hat{y}_i}{y_i} \right| \quad (6)$$

$$\text{Coefficient of determination (R}^2\text{)} = 1 - \frac{\sum_{i=1}^n (y_i - \hat{y}_i)^2}{\sum_{i=1}^n (y_i - \bar{y})^2} \quad (7)$$

Table 1. Experimental and predicted values using different regression models

S.NO.	X1	X2	X3	X4	TEST DATA	PREDICTED (1) XGBOOST	PREDICTED (2) Support Vector Regression	PREDICTED (3) Random Forest Regression	PREDICTED (4) Response Surface Method	PREDICTED (5) Taguchi based linear model
1	-1	-1	-1	-1	3.45	3.450144	3.385071	3.5339	3.253852	3.405593
2	-1	-1	0	0	3.506	3.505383	3.570623	3.7098	3.816685	3.840481
3	-1	-1	1	1	5.04	5.037933	4.354874	4.6458	4.893519	4.27537
4	-1	0	-1	0	2.96	2.96144	3.024591	3.2573	2.864519	3.430481
5	-1	0	0	1	3.82	3.819308	3.755149	3.8261	3.972685	3.86537
6	-1	0	1	-1	3.382	3.380852	3.43307	3.562	3.498352	3.444259
7	-1	1	-1	1	3.982	3.981829	3.505373	3.8274	3.778519	3.45537
8	-1	1	0	-1	3.41	3.410325	3.34553	3.4301	3.391519	3.034259
9	-1	1	1	0	4.25	4.250471	3.918938	3.992	4.330352	3.469148
10	0	-1	-1	-1	2.876	2.877862	3.227187	3.133	3.13763	3.179926
11	0	-1	0	0	3.26	3.260162	3.324519	3.3564	3.24763	3.614815
12	0	-1	1	1	4.35	4.34951	4.012648	3.9123	3.87163	4.049704

S.NO.	X1	X2	X3	X4	TEST DATA	PREDICTED (1) XGBOOST	PREDICTED (2) Support Vector Regression	PREDICTED (3) Random Forest Regression	PREDICTED (4) Response Surface Method	PREDICTED (5) Taguchi based linear model
13	0	0	-1	0	2.62	2.619565	2.656429	2.7106	2.49363	3.204815
14	0	0	0	1	2.48	2.482483	3.337663	2.9611	3.088963	3.639704
15	0	0	1	-1	2.74	2.739819	2.804515	2.8173	2.556296	3.218593
16	0	1	-1	1	2.87	2.87007	3.104081	2.9498	3.032963	3.229704
17	0	1	0	-1	2.57	2.571641	2.752938	2.741	2.58763	2.808593
18	0	1	1	0	3.264	3.262257	3.199616	3.1598	3.07363	3.243481
19	1	-1	-1	-1	4.14	4.13794	3.548494	3.8808	4.074519	2.954259
20	1	-1	0	0	4.03	4.030339	3.491223	3.9103	3.731685	3.389148
21	1	-1	1	1	3.278	3.278382	3.541257	3.6593	3.902852	3.824037
22	1	0	-1	0	2.774	2.773647	2.989197	2.9778	3.055852	2.979148
23	1	0	0	1	4.02	4.017954	3.368106	3.6101	3.258352	3.414037
24	1	0	1	-1	2.6	2.603139	2.66463	2.7612	2.667352	2.992926
25	1	1	-1	1	3.3	3.300311	3.23555	3.2762	3.340519	3.004037
26	1	1	0	-1	2.836	2.834259	2.771423	2.852	2.836852	2.582926
27	1	1	1	0	2.76	2.76097	2.824781	2.8824	2.870019	3.017815

The error analysis confirms that XGBoost is the most accurate and robust model among all considered. Its ability to consistently predict values very close to the actual test data across all conditions makes it the best choice for this dataset. Traditional linear or response surface methods show significant limitations, especially in handling nonlinearities or interactions in the input features.

- XGBoost clearly outperforms all other models, achieving the lowest error (MAE: 0.007, RMSE: 0.009) and a near-perfect R^2 score (~ 0.9999), indicating that it almost perfectly fits the data. Highly sensitive to input variation but robust against noise.
- Support Vector Regression (SVR) and Random Forest Regression also show strong predictive capabilities with high R^2 scores and relatively low errors, making them reliable alternatives. Acceptable performance but moderately less robust under varying conditions.
- Response Surface Method (RSM) performs moderately well but not as accurately as SVR or XGBoost. It Performs reasonably well but limited in modeling nonlinear effects.
- The Taguchi-based Linear Model shows the least accuracy, indicating that linear assumptions may not capture the complexity of the data.
- The dataset contains only 27 design points. XGBoost's boosting mechanism is better suited to extracting signal from limited data, whereas Random Forest may suffer from variance and instability due to its reliance on random sampling.

Table 2. Comparison of prediction errors for different modelling approaches

S.NO.	X1	X2	X3	X4	TEST DATA	ERROR (1) XG BOOST	ERROR (2) Support Vector Regression	ERROR (3) Random Forest Model	ERROR (4) RSM	ERROR (5) Taguchi based linear model
1	-1	-1	-1	-1	3.45	0.00417	1.881992	2.4313	5.6855	1.2872
2	-1	-1	0	0	3.506	0.01759	1.843218	5.8118	8.8615	9.5403
3	-1	-1	1	1	5.04	0.04101	13.593771	7.8218	2.9064	15.171
4	-1	0	-1	0	2.96	0.04866	2.182137	10.044	3.2257	15.895
5	-1	0	0	1	3.82	0.01813	1.69768	0.1592	3.997	1.1877
6	-1	0	1	-1	3.382	0.03393	1.510066	5.3223	3.4403	1.8409
7	-1	1	-1	1	3.982	0.00429	11.969532	3.882	5.11	13.225
8	-1	1	0	-1	3.41	0.00954	1.890602	0.5894	0.542	11.019

S.NO.	X1	X2	X3	X4	TEST DATA	ERROR (1) XG BOOST	ERROR (2) Support Vector Regression	ERROR (3) Random Forest Model	ERROR (4) RSM	ERROR (5) Taguchi based linear model
9	-1	1	1	0	4.25	0.01109	7.789693	6.0701	1.8906	18.373
10	0	-1	-1	-1	2.876	0.06474	12.210941	8.9374	9.097	10.568
11	0	-1	0	0	3.26	0.00498	1.97912	2.9571	0.3795	10.884
12	0	-1	1	1	4.35	0.01126	7.755219	10.061	10.997	6.9034
13	0	0	-1	0	2.62	0.01662	1.390413	3.4595	7.1134	22.321
14	0	0	0	1	2.48	0.10011	34.583173	19.399	24.555	46.762
15	0	0	1	-1	2.74	0.00662	2.354546	2.8204	6.7045	17.467
16	0	1	-1	1	2.87	0.00243	8.156142	2.7812	5.6782	12.533
17	0	1	0	-1	2.57	0.06384	7.118204	6.6553	0.686	9.2838
18	0	1	1	0	3.264	0.05341	1.972538	3.1918	5.8324	0.6286
19	1	-1	-1	-1	4.14	0.04976	14.287578	6.2609	1.5817	28.641
20	1	-1	0	0	4.03	0.00841	13.369167	2.9692	7.4024	15.902
21	1	-1	1	1	3.278	0.01165	8.031029	11.632	19.062	16.658
22	1	0	-1	0	2.774	0.01273	7.75765	7.3453	10.16	7.3954
23	1	0	0	1	4.02	0.05089	16.216281	10.197	18.946	15.074
24	1	0	1	-1	2.6	0.12073	2.485776	6.2008	2.5905	15.113
25	1	1	-1	1	3.3	0.00941	1.953034	0.7224	1.2278	8.9686
26	1	1	0	-1	2.836	0.0614	2.277031	0.5642	0.03	8.9236
27	1	1	1	0	2.76	0.03513	2.347153	4.4355	3.9862	9.3411

Table 3. Comparison of key performance metrics

S. No.	MODEL	MAE ↓	MSE ↓	RMSE ↓	MAPE ↓	R ²
1	Taguchi based Linear Model	0.4292	0.2690	0.5186	13%	0.355
2	Response Surface Method	0.2101	0.0819	0.2862	6.36%	0.8037
3	Support Vector regression	0.244	0.115	0.338	7.06%	0.7247
4	XGboost Regression	0.001	0.000002	0.001326	0.0323%	0.9999
5	Random Forest Regression	0.5463	0.4304	0.6561	17.58%	0.1755

5. Conclusion

The results clearly demonstrate that XGBoost Regression offers the best predictive performance among the evaluated models, with exceptionally low error values (MAE: 0.001, MSE: 0.000002, RMSE: 0.001326, MAPE: 0.0323%) and an R² of 0.9999, indicating near-perfect correlation with actual values. Importantly, the predicted optimum value from XGBoost is 2.48, which falls well within the target range of 2.349 to 2.768 established by previous studies. This value is also closer to the ideal surface roughness compared to prior research, which reported an optimum of 2.62. This suggests that XGBoost not only provides a more accurate model but also identifies more optimal conditions all 3' factors at medium levels for minimizing surface roughness. Furthermore, although other models (like RSM and SVR) also produced predictions within the acceptable range, some yielded slightly higher optimum response values than desired. XGBoost can be considered the most accurate model for this specific dataset, showing the strongest agreement with the

experimental results. However, its generalization to new or unseen data cannot be guaranteed without further validation steps such as:

- Cross-validation or external test datasets,
- Sensitivity and robustness analyses, and
- Hyperparameter optimization with regularization to prevent overfitting.

From an engineering point, while XGBoost offers high accuracy and flexibility, it should be applied with caution in experimental studies involving limited data. Incorporating larger datasets, applying regularization techniques, and conducting sensitivity or uncertainty analyses are essential to ensure reliable predictions. Future work should focus on integrating hybrid models and ensemble averaging to achieve a balance between prediction accuracy and physical interpretability.

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References

- [1] Bhattacharya S. Central composite design for response surface methodology and its application in Pharmacy. London: IntechOpen; 2021. <https://doi.org/10.5772/intechopen.95835>
- [2] Cui F, Su Y, Xu S, Liu F, Yao G. Optimization of the physical and mechanical properties of a spline surface fabricated by high-speed cold roll beating based on Taguchi theory. Math Probl Eng. 2018;2018:1-12. <https://doi.org/10.1155/2018/8068362>
- [3] Mounika G, Rajyalakshmi K, Rajkumar GVS, Sravani D. Prediction and optimization of process parameters using design of experiments and fuzzy logic. Int J Interact Des Manuf. 2024;18(4):2333-2343. <https://doi.org/10.1007/s12008-023-01446-x>
- [4] Nikhila Sri D, Kottapalli R, Pavani A, Ganteda C, Gouthami E, Abd-Elmonem A, et al. Comparison between response surface methodology and Taguchi method for dyeing process parameters optimization in fabric manufacturing by empirical planning. Sci Rep. 2025;15(1):1-10. <https://doi.org/10.1038/s41598-025-94919-w>
- [5] Rajyalakshmi K, Medatati V, Boggarapu NR. Optimal Fenton Process Using the Modified Taguchi Approach. Thai Stat. 2025;23(2):377-392.
- [6] Rajyalakshmi K, Nageswara Rao B. Modified Taguchi approach to trace the optimum GMAW process parameters on weld dilution for ST-37 steel plates. ASTM Int J Test Eval. 2019;47(4):3209-3223. <https://doi.org/10.1520/JTE20180617>
- [7] Romelin C, Zahedi, Nusantara B. Comparative Analysis of Response Surface Methodology (RSM) and Taguchi Method: Optimization Hydraulic Ram Pump Performance. Oper Res Forum. 2024;5:85. <https://doi.org/10.1007/s43069-024-00359-z>
- [8] Panigrahi BN, Nayak J, Bhatra J, Sahoo SS, Gountia D, Choudhury BK. Experimentation, Taguchi and machine learning-based optimisation of parameters concerning heat losses from the trapezoidal box-type solar cooker. Energy Sources Part A. 2024;46(1):60-76. <https://doi.org/10.1080/15567036.2024.2416084>
- [9] Ross PJ. Taguchi techniques for quality engineering. 2nd ed. New York: Tata McGraw Hill; 2005.
- [10] Rajyalakshmi K, Boggarapu NR. Expected range of the output response for the optimum input parameters utilizing the modified Taguchi approach. Multidiscip Model Mater Struct. 2018;15(2):508-522. <https://doi.org/10.1108/MMMS-05-2018-0088>
- [11] Rajyalakshmi K, Nageswara Rao B. Modified Taguchi approach to trace the optimum GMAW process parameters on weld dilution for ST-37 steel plates. ASTM Int J Test Eval. 2019;47(4):3209-3223. <https://doi.org/10.1520/JTE20180617>
- [12] Samuel BO, Alabi AA, Lawal SA, Peter E, Ibrahim TK. Optimizing the effect of heat treatment on the mechanical properties (tensile Strength and hardness) of Hyphaene Thebaica nut; A machine learning and Taguchi approach. Heliyon. 2024;10(19):e38899. <https://doi.org/10.1016/j.heliyon.2024.e38899>
- [13] Athreya S, Venkatesh YD. Application of Taguchi method for optimization of process parameters in improving the surface roughness of lathe facing operation. Int Ref J Eng Sci. 2012;1(3):13-19.
- [14] Varalakshmi M, Rajyalakshmi K, Nageswara Rao B. Optimization of process parameters using different statistical designs. J Eng Sci Technol. 2022;17(1):523-533.
- [15] Varalakshmi M, Rajyalakshmi K. Optimization of responses using balanced ternary designs. Int J Adv Sci Technol. 2020;29(5):4771-4775.

- [16] Varalakshmi M, Rajyalakshmi K, Charankumar G, Vu VD, Nguyen VD, Ngoc-Hung, et al. Multiresponse optimization of agricultural residues using modified taguchi approach and statistical conditioning of the data. *Int J Agricult Stat Sci.* 2021;17(1):1387-1394.
- [17] Lestari WD, Adyono N, Faizin AK, Haqiyah A, Sanjaya KH, Nugroho A, et al. Optimization of 3D printed parameters for socket prosthetic manufacturing using the taguchi method and response surface methodology. *Results Eng.* 2024;21:101847. <https://doi.org/10.1016/j.rineng.2024.101847>
- [18] Jou YT, Silitonga RM, Sukwadi R. A study on the construction of die-casting production prediction model by machine learning with Taguchi methods. *J Chin Inst Eng.* 2023;46(5):540-550. <https://doi.org/10.1080/02533839.2023.2204880>

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Unravelling the nonlinear behavior of stabilized mud bricks through inclusion morphology and mineralogical composition analysis

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Abstract

Compressed earth bricks remain a cost-effective and widely available construction material, and their stabilized forms are increasingly adopted worldwide. In this article, the nonlinear behavior of stabilized mud bricks is assessed, giving particular emphasis to the effect of inclusion morphology and mineralogical composition. The work combines the geotechnical and mineralogical characterization of the soil and silica fume with a micromechanical analysis based on numerical homogenization. Experimental tests will be compared to numerical predictions with the aim of assessing the material response. The results reveal that the increment of silica fume content from 8% to 10% provides a denser mix (17.97 to 18.18 kN/m³), with improved compressive performance. Inclusion morphology is also a significant factor in nonlinear behavior, as spherical inclusions provided stiffness predictions that were 12% and 18% closer to experimental curves for the 8% and 10% mixes, respectively. These observations underline the importance of accurate microstructural representation when modeling stabilized earth bricks. This integrated experimental-numerical methodology will provide important insight into the predominant mechanisms in stabilized mud bricks and will contribute to establishing more reliable and economic earthen construction materials.

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1. Introduction

Compressed stabilized earth bricks CEBs are gaining popularity worldwide due to their cost-effectiveness and remarkable hygrothermal performance. However, raw earth often requires stabilization to improve its mechanical behavior and durability. Stabilization techniques may include mechanical compaction, particle-size correction, or chemical stabilization through hydraulic binders or mineral additives, each contributing differently to compactness and porosity reduction. Numerous studies have therefore explored various stabilizers to enhance the performance of CEBs. For instance, ceramic wastes (50%) were used to improve compressive strength [1], while esparto fibres were incorporated to reduce thermal conductivity and increase flexural strength [2]. Other chemical stabilizers, including cement- and sand-based mixtures, have also been widely studied [3–6]. Other researchers have used natural pozzolan as an environmentally friendly alternative to cement in the manufacture of compressed earth blocks (CEBs) [4], others who have used Ground Granulated Blast Furnace Slag (GGBFS) as a binder [5]. Overall, these works show that additives can improve strength and durability; however, the

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resulting materials remain heterogeneous geomaterials whose macroscopic response is governed not only by stabilizer content but also by microstructural features.

Heterogeneous geomaterials exhibit complex micromechanical interactions between their constituents, whose contrasting behaviors strongly influence macroscopic response. Understanding these mechanisms is challenging because their intrinsic mechanical behavior cannot be directly assessed experimentally at the microscopic scale. Numerical homogenization therefore provides an efficient alternative for studying micro-macro interactions. Multiscale strategies have been successfully applied to a broad range of problems, including creep behavior in clays [8–10], micro-macro deformation in embankments [11], mechanical properties of fibre-reinforced composites [12], and nonlinear computational homogenization frameworks [13–17]. These contributions confirm the relevance of homogenization methods, yet only a limited number of studies have applied them to stabilized earth materials, particularly to quantify how inclusion morphology and mineralogical composition jointly influence nonlinear behavior. Although many studies have examined stabilizers for CEBs, and others have developed homogenization models for heterogeneous geomaterials, there is a lack of integrated experimental-numerical studies that explicitly link silica fume content, inclusion morphology, and mineralogical composition to the nonlinear macroscopic behavior of stabilized earth bricks. In particular, the effect of inclusion shape on the micromechanical response of silica-fume-stabilized mixtures remains insufficiently documented.

The present study addresses this gap by investigating the mechanical behavior of a tuff-silica fume mixture for CEB production through a combined experimental and numerical approach. The experimental program involves geotechnical, microstructural (XRD-TGA), and mechanical characterization of the constituents and their mixtures, which serves as a basis for constructing a representative volume element (RVE). A nonlinear numerical homogenization scheme is then applied to predict the macroscopic behavior, and the numerical results are systematically compared with experimental stress-strain curves to assess the accuracy of the proposed model.

The novelty of this work lies in the combined assessment of silica fume morphology, mineralogical composition, and nonlinear micromechanical modelling within the same framework, enabling a more accurate understanding of the mechanisms controlling stabilized earth behavior. The approach provides quantitative evidence of the influence of inclusion morphology on stiffness and strength, which is rarely addressed in previous CEB studies. The results show that the macroscopic behavior is strongly influenced by both the volume fraction and morphology of silica fume. The proposed homogenization approach offers a reliable predictive tool that reduces the need for costly and time-consuming experimental campaigns and contributes to improved design strategies for stabilized earth bricks.

2. Materials and method

2.1. Description of Material

2.1.1 Soil

The soil analyzed in this research originates from Beni-Saf, a tiny town situated in the Wilaya of Ain Temouchent, which is approximately 520 km from the capital city of Algiers in northwestern Algeria. Table 1 lists the soil's geotechnical features, revealing that it has a low concentration of large, malleable particles. The soil is slightly organic and contains significant amounts of CaCO_3 .

2.1.2 Silica Fume

Silica fume powder is a highly prized product for its rich silica content, active micro-silicates, and high specific surface area of 220.000 cm^2/g . Its composition is mostly made up of amorphous spherical SiO_2 particles, accounting for 93-98% of its total makeup. These minute particles, which are a few fractions of a micron in size, give pozzolan a granulometry of 0.05 to 0.15 μm .

Table 1. Geotechnical soil parameters of soil

Properties	Values	Standards
Gravel (%)	12	NF P 94-041 [18]
Sand (%)	48	
Limon (%)	21	
Clay (%)	19	NF P 94-057 [19]
Liquid limit (%)	37	NF P 94-051 [20]
Plasticity Index (%)	17	
Specific density	2,74	NF P 94-054 [21]
Calcium carbonate content (%)	85	NF P 94-048 [22]
Organic meter content (%)	1,8	NF P 94-047 [23]

2.2. Physico-Chemical Analysis Using The DRX-ATG Technique

To conduct a mineral analysis, the X-ray diffraction (XRD) technique was selected. A Rigaku miniflex 600 diffractometer was employed, with a scan range between 2 to 80°, a pitch of 0.02°, and a speed of 4°/minute. Rigaku's PDXL2 software was utilized to process the gathered data. The SDT Q600 device was utilized to carry out both differential thermal analysis (DTA) and thermogravimetric analysis (TGA) concurrently. The temperature range for the analysis was 20 to 1250°C.

The XRD pattern presented in Fig. 1 reveals that the soil mineralogy is mainly composed of quartz and calcite, accompanied by minor amounts of clay minerals and silicate phases, such as kaolinite. Quartz contributes to the granular skeleton of the material, enhancing stiffness and load-bearing capacity, while calcite may act as a cementing agent, promoting particle bonding and improved compressive strength.

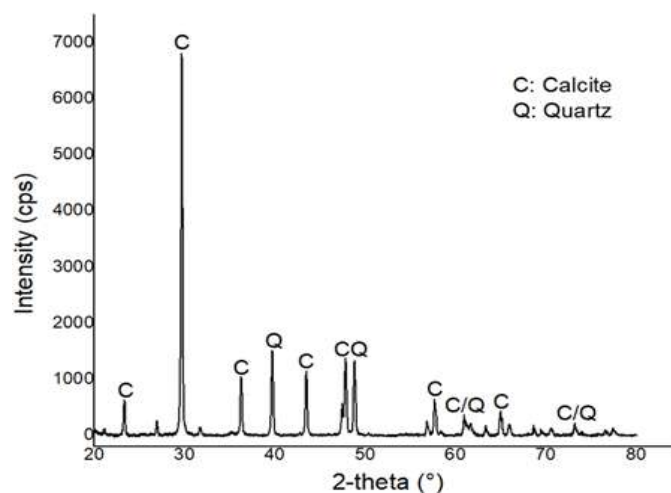


Fig. 1. Soil XRD Spectrum

The TGA/DTA curves shown in Fig. 2 exhibit several characteristic thermal events. The initial mass loss observed below approximately 150 °C is attributed to the evaporation of physically adsorbed water. A subsequent mass loss occurring between 450 and 650 °C is associated with the dehydroxylation of clay minerals, particularly kaolinite. The major mass loss detected at higher temperatures, typically between 700 and 900 °C, corresponds to the thermal decomposition of calcite (CaCO_3), confirming its presence as identified by XRD analysis.

The combined XRD and TGA results demonstrate the mineralogical stability of the soil constituents within the temperature range relevant to mechanical testing. The coexistence of a rigid quartz framework, calcite-induced cementation, and limited clay content explains the mechanical performance observed experimentally, particularly in terms of compressive strength and

structural integrity. These physico-chemical characteristics justify the use of the identified phases in the subsequent mechanical interpretation and numerical homogenization approach.

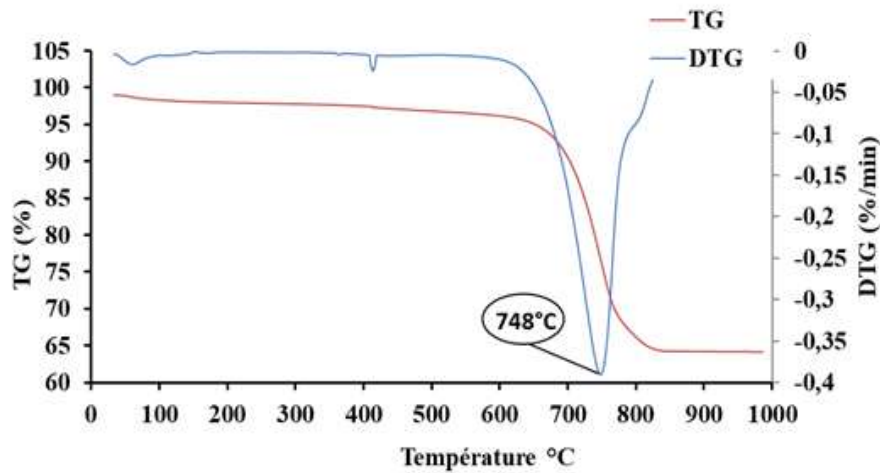


Fig 2. Soil thermogravimetric analysis

2.3. Preparation of Samples

In the industrial sector, compressed earth bricks are typically produced using static compaction. This manufacturing process renders the PROCTOR test unsuitable for determining optimal parameters, as the energy provided differs from that of static compaction. As such, a decision was made to utilize the static compaction method. Initially, a predetermined amount of dry soil was blended with varying amounts of water (ranging from 8% to 18%, with a 2% increment). The mixture was then homogenized and introduced into a double-piston mold for static compaction, ultimately achieving a 4MPa. Finally, density calculations were made for each water content value, based on the weight and volume of the compacted sample. As depicted in Fig 3, the water content fluctuates in relation to the dry density for both mixtures. The graph showcases a bell-shaped curve with a peak dry density that mirrors that of the PROCTOR test. It's important to note that the optimum water content for the mixture of soil and 8% silica fume was 11.42%, resulting in a maximum dry density of 17.97 KN/m³. In contrast, the optimum water content for the mixture of soil and 10% silica fume was 12.3%, yielding a maximum dry density of 18.18 KN/m³. The silica fume contents of 8% and 10% were selected based on the static compaction results presented in Fig. 3, which showed well-defined compaction curves and high maximum dry densities for both mixtures.

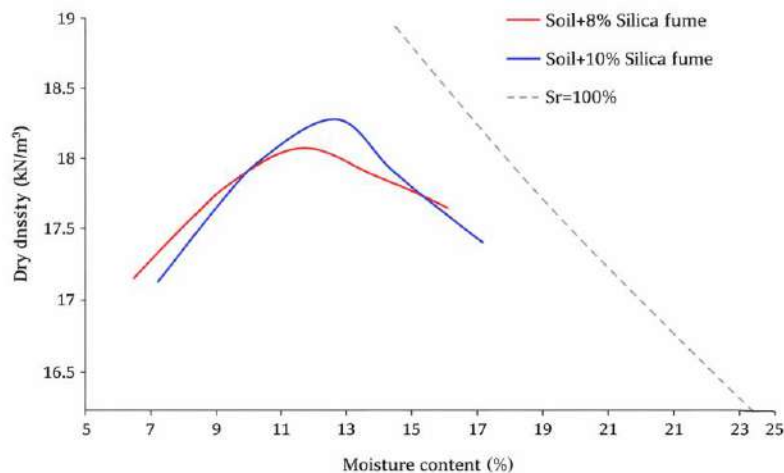


Fig. 3. Static compaction curve

To prepare the samples, the static compaction method was employed with predetermined optimal parameters of dry density and water content. Cylindrical specimens, measuring 10 cm in height and 5 cm in diameter, were manufactured, with a total of five identical samples prepared for each formulation in order to ensure statistical reliability and experimental reproducibility. The specimens were then cured under open-air laboratory conditions for 28 days at a temperature of $(20 \pm 2)^\circ\text{C}$ and a relative humidity of $(50 \pm 2)\%$. These curing conditions were selected to promote moisture stabilization and to guarantee the reproducibility of the mechanical properties prior to compression testing. After the curing period, uniaxial compression tests were performed.

2.4. Compressive Behavior Method

To conduct a compressive strength test, a hydraulic press equipped with a 50KN force sensor was used in accordance with the Standard EN 1015-11/A1[24]. The test was conducted at a steady speed of 1mm/min. Prior to the test, the sample surfaces were meticulously cleaned to ensure consistent contact with the compression platens.

3. Modelling Procedure

The numerical homogenization framework was employed to predict the macroscopic mechanical behavior of the tuff-silica fume composite based on its microstructural characteristics. The spatial distribution of silica fume particles within the tuff matrix was derived from scanning electron microscopy (SEM) observations (Fig. 4). The porosity of the matrix was explicitly incorporated into the numerical model to ensure consistency with the experimentally compacted material. Three representative volume element (RVE) configurations were investigated to evaluate the influence of inclusion morphology: a spherical inclusion, a cubic inclusion, and a configuration composed of eight randomly distributed spherical inclusions (Fig. 5). The dimensions of the RVE were defined to ensure phase partitioning between the tuff matrix and the silica fume inclusions.

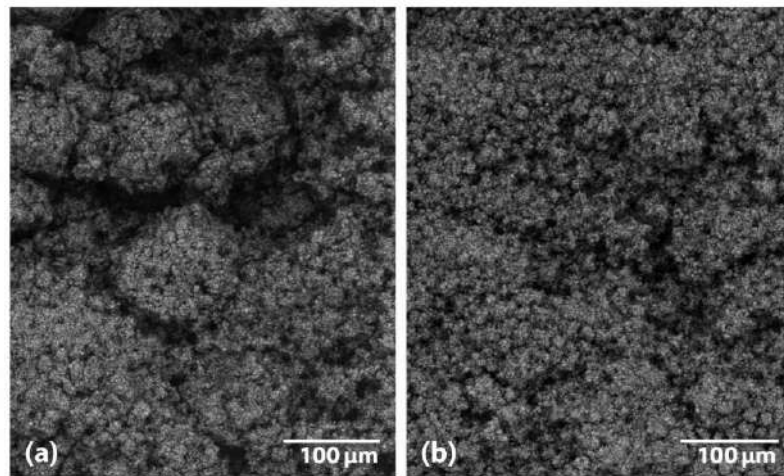


Fig 4. Scanning electron microscopy: (a) Tuff + 8% silica fume, (b) Tuff + 10% silica fume

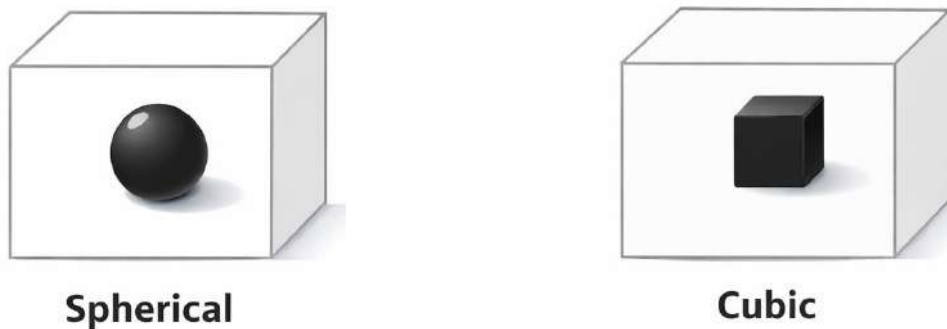




Fig. 5. 3D schematic representation of unit cell for spherical, cubic, and random 8-sphere reinforcement

The mechanical behavior of the tuff matrix was modeled using an elastoplastic formulation with isotropic hardening (NLISO), governed by the Voce law expressed in Eq. (1):

$$H(\varepsilon_e^p) = R_0 \cdot \varepsilon_e^p + R_{inf} \cdot (1 - e^{-b \cdot \varepsilon_e^p}) \quad (1)$$

where σ_0 represents the initial yield stress, R_0 is the initial hardening modulus, R_{inf} denotes the saturation value of isotropic hardening, and b controls the rate of hardening evolution. These parameters characterize the nonlinear compressive response of the tuff matrix and were identified through a least-squares fitting procedure based on experimental uniaxial compression data. The calibrated parameters are summarized in Table 2.

The silica fume phase was assumed to behave as a linear elastic material, consistent with its mineralogical nature and previous studies. Elastic properties for both phases were adopted from the literature [25] and are reported in Table 2.

Table 2. Mechanical parameters of the material

Mechanical properties		Soil matrix	Silica fume
Elastic parameters [25]	E (MPa)	84000	96400
	ν	0,3	0,08
Voce's plastic model parameters	σ_0 (MPa)	0.009	
	R_0 (MPa)	50.47	
	R_{inf} (MPa)	1.515	
	b (MPa)	100.05	

Macroscopic stress and strain tensors were obtained through volume averaging of the microscopic fields according to Eq. (2):

$$\bar{\sigma}_{ij} = \frac{1}{V} \int_V \sigma_{ij} dV \quad \bar{\varepsilon}_{ij} = \frac{1}{V} \int_V \varepsilon_{ij} dV \quad (2)$$

The RVE geometry was defined such that the volume fractions of the matrix and the reinforcement phases satisfy the unit partition condition given by Eq. (3):

$$f_t + f_s = 1 \quad (3)$$

Numerical simulations were performed using the ANSYS finite element software. Symmetry and periodic boundary conditions were applied following the methodology proposed by Belayachi et al. [15], ensuring a homogeneous macroscopic strain state within the RVE. A uniform displacement was imposed to reproduce uniaxial compression loading conditions consistent with the experimental tests.

The finite element mesh consisted of 10-node tetrahedral elements (SOLID187). A mesh sensitivity analysis was conducted by progressively refining the mesh, particularly around the silica fume inclusions, until variations in the macroscopic stress-strain response became negligible. The

adopted mesh (Fig. 6) ensures numerical convergence while maintaining reasonable computational efficiency. The numerical stress-strain responses obtained from the homogenization procedure were quantitatively compared with experimental results, showing good agreement in terms of elastic stiffness, yield stress, and post-yield behavior, thereby validating the proposed numerical model.

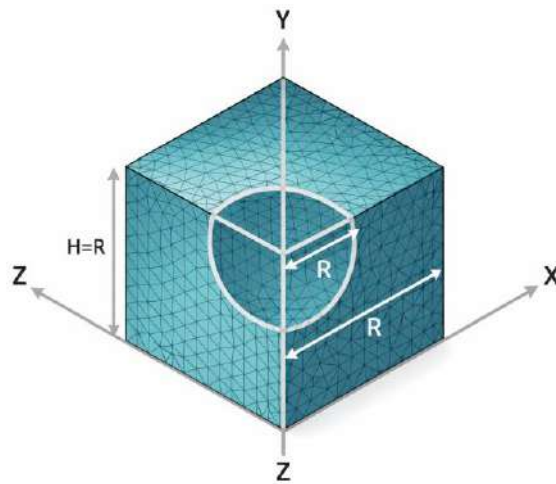


Fig. 6. Example of mesh for the case of a spherical inclusion

4. Results and Discussions

4.1. Compressive Behavior

The stress-strain outcomes are depicted in Fig 7 for the two mixtures that were investigated. It is apparent that the mixture containing soil and 10% silica fume exhibits greater strength than the mixture with soil and 8% silica fume, for the same displacement value. This finding suggests that while silica fume content can improve compressive strength, it is not the sole determinant. In addition, silica fume can bolster soil strength via densification mechanisms. Firstly, silica fume particles fill soil voids, thereby reducing porosity. Secondly, a chemical reaction between silica fume and soil minerals can stabilize the soil, improving its strength and stiffness. This densification was evident in the samples, which were able to endure more stress before breaking.

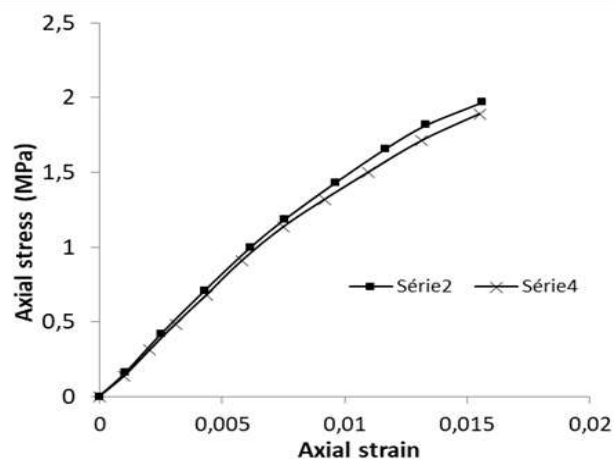


Fig 7. Stress-strain curve of the two mixtures

4.2. Effect of Silica Volume Fraction on Numerical Response

To investigate the impact of silica fume content on the mechanical properties of the material, uniaxial test simulations were performed using various inclusion types, as shown in Table 2. The stress-strain curves of each inclusion type are depicted in Fig 8. The results show a slight difference in the case of a spherical inclusion, but a cubic inclusion's rigidity increased with a 2% silica fume

variation. Interestingly, the curves for 8 spherical inclusions had overlapping results for both silica fume contents. These findings emphasize that silica fume content and inclusion type are crucial factors impacting the material's mechanical properties.

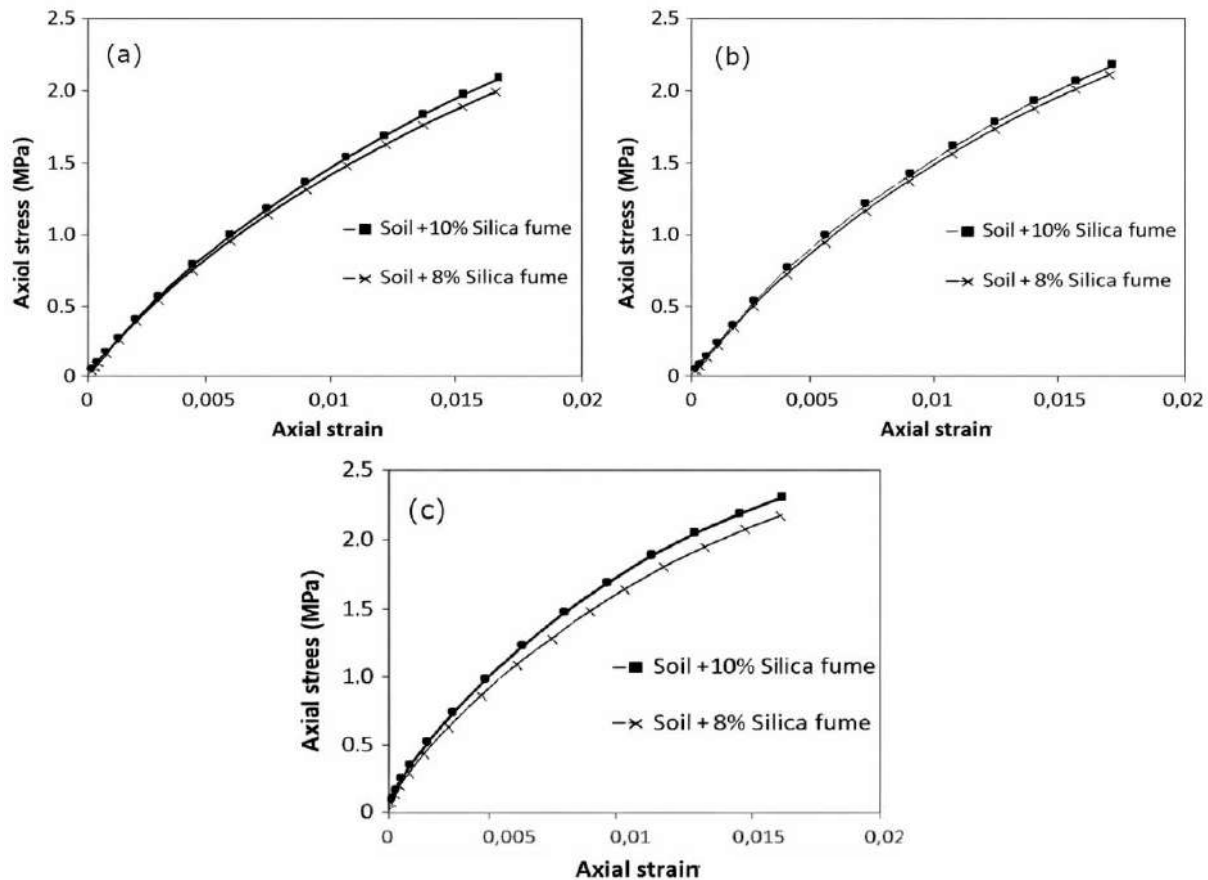


Fig. 8. Macroscopic numerical stress-strain behavior for: (a) One spherical inclusion, (b) Eight spherical inclusions, (c) One cubic inclusion

4.3. Effect of Inclusion Morphology

Fig (9) presents the macroscopic stress-strain curves for different inclusion shapes. The data clearly shows how significant the shape of the silica inclusion can be. For instance, when comparing the spherical inclusion to the cubic inclusion in the mixture of soil+8% silica fume, there was a 12% difference.

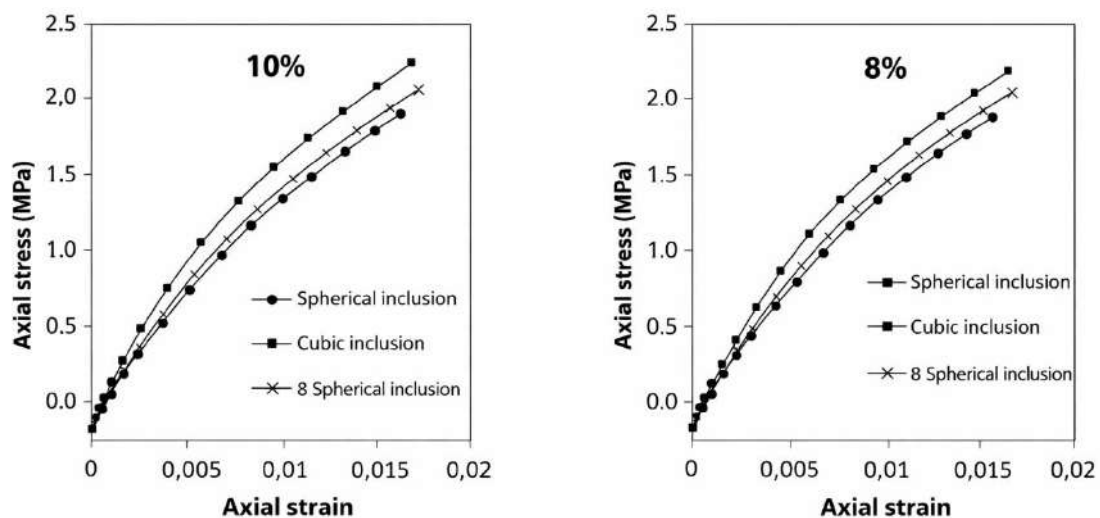


Fig. 9. Effect of inclusion morphology on the stress-strain behavior for the two silica fume contents

This gap was even more pronounced, at 18%, in the case of soil+10% silica fume mixture, particularly at high stress and strain levels. The impact of inclusion shape on the effective response of the material was also more significant. This observation was also supported by the study conducted by Belayachi N. et al. [15]. Recent research has revealed that hard inclusions bear less stress than their softer counterparts. Additionally, shapes with corners tend to create a plastic zone, which in turn generates a strong plasticity gradient, resulting in a reduction of stress at these corners. These findings highlight the significance of considering the shape of inclusions while designing composite materials.

4.4. Comparison of Experimental and Numerical Macroscopic Responses

The stress-strain curves for three different shapes of inclusions, as well as the experimental curves, are displayed in Fig 10. It is noteworthy that the responses for all shapes, and for the two silica fume contents, are almost identical, except for the cubic inclusion. In this case, it was observed that the rigidity increased with the silica fume content. The numerical and experimental stress-strain curves for the spherical inclusion, for the two silica fume contents, overlap, indicating that the spherical inclusion is a better representation of the actual microstructure of silica fume grains. This finding has been echoed in other research studies, including Armengaud Julie's work [26], which analyzed scanning electron microscopy images of silica smoke particles and found that they were approximately 0.1 μm in spherical shape. These findings are consistent with prior experimental research by Aïtcin and Mindess [27], as well as Vivek Bindiganaville and Banthia [28].

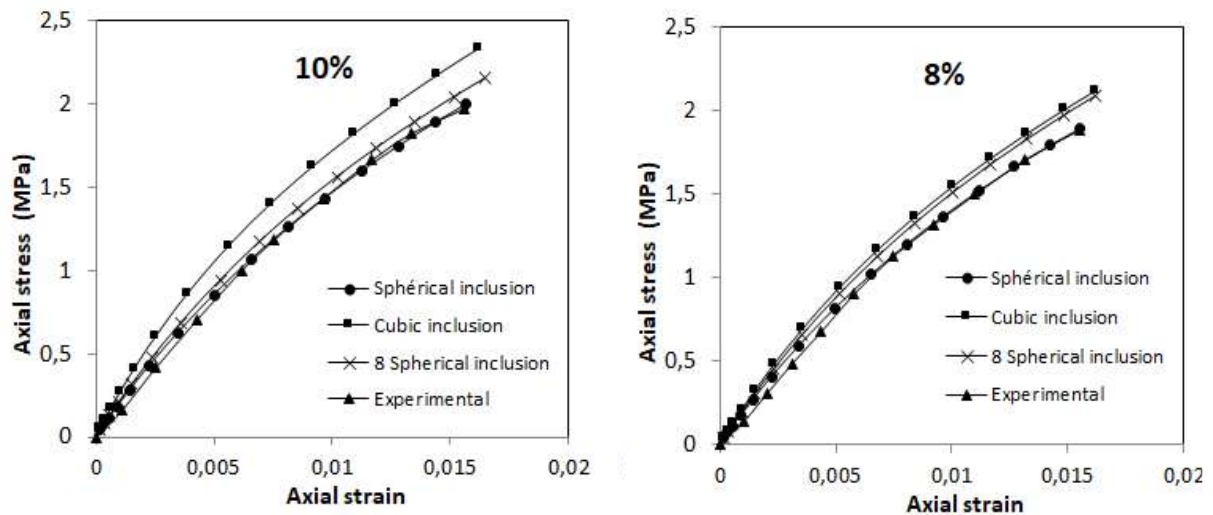


Fig. 10. Comparison of stress/strain behavior for the different inclusions with experimental results

To quantitatively assess the agreement between experimental and numerical responses (Fig. 10), error indicators were computed, including RMSE, MAPE and the coefficient of determination (R^2). For the 8% silica fume mixture, the best matching numerical configuration yields $\text{RMSE} \approx 0.12 \text{ MPa}$, $\text{MAPE} \approx 9.8\%$ and $R^2 \approx 0.92$, indicating a strong agreement over the investigated strain range. For the 10% mixture, the deviation is higher ($\text{RMSE} \approx 0.16 \text{ MPa}$, $\text{MAPE} \approx 18.6\%$ and $R^2 \approx 0.86$), suggesting a greater sensitivity of the response to microstructural representation and inclusion morphology. Overall, the quantitative metrics confirm that the proposed homogenization framework captures the global nonlinear trend, with a higher accuracy for the 8% mixture under the present modelling assumptions.

5. Conclusions

This study assessed the nonlinear mechanical behavior of stabilized mud bricks by coupling experimental characterization with micromechanical modelling based on nonlinear numerical homogenization. The experimental program and microstructural observations confirmed that the macroscopic response of the stabilized material depends not only on silica fume content but also

on inclusion morphology and mineralogical features, which must be accounted for when interpreting or predicting stress–strain behavior.

The proposed homogenization framework proved capable of reproducing the overall nonlinear trend observed experimentally. Quantitative validation against the experimental curves (Fig. 10) showed good agreement for the 8% mixture (RMSE $\approx$ 0.12 MPa, MAPE $\approx$ 9.8%, $R^2 \approx$ 0.92) and a larger deviation for the 10% mixture (RMSE $\approx$ 0.16 MPa, MAPE $\approx$ 18.6%, $R^2 \approx$ 0.86), indicating an increased sensitivity to microstructural representation at higher stabilizer content. These results provide evidence that inclusion morphology plays a non-negligible role in the predicted macroscopic behavior and support the relevance of microstructure-informed modelling for stabilized earth materials.

The main limitations of the present work are related to the simplified microstructural representation and the modelling assumptions adopted for the phases. Future work will focus on improving microstructural realism (e.g., inclusion size distribution, porosity and phase connectivity), refining phase constitutive behavior, and extending the approach to durability-related performance (wetting–drying cycles, thermal effects and long-term behavior).

Acknowledgement

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References

- [1] Seco A, Omer J, Marcelino S, Espuelas S, Prieto E. Sustainable unfired bricks manufacturing from construction and demolition wastes. *Constr Build Mater.* 2018;167:154-165. <https://doi.org/10.1016/j.conbuildmat.2018.02.026>
- [2] Ajouguim S, Talibi S, Djelal-Dantec C, Hajjou H, Waqif M, Stefanidou M, et al. Effect of Alfa fibers on the mechanical and thermal properties of compacted earth bricks. *Constr Build Mater.* 2020;239:117669. <https://doi.org/10.1016/j.matpr.2020.07.539>
- [3] Islam MS, Tausif-E-Elahi, Shahriar AR, Nahar K, Hossain TR. Strength and durability characteristics of cement-sand stabilized earth blocks. *J Mater Civ Eng.* 2020;32(5):04020087. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0003176](https://doi.org/10.1061/(ASCE)MT.1943-5533.0003176)
- [4] Bennis A, Zniker H, Ghassane A, et al. Eco-friendly production of compressed earth blocks with Moroccan pozzolana as a cement alternative. *Res Eng Struct Mater.* 2025;11(3):1-19. <https://doi.org/10.17515/resm2025-1001ma0702rs>
- [5] Das S. Mix screening study of laterite soil stabilization using one-part geopolymer binders for earth blocks. *Res Eng Struct Mater.* 2025. <http://dx.doi.org/10.17515/resm2025-1008ma0706rs>
- [6] Taallah B, Guettala A. The mechanical and physical properties of compressed earth block stabilised with lime and filled with untreated and alkali-treated date palm fibers. *Constr Build Mater.* 2016;104:52-62. <https://doi.org/10.1016/j.conbuildmat.2015.12.007>
- [7] Norouzi E, Li B, Erkmen RE. Modelling equivalent elastic properties of imperfectly bonded soil-rock mixtures using an XFEM-based computational homogenization. *Comput Geotech.* 2022;144:104638. <https://doi.org/10.1016/j.compgeo.2022.104638>
- [8] Dahhaoui H, Belayachi N, Zadjiaoui A, Duc M. Modélisation du Comportement Différé de la Bentonite Compacte par une Méthode d'Homogénéisation Numérique. *Acad J Civ Eng.* 2017;35(1):35-38.
- [9] Dahhaoui H, Belayachi N, Zadjiaoui A, Nishimura T. One-dimensional compression creep change under temperature and suction effects. *Int J Geotech Eng.* 2022;16(6):670-681. <https://doi.org/10.1080/19386362.2021.2025306>
- [10] Dahhaoui H, Belayachi N, Zadjiaoui A. Modeling of Creep Behavior of an Argillaceous Rock by Numerical Homogenization Method. *Period Polytech Civil Eng.* 2018;62(2):462-469. <https://doi.org/10.3311/PPci.11697>
- [11] Xu WJ, Wang S, Zhang HY, Zhang ZL. Discrete element modelling of a soil-rock mixture used in an embankment dam. *Int J Rock Mech Min Sci.* 2016;86:141-156. <https://doi.org/10.1016/j.ijrmms.2016.04.004>
- [12] Dehaghani PF, Ardakani SH, Bayesteh H, Mohammadi S. 3D hierarchical multiscale analysis of heterogeneous SMA based materials. *Int J Solids Struct.* 2017;118-119:24-40. <https://doi.org/10.1016/j.ijsolstr.2017.04.025>
- [13] Yuan Z, Fish J. Toward realization of computational homogenization in practice. *Int J Numer Methods Eng.* 2008;73(3):361-380. <https://doi.org/10.1002/nme.2074>

- [14] Zhang F, Wang T, Liu F, Peng M, Furtney J, Zhang L. Modeling of fluid-particle interaction by coupling the discrete element method with a dynamic fluid mesh: Implications to suffusion in gap-graded soils. *Comput Geotech.* 2020;124:103617. <https://doi.org/10.1016/j.compgeo.2020.103617>
- [15] Belayachi N, Do DP, Hoxha D. A note on the numerical homogenisation of the mechanical behaviour of an argillaceous rock. *Comput Geotech.* 2012;41:70-78. <https://doi.org/10.1016/j.compgeo.2011.11.007>
- [16] Abou-Chakra Guéry A, Cormery F, Shao JF, Kondo D. A micromechanical model of elastoplastic and damage behavior of a cohesive geomaterial. *Int J Solids Struct.* 2008;45(5):1406-1429. <https://doi.org/10.1016/j.ijsolstr.2007.09.025>
- [17] Ghosh S, Lee K, Moorthy S. Two scale analysis of heterogeneous elastic-plastic materials with asymptotic homogenization and Voronoi cell finite element model. *Comput Methods Appl Mech Eng.* 1996;132(1-2):63-116. [https://doi.org/10.1016/0045-7825\(95\)00974-4](https://doi.org/10.1016/0045-7825(95)00974-4)
- [18] AFNOR Standards XP P 94-041. Soils investigation and testing. Granulometric description. Wet sieving method. Saint-Denis: AFNOR; 1995.
- [19] AFNOR Standards NF P 94-057. Soils investigation and testing. Granulometric analysis. Hydrometer method. Saint-Denis: AFNOR; 1992.
- [20] AFNOR Standards NF P 94-051. Soils investigation and testing. Determination of Atterberg's limits. Liquid limit test using Cassagrande apparatus. Plastic limit test on rolled thread. Saint-Denis: AFNOR; 1993.
- [21] AFNOR Standards NF P 94-054. Soils investigation and testing. Determination of particle density. Pycnometer method. Saint-Denis: AFNOR; 1991.
- [22] AFNOR Standards NF P 94-048. Soils investigation and testing. Determination of the carbonate content - Calcimeter method. Saint-Denis: AFNOR; 1996.
- [23] AFNOR Standards NF P 94-047. Soils investigation and testing - Determination of the organic matter content - Ignition method. Saint-Denis: AFNOR; 1998.
- [24] AENOR. UNE EN 1015-11/A1: Methods of test for mortar for masonry. Part 11: Determination of flexural and compressive strength of hardened mortar. Madrid: AENOR; 2007.
- [25] Giraud A, Hoxha D, Do DP, Magnenet V. Effect of pore shape on effective thermo-poroelastic properties of isotropic rocks. *Int J Solids Struct.* 2008;45(1):1-23. <https://doi.org/10.1016/j.ijsolstr.2007.07.005>
- [26] Armengaud J. Optimisation des propriétés des bétons projetés par voie sèche [dissertation]. Toulouse: Université Toulouse 3 Paul Sabatier; 2016.
- [27] Aïtcin PC, Mindess S. Ecostructures en béton: Comment diminuer l'empreinte carbone des structures en béton. Mayenne: Eyrolles; 2013.
- [28] Bindiganavile V, Banthia N. Rebound in Dry-mix Shotcrete: Influence of Type of Mineral Admixture. *ACI Mater J.* 2000;97(2):115-119. <https://doi.org/10.14359/813>

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Stress analysis of aircraft tow hook using non-destructive testing techniques and numerical simulation

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Abstract

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Non-destructive testing was applied to an aircraft tow hook made of forged AISI 4130 alloy steels to evaluate its structural integrity under service conditions. Five inspection methods including magnetic particle, liquid penetrant, eddy current, ultrasonic, and radiographic testing were applied to the same component and correlated with finite element stress analysis. Eddy current, ultrasonic, and radiographic methods proved ineffective due to material ferromagnetism, complex geometry, and field equipment limitations, while liquid penetrant testing detected surface indications but suffered from a high level of non-relevant responses. In contrast, magnetic particle testing reliably identified a linear surface crack approximately 13 mm long at the inner curvature near the attachment region. Finite element analysis indicated a yield utilization of approximately 43%, corresponding to a safety factor of about 2.3 under nominal towing loads, and fracture-mechanics evaluation yielded a critical crack length of 19.6 mm, indicating that the observed defect had reached about 66% of the critical value. The strong agreement between experimental crack detection and numerical stress predictions demonstrates the value of integrating targeted NDT with finite element analysis and confirms magnetic particle testing as the most reliable inspection method for forged AISI 4130 aircraft tow hooks under realistic field conditions.

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1. Introduction

Aircraft ground handling includes towing operations in which the tow hook must safely withstand operational forces to prevent structural failure [1]. If a tow hook fractures or fails under load, the aircraft cannot be moved and may suffer Foreign Object Damage from debris, potentially leading to serious incidents [2,3]. Therefore, maintaining the integrity of tow hooks through proper design, material selection, and routine inspections is essential for flight safety [4,5]. Aviation authorities emphasize that ground equipment like tow bars and tow hooks receive regular maintenance and non-destructive inspections to prevent costly accidents [5,6].

Numerous studies have highlighted the importance of non-destructive testing (NDT) techniques in the aviation industry. For instance, Ulus et al. surveyed common NDT methods used in aircraft maintenance and showed their effectiveness in detecting surface cracks on airframe components [7]. Demircioğlu et al. focused on composites and demonstrated how methods like ultrasonic and radiographic inspection can reveal subsurface flaws in complex materials [8]. Ibrishev et al. emphasized that although effective NDT techniques are available for finished products, repairing damage that occurs during aircraft operation remains complex [9]. Broader reviews by Gupta et al.

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and Rao document advances in NDT technology and applications, underscoring that timely detection of cracks or damage can avert failures in service [10,11]. In particular, magnetic particle and liquid penetrant inspections are noted for their success in identifying fine surface discontinuities in steel parts [12]. Magnetic particle testing is widely used to detect surface and near-surface cracks in ferromagnetic materials [13]. It is an essential, highly sensitive, and user-friendly non-destructive technique for such applications [14]. On the numerical side, the Finite Element Method (FEM) has become a powerful tool for stress analysis of mechanical and aerospace components. Petrici et al. used FEM to obtain the stress and strain field in the towbar and the towing system. The experimental tests validate the theoretical results obtained [15]. In the context of vehicle and aircraft hardware, FEM-based studies have been conducted on towing assemblies and related parts. For instance, Aftab et al. developed a numerical model for a Boeing 737 nose landing gear to ensure its structural safety under operational loads [16]. Yadav et al. used structural finite element analysis (FEA) to analyze the nose gear's stress behavior and displacement during landing, as well as to assess the effect of the runway gradient [17]. More recently, Lin et al. combined finite element simulations with experimental modal testing for a B-727 aircraft nose landing gear, evaluating its strength and vibration performance during towing operations [18]. These works show the state-of-the-art in experimental NDT and computational analysis; however, most studies treat these approaches separately. There is a research gap in integrating multiple NDT methods with FEM for the same component to cross-verify findings. While some attempts at validation of experimental results with modelling exist in other fields [19], in aerospace components such combined studies are scarce. This gap is critical: a unified approach could improve defect detection and structural assessment by leveraging the strengths of both NDT and FEM.

In light of the above, the present work provides an integrated evaluation of an aircraft tow hook using five different NDT techniques in conjunction with finite element stress analysis. This dual approach is novel, as previous literature has not extensively reported correlating NDT findings with FEM predictions for towing hardware. By comparing experimental NDT results with simulation, the study identifies critical stress regions and verifies whether detected discontinuities align with high-stress zones – an important cross-check that enhances confidence in the findings. Unlike prior works that might focus only on a single NDT method or purely on analysis, this study uses a comprehensive suite (magnetic particle, liquid penetrant, eddy current, ultrasonic, and radiographic testing) on the same component, providing a broad assessment of each method's effectiveness. The key outcomes, including the superior crack-detection capability of magnetic particle testing and the FEM-calculated safety factor of the tow hook, fill the knowledge gap by demonstrating how numerical simulation can validate and complement NDT in identifying potential failure points. In comparison to similar studies, our results show agreement in general trends (e.g., known sensitivity of magnetic particle tests [13]) but also highlight where certain methods (ultrasonic, radiography) may fall short on complex geometries – a nuance not thoroughly documented before for tow hooks. The scientific contribution of this work lies in this holistic inspection methodology. It provides aviation maintenance engineers with a validated inspection strategy: focusing on the most sensitive NDT method for the given material and geometry, and using FEM as a diagnostic tool to interpret and reinforce the experimental observations. Ultimately, the findings of this study can contribute to the development of more reliable tow hook inspection routines and inform design improvements, thereby enhancing safety margins and reducing the risk of towing-related incidents.

2. Materials and Methodology

2.1 Material Properties

Forged components made of AISI 4130 alloy steel are widely used in the oil and gas, aerospace, agricultural, and defense industries, where reliable strength and toughness are required. Typical applications include forged valve bodies, pumps, and other medium- to low-stress structural parts. Fig. 1 shows the aircraft tow hook examined in this study, which was manufactured from AISI 4130 steel by hot forging at approximately 1000 °C. After forging, the hooks were oil-quenched from 850

°C and subsequently double-tempered at 560 °C and 530 °C, respectively, to achieve a balanced combination of strength and ductility.



Fig. 1. Aircraft tow hook

The typical mechanical properties of the heat-treated AISI 4130 alloy steel used in this study are presented in Table 1.

Table 1. Mechanical properties of AISI 4130 steel [20,21]

Property	Value	Unit
Yield strength	630	MPa
Ultimate tensile strength	800	MPa
Hardness	380	HV
Young's modulus	205	GPa
Poisson's ratio	0.29	–
Density	7.85	g/cm ³
Fracture toughness	75	MPa·m ^{1/2}

According to Linear Elastic Fracture Mechanics (LEFM), the critical crack length for AISI 4130 can be determined using Eq (1) [22].

$$K_I = Y \sigma \sqrt{\pi a} \Rightarrow a_c = \frac{1}{\pi} \left(\frac{K_{IC}}{Y \sigma} \right)^2 \quad (1)$$

where, K : Stress intensity factor, σ : Applied normal stress, a : Crack length, Y : Geometry factor, a_c : Critical crack length, K_{IC} : Fracture toughness

For a conservative estimation, the crack observed on the tow hook can be assumed to behave as an edge or surface crack in a wide plate. Under this assumption, the geometry factor Y is taken as 1.12. For the maximum stress of 270 MPa obtained in this study, the corresponding critical crack length is calculated to be 19.6 mm. To account for uncertainty in crack geometry and depth, a brief sensitivity consideration was conducted by varying the geometry factor within a reasonable range ($Y = 1.0$ – 1.25), consistent with shallow-to-moderate surface crack configurations reported in the literature. This variation yields a critical crack length range of approximately 17.5–22 mm. Within this range, the experimentally observed crack length of approximately 13 mm corresponds to about 60–75% of the critical value, confirming that the crack has progressed significantly toward instability even under conservative assumptions.

It should be noted that magnetic particle inspection provides reliable information on surface crack length, while the crack depth and exact crack shape cannot be directly determined using this method. Therefore, the fracture-mechanics evaluation presented here is intended as a screening-level assessment, based on a conservative surface-crack assumption. The objective of this analysis is not to predict the exact remaining life, but to evaluate whether the detected crack length approaches a critical threshold under nominal towing loads.

2.2. Research Methodology

The methodology combines standardized non-destructive testing (NDT) on the actual part with finite element method (FEM) on a geometrically accurate model under towing loads. After material/property confirmation (Table 1), five NDT methods were applied per Table 2, then results were compared with ANSYS-based stress/deformation maps to confirm whether indications align with predicted hot-spots. Using complementary methods mitigates false negatives from any single technique; FEM supplies a physics-based expectation of crack-prone zones to guide interpretation and future inspections. The brands and models of the test devices used in this study and the standards followed for each test are summarized in Table 2.

Table 2. Testing devices and relevant standards

Non-destructive testing method	Standards	Test device brand and model
Magnetic particle	ISO 9934-1/2/3, ISO 3059, ASTM-E1444	Magnaflux ZA-1633
Ultrasonic	ISO 17640, 16809, EN 12668-1	Eddyfi Panther 2/Mantis/Cypher
Liquid penetrant	ISO 3452-1, ISO 3059	Magnaflux ZA-28/37/915
Radiographic	ISO 17636-1/2, 19232	TroyMet
Eddy current	ISO 17643	Olympus ECT systems

2.2.1 Eddy Current Testing

Eddy current testing was performed using a surface probe with a differential ferrite core in accordance with ISO 17643. 100 kHz was selected as the nominal operating frequency due to the ferromagnetic nature of AISI 4130 steel. Under typical inspection conditions, DC pre-magnetization is applied to stabilize magnetic permeability and improve signal-to-noise ratio; however, the required equipment was not available in the present study, and the inspection was therefore conducted without pre-magnetization. Calibration was performed using an AISI 4130 reference block containing 0.5, 1.0, and 2.0 mm electrical-discharge machining notches, and the amplitude and phase responses were verified across the frequency sweep. This configuration was selected due to known limitations of eddy current testing in ferromagnetic steels.

2.2.2 Ultrasonic Testing

Ultrasonic testing was performed under the best achievable field conditions permitted by the available equipment. Ultrasonic testing was conducted using a portable flaw detector equipped with a 5 MHz straight-beam longitudinal-wave contact probe with a 10 mm active diameter. Calibration was performed on a steel reference block containing side-drilled holes in accordance with ISO 17640 and EN 12668-1. A glycerin-based couplant was applied to ensure acoustic coupling, and the inspection gate was set from the surface echo to a sound-path depth of 60 mm.

Despite proper calibration and parameter control, the complex curved geometry and variable cross-section of the forged tow hook caused beam distortion, multiple internal reflections, and poor signal-to-noise ratio. Advanced ultrasonic configurations such as phased-array, immersion testing, or custom angle-beam wedges were not available at the test facility. Under these practical conditions, conventional straight-beam ultrasonic testing did not yield interpretable flaw indications and was therefore deemed unsuitable for reliable inspection of this component.

2.2.3 Radiographic Testing

Radiographic inspection was performed in accordance with ISO 17636-1:2021 (film technique for metallic materials) and ISO 19232 (IQI classification). The TroyMet industrial X-ray unit (tungsten target, constant potential) was operated at the parameters with their values are shown in Table 3.

The film type used in the radiographic test was AGFA D7 (Class C, medium-speed, EN 584-1), used together with 0.03-mm lead intensifying screens placed on both the front and back sides. Processing was performed under red-safe light using manual developer and fixer cycles, including 5 minutes of development, followed by fixing, washing, and drying. A Type 2 ISO 19232-1 IQI (wire

type, Fe) was placed on the source side; the smallest visible wire was No. 10 (1.0 mm), giving a basic IQI sensitivity of 2 %. The processed film is then examined on a light inspection panel.

Table 3. Values of parameters for radiographic testing

Parameters	Current	Duration	Voltage	Application Distance	Focal-spot size
Test Values	4 mA	90 s	200 kV	915 mm	2 mm

2.2.4 Liquid Penetrant Testing

Fluorescent penetrant inspection was conducted in accordance with ISO 3452-1:2021 and ISO 3059. A post-emulsifiable fluorescent penetrant system comprising Magnaflux ZL-37 penetrant, ZE-4B emulsifier, and ZP-9F non-aqueous wet developer was used. Prior to testing, the surface was cleaned with the SKC-S solvent cleaner and air-dried. The penetrant was applied and allowed to dwell for 20 ± 2 minutes at 22 ± 3 °C. Excess penetrant was removed by post-emulsification for 60 seconds, followed by rinsing with a low-pressure water spray (< 0.35 MPa, 25–40 °C). The component was then dried using hot air at 50 ± 5 °C for 5 minutes. The developer was subsequently applied to achieve a coating thickness of 0.02–0.05 mm, and a developer dwell time of 10 minutes was observed. Inspection was performed under UV-A illumination at 365 nm with an intensity greater than $1000 \mu\text{W}/\text{cm}^2$ at a distance of 38 cm, while ensuring that ambient white-light levels remained below 20 lx, as specified in ISO 3059. System sensitivity was verified using a Type 1 NiCr cracked reference panel (JIS K C-1 equivalent) before examination.

2.2.5 Magnetic Particle Testing

Magnetic particle testing was selected due to its high sensitivity to surface and near-surface discontinuities in ferromagnetic steels. Following the previous four tests, a magnetic particle test was performed, the method recommended by the manufacturer.

Prior to testing, the hook was thoroughly cleaned to remove all grease and contaminants. The main purpose of this cleaning was to prevent any residual substances from concealing potential discontinuities. It was then positioned within a device equipped with a five-turn coil (Fig. 2).



Fig. 2. Aircraft tow hook sample washing with magnetic particle testing solution

Magnetic particle inspection was performed according to ASTM E1444/E1444M-25 and ISO 9934-1/-3 using a Magnaflux ZA-1633 wet horizontal bench unit. The tow hook was magnetized circularly by-passing current through a five-turn coil, inducing circumferential fields across the hook section. ASTM-E1444 refers to ASTM-E709-21 for calculation. The magnetizing current was calculated utilizing Eq (2) [23].

$$NI = \frac{K}{L/D} \quad (2)$$

Where, N : Number of turns in the coil, I : Coil current to be used, K : Empirically derived constant, L : Part length, D : Part diameter, NI : Ampere-turns.

The calculation is provided to document compliance with ASTM E1444 requirements; it does not represent a parametric optimization but ensures adequate magnetization under field conditions. K values in Eq. (2) have three options as presented in Table 4. The ampere-turns and ampere values generated for $N=5$ and length-to-diameter ratio (L/D) of 5. The tow hook cross-section is <0.1 of the coil area (Table 4, Option 1), so the highest K ensures adequate field strength and flaw visibility; lower K values are intended for larger parts occupying more of the coil.

Table 4. K values related to part cross-sectional area and coil cross-section

Option	Part cross-sectional area	K value	NI	I
1	< 0.1 of coil cross-section	45000	9000	1800
2	$(> 0.1$ and $< 0.5)$ of coil cross-section	43000	8600	1720
3	< 0.5 of coil cross-section	35000	7000	1400

A fluorescent wet suspension (Magnaflux 14A in Carrier II oil) was used at $1.2 \text{ g/L} \pm 0.2 \text{ g/L}$ particle concentration, verified daily with a centrifuge tube test per ISO 9934-2. The bath was continuously agitated and maintained at $25 \pm 3 \text{ }^\circ\text{C}$. Field adequacy was confirmed using Type QQI shims (Quantitative Quality Indicators, 0.004 in) placed on flat zones of the hook and with a digital gaussmeter, verifying surface field strength between 2.0–3.5 mT (20–35 G) at magnetization.

Fluorescent inspection was conducted in a darkroom with UV-A intensity $> 1000 \text{ } \mu\text{W cm}^{-2}$ at 38 cm and ambient white light $< 20 \text{ lx}$, complying with ISO 3059. After inspection, the tow hook was demagnetized by gradually reducing AC coil current from 1800 A to 0 A while moving the part axially through the field. Residual magnetism was checked ($< 0.5 \text{ G}$) with a gaussmeter before re-iling the surface.

2.2.6 Towing Force

During aircraft towing, the tow bar exerts a horizontal force on the tow hook that must overcome the rolling resistance of the aircraft. To estimate this towing force, the resistance was modeled using friction coefficients corresponding to different ground surfaces. Typical friction coefficients (μ) for runways are given in Table 5.

Table 5. Friction coefficients of different materials [24,25]

Material	Friction coefficient (μ)
Concrete runway	0.02–0.05
Asphalt runway	0.03–0.06
Grass/soil	0.1–0.2

During towing, aircraft weight and fuel load are critical; heavier loads and excessive towing speed increase the risk of accidents. Safe towing requires controlled speed and adherence to FAA guidelines [4]. The General Dynamics F-16 Fighting Falcon was considered in full configuration [26]. Standard ground-handling procedures recommend that the towing speed does not exceed approximately 10 km/h. Under such controlled conditions, towing is performed in a quasi-steady manner, and sudden acceleration or deceleration events are intentionally avoided. Consequently, the forces associated with starting and stopping were neglected in the present analysis. Assuming a constant towing velocity, dynamic inertial effects were not considered, and at these low operating speeds aerodynamic drag forces are negligible. Similar assumptions have been reported by Lin and Yang (2024), who analyzed commercial aircraft nose landing gear under towing (taxi-out) conditions and concluded that, for towing speeds on the order of 10 km/h, the influence of dynamic effects is negligible [18].

Nevertheless, it is acknowledged that real towing operations may involve minor transient effects such as gentle braking, turning maneuvers, small impacts, or slight misalignment between the tow bar and the aircraft. These effects may cause short-term load amplification beyond the nominal straight-tow condition considered here. From an engineering assessment perspective, the static towing force adopted in this study should be interpreted as a conservative baseline load, suitable for assessing structural robustness and correlating experimentally observed damage with numerically predicted stress concentrations.

The aircraft being serviced via the tow hook was analyzed in two weight configurations: empty (basic operating weight) and fuel-loaded (maximum taxi weight). The parameters used in the towing-force calculations are presented in Table 6, and all computations were performed assuming an asphalt surface. The angular position (θ) between tow hook and ground is illustrated in Fig. 3.

Table 6. Parameters of towing force calculations

Parameter	Value
Empty aircraft mass (m)	12,473 kg
Fuel-loaded aircraft mass (m)	19,187 kg
Average friction coefficient of asphalt runway (μ)	0.04

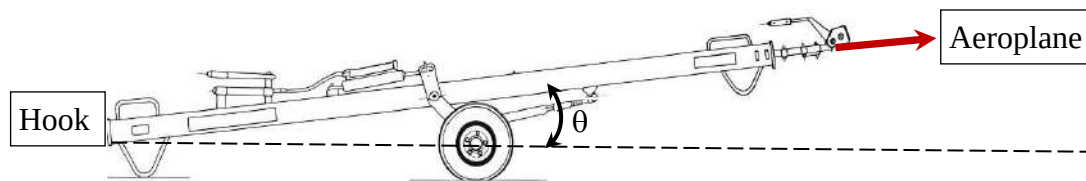


Fig. 3. Angular position of tow hook relative to ground

2.2.7 Numerical Analysis

The finite element analysis was conducted using ANSYS Mechanical. Boundary conditions were defined with fixed supports at the connection points and applied towing loads in the forward direction. Although the tow hook is mechanically connected through a pin-clevis joint, complete rotational freedom is not realized in practice due to contact friction, load-induced constraint, and manufacturing tolerances. To evaluate the influence of this modeling assumption and to avoid bias associated with an arbitrary boundary definition, a simplified parametric comparison was conducted between a fully fixed support and a rotationally released support condition. The fixed support produced stress values approximately 10–15% higher than the released case at the critical region. Therefore, the fixed boundary condition adopted in this study represents a conservative upper-bound estimate of stress, ensuring that the reported results do not underestimate the structural demand and improving the reproducibility of the analysis under realistic field conditions. Load magnitudes were calculated based on aircraft weight conditions (empty and fuel-loaded).

According to Fig. 4, the mesh was refined up to approximately 700,000 elements, after which the maximum equivalent stress stabilized at around 160 MPa, indicating that further mesh refinement had a negligible effect on the stress results. In the structural analysis of the tow hook, a three-dimensional mesh model consisting entirely of tetrahedral elements was generated using the finite element method (FEM). Tetrahedral elements are widely preferred in such analyses due to their high adaptability to complex geometries and compatibility with automatic meshing algorithms. During the modelling process, a high-resolution mesh was constructed, and the geometric quality of each element was evaluated using the element quality parameter. Element quality, which represents the overall distortion of an element relative to its ideal finite element shape, is a critical indicator of mesh performance and has a significant influence on the accuracy and stability of numerical solutions.

The mesh quality analysis revealed that the element quality values ranged between 0.2132 and 1, as shown in Fig. 5. The majority of the elements exhibited quality values between 0.4 and 0.7,

indicating that the overall mesh maintained an acceptable level of accuracy for numerical computation. Locally lower element quality values (approaching 0.2) were observed in the regions with complex geometry and tight curvature, particularly near the inner fillets and hook transitions. Although these areas may slightly reduce local solution accuracy due to geometric complexity, the limited extent of such elements ensures that the global model accuracy remains within acceptable limits.

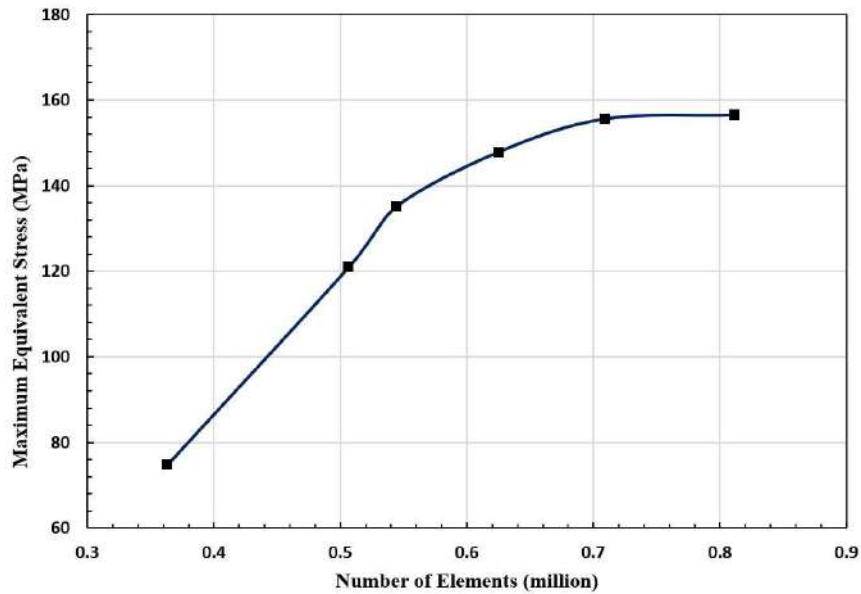


Fig. 4. Mesh convergence study

Visual evaluations based on color contour maps (Fig. 5) clearly demonstrated the spatial distribution of element quality. In the mesh quality plot, blue and green zones represent elements with high quality values (0.6–0.9), which dominate most regions of the model. Conversely, yellow to red zones, indicating lower element quality (below 0.4), were confined to limited areas, mainly around the inner fillets and sharp curvature transitions. This distribution confirms that the overall mesh exhibits high numerical quality with well-shaped tetrahedral elements. Particularly at load application points and structurally critical regions, the element transitions were smooth, minimizing geometric distortion and improving both the stability and accuracy of the analysis results.

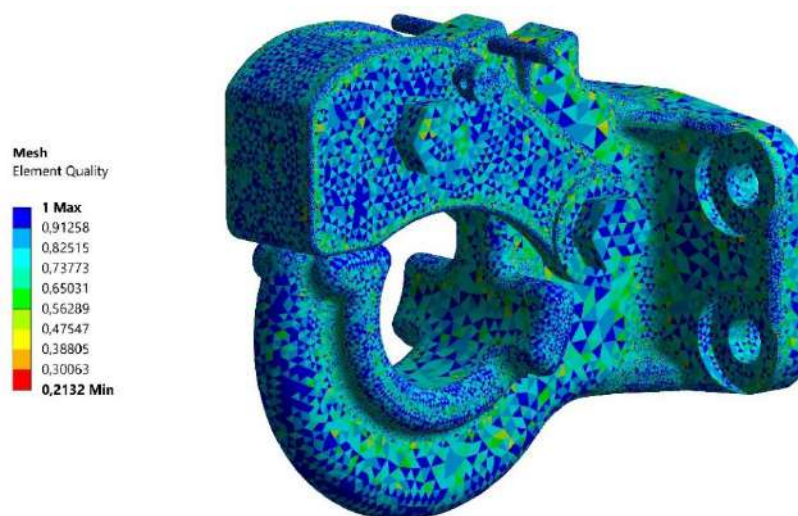


Fig. 5. Distribution of element quality values of tow hook

In conclusion, the tetrahedral mesh structure applied in the structural analysis of the tow hook, a geometrically complex mechanical component, provides both numerical accuracy and engineering reliability. The homogeneous distribution of element quality values and the low distortion levels in critical regions demonstrate that the mesh employed offers an effective and robust basis for FEM-based structural analysis. Fig. 6. illustrates the research methodology, including material characterization, application of five NDT methods on the tow hook, analysis of NDT results, finite element simulation of the tow hook under towing loads, and correlation of the experimental and numerical findings. It provides a visual overview of all steps in this methodology, from initial inspections to final analysis.

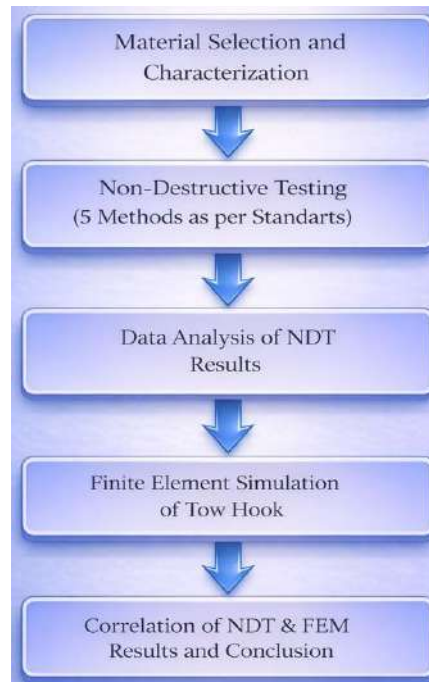


Fig. 6. The research methodology

3. Results and Discussion

3.1. Results

A total of five different tests were performed to investigate the damage analysis of the tow hook. The findings from these tests yielded highly informative results. Reviewing the outcomes of each test individually provides valuable insight.

3.1.1 Eddy Current Test

The tested tow hook is ferromagnetic in nature. Compared to non-ferromagnetic materials, the magnetic permeability of ferromagnetic materials varies depending on their internal position, which may adversely affect the signal-to-noise ratio. It is widely accepted that the use of a DC magnetization technique—where magnetization is partially or fully directed in one orientation—in combination with eddy current testing ensures reliable inspection [27]. This necessitates that the material reaches magnetic saturation before testing [28]. However, while this may appear effective theoretically, it is rarely preferred in practice. This is primarily because, even if the material is brought to magnetic saturation, its residual magnetism is short-lived and easily influenced by even minor magnetic fields. For instance, the Earth's magnetic field itself can have an impact on the material. As the testing facility lacked the equipment required for pre-magnetization, the eddy current test was performed without this step. No interpretable eddy current indications were obtained under the available inspection conditions.

3.1.2 Ultrasonic Test

Preliminary visual inspection indicated that the complex curved geometry and uneven surface condition of the tow hook posed challenges for reliable ultrasonic inspection using conventional straight-beam techniques. In stressed metallic components, ultrasonic wave propagation may exhibit non-linear behavior due to interactions with dislocation structures, leading to waveform distortion and degraded echo interpretation [29].

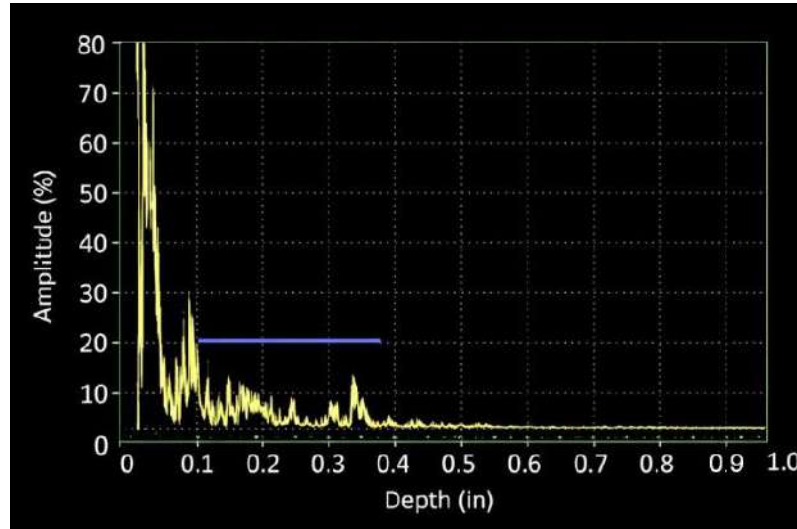


Fig. 7. Ultrasonic A-scan showing geometry-induced reflections and low signal-to-noise ratio (estimated SNR $\approx$ 3–4 dB)

As illustrated in Fig. 7, the ultrasonic A-scan is characterized by multiple overlapping reflections, with the back-wall echo amplitude comparable to the background noise level. The signal-to-noise ratio (SNR), estimated by comparing the peak back-wall echo amplitude to the mean background noise within the inspection gate, did not exceed approximately 3–4 dB across multiple probe positions. This value is below the commonly accepted threshold (≥ 6 dB) required for reliable flaw detection in conventional ultrasonic testing. Reflections from internal curvature further overlapped with back-wall echoes, preventing definition of a meaningful detection limit; consequently, the effective minimum detectable flaw size was conservatively assumed to exceed 3–4 mm for surface-connected discontinuities.

Ultrasonic inspection performance is strongly influenced by the compatibility between probe size, geometry, and the inspected material [30], and the technique exhibits limited sensitivity to surface-connected flaws when used as a standalone method [31]. Despite proper calibration, controlled couplant application, and optimized gate settings in accordance with relevant standards, the recorded signals were dominated by geometry-induced mode conversions and surface-scatter noise. Under these practical field conditions, conventional single-element straight-beam ultrasonic testing did not yield interpretable indications and was therefore deemed unsuitable for reliable inspection of the examined tow hook in the absence of advanced phased-array or immersion-based configurations.

3.1.3 Radiographic Test

As illustrated in Fig. 9, no interpretable discontinuities were identified on the radiographic film obtained under the applied exposure conditions. At an operating voltage of 200 kV, X-ray penetration through the 22 mm thick quenched-and-tempered AISI 4130 steel approaches the upper limit of Class C film radiography. Even at maximum voltage, the resulting film density gradient remained low, with a mean optical density of approximately 1.4 ± 0.2 .

Scatter radiation and geometric unsharpness, estimated to be approximately 1.8 mm at a source-to-film distance of 915 mm, further reduced image contrast and masked potential small planar flaws. Based on the applied exposure parameters (200 kV, AGFA D7 Class C film, and ISO wire-type

IQI), the achieved basic image quality indicator sensitivity was approximately 2%, corresponding to a minimum detectable thickness variation of about 0.4–0.5 mm in a 22 mm steel section. Under these conditions, planar cracks with an opening width below approximately 1.0 mm are considered undetectable.

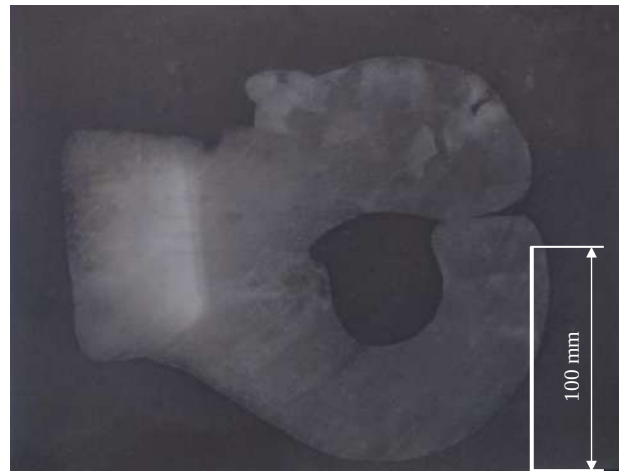


Fig. 8. Radiographic image of the aircraft tow hook

The absence of visible flaw indications in Fig. 9 is therefore attributed to the combined effects of material thickness, high density, and complex overlapping geometry, rather than insufficient inspection execution. Reliable crack detection for this component would require either higher-energy radiation sources (≥ 1 MeV X-ray or Co-60/Ir-192 γ -ray) or digital radiography with enhanced contrast resolution, which were not available under the present field conditions.

3.1.4 Liquid Penetrant Test

As shown in Fig. 10, fluorescent penetrant testing revealed both relevant and non-relevant surface indications on the forged tow hook. Linear or rounded indications conforming to ISO 23277 Level 2 criteria were classified as relevant, while isolated fluorescence below 1 mm and diffuse background responses were disregarded. Surface roughness associated with the forging process produced scattered fluorescence, which limited interpretability despite careful surface preparation. To mitigate this effect, oxide scale was removed by light abrasive polishing, post-wash pressure was reduced to prevent penetrant retention in surface pores, and an etched control coupon was processed concurrently to monitor background intensity. Nevertheless, diffuse non-relevant indications remained present.



Fig. 9. Observed discontinuities after liquid penetrant testing

Quantitative evaluation showed an average false-indication density of approximately 6–8 non-relevant indications per 100 cm² outside the stress-critical regions, primarily associated with surface roughness and forging marks. In contrast, relevant linear indications were sparse and localized. This resulted in a low signal-to-background ratio, demonstrating that while fluorescent penetrant testing is sensitive to surface-connected discontinuities, its reliability is strongly dependent on surface finishing quality for forged steel components.

3.1.5 Magnetic Particle Test

Prior to inspection, a gaussmeter check confirmed the absence of residual magnetic field. The tow hook was then positioned within the test coil, and a current of 1,800 A was applied to induce magnetization, while a fluorescent magnetic particle suspension was uniformly applied. Adequate magnetization was verified by post-test gaussmeter readings, and inspection was conducted in a darkroom under UV-A illumination.

As illustrated in Fig. 11, a distinct linear indication with a measured length of approximately 13 mm was detected on the inner curvature near the attachment lug. The indication was documented using a calibrated scale and a 10 mm reference grid and exceeded ISO 23278 Level 2 acceptance criteria for critical hardware, leading to its classification as rejectable.

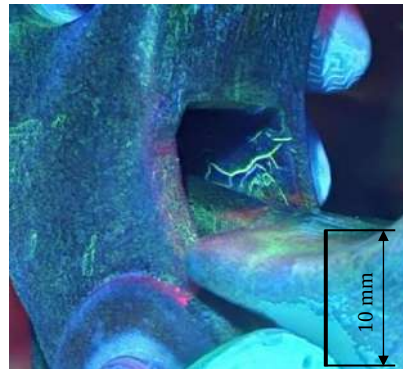


Fig. 10. Component inspected after magnetic particle test

No comparable linear indications were observed outside the stress-critical region predicted by the finite element analysis, indicating a low false-indication density. The indication was resolved with high contrast relative to the background particle distribution, resulting in a high signal-to-background ratio and confirming the superior sensitivity and specificity of magnetic particle testing for this component.

3.2. Discussion

3.2.1 Towing Force

During aircraft towing, the positioning and angle of the tow hook relative to the aircraft are of critical importance. The positioning directly affects the applied forces; incorrect alignment may shorten the service life of the component, cause material fatigue, or lead to undesired cracks and fractures. In the present scenario, the effects of the aircraft being either fuel-loaded or empty were evaluated in conjunction with the angular positioning of the hook. In practice, the angle between the two hook and ground ranges from 0° to 20°. Table 7 presents the calculated towing force values.

Table 7. Towing force values with ground angle θ

Tow hook angle, θ		0°	10°	20°
Towing force [kN]	Empty	4.9	4.98	5.21
	Fuel-loaded	7.53	7.65	8.01

3.2.2 Deformation and Stress

The analysis results included total deformation, maximum principal stress, and von Mises stress distributions. Total deformation analysis demonstrates the vertical displacement resulting from the applied force. Fig. 11 indicates the region experiencing maximum deformation (red) corresponding to the highest stress concentration. Areas experiencing minimal deformation are indicated in blue. The affected regions remain consistent across all angular variations. The distribution of maximum principal stress obtained from the finite element analysis is shown in Fig. 12. The transition from blue to green towards red corresponds to increasing stress levels.

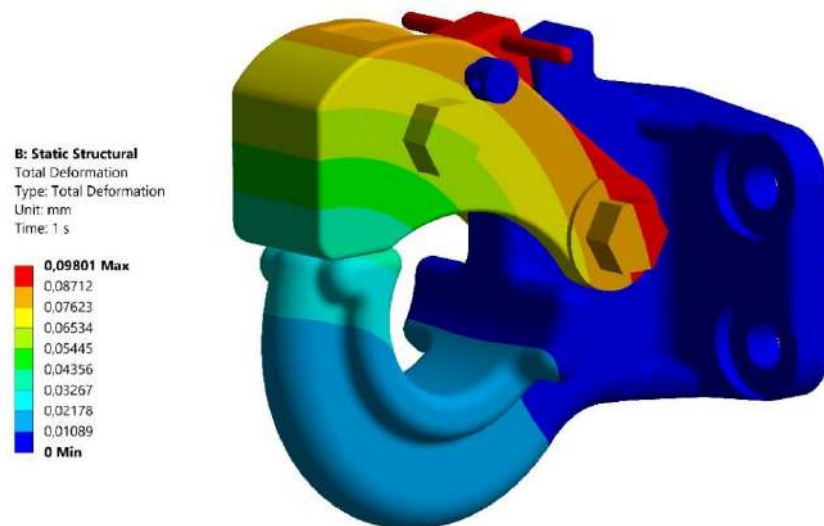


Fig. 11. Total deformation

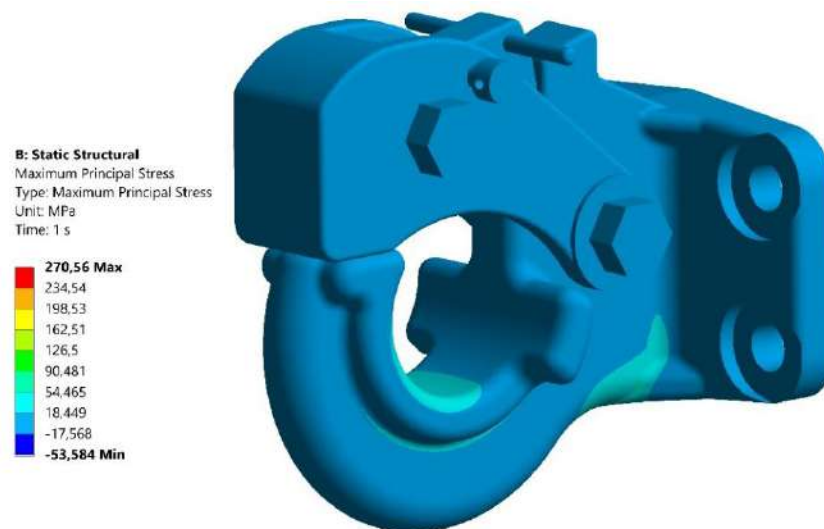


Fig. 12. Distribution of maximum principal stress in the aircraft tow hook under nominal towing loads

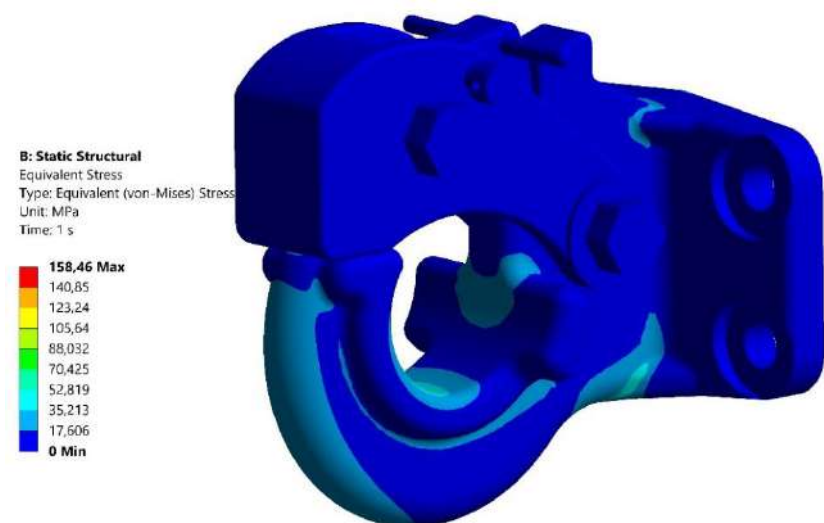


Fig. 13. Equivalent von mises stress

The identified high-stress regions correlate strongly with the cracks observed during non-destructive inspection, further supporting the structural assessment. For all angular variations, the analysis consistently revealed a localized high-stress concentration in the same region, indicating a potential failure zone. To quantify the stress level relative to material strength, a yield utilization ratio (UR) was defined as the ratio of the maximum principal stress obtained from the finite element analysis ($\sigma_{\max}$) to the material yield strength (σ_y), i.e., $UR = \sigma_{\max} / \sigma_y$. The corresponding safety factor was calculated as its reciprocal ($SF = \sigma_y / \sigma_{\max}$).

The finite element analysis yielded a maximum principal stress of approximately 270 MPa, corresponding to a yield utilization ratio of $UR \approx 0.43$ and a safety factor of approximately 2.3. Equivalent von mises stress analysis (Fig. 13) shows that stress is highest at the connection (end) section of the tow hook and lowest in the blue-colored regions. This aligns with the results obtained from the tests—particularly the indications observed in the liquid penetrant test (Fig. 9) and the distinct discontinuities revealed in the magnetic particle test (Fig. 10). The analysis confirms that the connection areas are the most susceptible to cracking or fracture.

3.2.3 Limitations of Non-Destructive Testing Methods

Several non-destructive testing methods, including eddy current, ultrasonic, and radiographic inspection, did not yield reliable flaw detection for the examined tow hook due to the combined effects of material properties, complex geometry, and practical equipment constraints. These outcomes emphasize that the effectiveness of an NDT method is highly dependent on its compatibility with the component geometry and available inspection configurations.

In particular, the limitations observed in ultrasonic testing did not arise from inadequate parameter control or non-compliance with standards, but from the inherent mismatch between the curved geometry of the tow hook and the conventional straight-beam UT setup employed. This clarification reconciles the controlled experimental methodology with the inconclusive inspection results.

3.2.4 Crack Criticality and Safety Margin Assessment

The fracture-mechanics assessment provides an important context for interpreting the detected magnetic particle indication. Given that crack depth information is not available from magnetic particle inspection, the present fracture-mechanics evaluation should be interpreted as a conservative screening tool rather than a detailed life-prediction model. From an engineering maintenance perspective, this level of assessment is sufficient to determine whether a detected surface crack has approached a critical condition requiring rejection or repair. More detailed crack-shape-dependent analyses would require advanced inspection techniques, which were beyond the scope of the present field-based study. Assuming a surface crack geometry with a geometry factor $Y = 1.12$, the critical crack length corresponding to the maximum principal stress of 270 MPa was calculated as 19.6 mm. The experimentally observed crack length of approximately 13 mm therefore corresponds to about 66% of the critical crack length under static towing conditions.

This comparison indicates that the detected crack has not yet reached the critical size for unstable fracture but has significantly reduced the remaining safety margin. A $\pm 10\%$ variation in the geometry factor ($Y = 1.0\text{--}1.25$) results in a critical crack length range of approximately 17.5–22 mm, which still places the observed crack within 60–75% of the critical threshold. From an engineering perspective, this proximity highlights the necessity of immediate rejection or repair, as further crack growth under cyclic towing loads could rapidly lead to fracture.

Although the global safety factor remains acceptable ($SF \approx 2.3$), the corresponding yield utilization indicates that local crack growth can significantly reduce the remaining safety margin well before yielding occurs. The combined FEM stress analysis and fracture-mechanics evaluation therefore demonstrate that the tow hook operates with an acceptable global safety factor under nominal loading, while local crack growth can critically compromise structural integrity.

4. Conclusion

This study presented an integrated structural integrity assessment of an aircraft tow hook by combining multiple non-destructive testing techniques with finite element stress analysis and fracture-mechanics-based evaluation. Rather than focusing solely on defect detection, the work aimed to clarify the capabilities and limitations of commonly applied inspection methods when used on a forged, geometrically complex aerospace component operating under service-relevant loads.

Among the investigated inspection techniques, magnetic particle testing demonstrated the highest effectiveness in identifying surface-connected defects at critical locations. In contrast, eddy current, ultrasonic, and radiographic inspections were shown to be inherently limited by the component geometry, material characteristics, and available field configurations. Importantly, these limitations did not arise from inadequate test execution or parameter control, but from a fundamental incompatibility between the inspection methods and the physical characteristics of the tow hook. This distinction underscores the necessity of method selection based on component-specific constraints rather than generalized inspection capability.

The finite element analysis revealed a consistent high-stress concentration in the same region where the relevant magnetic particle indication was detected, confirming the structural relevance of the observed defect. Although the tow hook exhibits an acceptable global safety factor under nominal towing loads, the associated yield utilization highlights that stress-based margins alone do not fully capture the risk posed by localized damage. The fracture-mechanics-based assessment provided essential context by demonstrating that the detected surface crack occupies a substantial fraction of the critical crack size for unstable fracture, indicating that local damage can significantly erode the remaining safety margin well before yielding occurs.

From an engineering and maintenance perspective, these findings emphasize that reliance on global safety factors or nominal stress criteria may lead to non-conservative conclusions for towing hardware subjected to cyclic service conditions. The integration of magnetic particle inspection with finite element stress analysis and crack-criticality evaluation enables a more realistic assessment of serviceability and supports timely rejection or repair decisions before catastrophic failure. The fracture-based crack assessment presented in this study represents a conservative, inspection-driven screening approach, intended to support maintenance decision-making rather than detailed crack-growth prediction. Beyond the specific case of the tow hook, the methodology presented in this study is applicable to other forged aerospace components with complex geometry, where local damage mechanisms may govern structural integrity despite apparently adequate global design margins.

References

- [1] IATA. Aircraft operational availability 2022: 1–47. <https://www.iata.org/contentassets/bf8ca67c8bcd4358b3d004b0d6d0916f/aoa-2nded-2022.pdf> (accessed October 7, 2025).
- [2] Chauhan T, Goyal C, Kumari D, Thakur AK. A review on foreign object debris/damage (FOD) and its effects on aviation industry. In: Sakthivel S, Karthikeyan S, Palani A, editors. International Conference on Nanotechnology: Ideas, Innovation and Industries, vol. 33, Anna Univ, Syed Ammal Engn Coll, Ramanathapuram, INDIA: Materials Today: Proceedings, Elsevier Ltd.; 2020, p. 4336–4339. <https://doi.org/10.1016/j.matpr.2020.07.457>.
- [3] Masniza Y, Zulfabli HM, Amarul T, Khairunnisa SN, Zilawati NARNN, Nazera D, et al. Potential FOD defects on the composite material. 5th International Conference on Green Design and Manufacture (IConGDM 2019), Jawa Barat, Indonesia: 2019, p. 020056. <https://doi.org/10.1063/1.5118064>.
- [4] Federal Aviation Administration. Towbar and towbarless movement of aircraft. US Department of Transportation 2023: 1–16. https://www.faa.gov/documentLibrary/media/Advisory_Circular/AC_00-65A.pdf (accessed October 7, 2025).
- [5] Federal Aviation Administration. ENR1. General rules and procedures. US Department of Transportation 2025. https://www.faa.gov/air_traffic/publications/atpubs/aip_html/part2_enr_section_1.1.html (accessed October 7, 2025).
- [6] Yilmaz AA. Critical connections: network analysis of human errors in aviation accidents. The International Journal of Aerospace Psychology 2025; 35: 203–229. <https://doi.org/10.1080/24721840.2025.2531741>.

- [7] Ulus Ö, Davarcı FE, Gültekin EE. Non-destructive testing methods commonly used in aviation. *International Journal of Aeronautics and Astronautics* 2024; 5: 10–22. <https://doi.org/10.55212/ijaa.1418742>.
- [8] Demircioglu P, Seckin M, Seckin AC, Bogrekci I. Non-destructive testing methods in composite materials. In: Kumar A, Kumar Singla Y, R. Maughan M, editors. *Fracture Behavior of Nanocomposites and Reinforced Laminate Structures*, Cham: Springer Nature Switzerland; 2024, p. 487–516. https://doi.org/10.1007/978-3-031-68694-8_21.
- [9] Ibrishv K, Prilipko E, Soldatkin M, Kononov N. Experimental research of quality of the adhesive repairs of composite structures using non-destructive testing. *Research on Engineering Structures and Materials* 2025; 11: 1051–1062. <https://doi.org/10.17515/resm2024.351ma0712rs>.
- [10] Gupta M, Khan MA, Butola R, Singari RM. Advances in applications of non-destructive testing (NDT): a review. *Advances in Materials and Processing Technologies* 2022; 8: 2286–2307. <https://doi.org/10.1080/2374068X.2021.1909332>.
- [11] Purna Chandra Rao B. Non-destructive testing and damage detection. In: Prasad NE, Wanhil RJH, editors. *Aerospace Materials and Material Technologies*, vol. 2, Springer; 2017, p. 209–228. https://doi.org/10.1007/978-981-10-2143-5_11.
- [12] Jarvis R, Cawley P, Nagy PB. Performance evaluation of a magnetic field measurement NDE technique using a model assisted probability of detection framework. *NDT & E International* 2017; 91: 61–70. <https://doi.org/10.1016/j.ndteint.2017.06.006>.
- [13] Sözeri V, Keskin O, Harmanşah C, Kozak İ, Kibar EN. Investigation of crack detection in magnetic particle testing images based on image process algorithms (in Turkish). *Afyon Kocatepe University Journal of Sciences and Engineering* 2023; 23: 115–122. <https://doi.org/10.35414/akufemubid.1173473>.
- [14] Wu Q, Dong K, Qin X, Hu Z, Xiong X. Magnetic particle inspection: status, advances, and challenges – demands for automatic non-destructive testing. *NDT & E International* 2024; 143: 103030. <https://doi.org/10.1016/j.ndteint.2023.103030>.
- [15] Petrici AV, Scutaru ML, Gheorghe V, Vlase S. Alternative solution for towing systems used in the automotive industry. *Applied Sciences* 2024; 14: 9131. <https://doi.org/10.3390/app14199131>.
- [16] Aftab SG, Sirajuddin, Sreedhara B, Ganesh E, Babu NR, Aithal SK. Finite element analysis of a passenger aircraft landing gear for structural and fatigue safety. In: Ramesh MR, Dasarath S, Prashatha K, editors. *5th International Conference on Advanced Research in Mechanical, Materials and Manufacturing Engineering-2021*, Elsevier; 2022, p. 152–158. <https://doi.org/10.1016/j.matpr.2021.08.199>.
- [17] Yadav BK, Yadav R, Sudhakar K. Stress distribution analysis using ANSYS of the nose landing gear of STOL aircrafts considering runway gradient: a simulation study. *Orchid Academia Siraha* 2023; 2: 1–20. <https://doi.org/10.3126/oas.v2i1.65618>.
- [18] Lin Q, Yang C, Bai Y, Qin J. Structural strength analysis and optimization of commercial aircraft nose landing gear under towing taxi-out conditions using finite element simulation and modal testing. *Aerospace* 2024; 11: 414. <https://doi.org/10.3390/aerospace11050414>.
- [19] Chakravarti BJ, Rajasekhar K. Experimental investigation and modelling of the layered concrete with different proportions of steel fibers. *Research on Engineering Structures and Materials* 2025; 11: 957–968. <https://doi.org/10.17515/resm2025-764me0316rs>.
- [20] Kurt Ö. Damage analysis of an aircraft tow hook (in Turkish). Master of Science. The Graduate School of Natural and Applied Science of Necmettin Erbakan University, 2025.
- [21] ASM International Handbook Committee. *Fracture Mechanics Properties of Carbon and Alloy Steels. Fatigue and Fracture*, vol. 19, ASM International; 1996, p. 614–654. <https://doi.org/10.31399/asm.hb.v19.a0002397>.
- [22] Jones DRH., Ashby MF. *Engineering Materials 1: An Introduction to Properties, Applications and Design*. 5th ed. Butterworth-Heinemann; 2019.
- [23] ASTM–E709-21. Standard guide for magnetic particle testing. ASTM Volume 03.03: Nondestructive Testing (I): B548 – E2373, vol. 03.03, West Conshohocken, PA: ASTM International; 2021. <https://doi.org/10.1520/E0709-21>.
- [24] Sofie M, Winiasri L, Triyono A. Planning for the extension of runway 01 for A330-200 aircraft type at Kalimantan Berau airport. *Proceeding of International Conference of Advanced Transportation, Engineering, and Applied Social Science*, vol. 3, 2024, p. 23–34.
- [25] Wong JY. *Theory of Ground Vehicles*. Wiley; 2022. <https://doi.org/10.1002/9781119719984>.
- [26] Bushell S, Willis D. *Jane's All the World's Aircraft: In Service*. JANES; 2025.
- [27] Deng Z, Kang Y, Zhang J, Song K. Multi-source effect in magnetizing-based eddy current testing sensor for surface crack in ferromagnetic materials. *Sens Actuators A Phys* 2018; 271: 24–36. <https://doi.org/10.1016/j.sna.2018.01.009>.
- [28] Jin X, Yang L. Research on eddy current-based detection method for pipeline hard spots. *Measurement* 2026; 257: 118952. <https://doi.org/10.1016/j.measurement.2025.118952>.

- [29] Wang JJ, Wen ZX, Pei HQ, Gu SN, Zhang CJ, Yue ZF. Thermal damage evaluation of nickel-based superalloys based on ultrasonic nondestructive testing. *Applied Acoustics* 2021; 183: 108329. <https://doi.org/10.1016/j.apacoust.2021.108329>.
- [30] Görgün E. Investigation of the effect of ultrasonic inspection probe diameters on impact echo value (in Turkish). *Karadeniz Fen Bilimleri Dergisi* 2022; 12: 381–389. <https://doi.org/10.31466/kfbd.1077386>.
- [31] Sevim MA, Çelt AC, Kabar S, Akgünlü Ö. Applications of non-destructive testing methods on rails in railways (In Turkish). *Demiryolu Mühendisliği* 2020: 60–74. <https://dergipark.org.tr/en/pub/demiryolu/issue/50417/658939>

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Enhancing the mechanical and durability quality of fibre-modified concrete using eco-sustainable wastes and lime powder

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Abstract

The use of agro-waste as reinforcement can enhance the mechanical and water absorption properties of concrete. The study investigates the use of agro-wastes, such as coconut fiber and plantain peel ash (PPA), combined with lime powder to enhance concrete properties and mitigate environmental harm. Coir fiber was added to the concrete at 0%, 5%, and 10% of the concrete's total density, while the cement was partially substituted with lime powder at 0%, 10%, and 20%, and with PPA at 1.5%. The concrete mix's qualities were evaluated at different ratios. A concrete mix containing 10% lime powder, 0.5% coconut fiber, and 1.5% plantain peel ash yielded higher flexural and split tensile strengths, while the highest compressive strength compared with the control (Mix 1) was achieved with fiber content ranging from 0.5% to 1%. The concrete with 0.5% coir fiber, 10% lime, and 1.5% PPA showed better water absorption than other mixes. The study reveals that alkali-treated coconut fiber enhances the tensile strength of concrete, while plantain peel ash, lime powder, and coconut fiber decrease workability and increase compressive, split tensile, and flexural strengths. Nigeria's sustainable management of agricultural waste, such as coconut fiber and plantain peels, can improve concrete workability, reduce Portland cement usage, and decrease CO₂ emissions.

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1. Introduction

The growing population and modernization have led to increased waste production, raising environmental concerns [1]. This has necessitated a paramount drive for sustainable construction in the construction industry [2,3]. Recycling of agricultural and industrial waste into useful resources can significantly promote a sustainable environment [4]. Fibers are being incorporated into concrete to enhance its strength and toughness, and also reduce pollution [5]. Natural, synthetic, glass, and steel fibers are being utilized to reduce environmental pollution and enhance concrete strength. Utilizing locally available, sustainable non-abrasive materials such as coconut, bamboo, sugarcane, palm, hemp, banana, pineapple leaf, and cotton has been the focus of researchers lately. Concrete, a durable and cost-effective construction material, is widely used for its versatility, resulting in annual consumption of approximately 10 billion tons. Concrete strength is influenced by various factors, including curing, water-cement ratio, compaction, raw material

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quality, age, additives/admixtures, fiber content, and temperature, all of which are crucial for designing structures and selecting materials.

The use of reinforcement does not stop crack growth in cement-based materials. However, it has been reported that the addition of short and discrete fibers to the concrete matrix will significantly improve the concrete's toughness, ductility, and post-cracking strength of concrete due to the bridging mechanism fibers deplores when loaded [6,7]. Fibers, hair-like structures found in animals, plants, minerals, and synthetic materials, are used as reinforcements in cementitious composites. Concrete uses fibers to control shrinkage and drying shrinkage cracking. Steel reinforcement enhances strength, while short fibers provide uniform tensile properties and increased resistance to microcracking. Natural fibers are used in construction for environmental, waste-reduction, insulation, safety, and economic benefits [8]. Reinforced concrete, including steel, polyester, glass, and polypropylene fibers, improves impact strength, strain capacity, plastic shrinkage resistance, segregation, fire resistance, and durability. Fiber-reinforced concrete is popular due to its high tensile strength and low maintenance costs [9].

The incorporation of coconut fiber derived from recycled coconut residue into concrete offers an avenue for environmental sustainability in the construction sector. It is a natural fiber with high strength and scraping resistance. Coconut fiber is readily available in Nigeria due to the abundance of coconut trees, especially in the country's coastal regions [10]. In recent times, coconuts have been grown and used for commercial purposes in more than 90 countries, contributing to the 64 million tons of coconuts produced globally each year [11].

It offers a 25% improvement in flexural strength, making it a potential reinforcement material in concrete due to its ductility and affordability. Examining the strength properties of coir fiber in concrete, used to replace fine and coarse aggregate replacement of 0.5% and 1%. With a design strength of 20.59 MPa, cylinders with a 1% fiber addition performed better than those without fibers [12]. In another study by Ede and Agbede [10] the findings demonstrated that the compressive strength of coir fiber-reinforced concrete increased with curing age and increased with the coir fiber proportion up to 0.5%, before progressively declining from 0.75% to 1.0%. In comparison to the control sample, the percentage strength improvement at 28 days for fiber contents of 0.25%, 0.5%, 0.75%, and 1.0% is 4.58%, 38.13%, 8.56%, and -2.42%, respectively. According to the data, the concrete's flexural strength increased by 28.82%, 22.15%, and 0.42% at 28 days for 0.25%, 0.5%, and 1.0% of coir fiber, respectively. This proves that Coir fiber enhances concrete properties and reduces pollution by minimizing waste from coir-based manufacturing units, making it a sustainable waste management technique.

Several researchers have explored the incorporation of industrial by-products and agricultural residues into concrete to enhance its strength and durability [13]. Plantain peels, non-biodegradable by-products from plantain processing industries, are a significant contributor to environmental pollution. A study by David et al. [14] identified the PPA as a water-reducing admixture that prevents bleeding and segregation in freshly mixed concrete and found that adding 5% PPA to concrete boosts its compressive strength, which was 27.30 N/mm². Additionally, the concrete's standard workability is 45 mm for the control and 60 mm for the 5% PPA. The application of PPA as a concrete admixture has been shown to significantly enhance the compressive strength, workability, and durability of concrete structures. It was found that adding ripe and unripe plantain peel ash to concrete increased its workability and late strength development, such that concrete containing 1.5 and 2.5% ripe PPA has strengths of 22.77N/mm² (28.50% increase) and 20.64N/mm² (32.40% increase), respectively, while concrete containing 1.5 and 2.5% unripe PPA has strengths of 20.86N/mm² (31.00% increase) and 19.02N/mm² (35.70% increase) at 90 days, prompting the study to investigate the use of PPA as a partial replacement for cement [15]. Aderinola et al. [16] examined the impact of partial cement replacement with PPA and polystyrene (POL) on concrete compressive strength. Results showed that adding PPA and polystyrene to concrete improved workability but reduced compressive strength.

The effect of replacing Portland cement with fine limestone powder improves the compressive strength of high early-strength cement but decreases its compressive and flexural strengths [17]. Studies show that increasing the CaCO₃ content and fineness accelerates the hydration rate of C₃S

in Portland cement, while limestone powder enhances strength without affecting mortar workability. When working with lower-reactivity pozzolans, such as plantain ash, lime increases the availability of $\text{Ca}(\text{OH})_2$, thereby accelerating and improving ash reactivity and concrete workability [18]. Hence, combining Portland cement with lime also increases early strength.

It has been established in the literature that coir fibres increase the strain capacity and toughness of concrete by acting as crack arresters, bridging microcracks before they spread and enhancing service life under dynamic conditions and lessening brittleness [19, 20]. Plantain peels contain silica (SiO_2), alumina (Al_2O_3), and other oxides, which are comparable to the conventional pozzolans used in concrete when burned to ash. Thus, by taking part in pozzolanic interactions with lime/cement hydration products, PPA strengthens the paste by forming additional binding gel (C-S-H) [21]. When working with lower-reactivity pozzolans, such as PPA, lime increases the availability of $\text{Ca}(\text{OH})_2$, which speeds up and improves ash reactivity and concrete workability [18]. Therefore, the fibers prevent cracking, making the matrix more resilient to pressure without brittle failure. As lime/ash densifies the cementitious matrix, decreasing porosity, the matrix becomes more resilient to pressure. This is more beneficial than employing any single additive alone; this type of hybrid system (fibre + pozzolanic + activator) is more sophisticated, as it concurrently enhances mechanical performance, durability, and sustainability. In terms of pozzolanic Reactions, Calcium hydroxide is less significant, but calcium silicate hydrates produced during cement hydration are substantial. Reactive silica and alumina found in plantain ash react with $\text{Ca}(\text{OH})_2$ from cement and lime to form additional C-S-H, increasing strength and decreasing free $\text{Ca}(\text{OH})_2$ that could cause efflorescence [21].

Previous studies have focused more on mechanical characteristics, using separate SCMs or a combination of two, with little attention paid to the strength and durability of concrete blended with coir fiber and agro-based materials, such as plantain peel ash. Currently, to the best of the author's knowledge, there are no published works on the combination of lime and PPA, indicating a grey area as far as the binary use of SCMs in concrete is concerned. This further demonstrates that there is still much to explore in the use of agro-waste materials for construction, especially in Nigeria, where an acute housing shortage persists. The dumping of agricultural wastes, such as coconut fiber and plantain peel, at landfills is a significant problem. Utilizing these eco-sustainable wastes can improve concrete strength and reduce environmental degradation [22]. The novelty of this study is shown in the synergistic application of low-energy, agro-based by products; plantain peels, lime, and coir fiber as either waste or requiring far less processing than cement, and also the sustainability of the product. A significant environmental problem is addressed by substituting some of the cement component with these materials, thereby lowering greenhouse gas emissions associated with cement manufacture. Therefore, the study evaluates the mechanical and water absorption properties of coconut fiber reinforced concrete incorporating plantain peel ash as an admixture and lime powder as a supplementary cementitious material.

2. Materials and Methods

2.1 Materials Preparation

The materials used included cement, fine aggregates, coarse aggregates, superplasticizer, water, coconut fiber, plantain peel ash, and lime powder, all of which were locally sourced. The study's findings determine the quality and features of the concrete. Dangote 3x cement, a crucial component of concrete, was procured. The concrete was then made from crushed granite stones, ranging in size from 10 to 20 mm, after thorough cleaning to remove dirt and dust. The fine aggregates were locally sourced in Ado-Ekiti, washed with potable water, and sieved to remove large particles and lumps, with a maximum particle size of 4.75 mm. Coconut fiber, an agricultural waste material obtained from the outer husk of coconuts, was used in the study. According to Adetayo and Jubril [15], unripe plantain peel ash contains a high content of silica oxide (54%) as well as other oxides such as aluminum oxide (3.080%), magnesium oxide (6.633%), calcium oxide (4.067%), ferric oxide (2.887%), and sulfur trioxide (0.583%). Plantain Peel Ash (PPA) is produced by sun-drying and burning plantain peels at 700°C in a furnace, as presented in Figures 1a and 1b.

The study used COSTAMIX 200 super plasticizer (SP), a high-range water-reducing admixture, according to ASTM C494 04 [23], and hydrated lime powder (LP), a hydrate-based product, to enhance the workability and water-retention properties of cement, as illustrated in Figures 1c and 1d. According to Awolusi et al. [24], lime powder contains silica oxide (5.69%), aluminum oxide (7.65%), magnesium oxide (4.69%), calcium oxide (80.71%), ferric oxide (0.24%), and sulfur trioxide (0.07%). The treatment process for coconut fiber (CF) began with washing in clean water, soaking for 30 minutes, and removing the coir dust. The fiber was then soaked in NaOH water for 45 minutes, washed with fresh water, and the color changed from light brown to dark brown, as shown in Figures 1e to 1h. Coconut fibers treated with sodium hydroxide exhibit improved cement performance, chemical stability, splitting tensile strength, and resistance to degradation in alkaline environments [25]. The untreated and treated are shown in Figure 1g and 1h, respectively.

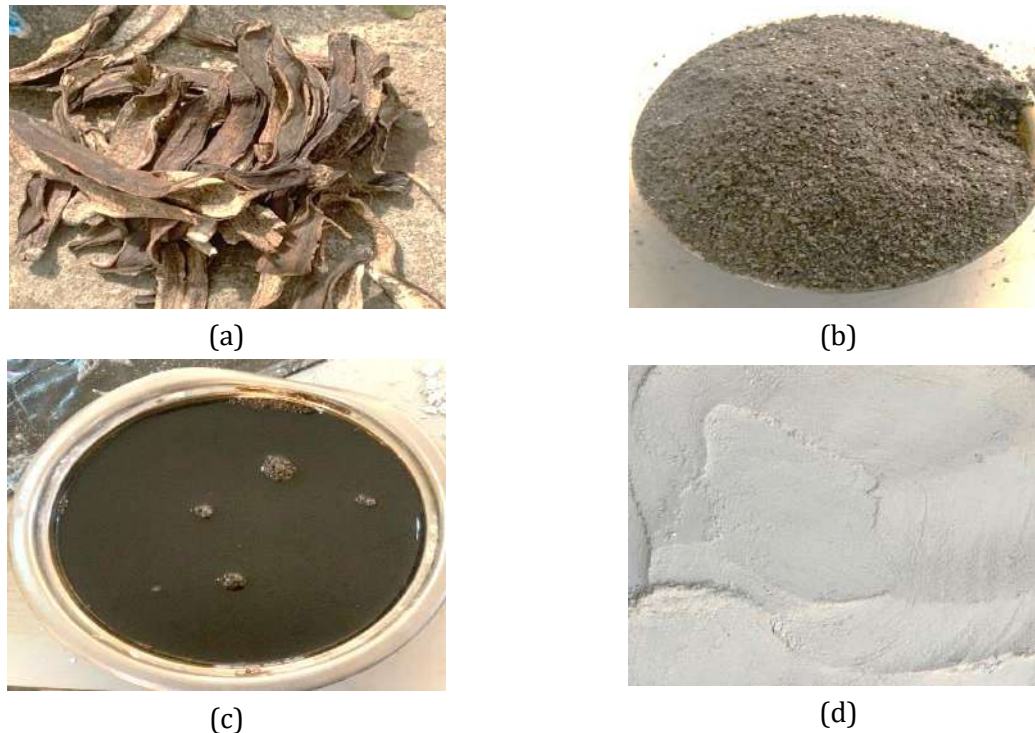


Fig. 1. Material preparation process (a) Sun-dried plantain peels (b) Plantain peel ash (c) Super plasticizer (d) lime Powder

2.2 Experimental Procedure

The coconut fiber-reinforced concrete was thoroughly mixed with various materials, including coarse aggregates, cement, fine aggregates, lime powder, and plantain peel ash, as shown in Table 1. The lime powder was incorporated as a percentage replacement for cement. The plantain peel ash was incorporated into the concrete mix as an admixture. To achieve a workable concrete and maintain a constant water-cement ratio of 0.3, the superplasticizer was mixed with water before adding it to the concrete mix. The fibers were cut into a length of 20 mm with an aspect ratio of 100. Half of the fibers were randomly dispersed in the concrete mix, which was then thoroughly mixed for 1 minute. The other half was randomly dispersed, ensuring a homogeneous concrete was achieved. Concrete was placed in moulds, cubes (100 mm), cylinders (200 mm x 100 mm), and beams (400 mm x 100 mm), compacted with 25 blows, demolded 24 hours after casting, and transferred into a water curing tank at a laboratory temperature of $29^{\circ}\text{C} \pm 1$. Figure 2 illustrates the flowchart of the experimental procedure and results analysis. The slump measurement and concrete cubes, cylinder, and beam casting are shown in Figures 3a and 3b, whereas the result of the concrete cube and failed samples after curing are presented in Figures 3c and 3d. The data were computed for each mix proportion as the average of three specimens. Three (3) samples of cube, cylinder, and prism for each mix were prepared, along with corresponding control samples (Mix 1). The quantity of lime powder required for each mix was incorporated into the concrete as a percentage replacement of the binder content by mass. The PPA used as admixture was also added

to the concrete mix by mass of the binder content. The coir fibre content (0.5%, 1%, 1.5%) were incorporated into the mix as a percentage of the volume fraction of concrete.

Table 1. Mix proportion for constituent materials

Mix No.	Lime powder %	Coconut fibre (%)	PPA (%)	Super Plasticizer (%)	Binder content (kg/m ³)	Cement (kg/m ³)	Coarse Aggregate (kg/m ³)	Fine Aggregates (kg/m ³)	Water (kg/m ³)
1	0	0.0	0	1.5	400	400	1200	597	120
2	0	0.5	1.5	1.5	400	400	1200	597	120
3	0	1.0	1.5	1.5	400	400	1200	597	120
4	0	1.5	1.5	1.5	400	400	1200	597	120
5	10	0.5	1.5	1.5	400	360	1200	597	120
6	10	1.0	1.5	1.5	400	360	1200	597	120
7	10	1.5	1.5	1.5	400	360	1200	597	120
8	20	0.5	1.5	1.5	400	320	1200	597	120
9	20	1.0	1.5	1.5	400	320	1200	597	120
10	20	1.5	1.5	1.5	400	320	1200	597	120

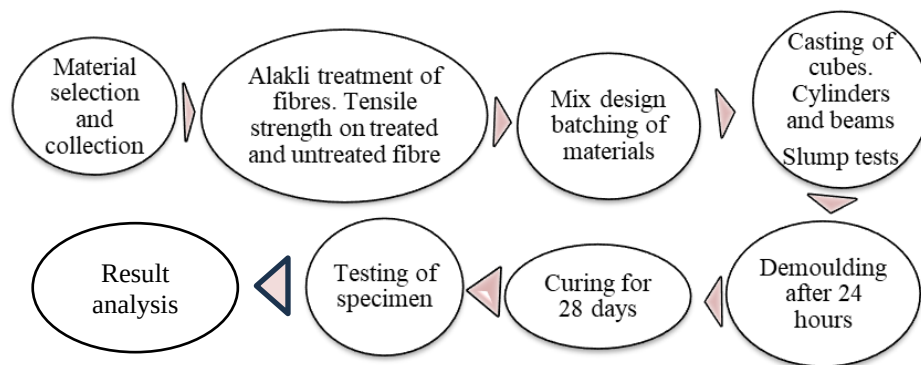


Fig. 2. Experimental design flow chart

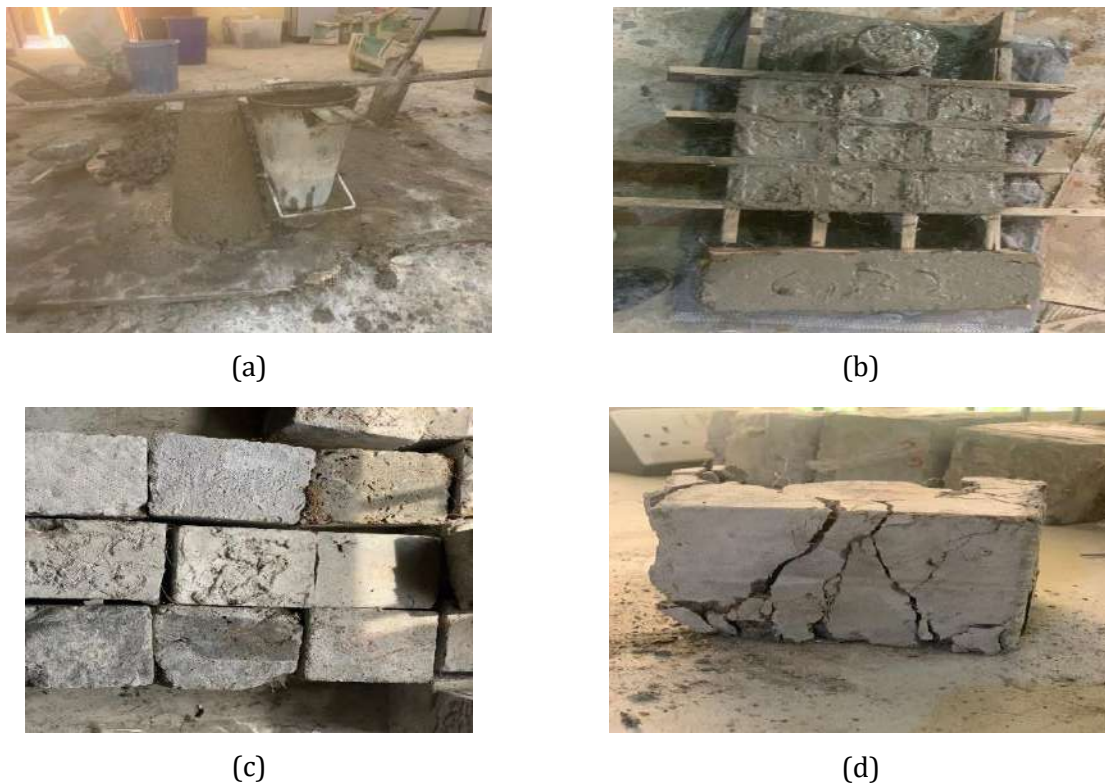


Fig. 3. (a) Slump measurement (b) Concrete cubes, cylinder, and beams casting (c) Concrete cube after curing (d) Failed concrete

2.3.1 Specific Gravity Test (SGT) and Particle Size Distribution (PSD)

Sieve analysis was conducted on fine and coarse aggregates, PPA, and lime powder, focusing on the granular material through progressively smaller mesh sizes. The specific gravity was determined using equations 1, 2, 3, and 4:

$$S_g = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \quad (1)$$

$$R = \frac{W_{far}}{m} \times 100 \quad (2)$$

$$P = 100 - \Sigma R \quad (3)$$

$$F_{mfa} = \frac{\Sigma R}{100} \quad (4)$$

2.3.2 Tests on Fresh Sample Properties (FSP)

The slump test is a control method used to assess the consistency and workability of the concrete, ensuring it meets the BS EN 12390 standard [26]. It involves filling a cone with three layers of concrete, tamping 25 times, and measuring the slump level.

2.3.3 Test on Hardened Concrete Samples

In this section, test procedures for cube, cylindrical, and prism samples were illustrated based on existing standard test methods for concrete samples.

2.3.3.1 Compressive Strength Test (CST)

Compressive strength measures the material's ability to withstand axial loads. Concrete cubes (100 mm x 100 mm), both plain and fiber-reinforced, were tested using an ELE machine at a loading rate of 0.6 kN/s, in accordance with BS EN 12390-3 [27]. The strength was calculated by dividing the maximum crushing load by the effective area as stated in Equation 5:

$$C_s = \frac{L}{A} \quad (5)$$

2.3.3.2 Split Tensile Strength Test (STS)

The tensile strength of concrete, measured on 200 mm by 100 mm concrete cylinders, containing coconut fiber, lime powder, and plantain peel ash, was obtained after 28 days of curing using an ELE compression testing machine at a loading rate of 0.940 kN/sec according to BS EN 12390-6 [28]. The split tensile strength (STS) was determined using Equation 6, and the result is presented in Figure 4a.

$$F_t = \frac{2P}{(\pi DL)} \quad (6)$$

2.3.3.3 Flexural Strength Test (FST)

The flexural strength of the concrete was determined using 400 mm by 100 mm beams under a central line load and a three-point test. The beams were subjected to a displacement control condition of 0.10 mm/min, as specified in BS EN 12390-5 [29], 28 days after casting, as per equation 7, and the result is presented in Figure 4b.

$$F_{St} = \frac{3PL}{2BD^2} \quad (7)$$



(a)



(b)

Fig. 4. (a) Split tensile test on concrete cylinders (b) Flexural test on concrete beams

2.3.4 Water Absorption Test (WAT)

The water permeability of concrete was assessed using ASTM C642 [30], with water absorption tests on cubes conducted to determine initial surface absorption (ISA) and final water absorption (FWA) after 90 minutes and 24 hours, as shown in Equations 8 and 9.

$$I_{SA} = \frac{W_{@90 \text{ mins}} - W_{OD}}{W_{OD}} \times 100 \quad (8)$$

$$F_{WA} = \frac{W_{@24 \text{ hours}} - W_{OD}}{W_{OD}} \times 100 \quad (9)$$

3. Results and Discussion

This research involved ten runs of concrete with coconut fibre, PPA, and lime powder, cured in potable water for 28 days. The fresh and hardened properties were determined through compressive, flexural, and split tensile strength tests, as well as the Initial surface absorption and final water absorption.

3.1 Particle Size Distribution and Specific Gravity Test

The distribution of sieve analysis was conducted on fine aggregates, coarse aggregates, and lime powder, with results of 2.65, 2.50, and 2.62, respectively. The average fine moduli for the three samples considered are 2.20, 2.87, and 2.5, as presented in Tables 2, 3, and 4. The deflection outcomes for varying loads of untreated and treated alkali coconut fiber are presented in Figures 5 and 6.

Table 2. Sieve analysis of fine aggregate

Sieve Size (mm)	M ₁ (g)	M ₂ (g)	Fine aggregate Retained (g)	Retained (%)	Cumulative Retained (%)	Passing (%)
4.75	540	540	0	0.0	0.0	100.0
2.36	535	538	3	1.5	1.5	98.5
1.18	487	500	13	6.5	8.0	92.0
0.60	479	530	51	25.5	33.5	66.5
0.30	412	451	39	19.5	53.0	47.0
0.15	383	397	14	7.0	60.0	40.0
0.063	397	404	7	3.5	63.5	36.5
Fine modulus = 2.20						

Table 3. Sieve analysis of coarse aggregate

Sieve Size (mm)	M1 (g)	M2 (g)	Retained Coarse Aggregates	Retained (%)	Passing (%)
28	1500	1500	0	0	100
20	1510	1568	58	29	71
14	1270	1334	64	32	68
10	1270	1391	121	60.5	39.5
6.3	1198	1349	151	75.5	24.5
4.75	800	979	179	89.5	10.5

Fine modulus = 2.87

Table 4. Sieve analysis of lime powder

Sieve Size (mm)	M1 (g)	M2 (g)	Lime Powder Retained	Percentage Retained (%)	Percentage Passing (%)
5.00	545	545	0	0	100
2.36	535	535	0	0	100
1.18	487	505	18	9	91
0.60	478	520	42	21	79
0.30	412	522	110	55	45
0.15	383	533	150	75	25
0.06	397	577	180	90	10

Fine modulus = 2.5

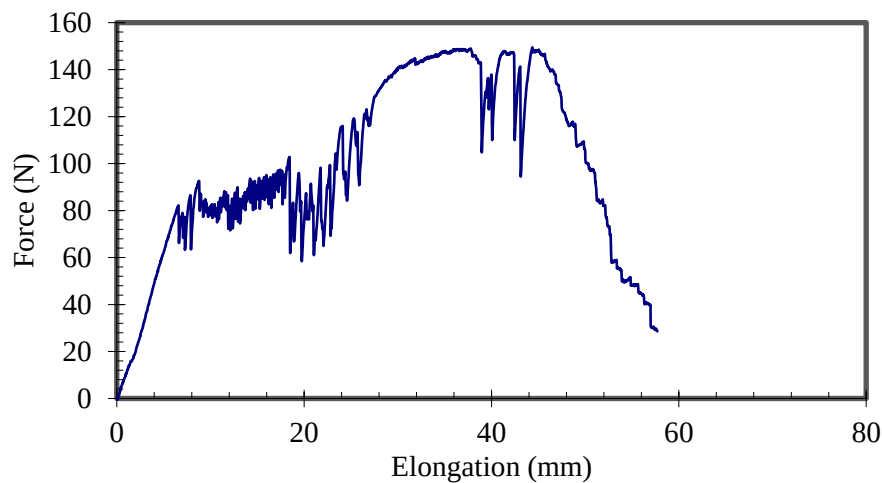


Fig. 5. Load deflection of untreated coconut fibre

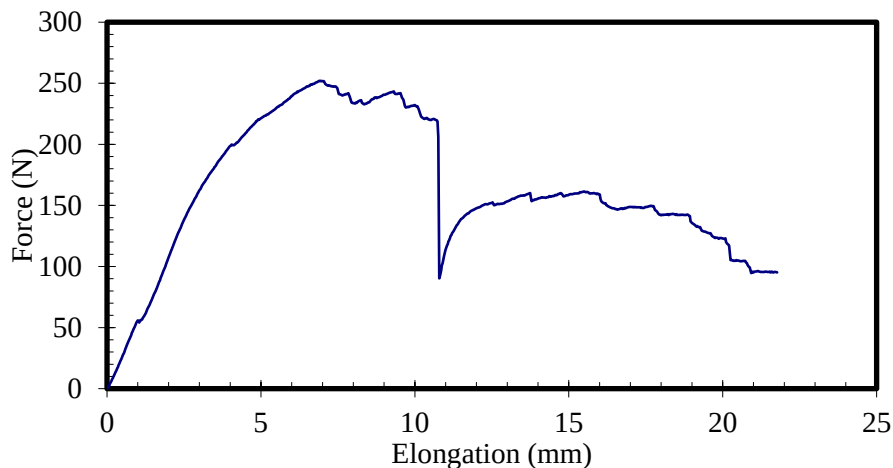


Fig. 6. Load deflection of treated coconut fibre

3.2 Tensile Strength of Untreated and Alkali-Treated Coconut Fibre

Table 5 presents the tensile strength and elongation of untreated and alkali-treated coconut fibers, tested at a rate of 50 mm/min with a sample length of 100 mm and a diameter of 0.2 mm. The tensile strength of untreated coconut fiber is 7.48 N/mm², while the tensile strength of alkali-treated coconut fiber is 12.6 N/mm². The tensile strength of alkali-treated coconut fiber significantly increases by 68% compared to that of untreated coconut fiber. According to Aluko et al. [31], treated natural fibers have been observed to exhibit higher tensile strength than untreated ones.

Table 5. Maximum force, tensile strength, and elongation

Treatment	Maximum Force (N)	Tensile Strength (N/mm ²)	Elongation
Untreated	149.50	7.48	57.70
Alkali treated	252.00	12.60	21.77

3.3 Tests on Fresh Properties of Coconut Fiber-Reinforced Concrete

Table 6 displays the mix proportions and slumps; the controlled concrete has the highest slump of 35 mm, excluding coconut fiber, plantain peel ash, and lime powder. Concrete with 0% lime powder, 0.5% coconut fiber, and 1.5% PPA had a slump of 38 mm, while concrete with 20% lime powder, 1.5% coconut fiber, and 1.5% PPA had a slump of 25 mm. The addition of increased coconut fiber content to the concrete mix decreases workability due to its high-water absorption during mixing [32,33]. Workability was observed to decrease in the presence of high fiber content combined with PPA and high lime powder content.

Table 6. Mix proportions and their varying slumps

Mix Number	Lime powder (%)	Coconut fibre (%)	PPA (%)	Slump (mm)
1	0	0.0	0.0	35±1.2
2	0	0.5	1.5	38±0.5
3	0	1.0	1.5	40±0.7
4	0	1.5	1.5	45±0.3
5	10	0.5	1.5	40±0.5
6	10	1.0	1.5	42±0.4
7	10	1.5	1.5	45±0.5
8	20	0.5	1.5	44±0.6
9	20	1.0	1.5	44±0.5
10	20	1.5	1.5	25±0.3

3.4 Mechanical Properties of Coconut Fibre Reinforced Concrete

The mechanical properties of concrete cubes, beams, and cylinders were tested. The compressive, flexural, and split tensile strengths result alongside the standard error is presented in Figure 7.

3.4.1 Compressive Strength (CS)

Based on Figure 7, the concrete's compressive strength ranged from 6.167 N/mm² to 13.171 N/mm². Mix 2, which contained 0% Lime Powder, 0.5% Coconut powder and 1.5% PPA gave a compressive strength surge of 12.1 N/mm², probably due to 1.5% PPA content, even with no lime powder added, the strength falls with Mix 3, however, the strength gradually rose and topped its strength with Mix 6 giving the highest value of average compressive strength at 13.171 N/mm², with 10% Lime Powder, 1% Coconut fiber and 1.5% PPA as compared to the control mix. This can be attributed to the synergistic effect of Lime Powder and PPA in improving the concrete performance. This is supported by a previous study on the effects of SCMs on concrete mechanical properties. The lowest compressive strength, mix 9, was 6.167 N/mm² and comprised 20% Lime Powder, 1% coconut fiber, and 1.5% PPA. The low compressive strength of the concrete specimens could be attributed to the coconut fiber's orientation, modulus, and specific gravity. Coir fiber is a

low-modulus fiber, unlike steel fiber, which is a high-modulus fiber [7,34]. According to a previous study by Velez et al. [12], the presence of coir fibers tends to reduce the compressive strength of composite concrete. Based on the compressive strength results of this study, it can be inferred that concrete can be beneficial for non-structural applications.

3.4.2 Split Tensile Strength (STS)

According to Figure 7, the split tensile strength of the mix's ranges from 1.348 N/mm² to 2.604 N/mm². The split tensile strength of Mix 2 increased to 2.604 N/mm², as compared to the control sample (Mix 1) of 2.08 N/mm², which indicates that 0.5% is the optimum coir fiber content, as 1% and 1.5% of coir fiber did not improve the split tensile strength. The decline in strength could be attributed to the reduction in fiber-matrix affinity. As the fiber content increases, the surface area requiring bonding between the concrete mixture and the fibers increases. Mix 4 has the lowest split tensile strength of 1.348 N/mm², and the decline in split tensile strength may be attributed to both the high fiber content of 1.5%. The addition of 0.5% coir fiber and 0% Lime Powder with a PPA content of 1.5% to coconut fiber-reinforced concrete significantly enhances its tensile strength, thereby improving the material's behavior. The observation of the positive effect of coir fiber in enhancing the post-cracking tensile strength properties of concrete is corroborated by previous studies [12, 35].

3.4.3 Flexural Strength (FS)

Concrete Prisms with dimensions of 400 mm by 100 mm by 100 mm were tested for flexural strength, and the concrete's strength ranged from 3.24 N/mm² to 6.96 N/mm² as presented in Figure 7. A minimum strength of 3.24 N/mm² was attained with 0% lime powder, 1.5% coconut fiber, and 1.5% PPA while the flexural test yielded highest strength with Mix 5, which consisted of 10% Lime Powder, 0.5% coir fiber, and 1.5% PPA, as compared to the control sample (Mix 1). The flexural strength of 6.96 N/mm² indicates improved performance for coconut fiber reinforced concrete. From this result it could be inferred that a 0.5% coir fiber content is an optimum level for enhancing concrete flexural properties, as it improves concrete ductility and prevents fractures by quickly transferring and distributing stresses to the concrete matrix through the interfacial bonding between the fibers and the concrete components [36].

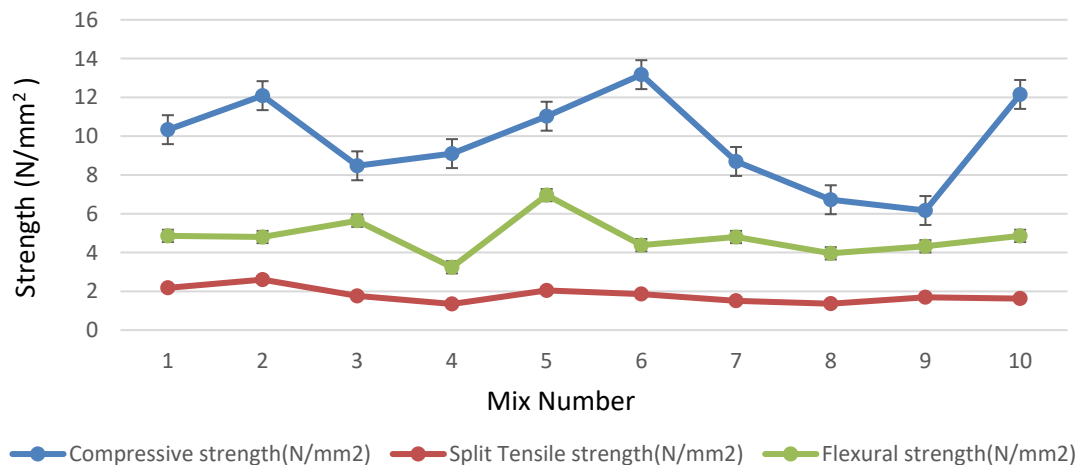


Fig. 7. Compressive, flexural, and split tensile strengths

3.5 Water Absorption Properties of Coconut Fibre Reinforced Concrete

The water absorption results alongside the standard errors is presented in Figure 8. This Figure presents the initial surface and final water absorption as a function of the mix proportions, and it is clear that both properties follow the same pattern. In controlled concrete without coir fiber, PPA, and lime powder, the absorption rates were 0.769% and 1.154%, respectively. The results of the analysis comparing the water absorption of concrete with different amounts of lime powder, coconut fiber, and PPA showed a range of 1.21% to 2.41% for the usual amount. In contrast, using

20% lime powder, 1.5% coconut fiber, and 1.5% PPA resulted in water absorption rates of 1.316% and 2.193%, respectively. Generally, plant-based fibers tend to absorb water due to their hydrophilicity and hygroscopic nature; however, alkaline treatment and chemical modifications have significantly reduced coconut fibers' tendency to absorb water. Therefore, Mix 5, with 10% Lime Powder, 1.5% PPA, and 0.5% Coir fiber among all the mixes, has a lower water absorption percentage at 0.8135% and 1.22% for ISA and FWA, respectively. Relating this to Figure 7, the mechanical characteristics, it is obvious that apart from Mix 1, Mix 5 has the lowest water absorption rate. Furthermore, the reason may not be far-fetched, since density could be related to strength, the compressive strength is 11.03 N/mm^2 , above the control sample (Mix 1), indicating the 10% lime powder and 1.5% PPA has remarkable impact on the Mix 5, thus, by taking part in pozzolanic interactions with lime/cement hydration products, PPA strengthens the paste by forming additional binding gel (C-S-H) [21]. Mix 5 has the highest flexural strength of 6.96 N/mm^2 , and the split tensile strength is marginal and comparable to that of Mix 1. Low resistance to water could be linked to the combined effect of Cement, Lime, and PPA. This finding aligns with previous studies on the water-absorbing tendency of organic fibers [37].

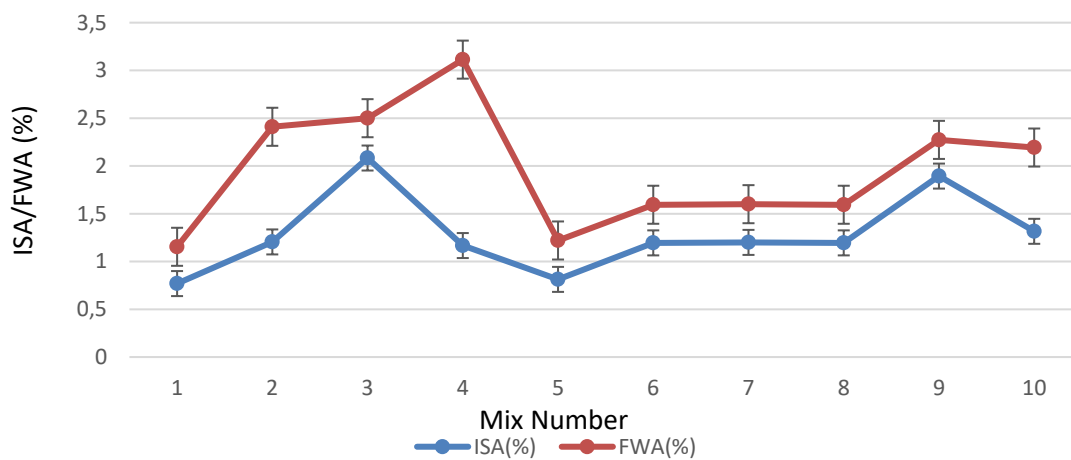


Fig. 8. Initial surface absorption and final water absorption

4. Conclusion

The ability of waste materials to either replace or supplement the conventional material in concrete requires that their performance in use is evaluated. The present and future performance of these alternative materials play a vital role in its acceptability hence the need for an experimental investigation on the mechanical and water absorption properties. The study examined the use of waste materials, including coconut fiber and plantain peel ash combined with lime powder in concrete. The study achieved improved flexural strength using the mechanical approach and demonstrated potential for improved resistance to water absorption with concrete mix containing 10% lime powder and 1.5% plantain peel ash. The maximum compressive, flexural and split tensile strengths was achieved with coconut fiber content of 0.5%. Based on maximum compressive strength of 13.17 N/mm^2 achieved in this study, this concrete can practically be applicable for non-structural or low-load purposes. The outcome reveals that alkali-treated coconut fiber enhances the tensile strength of concrete, while adding plantain peel ash, lime powder, and coconut fiber reduces workability. The study demonstrates that the combination of these aforementioned materials can enhance the concrete mechanical properties, minimize environmental impact, and reduce the water absorption rate in concrete. Agricultural waste, including coconut fiber and plantain peels, can be used to improve concrete performance, while reducing dependence on Portland cement and CO_2 emissions. The increasing generation of agricultural waste and the expansion of infrastructure in Nigeria necessitate sustainable management strategies to address growing waste and enhance the quality of construction materials. Further research is needed to examine the microstructural properties of coconut fiber reinforced concrete incorporating PPA and lime powder. This will provide detailed insight into the materials internal structure of the concrete.

In addition, a mechanism to enhance the separation of coconut fiber strands for easier mixing and compaction will promote the applicability of this natural fiber. Furthermore, a life-cycle cost analysis and sustainability metrics is required to promote the environmental benefits associated with sourcing and processing of waste materials.

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References

- [1] Singh M, Roy ABD, Waseem S, Singh H. Feasibility and performance analysis of carbonated recycled aggregate concrete. *Int J Sustain Eng* [Internet]. 2021;14(4):761–75. <https://doi.org/10.1080/19397038.2020.1856967>
- [2] Figueiredo F, Silva P, Botero ER, Maia L. Concrete with partial replacement of natural aggregate by PET aggregate — An exploratory study about the influence in the compressive strength. *Mater Sci*. 2022;9:172–183. doi: 10.3934/matricsci.2022011
- [3] Salem N. Effect of calcined clay on the properties of cementitious mortar reinforced by Posidonia Fiber . *Res Eng Struct Mater*. 2023, <http://dx.doi.org/10.17515/resm2023.759ma0505>
- [4] Maraveas C. Production of sustainable construction materials using agro-wastes. *Materials (Basel)*. 2020;13(2) 262. <https://doi.org/10.3390/ma13020262>
- [5] Golewski GL. Mechanical properties and brittleness of Concrete made by combined fly ash , silica fume and nanosilica with ordinary Portland cement. *Mater Sci*. 2023;10:390–404. <https://doi.org/10.3934/matricsci.2023021>
- [6] Al-ramthan A, Mezrakchi RA. Investigation of Cementitious Composites Reinforced with Metallic Nanomaterials , boric acid , and lime for infrastructure enhancement. *Mater Sci*. 2024;11:495–514. <https://doi.org/10.3934/matricsci.2024025>
- [7] Ayeni IS, Yatima JM., Abdul Shukor Lim NH. A Review of Hybridised use of fibres in shear behaviour of fibre-reinforced Concrete beams. *ASEAN Eng J*. 2024;1(14):145–56. <https://doi.org/10.11113/aej.V14.20314>
- [8] Affan Usmani M, Anas M. Study of Natural Fibre Reinforced Composites. *IOP Conf Ser Mater Sci Eng*. 2018;404(1) 012048. <https://doi.org/10.1088/1757899X/404/1/012048>
- [9] Joana M. Ferraz, Cláudio HS. Del Menezzi,, Divino E. Teixeira and SAM. Effects of treatment of coir fiber and cement/fiber ratio on properties of Cement-bonded composites. *Bioresources*. 2011;6(3):3481–3492.
- [10] Ede AN, Agbede JO. Use of Coconut Husk Fiber for Improved Compressive and Flexural Strength of Concrete. *Int J Sci Eng Res*. 2015;6(2):968–74.
- [11] Balaji GD, Abinaya C. Coconut : A Versatile And Multipurpose Crop. 2024;(06):26–30.
- [12] Velez E, Rodriguez R, Yanchapanta Gomez NB, Mora ED, Hernandez L, Albuja-Sanchez J.; Calvo, M.I. Coconut-Fiber Composite Concrete: Assessment of Mechanical Performance and Environmental Benefits. *Fibers* 2022; 10, (96) 1-12. <https://doi.org/10.3390/fib10110096> .
- [13] Ayoub Souileh, Achraf Mabrouk, Latifa Ouadif, Driss El Hachmi, Mohammed Chrachmy KB. Building earthquake resilience with clay shale: pioneering sustainable concrete solutions with eco-friendly materials. *Res Eng Struct Mater*. 2024;10(4). <http://dx.doi.org/10.17515/resm2024.479ea1007rs>
- [14] David, A. O., Bello, O. S., Akisanya, O. O., Orogade, B. O. and Amusan G. Effect of Plantain Peel Ash As An Admixture on Concrete Properties. *Sci Africana*. 2024;23(3):73–80. <https://dx.doi.org/10.4314/sa.v23i3.7>.
- [15] Adetayo OA, Jubril O. Effect of Ripe and Unripe Plantain Peel Ash on Concrete Workability and Compressive Strength. *FUOYE J Eng Technol*. 2019;4(1):145–8. <http://dx.doi.org/10.46792/fuoyej.v4i1.318>
- [16] Aderinola OO, Yusuf Y, Omotayo OO. Assessment Of Cement Concrete Partially Replaced with Polystyrene and Plantain Peel Ash. *Niger J Technol*. 2020;39(3):694–700. <http://dx.doi.org/10.4314/njt.v39i3.7>.
- [17] Nkem Ede A, Oluwabambi Ige A. Optimal Polypropylene Fiber Content for Improved Compressive and Flexural Strength of Concrete. *IOSR J Mech Civ Eng*. 2014;11(3):129–35. <https://doi.org/10.9790/1684-1134129135>
- [18] Jayashree Sengupta, Nirjhar Dhang AD. A Cost-Effective Slag-based Mix Activated with Soda Ash and Hydrated Lime: A Pilot Study. *Indian Institute of Technology, Kharagpur, West Bengal 721302, India;* 2024. <https://doi.org/10.1061/PPSCFX.SCENG-1426>

- [19] Martinelli B, Roger F, Ribeiro C, Marvila MT, Monteiro SN, Garcia C, et al. A Review of the Use of Coconut Fiber in Cement Composites. *Polymers* (Basel). 2023;1–15. <https://doi.org/10.3390/polym15051309>
- [20] Khan M, Ali M. Improvement in concrete behavior with fly ash, silica-fume and Coconut fibres. *Constr Build Mater* [Internet]. 2019;203:174–87. <https://doi.org/10.1016/j.conbuildmat.2019.01.103>.
- [21] Darweesh HHM. Effect of banana leaf ash as a sustainable material on the hydration of Portland cement pastes. *Int J Mater Sci*. 2023;4(1):1–11 .<https://www.mechanicaljournals.com/materials-science>.
- [22] Ahmad J, Majdi A, Al-Fakih A, Deifalla AF, Althoey F, El Ouni MH, et al. Mechanical and Durability Performance of Coconut Fiber Reinforced Concrete: A State-of-the-Art Review. *Materials* (Basel). 2022;15(10). <https://doi.org/10.3390/ma15103601>
- [23] C494-04 A. Standard Specification for Chemical Admixtures for Concrete. *Annu B ASTM Stand*. 2008;;1–9.
- [24] Awolusi T, Taiwo A, Aladegboye O, Oguntayo D. Optimisation of quinary blended supplementary cementitious material for eco-friendly paving unit using Taguchi orthogonal array design. *Materials Today : Proceedings*. 2022; 65(3); 2221-2227. <https://doi.org/10.1016/j.matpr.2022.06.263>
- [25] Shcherban EM, Stel SA, Beskopylny AN, Mailyan LR, Meskhi B, Shilov AA, Chemilnik A, Ozkilog YO, Aksoylu C. Normal-Weight Concrete with Improved Stress – Strain Characteristics Reinforced with Dispersed Coconut Fibers. *Appl Sci*. 2022;12. 11734. <https://doi.org/10.3390/app122211734>.
- [26] BS EN 12350-2. Testing fresh concrete, Part 2: Slump-test. BSI. 2009;
- [27] 12390-3:2019 BE. Testing hardened concrete Compressive strength of test specimens. *Br Stand Inst*. 2019;24.
- [28] BS EN 12390-6. Testing hardened concrete: Tensile splitting strength of test specimens. BSI. 2009;
- [29] BS EN 12390-5. Testing hardened concrete: Flexural strength of test specimens. BSI. *Br Stand Inst*. 2009;
- [30] 21 A-C-. Standard Test Method for Density , Absorption , and Voids in Hardened Concrete 1. :19428.
- [31] Aluko OG, Yatim JM, Kadir MAA, Yahya K. Residual Cube Strength and Microstructural Properties of Fire-Damaged Biofibrous Concrete with GEP-Based Prediction Model. *Arab J Sci Eng* [Internet]. 2023;48(10):13945–66. Available from: <https://doi.org/10.1007/s13369-023-08018-x>
- [32] Ali M. Coconut fibre : A versatile Material and its Applications in Engineering #. *J Civ Eng Constr Technol*. 2011;2(9):189–97. <http://www.academicjournals.org/icect>
- [33] Ali M, Chouw N. Experimental Investigations on Coconut-fibre rope Tensile Strength and pullout from Coconut fibre Reinforced Concrete. *Constr Build Mater*. 2013;41:681–90. <http://dx.doi.org/10.1016/j.conbuildmat.2012.12.052>
- [34] Parveen and Sharma A. Structural Behaviour of Fibrous Concrete Using Polypropylene Fibres. 2013;3(3):1279–82. <https://api.semanticscholar.org/CorpusID:15315240>.
- [35] Vagestan PK, Periyasamy M, Pathikonda P, Kiran U, Yadav SP, Zahid M, Avula D. Enhancing the Strength Properties of Concrete using Coconut fiber and Coconut fiber ash. *AIP Conf. Proc*. 2024; 3037, 020053. <https://doi.org/10.1063/5.0196073>
- [36] Zain M, Azffar M, Nurizaty Z, Sa N, Mujedu KA. Mechanical Properties of Concrete Containing Coconut Shell as Coarse Aggregate Replacement. *J Sustain Civ Eng Technol*. 2024;3(1):69–80. <https://doi.org/10.24191/jscet.v3i1.69-80>.
- [37] Ali NM. The Effect of Plantain and Banana Peel Ash on the Properties of Concrete. *Int J Innov Eng Res Technol*. 2022;9(4):31–37. <https://doi.org/10.17605/OSF.IO/8B4Y3>

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Optimized Al2011 hybrid composites reinforced with nano TiC and MoS₂ for enhanced mechanical and tribological performance

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Abstract

Hybrid metal matrix composites (HMMCs) are designed with the increased demand of light weight, strong and wear resistant materials in automotive and aerospace industry and structural fields. This paper reports the preparation and full characterization of Al2011-based HMMCs reinforced by different weight percent (2-10 wt%) of nano Titanium Carbide (TiC), and of a fixed composition of 4 wt% of nano Molybdenum Disulfide (MoS₂) prepared by stir casting. Nano TiC with its mechanical strength and boost in load-carrying strength was added, and MoS₂ was added as a solid lubricant to improve the tribology properties. During development, appropriate preprocessing, as well as drying and preheating of powder, led to an equal distribution of reinforcement and wettability. SEM, EDS and XRD microstructural studies affirmed homogeneous dispersion and integrity at interfaces. Density and porosity studies have shown very little defect in casting and the maximum densification was found at 8 wt% TiC. The tensile strength, hardness and impact resistance were significantly improved by mechanical testing and reached the maximum at 8 wt% TiC where agglomeration of particles decreased performance slightly. The dry sliding tribological tests showed exceptional decreases in wear speed, wear loss, and coefficient of friction with rising reinforcement level especially the synergistic impact of surface hardening TiC and lubricant MoS₂. Surface morphology due to wear indicated a shift in wear mechanisms to an oxidative and mild abrasive wear regime in composites compared with adhesive and abrasive wearing of the base alloy. The 8 wt% TiC + 4 wt% MoS₂ hybrid composites provided the most favorable mechanical and tribological performance. Although TiC/MoS₂ hybrids have been used in other Al matrices (e.g., AA6061/AA7075), there is still limited report on systematic optimization in the Al2011 matrix; this work fills this gap by bridging process controls with a complete microstructural-mechanical-tribological analysis.

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1. Introduction

The use of Aluminum Matrix Composites (AMCs) has been of particular interest over the past decades with its high mechanical, thermal, and tribological performance relative to traditional aluminum alloys [1]. AMCs are usually comprised of an aluminum or aluminum alloy matrix reinforced with hard ceramic particles, including SiC, Al₂O₃ or B₄C, which give them greater strength, stiffness, wear resistance, and thermal stability. Nano-reinforcements like TiO₂, graphene, and CNTs have enhanced its performance with a high surface area to facilitate load transfer, and low friction and wear due to the formation of lubricating films [2,3].

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To address the drawbacks of single-reinforced systems, Hybrid Metal Matrix Composites (HMMCs), i.e. systems combining two or more reinforcements, have been designed. One approach is to combine a hard ceramic, e.g., TiC, with a solid lubricant such as MoS₂, thus attaining high strength and enhanced wear resistance [4,5]. Further processing, such as powder metallurgy, ultrasonic-assisted stir casting and improved preheating, have resulted in improved dispersion and less porosity, and such hybrids are becoming more aerospace, automotive, and defense grade [9,10].

Al2011 is a favorable matrix among aluminum alloys due to its high machinability, moderate strength, and thermal/electrical conductivity [6]. Nevertheless, mechanical and tribological characteristics can be further advanced using hybrid nano-reinforcements. Titanium Carbide (TiC) is used to provide hardness, wear resistance, and thermal stability, and Molybdenum Disulfide (MoS₂) is used as a solid lubricant to reduce friction and alleviate brittleness [7,8].

Regardless of the vast research performed on the topic of AMCs, there are a limited number of studies concerning hybrid nano-reinforced systems in the context of the Al2011 alloy. The majority of previous literature has focused on traditional alloys like Al6061 or Al7075 and has generally neglected to consider the combined effect of both a ceramic reinforcement and a solid lubricant. So far, no systematic study has combined nano TiC with nano MoS₂ into the Al2011 matrix. The originality of the current study is the identification of such an initial combination, which is backed by the optimization of stir casting and multifaceted microstructural, mechanical, and tribological characterization. The methodology has not just filled gaps in the literature, but it has also shown the potential of Al2011-based HMMCs in aerospace, automotive, and metal-tooling applications that require a strength, toughness, and wear resistance balance.

1.1 Recent Related Work (2021–2025) And Positioning of This Study

Other studies have recently reported TiC moS₂ or MoS₂ + ceramic hybrids in Al alloys other than Al2011. In the case of AA7075, TiC+MoS₂ hybrids through stir casting yielded significant wear reduction and lowering of friction during dry sliding, where Taguchi-based optimization defined the operating range [11,12,13]. In AA6061 and the Al-Mg-Si-Cu alloy, MoS₂ and hard particles (Al₂O₃, SiC, WC) enhanced wear, but may compromise tensile response at higher MoS₂ concentrations, highlighting a strength-friction trade-off and content optimization requirement [14,15,16]. Larger reviews also highlight the fact that dispersion, porosity, and hardness / lubricity trade-off are controlled by the hybridization and stir-casting controls (powder pre-heat, melt temperature, vortex time), however [17].

1.2 Gap Addressed Here

Prior work largely targets AA6061/AA7075 or Al-Mg-Si-Cu matrices; Al2011, a Cu-rich, free-machining alloy with distinct solidification behavior, has not been systematically explored with nano-TiC + nano-MoS₂ hybrids across both mechanical (tensile, hardness, impact) and multi-regime wear testing with post-wear SEM. Our contribution is to (i) tailor a stir-casting window for Al2011 with nanoparticle pre-heat and controlled vortexing, (ii) map the property envelope 2–10 wt% TiC at fixed 4 wt% MoS₂, and (iii) rationalize the 8 wt% TiC + 4 wt% MoS₂ optimum relative to dispersion/agglomeration mechanisms and to trends reported for other Al matrices [11,12,16,18].

2. Methodology and Objectives

This study is an attempt to formulate and describe Al2011-based hybrid metal matrix composites (HMMCs) with nano Titanium Carbide (TiC) and nano Molybdenum Disulfide (MoS₂) reinforcement. The specific objectives are:

- To prepare Al2011TiCMoS₂ composites with different levels of TiC content (2-10 wt%) and constant 4 wt% of MoS₂ by an optimized stir casting pathway.
- To describe their physical, microstructural, mechanical and tribological characteristics.
- To determine the best combination of reinforcements to achieve maximum strength, toughness and wear resistance to be used in automotive, aerospace and tooling purposes.

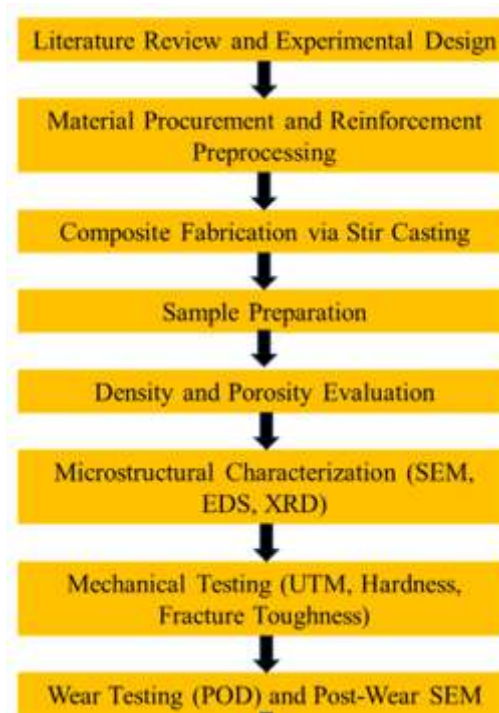


Fig. 1. Methodology flowchart

3. Materials and Methods

3.1 Materials

The alloy is Al2011 with the composition of 93.7% Al, 5.5% Cu, and 0.4% each of Bi and Pb. It has a density of 2.82 g/cm³, a melting point of 535 °C, thermal conductivity of 138-152 W/ m·K, and a modulus of elasticity of 71 GPa. For contextual reference, vendor datasheets report higher strength ranges for wrought Al2011 in aged tempers (e.g., T-series). In this work, all comparisons of mechanical properties are made only against the in-house as-cast Al2011 baseline fabricated and tested under identical conditions.

Nano-TiC (under 100 nm) has a melting point of around 3160 °C, density of 4.93 g/cm³, hardness of 3200-4000 HV, elastic modulus of 460 GPa, and thermal conductivity of 21 W/ m·K. Nano-MoS₂ (under 100 nm) has a thermal conductivity of about 85 W/ m·K, hardness of 1000-1200 HV.

Fig. 2. Nano TiC and nano MoS₂

3.2 Composite Fabrication

This research centers on production of Al2011 based hybrid metal matrix composites (HMMCs) reinforced with nano-sized Titanium Carbide (TiC) and Molybdenum Disulphide (MoS₂) in stir casting process (Fig. 3). Al2011 alloy chosen as the base matrix because of its superior machinability and moderate strength and relatively easy fabrication capabilities, with TiC (a hard ceramic nanoparticle, <100 nm) added because of its reinforcing effect that increases mechanical strength and wear resistance. To enhance the tribological performance in terms of decreased friction and wear, MoS₂ as a two-dimensional solid lubricant was used. Choice of optimal MoS₂ content was also a critical parameter in this design. After a thorough literature review and preliminary testing, 4 wt% MoS₂ was determined as the most promising loading that strikes the right balance between the two objectives, the reduction of friction and maintaining mechanical integrity. Dried at 100-120°C under a vacuum, Reinforcement powders were pre-heated to 300-400 °C to enhance wettability and limit associated defects caused by moisture. Al2011 alloy melt was heated to 750-800 °C in graphite crucible, and a mechanical stirrer set at 400-600 rpm was utilized to create a vortex where the reinforcements were progressively added and then mixed over 5-10 minutes. To be able to reproducibly achieve the desired results, the stir casting procedure has been rigorously standardized in all compositions. The stirring rate was kept constant at 500 ± 20 rpm until a stable vortex was formed (a total of 8 ± 1 minutes), then the nano-reinforcements were added steadily during the first three minutes to allow sufficient wetting and dispersion. The preheated steel moulds were cast at 250 ± 10 °C and allowed a time of approximately 15 minutes to stabilize prior to pouring which contributed to reducing the effects of thermal shock and shrinkage porosity. Once poured, the ingots were left to cool at ambient temperature (28 °C) in still air to solidify the ingots without forced cooling, which ensures uniform solidification of the grains. Casting was done in the normal laboratory air conditions and the preheating (300-400 °C for 1 hour) of the reinforcements was undertaken to get rid of moisture and the entrapment of gases. Weighed in a precision balance with ± 0.001 g accuracy gave the reinforcement powders a compositional tolerance of approximately ± 0.05 wt% and the Al2011 alloy charge was weighed with a ± 0.1 g accuracy so that the melt ratios could remain consistent. The slurry was poured into preheated steel casts to make ingots. 4 wt% MoS₂ is selected on the basis of previous literature that has compared the properties and performance of this concentration against others and has confirmed that 4 % is sufficiently good in reducing the coefficient of friction and the wear rate by forming a lubricating tribo-film without any adverse effect on hardness or tensile strength. The elevated concentration of MoS₂ (6 wt %) has been linked to low interfacial bonding and porosity, which degrade the mechanical property. Consequently, our combined TiC reinforced with 4 wt% MoS₂ provides an optimized hybrid reinforcement approach to engineer multifunctionality in an aluminum-based material.

3.2.1 Heat-Treatment/Temper

All cast ingots and machined coupons were tested in the as-cast laboratory condition. No solution heat treatment or artificial ageing was performed; specimens solidified in still air after pouring. Consequently, wrought temper designations (e.g., T-temper) are not applicable to these coupons.



(a)



(b)

Fig. 3. (a) Stirring of molten aluminium, adding of reinforcements, pouring the mixture in to the preheated dies and (b) Final cylindrical ingots

Table 1. Composition of fabricated hybrid Al2011 composites

Composite ID	Nano TiC Content (wt%)	Nano MoS ₂ Content (wt%)
Hybrid-2	2	4
Hybrid-4	4	4
Hybrid-6	6	4
Hybrid-8	8	4
Hybrid-10	10	4

3.3 Experimentation

To reduce variability, all test specimens were machined at the central locations of the cast ingots, and not on surface layers, which can contain segregation or oxide skin. Samples were first polished and all casting scale was removed before testing. In every condition, three or more replicates were made, and the reported values are averages plus a standard deviation normally between ± 2 -5%. This methodology made sure that the data is a representation of actual material behavior and any tiny experimental uncertainty is considered. All the tests were conducted in triplicates in all experimental tests such as hardness, tensile, impact and wear and the results are presented as means-standard deviation (SD). To make the presentation of data in figures and tables more precise, error bars indicating one standard deviation are used to show variability and have statistical reliability. The experimental study aimed to assess the physical, mechanical and tribological characteristics of Al2011-based hybrid metal matrix composites filled with nano Titanium Carbide (TiC) and Molybdenum Disulfide (MoS₂), with a fixed percentage 4 wt%.



Fig. 4. Density tester, hardness test setup, UTM for tensile properties

Evaluation of physical property involved the measurement of the theoretical and experimental density as well as the porosity. The Rule of Mixtures was integrated to ascertain theoretical density with reference to the weight fraction and densities of individual constituents whereas experimental density was measured through Archimedes principle of weighing a specimen in air and water. Porosity was calculated to gauge structural integrity in order to determine the cast samples. To characterize the mechanical properties, a Vickers Hardness based testing was conducted under 500-gf load with a dwell time of 10-15 seconds and was repeated five times per each sample to be precise in the results. Yield strength, ultimate tensile strength, and the elongation of tensile properties were obtained according to ASTM E8/E8M specifications on Universal Testing Machine

in dog-bone-shaped specimens. Volumes of energy that were absorbed by U-notched specimens in a sudden impact were measured by Charpy impact tests based on ASTM E23. These are the experimental procedures that allowed reaching an in-depth understanding of the role of nano TiC and MoS₂ reinforcements on structural integrity, strength, toughness, and wear resistance of the hybrid composites developed, and capable of being implemented in countless applications, including but not limited to bushings, sliding interfaces, and in aerospace structural components.

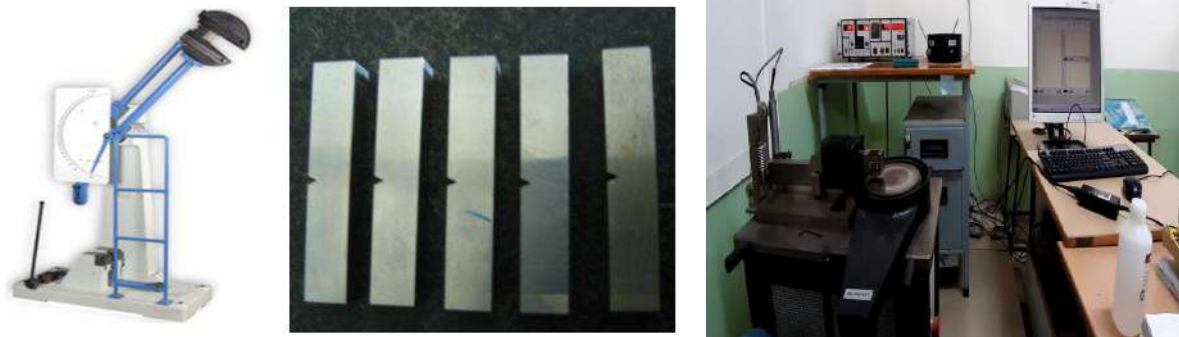


Fig. 5. Impact test setup and POD setup

Dry sliding tests were conducted on a pin-on-disc tribometer per ASTM G99. Composite pins (cylindrical, $\varnothing 8 \text{ mm} \times 25 \text{ mm}$) were machined from the central region of each cast, ground to P1200 SiC finish, ultrasonically cleaned in acetone (5 min), rinsed with ethanol, and air-dried. The counter face disc was bearing steel 100Cr6 (EN31), hardness $60 \pm 2 \text{ HRC}$, surface roughness $R_a = 0.05\text{--}0.10 \text{ }\mu\text{m}$, cleaned identically before each run. Tests used normal loads 10/30/50 N and speeds 100/300/700 rpm at a track radius of 30 mm, giving linear speeds of $0.314/0.942/2.199 \text{ m}\cdot\text{s}^{-1}$ ($v = 2\pi r \cdot \text{rpm}/60$). Each test covered a sliding distance of 1000 m (time adjusted per speed), with a 60 s run-in period excluded from analysis. A fresh track was used for every test to avoid cross-interaction. All tests were performed under laboratory air, $25 \pm 2^\circ\text{C}$ and $45 \pm 5\% \text{ RH}$, without lubrication.

The coefficient of friction (COF) was recorded continuously from the tribometer's tangential load cell and averaged over the steady-state segment (post run-in). Mass loss was measured to 0.1 mg (analytical balance) and converted to volume loss using the specimen's measured density (Archimedes method). Wear rate was reported as volume loss per unit load and distance:

$$\text{Wear rate} \left(\frac{\text{mm}^3}{\text{Nm}} \right) = \frac{\Delta m / \rho}{F_n s} \quad (1)$$

All conditions were tested in triplicate ($n = 3$). Results are reported as mean with $\pm \text{SD}$ ($n = 3$). One-way ANOVA (factor: TiC wt%) with Tukey HSD was used to assess composition effects for wear rate, wear loss, and COF; merged ANOVA results are summarized in Table 3

3.4 SEM Imaging and ImageJ Quantification

SEM was carried out at 15 kV, 10–12 mm WD under high vacuum. Dispersion was evaluated at $500\times$ (field width $\approx 1.0 \text{ mm}$; pixel $\approx 0.5 \text{ }\mu\text{m}$), with $n = 12$ fields/composition (3 specimens $\times$ 4 fields). Pixel calibration was set from the SEM scale bar. Images were processed in ImageJ: grayscale conversion $\rightarrow$ background subtraction $\rightarrow$ median filter $\rightarrow$ Otsu thresholding (guided by BSE contrast and spot EDS) $\rightarrow$ morphological clean-up $\rightarrow$ watershed separation. Features smaller than 3 px^2 were excluded as sub-resolution noise. Metrics reported are apparent areal fractions of resolved features $\geq d_{\text{res}}$ ($\sim 1\text{--}2 \text{ }\mu\text{m}$), equivalent-circular diameters, and particle counts.

Because the nano-reinforcements ($<100 \text{ nm}$) lie below the resolution limit at $500\times$, the measured area fractions ($\approx 0.8\text{--}1.6\%$) are lower bounds relative to the gravimetric target ($\sim 7 \text{ vol\%}$ at Hybrid-8). This underestimation is expected; high-magnification BSE/EDS confirmation ($10\text{k--}20\text{k}\times$) is noted for future work.

4. Results and Discussion

Mechanical and tribological values reported herein, unless mentioned to the contrary, are the mean of three independent tests, error bars in associated figures reflecting ± 1 standard deviation. With this presentation, it is possible to better compare trends and take into account natural experimental scatter.

- Reference baseline: Unless otherwise stated, improvements are reported relative to the as-cast Al2011 control produced and tested in this study (hardness 100 HV, UTS 365 MPa, YS 290 MPa, elongation 15%, Charpy 5.4 J).

4.1 Microstructural Characterization

Microstructural and compositional characterization of as-received materials was done so as to establish the suitability of such materials and use them in the fabrication of hybrid composites. SEM micrograph of Al2011 alloy 200x magnification helped to show a uniform microstructure with well distributed secondary phase particles and good structural integrity was reflected since there was no porosity or any other casting defects. The EDAX analysis did confirm that the major constituent was aluminum followed by copper, bismuth and lead, which are all components of the known alloy, whereas the XRD spectra based on the face forward cubic (FCC) crystal structure with the peaks at 2θ corresponding to (111), (200), (220) and (311) planes. Likewise, SEM morphology of nano TiC produced faceted, nanometric particles (40-80 nm) with little agglomeration, and high purity and the EDAX spectrum reported stoichiometric Ti and C with the trace of oxygen and Fe whereas XRD pattern suggested its FCC-type structure with maximum peaks corresponding to (111), (200), (220), and (311) planes in accordance with JCPDS card index no. 32-1383.

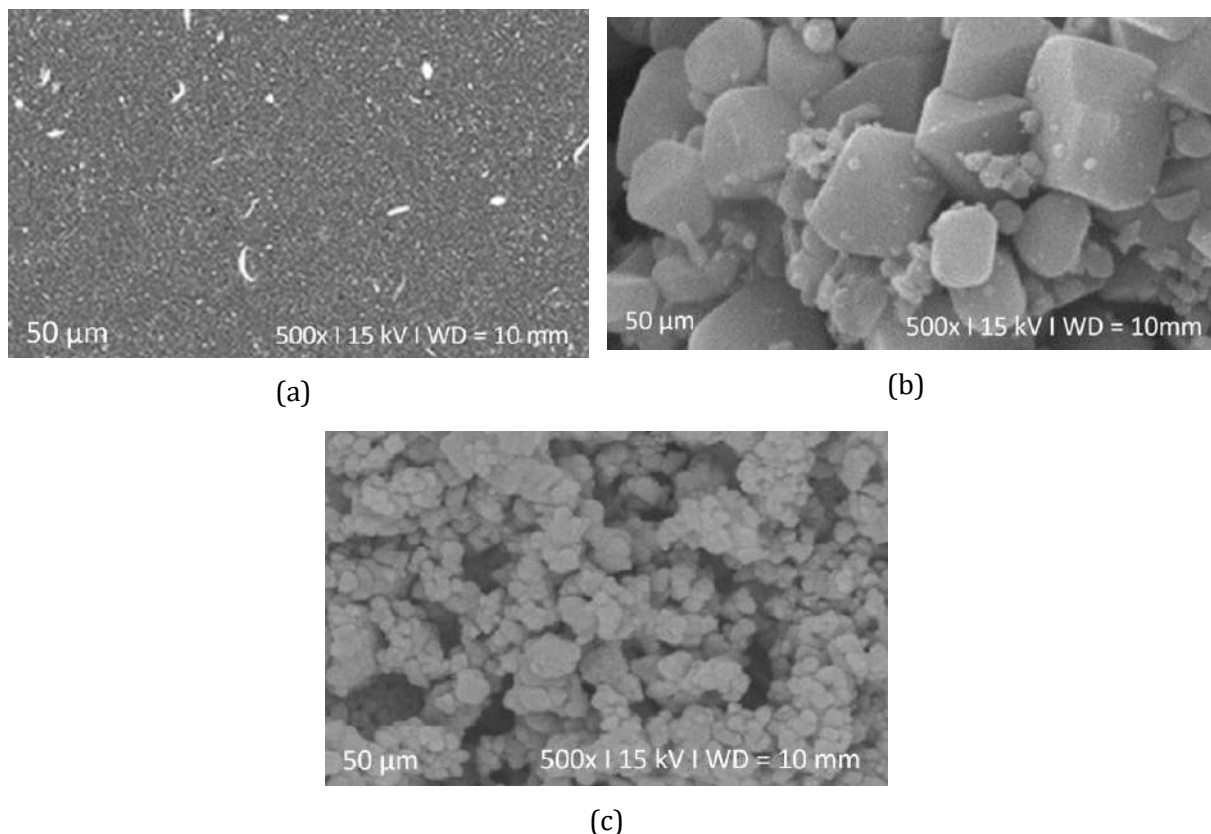


Fig. 6. SEM micrographs of (a) Al2011 base alloy, (b) nano-TiC powder, and (c) nano-MoS₂ powder acquired under identical imaging conditions

The EDAX verified that nano-MoS₂ particles had characteristic morphology and stoichiometry with peaks verified as in accordance with Mo and S, and according to XRD the structure of the particles was highly ordered and crystallized in hexagonal with very strong peaks distributed at $2\theta = 14.4^\circ$ and other related to the multi-planar nature of the crystal structure MoS₂ (JCPDS file No. 37-

1492). The lack of undesired peaks in the EDAX and XRD spectra of the materials tested justified their phase purity and chemical integrity that neither base alloy nor reinforcements would be structurally and chemically inappropriate towards the synthesis of a hybrid in metal matrix composite. Images were recorded at 500× magnification, with a 50 µm scale bar, using an accelerating voltage of 15 kV and a working distance of approximately 10 mm, enabling direct morphological comparison

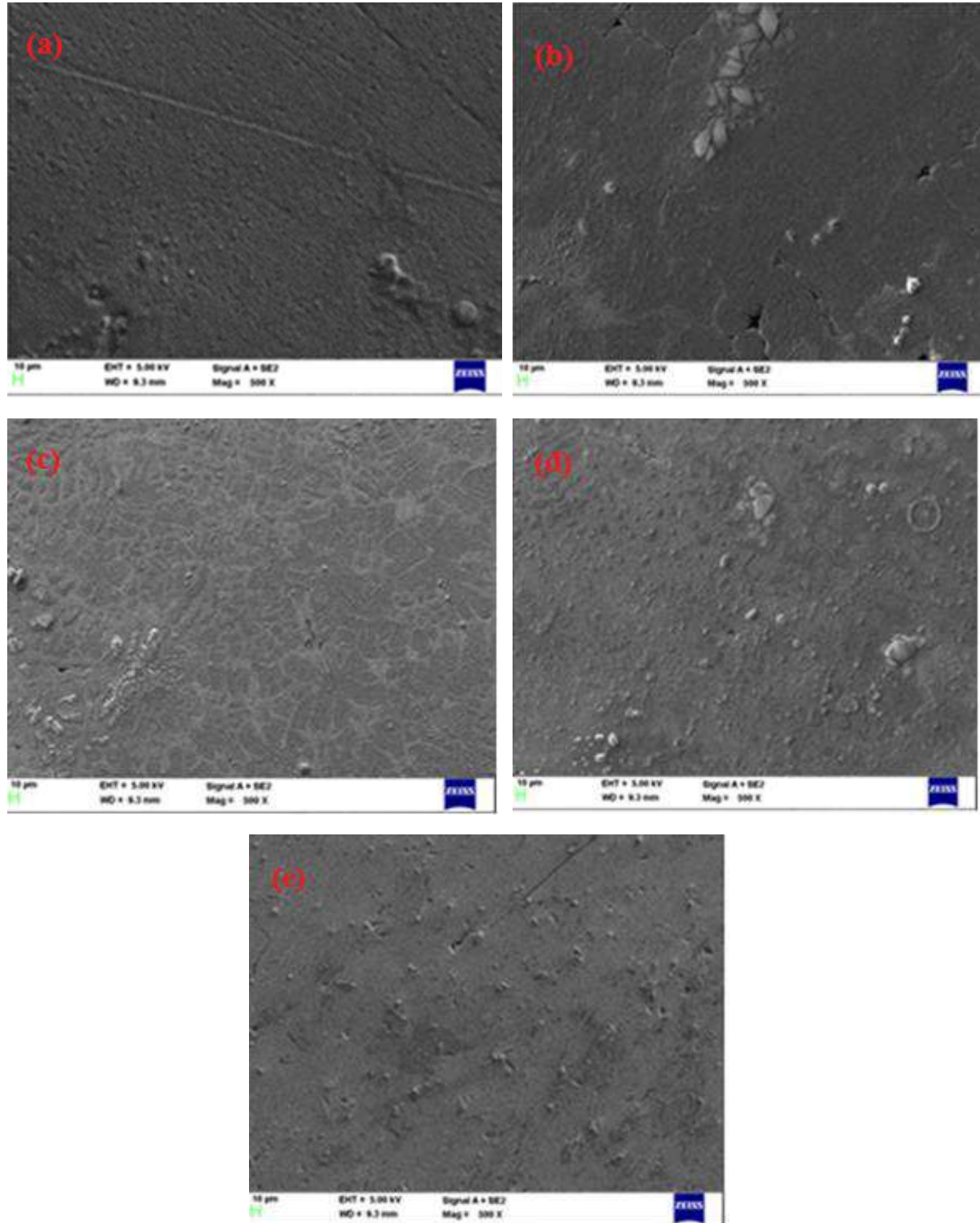


Fig. 7. SEM photographs of hybrid composites: a) 2% TiC and 4% MoS₂ b) 4% TiC and 4% MoS₂ c) 6% TiC and 4% MoS₂ d) 8% TiC and 4% MoS₂ and e) 10% TiC and 4% MoS₂

Fig. 6(a-e) shows SEM micrographs depicting the microstructural development of Al2011-based hybrid composites reinforced with different nano TiC contents (2-10 wt%) and 4 wt percent of nano MoS₂ at 500x magnification. The images provide insights into the dispersion of reinforcements, grain refinement, interfacial bonding and agglomeration behavior that are critical. The microstructure at 2 wt% TiC (Fig. 6a) has a relatively coarse matrix composition, a low density of TiC particles and flaky MoS₂ areas which are localized mainly along the grain boundaries. The distribution lies within a moderate range of uniformity, though the micro voids and cracks demonstrate a low interfacial bonding owing to the low amount of ceramic content. At 4 wt% TiC (Fig. 6b), the matrix seems to be finer and the dispersal of the reinforcement becomes more even, an indication of better wettability and bonding. TiC particles have a more even distribution in the matrix, whereas MoS₂ at the interfaces is probably involved in diversion of cracks and redistribution of stress. The 6 wt% TiC composite (Fig. 6c) shows a more subtle and homogenous microstructure, dense dispersion of TiC and grain boundary pinning, which show the effective inhibition of grain growth. The combination of MoS₂ contributes to interfacial stability, minimizing porosity and rendering it the most structurally robust. At 8 wt% TiC (Fig. 6d), although noticeable grain refinement is made, signs of agglomeration of TiC start to appear, in particular in localized agglomerates, which can potentially lead to initiation of voids and failure to achieve uniformity in the structure. Lastly, at 10 wt% TiC (Fig. 6e), an excess clustering and non-uniformity of the particle spacings prevail, creating more porosity and a hindered continuity of the matrix. The lower wettability and reinforcement oversaturation at this scale contribute to the degradation of the MoS₂ effectiveness, which compromises the interfacial integrity. All in all, the microstructural analysis demonstrates that the 6 wt% TiC + 4 wt% MoS₂ combination provides the highest reinforcements-matrix synergy effect with enhanced dispersion, reduced porosity, and good grain-refinement overall without the negative outcomes' higher degree of additions of ceramic additions.

Although the microstructural observations indicate that the 6 wt% TiC + 4 wt% MoS₂ composite exhibits the most homogeneous dispersion and the lowest porosity, subsequent property evaluations (density, hardness, tensile, impact, and wear) consistently identify the 8 wt% TiC + 4 wt% MoS₂ composition as the optimum in terms of combined mechanical and tribological performance. This apparent disparity highlights that microstructural uniformity alone does not directly translate to maximum property enhancement.

At 8 wt% TiC, a higher effective reinforcement content promotes stronger Orowan strengthening, enhanced load transfer, and greater resistance to plastic deformation and surface damage. Although localized TiC clustering is observed at this composition, its influence is insufficient to negate the benefits of increased ceramic content and improved load-bearing capability. In contrast, the 6 wt% composite, while structurally more uniform, contains a lower reinforcement volume fraction that limits the extent of strengthening and wear resistance achievable. Excessive agglomeration at 10 wt% TiC, however, introduces stress concentrations and porosity that deteriorate both mechanical and tribological performance.

Consequently, the 6 wt% TiC composite represents the most structurally uniform system, whereas the 8 wt% TiC + 4 wt% MoS₂ composite achieves the most favourable balance between reinforcement content, dispersion, and interfacial integrity, resulting in superior strength, toughness, and wear resistance. Hybrid-8 contains a higher reinforcement content (≈ 7 vol% total reinforcement, based on density conversion), which increases strengthening and load-bearing contribution. Hybrid-6 shows better dispersion uniformity, but its lower reinforcement volume limits the maximum attainable strengthening and wear resistance.

4.1.1 Quantitative Image Analysis of Reinforcement Distribution

Quantification confirmed the observed trends in dispersion (Table 2). Apparent areal fractions ranged from 0.85 to 1.62%, increasing with TiC addition. These values are lower than nominal because the 500 $\times$ overview resolves only micron-scale clusters, not individual nanoparticles. Gravimetric batching ensured actual reinforcement contents (± 0.05 wt%). Thus, the ImageJ values are best interpreted as relative indicators of clustering and distribution, not absolute vol%.

The area-fraction trend was consistent with the nominal reinforcement addition although magnification of the available SEM images could not directly measure the nanometric particle size.

This helps to achieve uniform integration of TiC/MoS₂ in the matrix during lower loadings whereas agglomeration in Hybrid-10 was also comparable to the increased dispersion in particle size distribution. This clustering is also consistent with the lower hardness and tensile strength observed at high amounts of TiC. The further improvement of the magnification (>20,000×) in the future will allow determining the nanometric size distribution of the particles more precisely, but the current approximate determination confers the dispersion trends that are being reported as qualitative.

Table 2. ImageJ quantitative metrics

Composite	Particles counted	Area fraction (%)	Mean ECD (pixels) ± SD
Hybrid-2	39	0.85	3.7 ± 2.0
Hybrid-4	42	1.12	4.1 ± 2.3
Hybrid-6	47	1.35	4.4 ± 2.1
Hybrid-8	52	1.48	5.0 ± 2.5
Hybrid-10	49	1.62	5.8 ± 3.0

Detailed particle size distribution histograms obtained from ImageJ analysis are provided in the Supplementary Material. Quantitative histogram analysis confirmed that Hybrid-6 exhibited a narrower particle size distribution, reflecting better dispersion and interfacial bonding. In contrast, Hybrid-10 showed a broader and skewed distribution due to reinforcement agglomeration, correlating with its reduced mechanical performance. Because the individual nano-reinforcements (<100 nm) lie below the resolution limit of 500× imaging, the reported area fractions represent comparative indicators of clustering rather than absolute nanoparticle volume fractions. Based on constituent densities, the Hybrid-8 composition corresponds to approximately ~7 vol% total reinforcement, a range reported to maximize load transfer and tribo-film stability without severe particle clustering. Accordingly, the ImageJ areal fraction and ECD results are used only for comparative dispersion/clustering trends, not for absolute nanoparticle quantification.

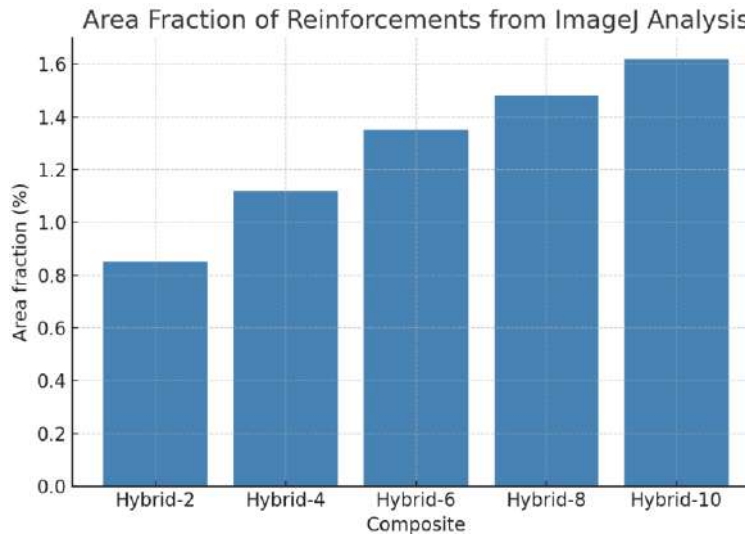


Fig. 8. Area fraction of reinforcements from ImageJ analysis

4.2 Density and Porosity Measurements

The theoretical density increased gradually with an increase in the percent of TiC, starting at 2.896 g/cm³ at 2 wt% of TiC to 3.002 g/cm³ at 10 wt% of TiC due to the relatively higher densities of the reinforcement relative to the matrix. Experimental density values also showed a similar trend with an increase in density values of 2.83 g/cm³ to 2.96 g/cm³ which indicates that the reinforcement has been effectively incorporated in matrix and that the stir casting process is reliable in producing a good integration of the matrix and the reinforcement. The porosity also decreased, based on the theoretical and experimental density, which was 2.28 % in the composite containing 2 wt% TiC

(Hybrid-2) to a minimum of 1.16% in the composite containing 8 wt% TiC (Hybrid-8), indicating a better wettability, less entrapment of gases, and better dispersion of reinforcement. Nonetheless, the further increase of porosity to 1.39% at 10 wt% TiC (Hybrid-10) indicates the reinforcement agglomeration and concentration of repeated nano-voids under higher loading. Hybrid-8 (8 wt% TiC + 4 wt% MoS₂) showed the most favorable physical properties, being high in experimental density and low in porosity, and hence, best selection for much improved mechanical and tribological properties.

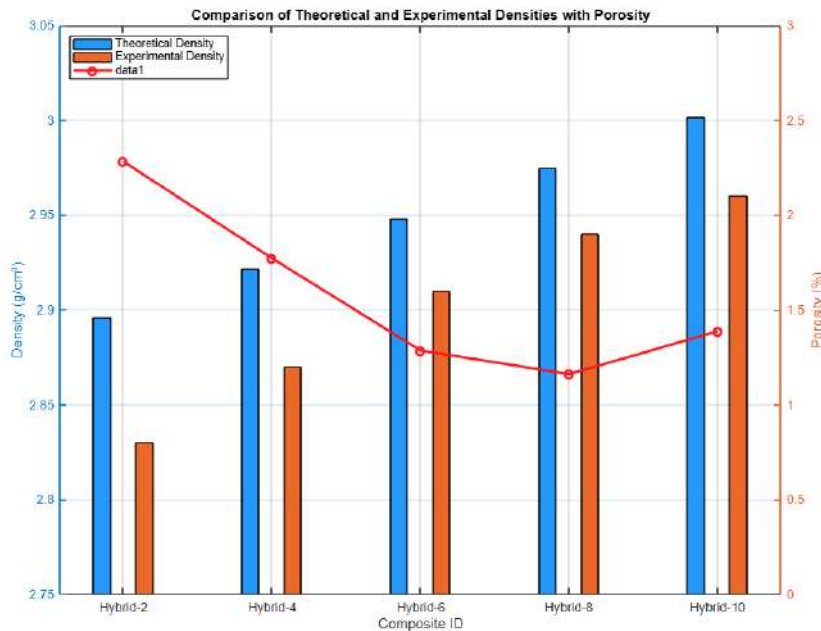


Fig. 9. Theoretical and experimental densities with porosity (%) for Al2011–TiC–MoS₂ hybrid composites

4.3 Hardness Measurements

The as-cast Al2011 baseline measured 100 HV; hardness increased with TiC content to 120 HV at Hybrid-8, followed by a slight decline at Hybrid-10. As the TiC content increased, a gradual increase in hardness was measured with a maximum reading of 120 HV at 8 wt% TiC as shown in Figure 10. Such improvement is attributed to several mechanisms: Orowan strengthening caused by the blockage of dislocation movements by hard TiC nanoparticles, efficient transfer of load between the ductile matrix and stiffer reinforcing, and grain refinement induced by TiC as nucleation sites, in line with the Hall-Petch relation. Even though MoS₂ is mainly a solid lubricant, its consistent occurrence at the grain boundaries helps with structural coherence, and it also aids in reinforcing dispersion homogeneity. Yet, a slight decrease in hardness to 117 HV was observed in the 10 wt% TiC composites, probably because the agglomeration of particles and localized porosity decrease the efficiency of bonding between the matrix and the reinforcement and facilitate the relaxation of stresses during indentation. All-in-all, the composite with 8 wt% TiC and 4 wt% MoS₂ (Hybrid- 8 composite) showed the best hardness response, depicting a synergetic balance between the reinforcement level, dispersion uniformity, and microstructural integrity. The composition is therefore added on tribological purposes where superior wear and indentation resistance is needed.

4.4 Impact Analysis

The results obtained by the Charpy impact test reported the impact strength of Al2011-based hybrid metal matrix composites reinforced with varying concentration of nano TiC (2 wt%, 6 wt%, 10 wt%) and the same proportion of MoS₂ (4 wt%) on U-notched specimens. An impact energy of 5.4J was measured on the base Al2011 alloy. The impact resistance also gradually improved with the content of TiC, and maximum impact resistance achieved by Hybrid-8 composition (8 wt% TiC + 4 wt % MoS₂) whose value reached 7.9 J, i.e. 46% higher. This is as a result of the crack-arresting

capability of hard TiC particles and stress-relieving effect of MoS₂, which absorbs impact shock energy collectively. It was however slightly reduced to 7.2 J in Hybrid-10 and this is probably attributed to agglomeration of particles and high porosity which may behave as stress concentrators and alter energy absorption. On the whole, the most favorable balance between toughness and reinforcing effectiveness was marked in Hybrid-8.

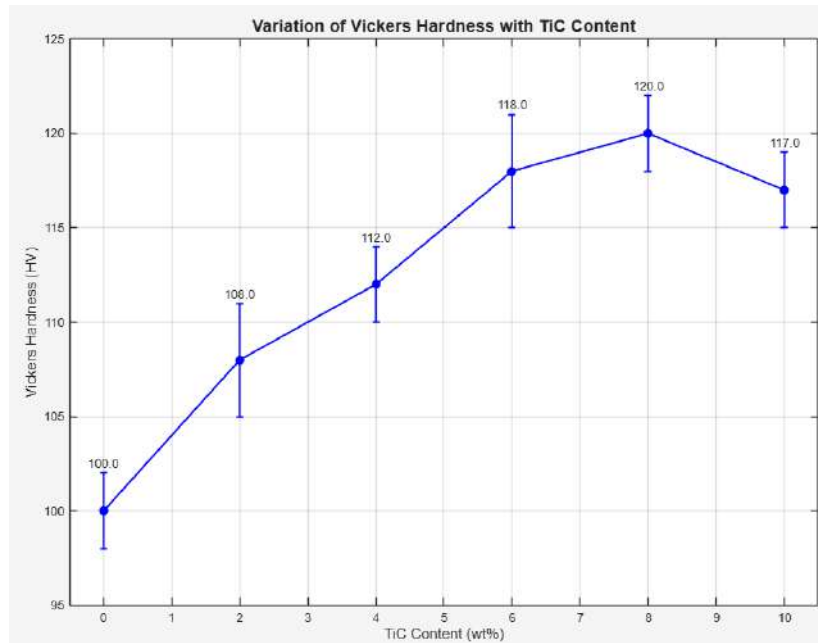


Fig. 10. Variation of Vickers hardness with TiC content in Al2011-based hybrid composites. Values represent mean $\pm$ standard deviation ($n = 3$ replicates). Error bars denote ± 1 SD

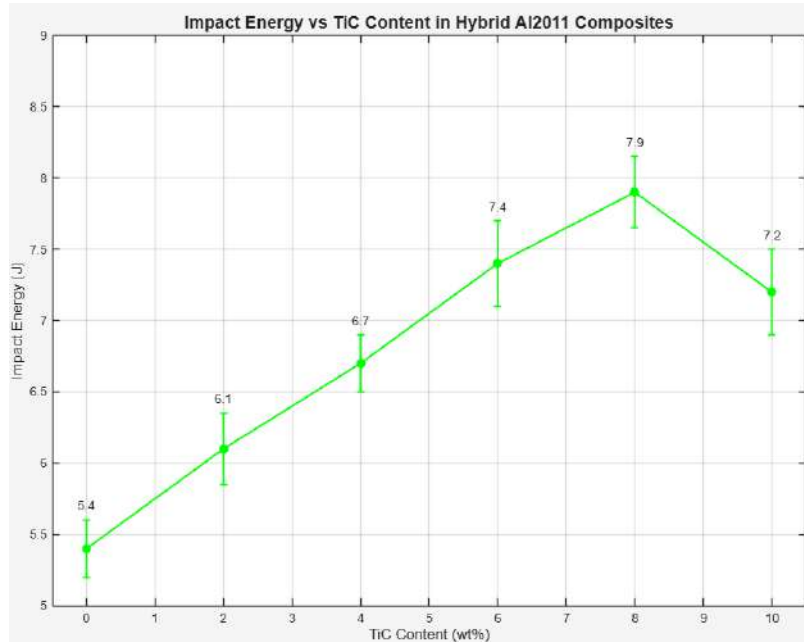


Fig. 11. Variation of impact energy with TiC content in Al2011-based hybrid composites. Values represent mean $\pm$ standard deviation ($n = 3$ replicates). Error bars denote ± 1 SD

4.5 Tensile Properties

Figure 12 summarizes the tensile properties Ultimate Tensile Strength (UTS), Yield Strength (YS) and Elongation (A50). The base Al2011 alloy had a UTS of 365 MPa with YS of 290 MPa and elongation of 15%. A significant change was witnessed through the introduction of reinforcements. UTS increased monotonically between 378 MPa (Hybrid-2 (2 wt. % TiC + 4 wt. % MoS₂)) and

414 MPa (Hybrid-8 (8 wt. % TiC + 4 wt. % MoS₂)), and YS increased monotonically between 328 MPa (Hybrid-2) and 352 MPa (Hybrid-8). Such enhancement is mainly through Orowan strengthening, load transfer effects, and grain refinement, with Titanium Carbide particles impeding the motion of dislocations and serving as nucleation points during solidifying. Although MoS₂ is a solid lubricant, it helped accommodate interfacial shearing stresses without undermining strength because of its fine dispersion and compatibility with the matrix. At 10 wt% TiC (Hybrid-10), UTS and YS were reduced by a small amount to 405 MPa and 319 MPa, respectively, which can be attributed to agglomeration of the particles, the growth of porosity, and microstructural defects that led to poorer reinforcement-matrix interfacial bonding. Ductility (A50) was reduced as TiC content was raised, falling by nearly 2 percentage point between the base alloy and Hybrid-8 because plastic deformation was limited by the presence of hard ceramic particles. Interestingly, Hybrid-10 also appeared to be subject to minor enhancement, to 13.1% possibly due to non-uniform agglomeration zones that allowed localized deformation of the matrix. Dislocation pile-up at reinforcement interfaces, thermal mismatch strengthening, and synergistic hard TiC and lubricating MoS₂ support the enhanced tensile response over compositions. On aggregate, Hybrid-8 displayed the best tensile behavior overall, providing a good trade-off between strength and ductility, and therefore a promising choice in structural applications as well as wear-sensitive applications.

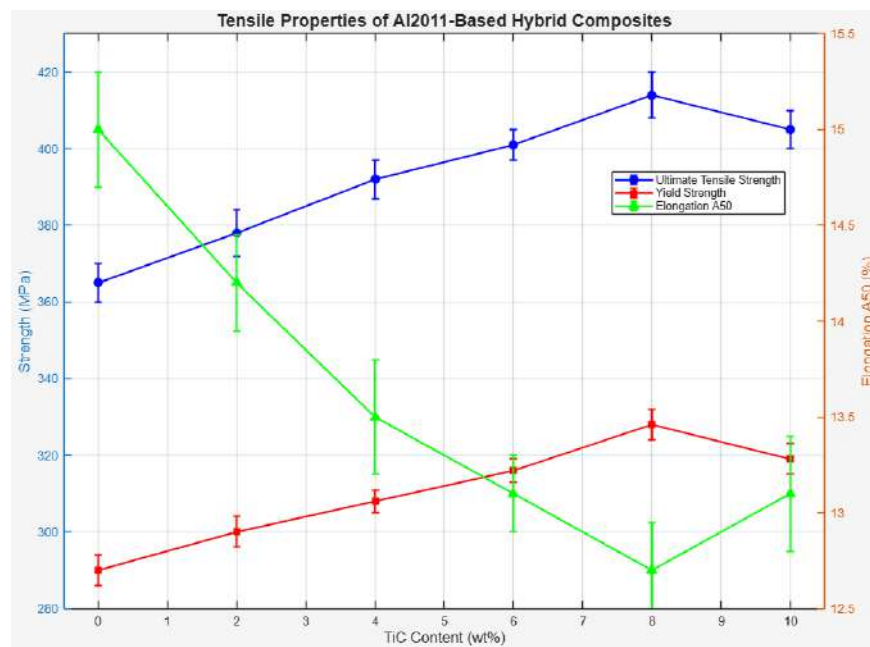


Fig. 12. Tensile properties of Al2011–TiC–MoS₂ hybrid composites as a function of TiC content (0–10 wt% at constant 4 wt% MoS₂): Ultimate Tensile Strength (UTS, MPa), Yield Strength (YS, MPa), and Elongation (A50, %). Values represent mean $\pm$ standard deviation from triplicate tests according to ASTM E8/E8M

4.5.1 Strengthening Mechanisms

The monotonic increase in YS/UTS up to 8 wt% TiC with a slight decrease at 10 wt% is in line with a combined effect of (i) Orowan bypassing by the nano-TiC, (ii) grain-size refinement (Hall -Petch), and (iii) load transfer/thermal-mismatch-induced dislocations. Though TEM/EBSD is not obtained in this work, the quantitative analysis of the SEM (Section 4.1.1) indicates progressive refinements and even distributions up to 68 wt% TiC, which are then related to the corresponding hardness and yield strength improvement. Recent EBSD/TEM research on Al-TiC and Al nanocomposites based on nano-TiC has directly imaged dislocation looping about nano-TiC and grain refinement as the primary strengthening mechanisms, which offer mechanistic evidence to this interpretation. As an example, TiC / Al -Cu composites are characterized by particle-size-dependent strengthening with the identification of grain refinement and Orowan as the most important mechanisms (EBSD/TEM-informed) [19]; wider scans and case studies report comparable behavior with nano-

reinforced Al (dislocation structures by TEM/EBSD, Hall-Petch response) [19]. In the case of MoS₂-containing Al matrices, recent experiments on Al-2024 + MoS₂ report hardness/strength improvements at 4 wt% and comment on interfacial load transfer and damage mitigation by tribo-mechanical lubrication, as in this case [20]. EBSD technique is also established to obtain grain maps, misorientation and geometrically necessary dislocation (GND) density in aluminum alloys [21,22]; these will be employed in follow-up studies to measure relative contributions of Orowan vs. grain-size strengthening in the current Al2011–TiC–MoS₂ system. Continuous stress–strain curves were not recorded during testing; therefore, tensile behavior is represented using yield strength, ultimate tensile strength, and elongation at fracture, which collectively provide an adequate description of the tensile response of the composites.

Fractographic examination of tensile fracture surfaces using SEM was not conducted in the present study. The tensile behavior and ductility trends were therefore interpreted based on quantitative tensile parameters (yield strength, ultimate tensile strength, and elongation at fracture) and their statistically validated trends across compositions. While fractography can provide additional insight into micro-mechanisms of failure, the current conclusions are supported by consistent mechanical data and microstructural observations. Detailed fracture-surface analysis is identified as part of future work. Fractographic examination of tensile fracture surfaces using SEM was not conducted in the present study. The tensile behavior and ductility trends were therefore interpreted based on quantitative tensile parameters (yield strength, ultimate tensile strength, and elongation at fracture) and their statistically validated trends across compositions. While fractography can provide additional insight into micro-mechanisms of failure, the current conclusions are supported by consistent mechanical data and microstructural observations. Detailed fracture-surface analysis is identified as part of future work.

Table 3. One-way ANOVA summary for mechanical properties of Al2011–TiC–MoS₂ composites

Property	F statistic	p-value	Significant ($\alpha=0.05$)	Note on optimum (Hybrid-8)
Hardness (HV)	170.1	1.0×10^{-10}	Yes	Higher than Hybrid-6 & -10
Impact energy (J/cm ²)	271.1	6.6×10^{-12}	Yes	Higher than Hybrid-6 & -10
UTS (MPa)	2684.6	$<1.0 \times 10^{-16}$	Yes	Higher than Hybrid-6 & -10
YS (MPa)	1943.5	$<1.0 \times 10^{-16}$	Yes	Higher than Hybrid-6 & -10
Elongation (%)	27	0.000004	Yes	Lower vs Base; no sig. diff. from Hybrid-6/-10

The optimum performance at 8 wt% TiC + 4 wt% MoS₂ was further supported by statistical analysis. One-way ANOVA across all mechanical properties confirmed significant composition effects (Table 2). Tukey's post-hoc tests consistently identified Hybrid-8 as superior to Hybrid-6 and Hybrid-10 in hardness, strength, and toughness ($p < 0.05$). For elongation, the expected reduction in ductility was significant compared with the base alloy, but not between Hybrid-6, -8, and -10.

4.6 Wear Analysis

This research adopted a pin-on-disc tribometer to measure the wear behavior in terms of important parameters: wear rate, wear loss and coefficient of friction (COF). Test at constant speeds (100, 300, and 700 rpm), as well as tests at constant loads (10, 30, and 50 N), were carried out to represent the various service conditions. Addition of hard TiC particles and lubricating MoS₂ was done to minimize the loss of material and to increase the stability of surface and to decrease friction. The resultant data aids in determining the best reinforcement mix that provides the highest wear resistance and the composites are well suited to harsh usage in automotive, aerospace and structural parts.

At a constant speed, a monotonic improvement of wear performance Al2011-based hybrid composites reinforced with nano TiC and MoS₂ with increasing reinforcement content was observed. Wear rate (fig 13) indicated a significant decrease across all load levels, especially at the highest load (100 rpm - 50 N) where it decreased by $2.177 \times 10^{-3} \text{ mm}^3/\text{N m}$ (0%) to $1.432 \times 10^{-3} \text{ mm}^3/\text{N m}$ (10%), or about 34%. At 300 and 700 rpm, same reductions were noted which supported the effectiveness of reinforcement in losing a minimum amount of material. This trend was reflected in wear loss values (fig 14) which decreased at 100 rpm and 10 N to 0.265 mg, 0.118 mg and at 700 rpm and 50 N to 0.261 mg and 0.109 mg respectively proving increased wear resistance. The coefficient of friction (COF) (fig 15) has also been reduced, though the greatest reduction was at 100 rpm and 10 N where it fell to 0.397 as compared to 0.629. This results in the enhancement due to the hard load-bearing titanium carbide particles, which reduce abrasion on the surface and the lubricating effect of MoS₂ that forms tribo-film that reduces friction between the interface. Small inaccuracies can be explained by momentary effects of agglomeration, e.g. at 300 rpm and 30 N. On the whole, the results confirm the fact that the hybrid reinforcement system significantly enhances wear resistance and frictional stability under the different loading conditions at constant sliding speeds.

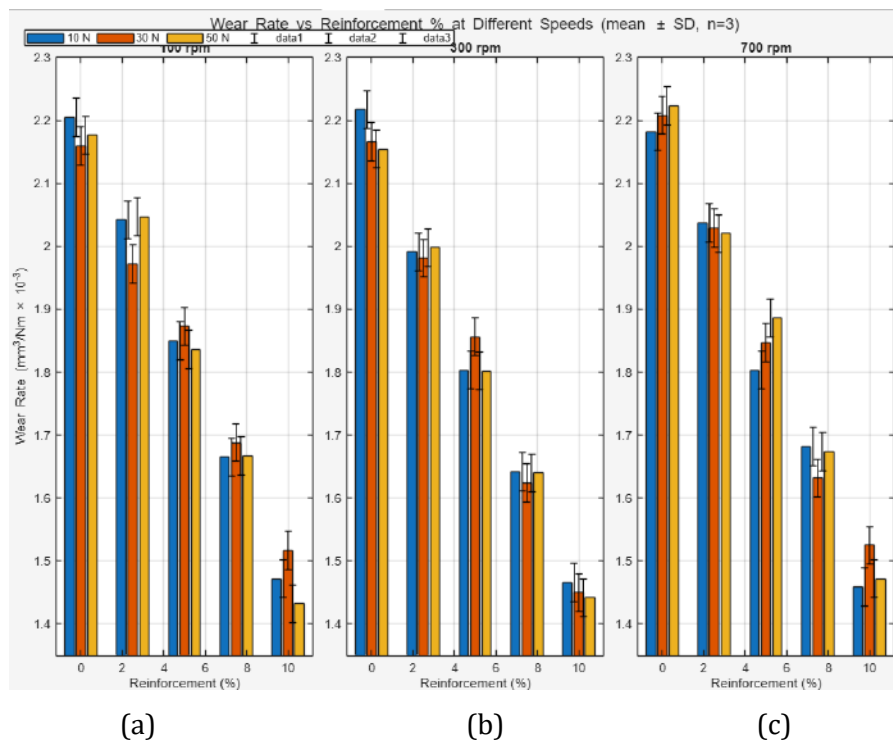


Fig. 13. (a) 100 rpm, (b) 300 rpm, (c) 700 rpm. Wear rate vs reinforcement percentage for loads 10/30/50 N. Mean $\pm$ SD (n = 3); error bars are ± 1 SD

At constant loads, the nano TiC and MoS₂ reinforced Al2011 hybrid composites showed a significant enhancement of tribological performance at all test speeds (100, 300, and 700 rpm). The wear rate decreased steadily with elevated reinforcement, falling by $\sim 33\%$ at 100 rpm as the percentage of reinforcement varied between 0 (unreinforced alloy) and 10% at a fixed 10 N load ($2.205 \times 10^{-3} \text{ mm}^3/\text{N m}$ to $1.472 \times 10^{-3} \text{ mm}^3/\text{N m}$ respectively). A similar trend was observed at 300 and 700 rpm which showed increased wear resistance which is attributed to the combined effect of TiC and MoS₂. This pattern continued at greater loads (30 N and 50 N). E.g. the wear rate reduced at 50 N and 700 rpm to $1.472 \times 10^{-3} \text{ mm}^3/\text{N m}$ compared to 2.223, further confirming the robustness of the reinforcement subjected to harsh conditions. The value of wear loss also trended in a similar manner. Wear loss decreased by 0.265 mg (0) to 0.118 mg (10) at 10 N 100 rpm and 0.261 mg (0) to 0.109 mg (10) at 50 N 700 rpm. These consecutive decreases indicate how efficient reinforcement was in reducing material removal through load redistribution and inhibiting plastic flow and delamination.

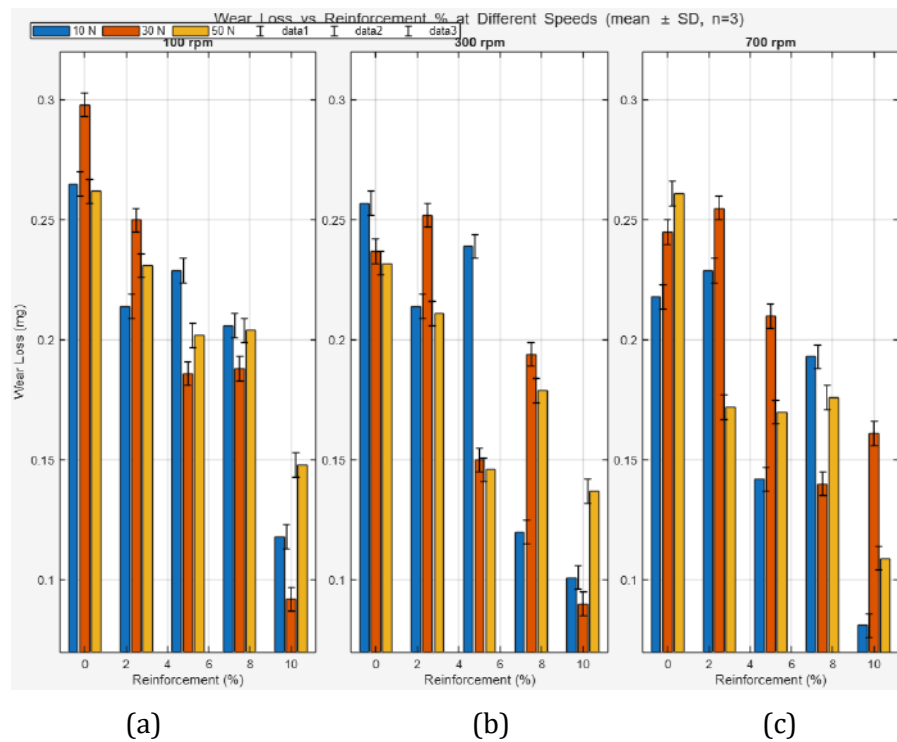


Fig. 14. (a) 100 rpm, (b) 300 rpm, (c) 700 rpm. Wear loss vs reinforcement percentage for loads 10/30/50 N. Mean $\pm$ SD (n = 3); error bars are ± 1 SD

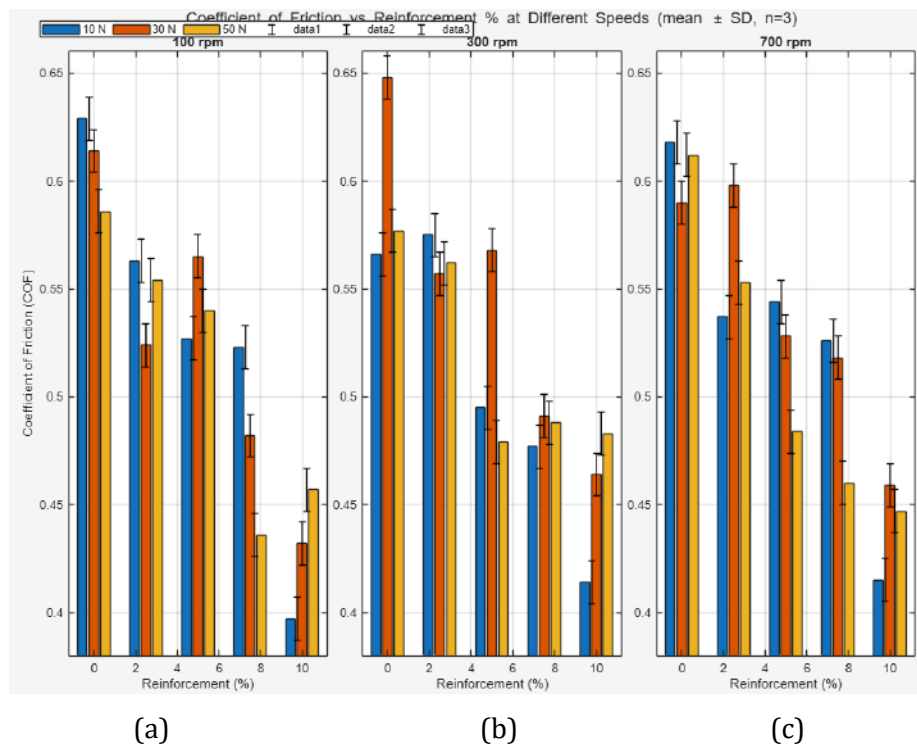


Fig. 15. (a) 100 rpm, (b) 300 rpm, (c) 700 rpm. COF vs reinforcement percentage for loads 10/30/50 N. Mean $\pm$ SD (n = 3); error bars are ± 1 SD

The coefficient of friction (COF) also decreased steadily as the reinforcements were added. Initially at 10 N and 100 rpm the reduction in COF was 0.629 (0%) to 0.397 (10%) and this pattern followed at all other speeds and loads. MoS₂ played a leading role in enhancing this tendency through a lubricating film and TiC improved surface integrity reducing interfacial shear. These findings show evident changes in dominant wear mechanisms. Both the adhesive wear and abrasive wear caused severe damage to the unreinforced alloy, whereas the hybrid composites exhibited a less intense

oxidative or boundary-lubricated wear regime attributed to the higher surface hardness value and a tribo-film generated by MoS₂. A combination of mechanical strengthening and tribological enhancement processes is the reason that limits wear behavior of Al2011-based hybrid composites reinforced with nano TiC and MoS₂. The first mechanism is the load bearing strength of the TiC nanoparticles which because of their high hardness and modulus strength, is prevented against plastic deformation and forms a barrier to ploughing and micro-cutting off the surface.

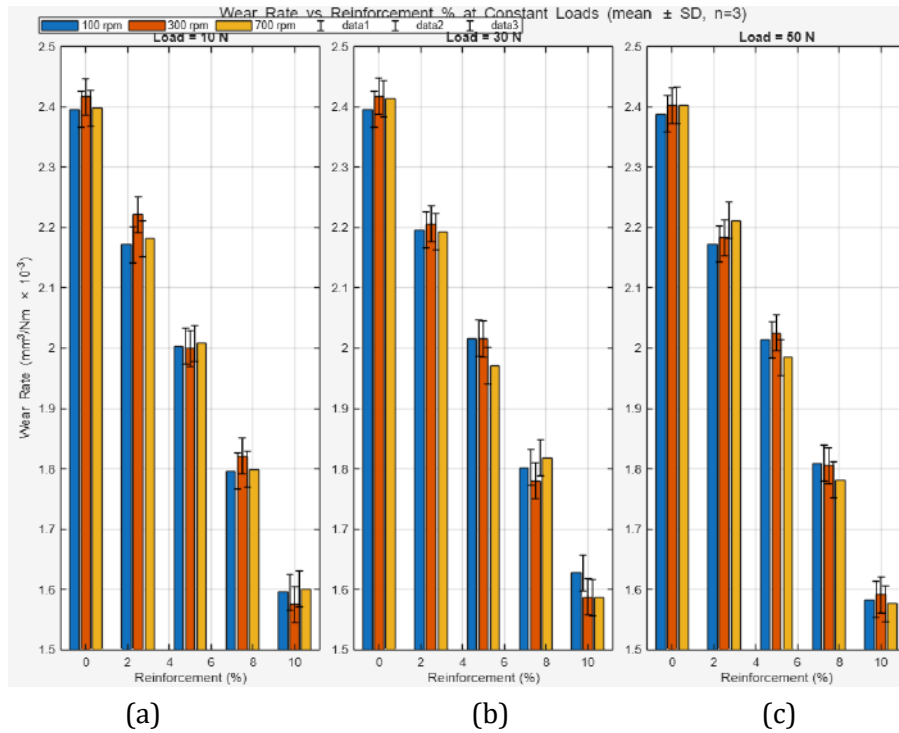


Fig. 16. (a) 10 N, (b) 30 N, (c) 50 N. Wear rate vs reinforcement percentage for speeds 100/300/700 rpm. Mean ± SD (n = 3); error bars are ±1 SD

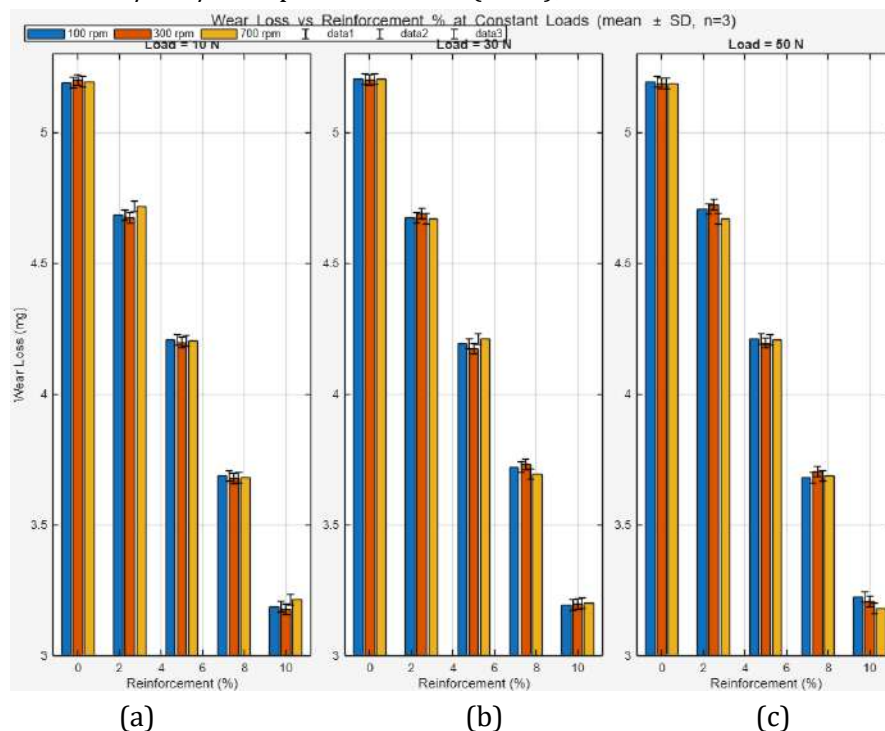


Fig. 17. (a) 10 N, (b) 30 N, (c) 50 N. Wear loss vs reinforcement percentage for speeds 100/300/700 rpm. Mean ± SD (n = 3); error bars are ±1 SD

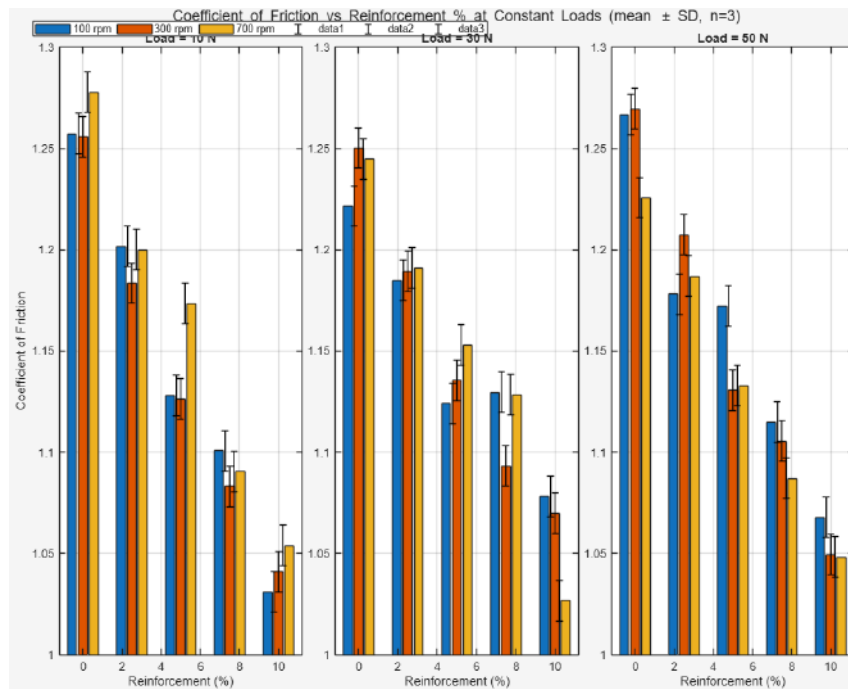


Fig. 18. (a) 10 N, (b) 30 N, (c) 50 N. COF vs reinforcement percentage for speeds 100/300/700 rpm. Mean $\pm$ SD ($n = 3$); error bars are ± 1 SD

This results in a significant decrease of abrasive wear. At the same time, MoS_2 , which also has a layered lamellar structure, is an important solid lubricant, providing a thin tribo-film with low shear strength at the sliding interface, which reduces interface shear stress, frictional heating, and adhesive wear. The synergistic effect of the combined wear contributions alters wear behavior shifting the limit of severe adhesive and abrasive wear common in the un-reinforced Al2011 alloy to a more stable oxidative and mild abrasive regime for the hybrid composites. Additionally, nano TiC also promotes grain refinement and Orowan strengthening via grains and surface hardening by thermal mismatch that led to better wear resistance and delamination inhibition. This improvement in wear rate, wear loss, and the coefficient of friction (COF) at all speeds and loads can therefore be explained by synergistic effect of reinforcement- induced hardening, interfacial film formation by MoS_2 , and enhanced thermal stability. All these effects contribute to the enhanced ability of the composite to resist material removal and interfacial deterioration, particularly in dry sliding and under high-load conditions, which proves their potential to perform highly demanding tribological applications.

Table 4. One-way ANOVA results for tribological properties of Al2011-TiC- MoS_2 composites across all test conditions.

Property	F statistic	p-value	Significant ($\alpha=0.05$)
Wear rate	204.49	9.57e-43	Yes
Wear loss	0.73	5.75e-01	No
COF	0.85	4.97e-01	No

It is vital to mention that the current wear tests were only carried out in dry sliding situations, which replicate extreme service conditions. Although it gives a consistent worst-case estimate of the performance of materials, most engineering applications like automotive engines, aircraft joints, and bearing systems are lubricated contacts or at a high temperature. Previous studies on Al-based nanocomposites have demonstrated that lubrication and temperature can have a significant effect on the shift between abrasive/adhesive wear and oxidative or delamination wear [23,24]. Therefore, current findings must be considered as minimum performance in severe conditions. The analysis will be extended to lubricated sliding and high-temperature environments in the future to develop a more comprehensive range of application of Al2011- TiC- MoS_2 alloys.

One-way ANOVA (Table 3) confirms a statistically significant composition effect for wear rate ($p < 0.001$). For wear loss and COF, the mean values decrease consistently with reinforcement, but the between-composition differences are not statistically significant at $\alpha = 0.05$ ($p > 0.05$). Therefore, these two responses are discussed in terms of practical magnitude of reduction rather than statistical significance. In addition to statistical significance, the magnitude of improvement is substantial. Compared with as-cast Al2011, the Hybrid-8 composite exhibited a $\sim 13.4\%$ increase in ultimate tensile strength, $\sim 20\%$ increase in hardness, and $\sim 34\%$ reduction in wear rate. These large effect sizes demonstrate the practical relevance of the optimized hybrid composition, even where statistical significance is limited by sample size. In addition to statistical significance, the magnitude of improvement is substantial. Compared with as-cast Al2011, the Hybrid-8 composite exhibited a $\sim 13.4\%$ increase in ultimate tensile strength, $\sim 20\%$ increase in hardness, and $\sim 34\%$ reduction in wear rate. These large effect sizes demonstrate the practical relevance of the optimized hybrid composition, even where statistical significance is limited by sample size. A schematic COF-time trace is included in Supplementary Fig. S1 to qualitatively illustrate run-in and steady-state behavior; continuous experimental COF time histories were not stored.

4.7 Microstructural Analysis of Worn Surfaces

SEM micrographs reveal solid demonstration of the wear behavior of the Al2011-based composites directly related to the composition of the reinforcement. The wear surface of the unreinforced Al2011 alloy is strongly degraded with deep grooves, ploughing marks, plastic deformation, and a large area of delamination.

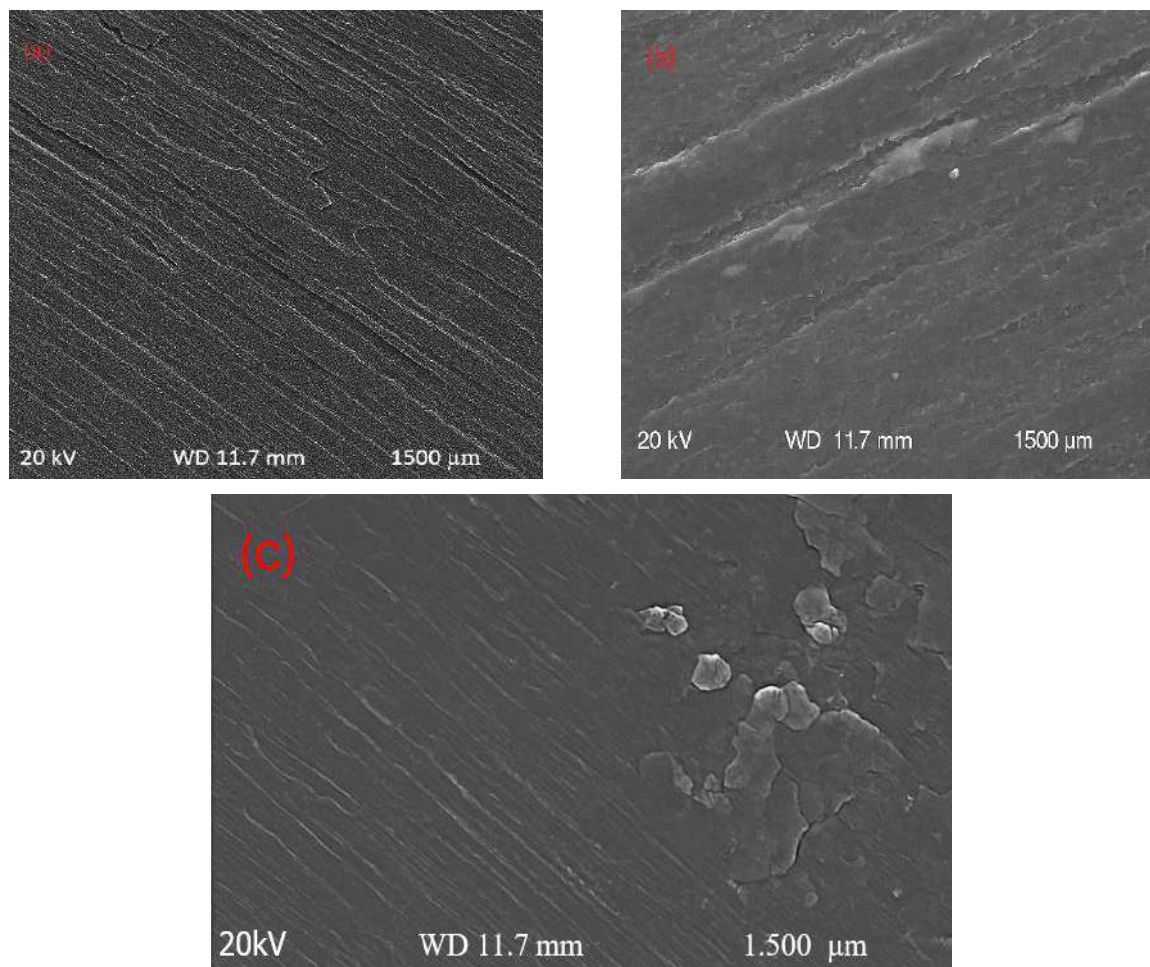


Fig. 19. SEM micrographs of worn surfaces a) Unreinforced Al2011 (0% TiC + 0% MoS₂) b) Hybrid-8 (8% TiC + 4% MoS₂) and c) Hybrid-10 (10% TiC + 4% MoS₂)

These characteristics indeed show the prevalence of adhesive and abrasive wear processes, which is characteristic of soft aluminum alloys with no protective and strengthening phases. By contrast, the Hybrid-8 composite (8 wt% TiC + 4 wt% MoS₂) shows a significantly smoother wear surface,

little deformation and evidence of a thin tribo film - hallmarks of a more stable oxidative and mild abrasive character. The TiC nanoparticles promote the matrix resistance to plastic flow and penetration, and MoS₂ adds a low shear strength qualifying lubricating layer, which lowers frictional heat and adhesive interaction. The combination of the two leads to decreased wear rate, lower friction coefficient and improved surface integrity. The Hybrid-10 composite (10 wt% TiC + 4 wt% MoS₂) however has a rougher morphology than Hybrid-8, showing evidence of reinforcement agglomeration, micro-cracking and pull-out of the particles. These characteristics are indicative of stress concentrations findings due to agglomeration and porosity, which eventually result in a decreased tribological performance. The comparison highlights the cruciality of the process of optimizing the reinforcement content: where Hybrid-8 manages to find the balance between hardening and lubrication, Hybrid-10 exceeds the limits where the process of reinforcement negatively affects the level of wear resistance as the highly processed content of reinforcement leads to flaws in the process and the occurrence of inhomogeneous areas. Complete ANOVA and Tukey HSD statistical summaries are provided in the Supplementary Information (Tables S1–S4).

5. Conclusion

The hybrid MMCs based on Al2011 reinforced with varying concentrations of nano Titanium Carbide (TiC) and constant concentrations of 4 wt% nano Molybdenum Disulfide (MoS₂) used in the present study were characterized and prepared by the stir casting technique.

- Addition of TiC resulted in tensile strength, hardness and impact toughness that were enhanced up to an optimum content value of 8 wt%, and with increasing contents, marginal degradation was experienced as a result of agglomeration and porosity.
- Even though MoS₂ did not produce a direct strength enhancement, friction reduction and enhanced interfacial bonding (which have a positive effect on tribological behaviour) were also observed.
- Wear testing indicated that the wear rate, wear loss and coefficient of friction reduced considerably with increasing reinforcement particularly under constant speed and load conditions.
- Hybrid-8 composition (8% TiC + 4% MoS₂) turned out to be the most optimal one, as it offers the best strength, ductility, and wear resistance.
- SEM of worn surfaces gave evidence to smoother surfaces and formation of tribo-films in Hybrid-8 but showed clusters and micro-cracks in Hybrid-10.
- The superiority of the Hybrid-8 composition was statistically validated using ANOVA, which confirmed significant improvements in hardness, strength, toughness, and tribological properties compared with neighboring compositions.

5.1 Engineering Implications

The Al2011-TiC- MoS₂ hybrid composites optimized exhibit great potential in automotive engine components, aerospace fittings, bearings, and tooling applications, which demand high strength and low wear. The stir casting route is also scalable and this adds even more relevance to industry.

5.2 Limitations and Future Research

The current research was restricted to SEM-based characterization and dry sliding wear experiments. EBSD /TEM analyses should also be conducted in future to measure the grain size distribution and dislocation density to have a better interpretation of Hall Petch and Orowan strengthening. Moreover, high-temperature and lubricated wear tests will also be required to approximate real-life use and verify its usability in commerce.

References

- [1] Surappa MK. Aluminium matrix composites: Challenges and opportunities. *Sādhanā*. 2003;28(1-2):319-334. <https://doi.org/10.1007/BF02717141>
- [2] Li J, et al. The tribological properties of nano-lubricants and their application on bearings: Recent research progress. *Int J Adv Manuf Technol*. 2024;134:3051-3082. <https://doi.org/10.1007/s00170-024-14351-8>
- [3] Bhane AB, Ramani HB. Mechanical and tribological properties of nano composites under dry sliding conditions: A study and investigation. *Int J Mech Eng*. 2022;7(10):137-144.
- [4] Sethi AK, et al. Fabrication and evaluation of aluminum hybrid metal matrix composite via powder metallurgy approach. *J Polymer and Composites*. 2024;12(2):205-211.
- [5] Teja VS, Sastry KSBSVS, Srinivas BNV, Krishnaji KV. Characterization of Cu-SiC-Gr hybrid metal matrix composite. *Int J Adv Mech Therm Eng*. 2019.
- [6] Singhal V, et al. Study of the influence of nanoparticle reinforcement on the mechanical and tribological performance of aluminum matrix composites-A review. *Lubricants*. 2025;13(2):93. <https://doi.org/10.3390/lubricants13020093>
- [7] Narayanasamy P, et al. Effect of hybridizing MoS₂ on the tribological behaviour of Mg-TiC composites. *Trans Indian Inst Met*. 2015. <https://doi.org/10.1007/s12666-015-0530-z>
- [8] Mao H, et al. Microstructure, mechanical properties, and reinforcement mechanism of second-phase reinforced TiC-based composites: A review. *Coatings*. 2022. <https://doi.org/10.3390/coatings12060801>
- [9] Malek A. Review of stir casting technique and technical challenges for ceramic reinforcement particulate and aluminium matrix composites. *Építőanyag - J Silicate Based and Composite Materials*. 2020;72(6):190-197. <https://doi.org/10.14382/epitoanyag-jsbcm.2020.32>
- [10] Bhanodaya KBN. Comprehensive analysis of stir casting process of aluminum-based composites with silicon carbide reinforcements. *NanoWorld J*. 2023;9(Suppl 4):S257-S261. <https://doi.org/10.17756/nwj.2023-s4-044>
- [11] Dhulipalla A, et al. Synthesis and machining characteristics of novel TiC ceramic and MoS₂ soft particulate reinforced aluminium alloy 7075 matrix composites. *Manufacturing Lett*. 2020;24:82-86. <https://doi.org/10.1016/j.mfglet.2020.04.001>
- [12] Umanath K, et al. Optimization of wear properties on AA7075/SiC/MoS₂ hybrid composites. *Mater Today: Proc*. 2021.
- [13] Gopalakrishnan R, et al. Influence of MoS₂ with TiC on the tribological and wear properties of hybrid aluminum composites (AA7075). In: *Advances in Materials and Manufacturing*. 2023. https://doi.org/10.1007/978-981-19-8497-6_27
- [14] Suchendra KR, Sreenivasa RM. Microstructure, mechanical and fracture behavior of Al₂O₃-MoS₂ reinforced Al6061 hybrid composite. *Frattura ed Integrità Strutturale*. 2022. <https://doi.org/10.3221/IGF-ESIS.61.16>
- [15] Sidharthan S, et al. Mechanical property analysis and tribological response optimization of SiC and MoS₂ reinforced hybrid aluminum functionally graded composite through Taguchi's DOE. *J Manuf Process*. 2023;102:965-984. <https://doi.org/10.1016/j.jmapro.2023.08.013>
- [16] Sharma SK, et al. Progress in aluminum-based composites prepared by stir casting: Mechanical and tribological properties for automotive, aerospace, and military applications. *Lubricants*. 2024;12:421. <https://doi.org/10.3390/lubricants12120421>
- [17] Sharath BN, et al. Enhancing tribological performance: A review of ceramic-reinforced aluminum hybrid composites. *Tribology Int Reports*. 2023. <https://doi.org/10.1016/j.hybadv.2023.100094>
- [18] Hemadri K, et al. Investigation on mechanical characterization of Al/MoS₂/WC hybrid composite. *Mater Today: Proc*. 2022. <https://doi.org/10.1016/j.matpr.2022.08.014>
- [19] Zhao D, et al. Influence of TiC particle size on microstructural evolution and fracture mechanism of TiC/Al-Cu composites. *J Alloys Compd*. 2024;1008:176763. <https://doi.org/10.1016/j.jallcom.2024.176763>
- [20] Carneiro Í, Simões S. Recent advances in EBSD characterization of metals. *Metals*. 2020;10:1097. <https://doi.org/10.3390/met10081097>
- [21] Zheng JH, et al. Quantifying geometrically necessary dislocation density during hot deformation in AA6082 Al alloy. *Mater Sci Eng A*. 2021;814. <https://doi.org/10.1016/j.msea.2021.141158>

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Influence of IRC vehicular loads on seismic response of composite I-Girder bridge

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Abstract

Heavy vehicles are often present on bridges during earthquakes, potentially affecting the seismic response of the structure. This study investigates the seismic behavior of a single span, two lane steel-concrete composite I-girder bridge subjected to vehicular loads specified by the Indian Roads Congress (IRC) in combination with seismic excitations. A finite-element model is developed using MIDAS Civil and validated in accordance with IRC provisions. Response spectrum analyses are conducted for Indian seismic Zones IV and V, along with linear time history analyses using recorded ground motions from the Bhuj (2001), Chamoli (1999), and India–Burma Border (1995) earthquakes. Stationary vehicular loads, including IRC Class A, Class AA, Class 70R, and Special Vehicle configurations, are applied at mid-span to obtain critical structural responses. The results indicate that stationary vehicles increase shear force, bending moment, and mid-span deflection, with heavier axle loads and closer axle spacing producing greater amplification. Seismic demands in Zone V are approximately 50% higher than those in Zone IV. Time history analyses show that the near-field Chamoli record generates responses up to three times greater than those from the Bhuj and India–Burma Border events. Overall, the findings demonstrate a non-linear relationship between axle load and seismic response, highlighting that both load magnitude and axle configuration significantly influence the dynamic demand.

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1. Introduction

Bridges are crucial to transportation infrastructure, and it is very necessary that they are reliable to handle multiple stress, especially the combined effects of heavy vehicular loading and seismic events. Traditionally, design standards have assessed the impacts of live loads and earthquake forces separately, often neglecting their simultaneous occurrence because of assumptions about low joint probability and the dominance of dead loads in seismic response [1-3]. However, past earthquake events have shown that automobiles can be present on bridges during seismic activity, which can significantly alter dynamic performance of a bridge and post-event serviceability [4,5].

Recent studies show that truck weight, axle configuration, and gross vehicle load directly affect bridge deterioration, fatigue, and the maintenance and renewal costs [6]. This highlights the importance of understanding how vehicular load impacts both existing and new bridge infrastructure. Furthermore, composite I-girder bridges are becoming more common because of their structural advantages, and they face changing requirements as heavier vehicles and complex traffic patterns become usual [7]. A specific challenge involves understanding the dynamics of interaction: advanced experimental, analytical, and numerical investigations indicate that vehicle-bridge interaction (VBI) can either enhance or reduce seismic response depending on factors such as mass distribution, vehicle modeling, and load position [2-4].

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Experimental as well as finite element analyses show that the dynamic correlation between vehicle movement and bridge superstructures, including inertia and damping effects, can significantly affect the seismic fragility and reliability of steel and composite bridges [1,2,8]. This effect is strongest when a stationary vehicle is positioned at the critical mid-span, where bending moment and deflection usually reach their maximum value [1, 3-5]. The presence of this concentrated load changes the distribution of static forces in the bridge and can also influence its dynamic and seismic behavior [3-5]. Notably, live load may act as a mass damper, reducing substructure forces under certain conditions, although excessive seismic excitation could reverse this advantage [3].

The response of bridge structures is influenced by factors such as geological structures and fault characteristics, which vary significantly across regions [9]. Near-fault ground motions lead to stronger amplification than far-field motions [8,10]. Wave passage effects and soil structure interaction further raise seismic demand, with the largest impact on soft soils and sites with low apparent wave velocities [11]. This highlights the necessity to examine the response of a structure on different seismic events or conditions.

The Indian Roads Congress (IRC) defines vehicle types and load levels based on Indian traffic conditions. This approach leads to live load models and combinations that differ from the more generalized design vehicles and load models adopted in AASHTO and Eurocode [12-14]. While AASHTO and Eurocode adopt site-specific spectra that more thoroughly account for soil-structure interaction and local amplification effects [15,16], Indian codes rely on zone-based response spectra with comparatively lower design acceleration values [12]. These variations underscore the importance of evaluating bridge and structural performance under Indian loading and seismic conditions.

Despite advances, established guidance remains inconsistent, making it difficult to incorporate live loads in seismic design [3]. Considering changing traffic demands and variations among seismic design codes regarding live loads, it is crucial to understand how a stationary vehicular load at mid-span affects shear force, bending moment, and deflection during earthquakes [1,3]. This article addresses this need by investigating these critical structural responses in single span composite I-girder bridges incorporating stationary IRC vehicular loads at mid-span and thereby providing insights to advance analysis, design, and reliability assessments for seismically active regions.

2. Objectives of the Study

This study evaluates how Indian Roads Congress (IRC) vehicle loads affect the seismic performance of a 25 m single span, two lane steel-concrete composite I-girder bridge. Using finite element modeling and IRC guidelines, response spectrum analyses for zones IV and V and linear time history analyses with three Indian ground motions (Bhuj 2001, Chamoli 1999, India–Burma Border 1995) are conducted, including effects from IRC Class A, Class AA (tracked), Class 70R (wheeled), and special vehicles. The primary objectives of present study are;

- To compute and compare key response quantities for each case: shear force, bending moment and mid-span deflection.
- To identify and interpret patterns how the presence of heavy vehicles influences the seismic demands on the composite span.

3. Methodology

3.1. Modelling and Validation

A finite element model of a composite I-girder bridge is developed as per IRC guidelines and checked for the validation of the serviceability conditions. The material properties, section properties, boundary and loading conditions used for modeling and validation are discussed as follows.

3.1.1. Material Properties

For the current model, structural steel of grade E350 is selected for main girders and other steel components such as cross bracing. For concrete components such as deck slab and crash barrier,

reinforced concrete of grade M40 is defined with reinforced steel Fe500. The properties of composite section are illustrated in Table 1.

Table 1 Material properties

Property	Structural Steel	Reinforced Concrete
Grade	E350	M40
Young's modulus (E)	2.0×10^5 MPa	33,345 MPa
Poisson's ratio (ν)	0.3	0.2
Density	76.98 kN/m ³	25.49 kN/m ³
Characteristic strength	350 MPa (Yield Strength, F_y)	40 MPa (Compressive Strength, f_{ck})
Thermal expansion coefficient	1.2×10^{-5} /°C	1.2×10^{-5} /°C

3.1.2. Time Dependent Material Properties

Time dependent material properties make it possible to examine long-term effects, particularly in concrete, such as creep, shrinkage and aging of stiffness. These phenomena are considered for modeling to ensure accurate construction-stage and long-term service life analyses. The creep, shrinkage and compressive strength for the present model is defined as per IRC 112, 2011 considering relative humidity 70% and normal cement.

3.1.3. Section Properties

Section properties define the geometric characteristics of the structural elements. These properties are basics for the accurate analysis and design of structural members as they directly influence parameters like stiffness, load-carrying capacity, deflection, and stress distribution. The geometric configuration and sectional details of the composite I-girder bridge, including the superstructure layout, composite section properties, and deck slab dimensions, are illustrated in Figures 1–4. and summarized in Table 2.

Table 2 Section properties

Parameters	Properties
Bridge deck	Reinforced concrete slab
Girders	Steel-plate girder
Span	25 m
Clear width of road way	7.5 m
Footpath	1.25 m on either side
Spacing of main girders	2 m
Cross bracing	4 @ 5 m

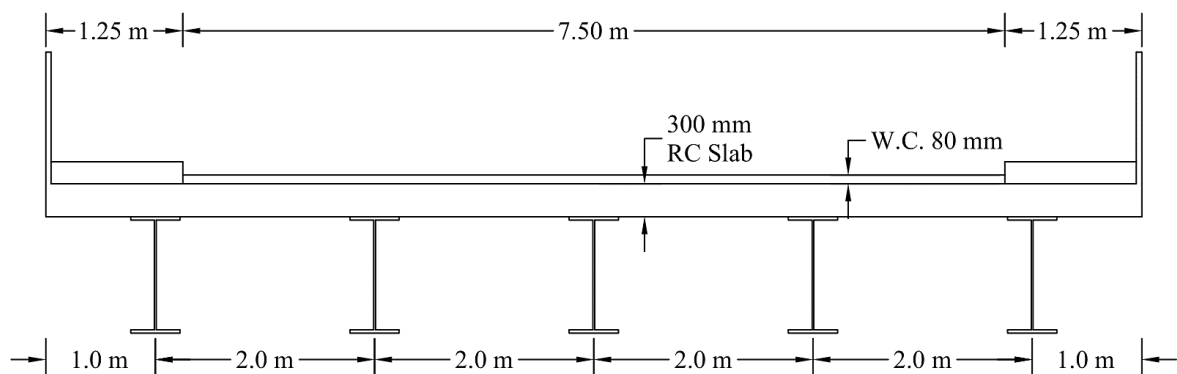
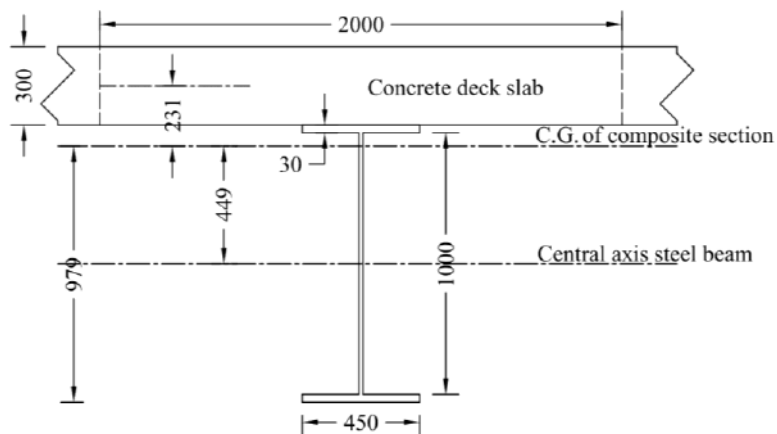


Fig. 1. Cross-sectional arrangement of the composite I-girder bridge superstructure showing deck slab, girders, and roadway configuration



Note: All dimensions are in mm

Fig. 2. Geometric and sectional properties of the steel-concrete composite I-girder section

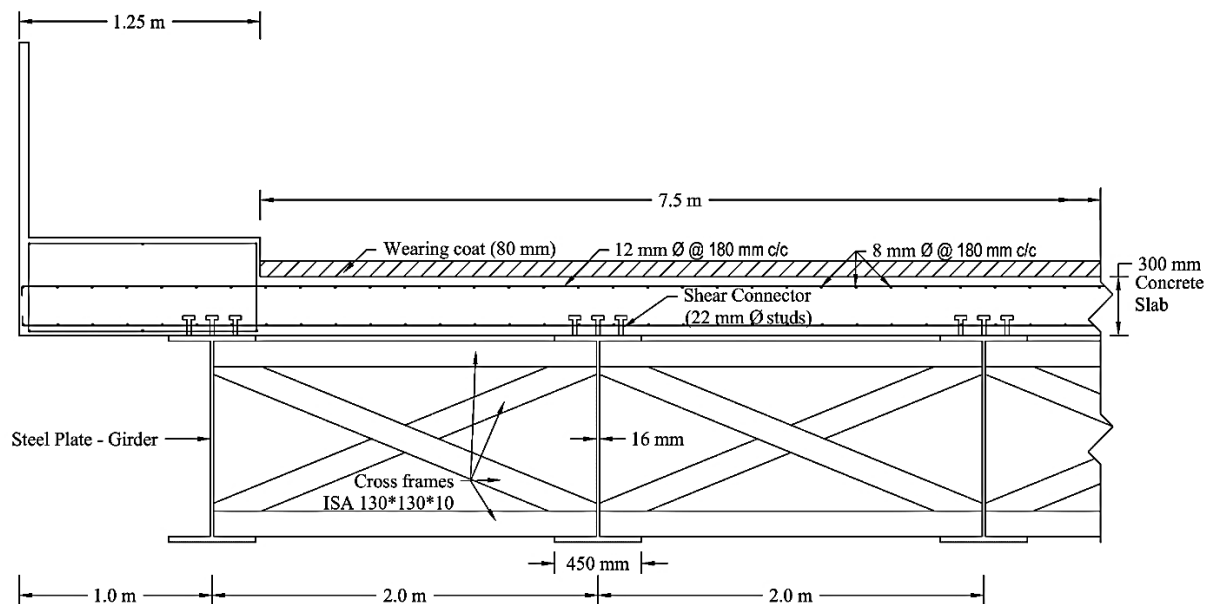


Fig.3 Cross section of the composite bridge superstructure

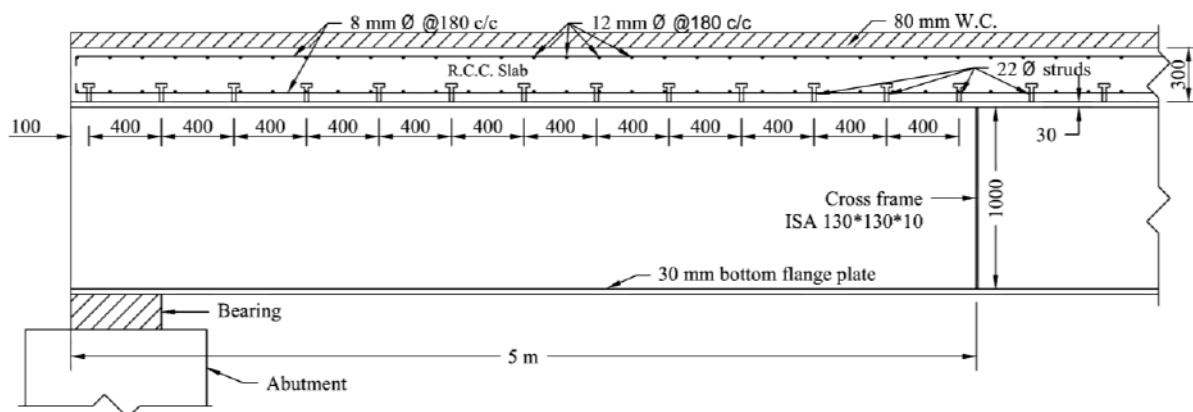


Fig. 4. Longitudinal section of the composite bridge superstructure

3.1.4 Finite Element Modeling

As per the material and section properties defined, the finite element model of the composite I-girder bridge is developed in MIDAS Civil using a three-dimensional grillage approach as portrayed

in Figure 5. Primary structural components, such as steel plate girders, cross bracings, and composite deck girder system, are modeled using frame (beam) elements as this modeling choice is well suited for capturing global flexure, shear, and torsion of girder bridges. The superstructure uses an average element length of about 1.0 m along the span. A mesh sensitivity study is conducted to refine the element size monitoring the key response quantities such as mid-span deflection, maximum bending moment, and shear force. The results showed negligible variation (less than 5%) between successive refinements and the final mesh balances numerical accuracy and computational efficiency for the analysis. The model is subjected to uniform 2% modal damping ratio enhanced by a multiplying factor of 1.6 in accordance with IRC:6-2017.

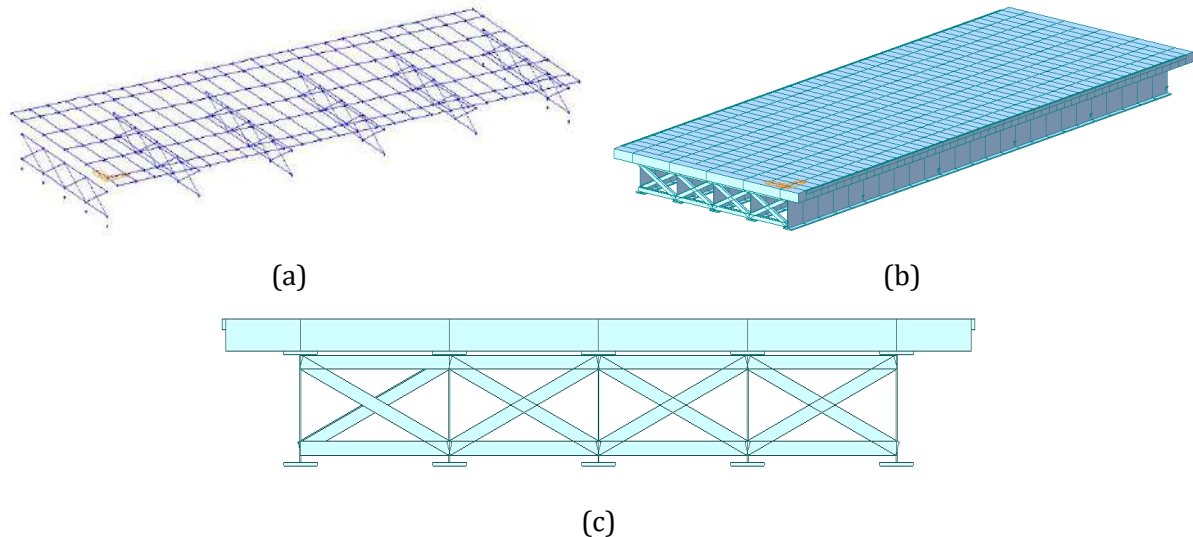


Fig. 5 Finite element model of the developed composite I-girder bridge superstructure (a) Grillage model, (b) 3D model and (c) Cross sectional view

3.1.5 Boundary Conditions

Firstly, to account for the rigid connection between the top slab and the girders, the nodes are connected using rigid links, which appropriately simulate relative deformation. Furthermore, the connection from the bottom of the girder to the bottom of the bearing (i.e., the top of the pier cap) is modeled using elastic links to represent the bearings themselves, with appropriate stiffness values assigned. Finally, since the substructure is not being modeled, the bottom of the bearings is assumed to be fixed. Therefore, fixed supports are assigned at these locations as shown in Figure 6, from which the reaction components can be extracted.

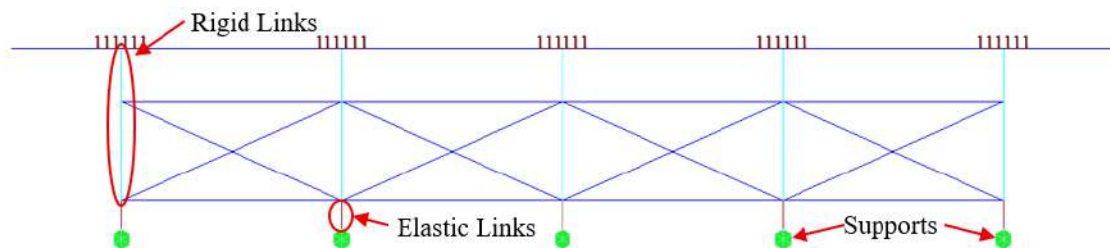


Fig. 6. Boundary condition idealization showing supports with rigid and elastic link

3.1.6 Dead loads

- **Self-weight:** Self-weight is applied to the analysis model using the self-weight command and with an option to adjust its intensity through a self-weight multiplier. The software calculates self-weight automatically from the defined section properties and material density in the model.

Self-weight factor = -1

- Super imposed dead load: In this model wearing course load and crash barrier loads are defined as the super imposed dead load. Wearing course load is computed as 3.52 kN/m/girder assuming the thickness of wearing course as 80 mm and unit weight of asphalt as 22 kN/m³.

Crash Barrier load is assumed as 10 kN/m.

3.1.7 Formulation of Construction Stage

A series of staged analyses during finite element modeling captures the incremental changes in structural behavior during the construction stage of a steel-composite I-girder bridge. This process is divided into three distinct steps, each reflecting the activation or deactivation of structural elements, boundary conditions, and loads to closely simulate field operations.

- Stage 1: Activation of steel I-girders and bracings with permanent supports. Once these primary members and restraints are in place, the self-weight of the steel elements is applied as a dead load.
- Stage 2: Applying wet concrete load as a construction load for 14 days to simulate deck slab casting.
- Stage 3: The temporary green concrete load is deactivated and the deck slab elements are activated. Additionally, the final service stage loads such as, crash barrier load and wearing course load are then applied and the final stresses are checked for the long-term case.

3.1.8 Applied Loadings

The loads for the validation of the developed model are computed and applied as per the IRC 6, 2017, discussed as follows.

- Temperature Loads: As per clause 215 of IRC 6, 2017 temperature loads in the model are defined as the uniform temperature load and temperature gradient. The overall temperature change of the bridge over a specific period, will have a minimum and a maximum temperature range. Assuming the region of the bridge site as New Delhi the maximum and minimum temperature load is computed as 23.7°C ± 17.05°C as per clause 215.2 of IRC 6, 2017. And, the temperature gradient is the difference in temperature between the top surface of the bridge and the levels below it through the depth of the superstructure [12]. As per clause 215.3 the positive and reverse temperature gradient is computed as in Table 3 and 4.

Table 3. Calculation of positive temperature gradient

Segment	H1 (m)	H2 (m)	T1 (°C)	T2 (°C)
1	0	0.18	20.5	4
2	0.18	0.3	4	3.077
3	0.3	0.33	3.077	2.846
4	0.33	0.7	2.846	0.000

Note: Taking reference as top of the bridge.

Table 4. Calculation of reverse temperature gradient

Segment	H1 (m)	H2 (m)	T1 (°C)	T2 (°C)
1	0	0.18	-6.800	0.000
2	0.3	0.33	0.000	-0.600
3	0.33	0.7	-0.600	-8.000
4	0.7	1.36	-8.000	-8.000

Note: Taking reference as top of the bridge.

- Wind Load: Considering the terrain type as obstructive and assuming the height of pier including pier cap 7 m, the wind load for the developed model located at New-Delhi is computed and following results in Table 5 are obtained as per Section 209 IRC 6, 2017.

Table 5. Wind load on bridge structure

Wind Load Component	Intensity (kN/m)
Transverse force on SS (F_T)	4.204
Longitudinal force on SS (F_L)	1.051
Uplift force per girder (F_U)	1.159
Transverse force on Vehicle (F_{TL})	1.774
Longitudinal force on Vehicle (F_{LL})	0.443

- Static seismic Load: For Indian seismic zone IV (i.e. New Delhi) considering important seismic class and medium type of soil, static seismic load for the steel-composite bridge super-structure is analyzed as per section 218 IRC 6, 2017. The computed seismic demand due to live load and dead load are presented in Table 6 and 7 respectively.

Table 6. Seismic demand due to live load

Seismic Load Components	Intensity (kN/m/girder)
In longitudinal direction	0 (Cl. 218.5.2 IRC 6, 2017)
In transverse direction	0.893
In vertical direction	0.596

Table 7 Seismic demand due to dead loads

Seismic Load Components	Intensity (as factors of dead load)
In longitudinal direction	0.504
In transverse direction	0.504
In vertical direction	-0.336

- Vehicular Load: For a two-lane carriageway, two load cases must be evaluated: one scenario with a single lane loaded by a 70R vehicle, and another with both lanes subjected to Class A vehicle loading [12]. However, in this study class AA and special vehicle are also considered. Lanes are designated in the model for the live load cases, and the eccentricities for these lanes are calculated according to Table 6A of IRC 6, 2017, illustrated as follows in Table 8.

Table 8. Definition of lanes for moving loads

Vehicle Class	Lane	Wheel spacing (m)	Eccentricity from left edge of the bridge (m)
Class A	1	1.8	2.55
	2	1.8	6.05
Class 70R	1	2.06	3.9
Class AA	1	2.05	3.9
Special Vehicle	1	1.8	5.3

- Braking Load: The braking effect acting on a simply supported span is assumed as per the clause 211.2 IRC 6, 2017. The critical case in present scenario for braking load is the class 70R wheeled vehicle with vertical load 1000 kN among Class A, Class AA and Class 70R vehicles. Thus, braking load due to 70R vehicle is 20% of vertical load i.e. 20 kN (i.e. 1.6 kN/m/girder). And, the load has an eccentricity of 1.2 m acting along a line parallel to the roadway, as specified in clause 211.3 IRC 6, 2017.

3.1.9 Validation of the Bridge Model

To ensure the structure and its elements perform adequately throughout their service life, the serviceability limit states (i.e. stress level, deflection and crack width) shall have to be satisfied [17-19].

- Stress Limitation

a. In the concrete of the slab (Cl. 12.2.1, IRC 112, 2011)

$$\begin{aligned} \text{The maximum stress } (\sigma_c) &= -7.72 \text{ MPa} \\ &\leq 0.48f_{ck}. \end{aligned} \quad \text{Hence, okay.}$$

b. In the reinforcement steel (Cl. 12.2.2, IRC 112, 2011)

$$\begin{aligned} \text{The maximum stress } (\sigma_c) &= -38.81 \text{ MPa} \\ &\leq 0.8f_y. \end{aligned} \quad \text{Hence, okay.}$$

c. In the structural steel (C.4.3.1, IRC SP 120, 2018)

$$\begin{aligned} \text{Nominal stress due to bending moment, } (f_b) &= 201.19 \text{ MPa} \\ &\leq 0.87 f_y / \gamma_{m,ser} \end{aligned}$$

$$\begin{aligned} \text{Nominal stress due to shear, } (\tau_b) &= 0.01 \text{ MPa} \\ &\leq 0.87 f_y / (\sqrt{3} \times \gamma_{m,ser}) \end{aligned}$$

$$\begin{aligned} \therefore \text{Combined stress, } \sqrt{f_b^2 + 3\tau_b^2} &= 201.19 \text{ MPa} \\ &\leq 0.87 f_y / \gamma_{m,ser} \end{aligned} \quad \text{Hence, okay.}$$

- Deflection Control (Cl. 604.3.2 IRC 22, 2015)

a. Deflection of composite girder under dead, super-imposed & live load impact,

$$d_{dl, ll} = -38.730 \text{ mm} \leq (L/600) \quad \text{Hence, okay.}$$

where, L = Span of the girder

- Crack Width (Cl. 12.3.4, IRC 112, 2011)

a. Crack width at top

Stress at top surface, $\sigma_{top} = -7.467 \text{ MPa}$. For compressive stresses, the crack check is skipped.

b. Crack Width at bottom

Stress at top surface, $\sigma_{bottom} = -0.887 \text{ MPa}$. For compressive stresses, the crack check is skipped.

3.2 Dynamic Analysis

Dynamic analysis of the composite I-girder bridge superstructure focusing on its response under vehicular loading and earthquake-induced ground motions is performed. The analysis begins with the identification of the critical vehicle position, converted to equivalent static loads, and modeled as lumped masses. Further, Eigenvalue analysis is performed to find the natural frequencies and mode shapes of the structure. Finally, Response Spectrum Analysis is performed for Indian seismic zones IV and V, and Time History Analysis using three recorded ground motions to assess seismic response.

3.2.1 Identification of Critical Vehicle Position

In bridge design and analysis, the position of moving vehicular loads on the span governs the magnitude of bending moment and deflection at any section. To determine the critical vehicle position, moving load analysis is done that produces the absolute maximum bending moment and mid-span deflection. The critical vehicular position for all the vehicles used for the analysis are portrayed in Figure 7. Once the critical position is established, the full moving-load model is replaced with an equivalent static load system that reproduces that worst-case effect. This equivalent static vehicular load typically consists of concentrated loads placed at the critical spacings.

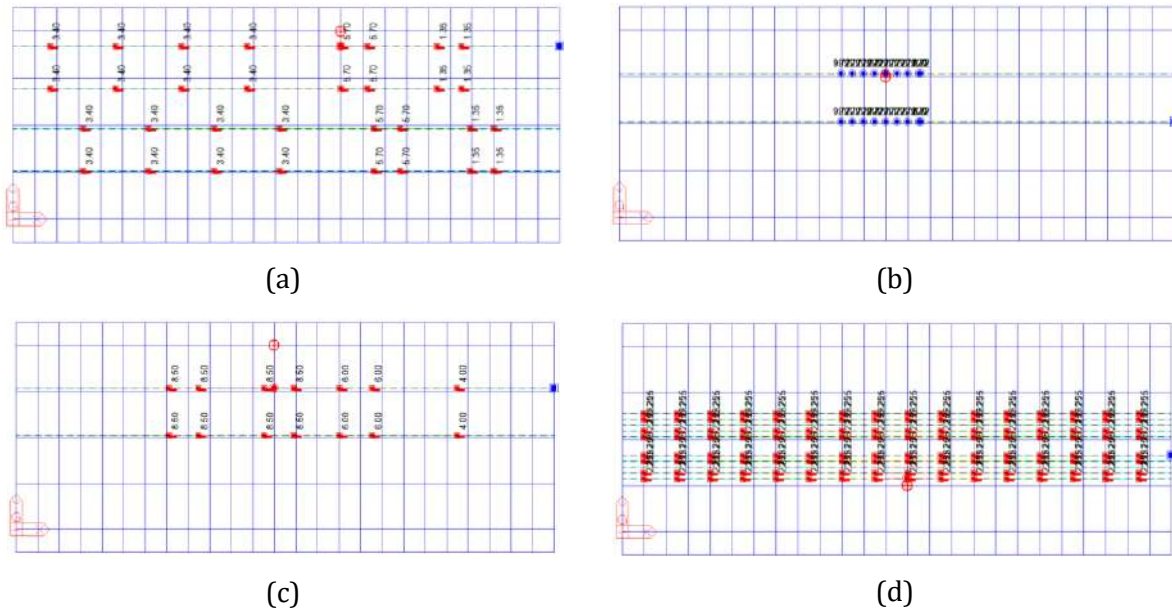


Fig. 7. Critical position of stationary IRC Vehicles on the bridge model(a) Dual class A, (b) Class AA (tracked), (c) Class 70R (wheeled) and (d) Special Vehicle

3.2.2 Application of Masses

Dynamic analysis is not performed using static loads directly, so each dead load and all other static load cases are firstly converted into equivalent nodal masses via the mass generation tool before carrying out any modal, response-spectrum, or time-history analysis. In this study, the “Load to Masses” function available in MIDAS Civil software is utilized to represent both the vehicular and structural masses.

3.2.3 Eigenvalue Analysis Control

Eigenvalue analysis determines a structure’s dynamic properties by solving the characteristic equation composed of its mass and stiffness matrices. These properties include natural modes or mode shapes, natural periods or frequencies, and modal participation factors. The natural frequencies and corresponding periods are summarized in Table 9, while the modal participation masses of the developed bridge model are presented in Table 10.

Table 9. Natural frequency and period of the model

Mode No.	Natural Frequency (cycles / s)	Natural Period (s)	Mode No.	Natural Frequency (cycles / s)	Natural Period (s)
1.	3.493	0.286	11.	25.670	0.039
2.	4.379	0.228	12.	31.888	0.031
3.	10.743	0.093	13.	32.452	0.030
4.	12.297	0.081	14.	34.292	0.029
5.	12.486	0.080	15.	34.601	0.028
6.	14.953	0.066	16.	36.312	0.027
7.	19.629	0.050	17.	37.393	0.026
8.	20.944	0.047	18.	38.060	0.026
9.	22.329	0.044	19.	38.351	0.026
10.	23.026	0.043	20.	43.919	0.023

Table 10. Modal participation masses

Mode No.	Transverse-X		Transverse-Y		Transverse-Z		Rotation-X		Rotation-Y		Rotation-Z	
	Mass (%)	Sum (%)	Mass (%)	Sum (%)	Mass (%)	Sum (%)	Mass (%)	Sum (%)	Mass (%)	Sum (%)	Mass (%)	Sum (%)
1.	2.27	2.27	0.00	0.00	78.93	78.93	0.00	0.00	0.00	0.00	0.00	0.00
2.	0.00	2.27	0.11	0.11	0.00	78.93	78.63	78.63	0.00	0.00	0.11	0.11
3.	0.00	2.27	92.90	93.01	0.00	78.93	0.15	78.78	0.00	0.00	0.00	0.11
4.	9.64	11.91	0.00	93.01	0.12	79.05	0.00	78.78	51.97	51.97	0.00	0.11
5.	0.00	11.91	2.03	95.05	0.00	79.05	0.14	78.91	0.00	51.97	4.04	4.15
6.	0.12	12.03	0.00	95.05	0.03	79.08	0.00	78.91	0.51	52.48	0.00	4.15
7.	17.71	29.73	0.00	95.05	0.06	79.15	0.00	78.91	0.18	52.67	0.00	4.15
8.	0.00	29.73	0.06	95.11	0.00	79.15	0.93	79.84	0.00	52.67	78.26	82.41
9.	17.83	47.57	0.00	95.11	1.96	81.11	0.00	79.84	3.07	55.74	0.00	82.41
10.	0.00	47.57	0.32	95.43	0.00	81.11	8.39	88.24	0.00	55.74	12.72	95.13
11.	39.13	86.70	0.00	95.43	2.66	83.77	0.00	88.24	1.25	56.99	0.00	95.13
12.	0.00	86.70	0.06	95.49	0.00	83.77	0.01	88.25	0.00	56.99	0.16	95.29
13.	0.73	87.42	0.00	95.49	0.03	83.80	0.00	88.25	7.22	64.21	0.00	95.29
14.	7.78	95.20	0.00	95.49	4.32	88.12	0.00	88.25	0.52	64.73	0.00	95.29
15.	0.00	95.20	0.38	95.87	0.00	88.12	0.00	88.25	0.00	64.73	0.00	95.29
16.	0.00	95.20	2.41	98.28	0.00	88.12	0.39	88.64	0.00	64.73	0.01	95.30
17.	0.00	95.20	0.13	98.41	0.00	88.12	0.11	88.75	0.00	64.73	0.25	95.55
18.	0.37	95.58	0.00	98.41	1.46	89.58	0.00	88.75	0.04	64.78	0.00	95.55
19.	0.00	95.58	1.25	99.65	0.00	89.58	1.91	90.66	0.00	64.78	0.02	95.57
20.	0.00	95.58	0.03	99.68	0.00	89.58	0.02	90.68	0.00	64.78	0.33	95.90

3.2.4 Response Spectrum Cases

This study performs response spectrum analysis for the seismic zones IV and V with zone factors 0.24 and 0.36 (Table 16, IRC 6, 2017) respectively considering the medium type of soil, importance factor 1.2 (Table 19, IRC 6, 2017) and response reduction factor as 1 (Table 20, IRC 6, 2017). Damping factor is taken as 2% for composite structure with a multiplying factor of 1.4 (Table 18-IRC 6, 2017). Both the cases are analyzed considering all the vehicular loads as per IRC 6, 2017. The response spectra used for seismic analysis in Zones IV and V are shown in Figure 8.

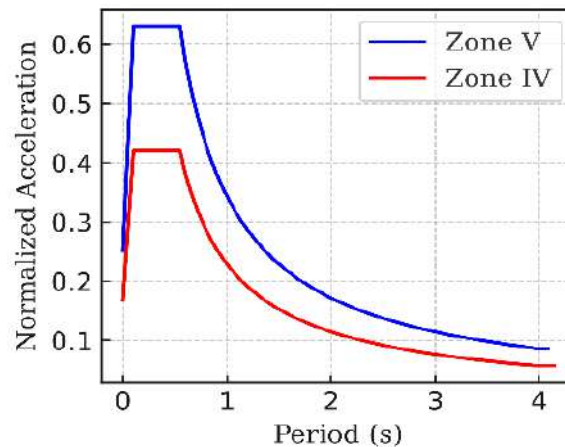


Fig. 8. Design response spectra for Indian seismic Zones IV and V

3.2.5 Time History Cases

Time History Analysis (THA) was performed for three cases corresponding to the Indian seismic Zone V, as summarized in Table 11. The earthquake acceleration records were obtained from the Strong Motion Virtual Data Center [20], and the corresponding accelerograms for each case are presented in Figures 9–11. This study excludes the vertical component of seismic motion as vertical peak ground acceleration is usually lower than the horizontal components and shows limited spectral amplification. Structural systems also have high vertical stiffness from gravity loads, which restricts vertical displacements and limits their impact on seismic response. And hence, only horizontal components are considered additionally improving computational efficiency.

Table 11. Time history cases

Earthquake	Station	Magnitude	Hypocentral distance (km)	Component	Peak ground acceleration (m/s ²)
Bhuj/Kachchh 2001-01-26 03:16:40	Ahmedabad, Gujarat	7.0	239.0	Longitudinal (78)	-1.0382
				Transverse (348)	-0.78236
Chamoli 1999- 03-28 19:05:11	Gopeshwar, Uttarakhand	6.6	17.3	Longitudinal (20)	-3.5283
				Transverse (290)	1.9507
India – Burma Border 1995- 05-06 01:59:07	Diphu, Assam	6.4	245.6	Longitudinal (180)	1.0030
				Transverse (90)	0.78988

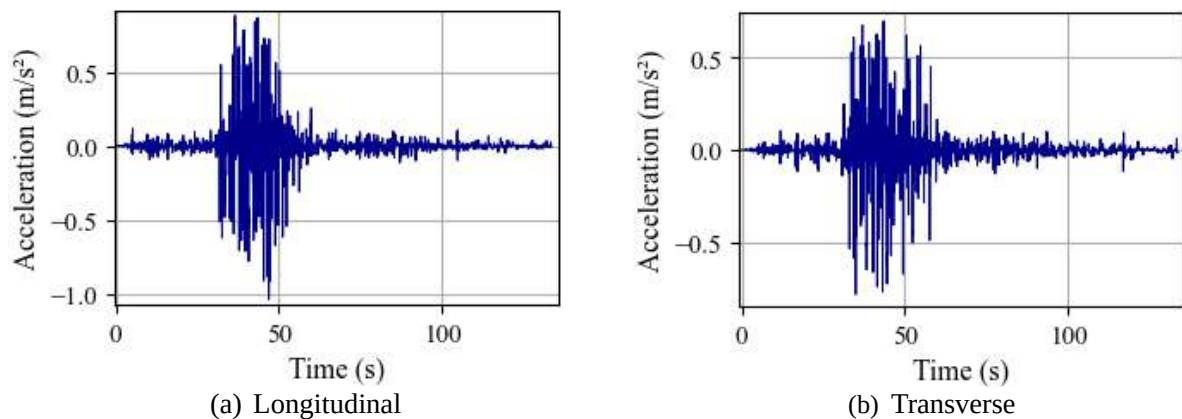


Fig. 9. Accelerogram of Bhuj/Kachchh earthquake

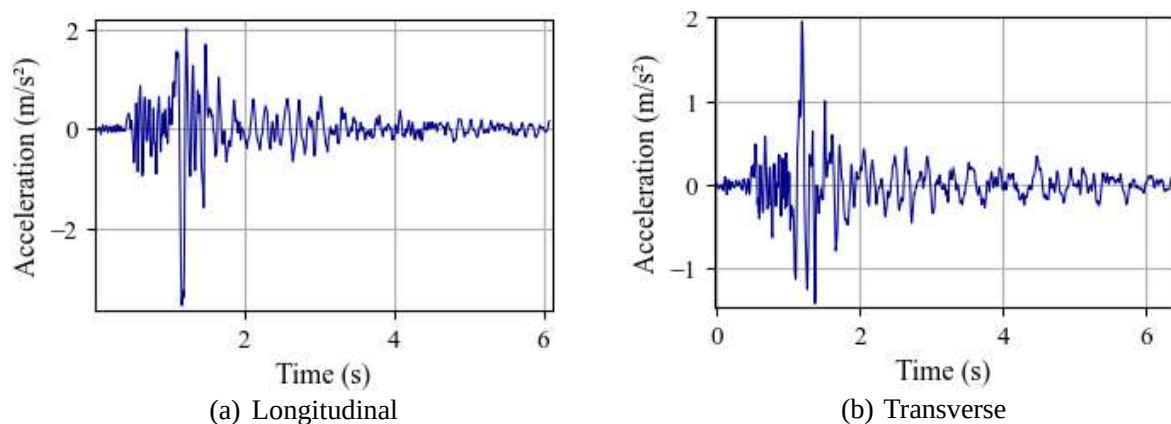


Fig. 10. Accelerogram of Chamoli earthquake

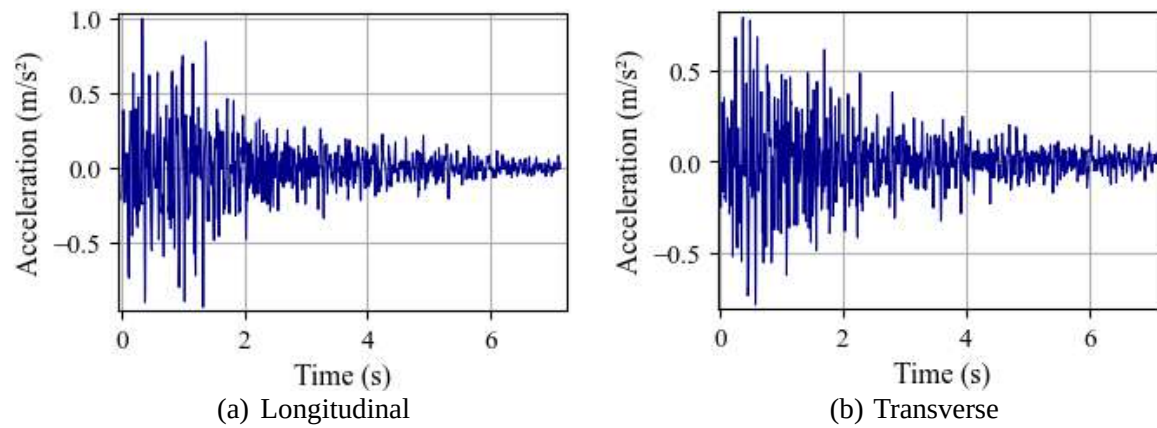


Fig. 11. Accelerogram of India - Burma Border earthquake

4. Results and Discussions

The comparative assessment of the bridge's seismic response under different vehicular load scenarios, employing both Response Spectrum Analysis (RSA) and Linear Time History Analysis (THA), reveals consistent patterns in the increase of structural demand caused by the presence of heavy vehicles at mid-span. In all seismic scenarios, findings indicate that vehicular loading enhances deflection, shear force, and the bending moment, with the degree of increase proportional to the vehicle's total axle load. This supports earlier studies on vehicle-bridge interaction Ghosh et al. [1], Kameshwar & Padgett [4], Liu et al. [5], and additionally measures the impact in relation to IRC-specified loads and Indian seismic conditions.

4.1 Response Spectrum Analysis Results

In both Seismic Zones IV and V, RSA outcomes (Figures 12-14) show a clear monotonic relationship between axle load and structural response. The special vehicle with an effective axle load of 288 tons consistently caused the largest increases across all three response parameters. With special vehicle, in Zone IV, shear force increased by 115%, bending moment increased by 112%, and the deflection increased by 106% compared to the case without any vehicle. The amplification is stronger in Zone V, with responses about 50% higher than those in Zone IV. However, the percentage increases caused by the presence of vehicles compared to the unloaded case were similar in both zones. The increase in axle load shows increment in structural response however the relation is not linear as the rate of increase reduces in the case of IRC special vehicle.

Table 12. Summary of response spectrum analysis

Vehicle	Total axle load on bridge (tons)	Zone IV			Zone V		
		Max. shear force (kN)	Max. bending moment (kN-m)	Max. deflection (mm)	Max. shear force (kN)	Max. bending moment (kN-m)	Max. deflection (mm)
Without Vehicle	-	38.4	137.5	1.869	57.6	206.3	2.804
Class AA (Tracked)	70	44.1	191.8	2.632	66.1	287.7	3.948
Class 70R (Wheeled)	100	50.7	222.1	3.008	76.1	333.2	4.511
Dual Class A	2 * 55.4 = 110.8	57.1	233.6	3.156	85.7	350.5	4.734
Special Vehicle	288	82.4	290.8	3.855	123.6	436.2	5.828

Note: Although the total axle load of the special vehicle is 385 tons, only 288 tons is considered in analysis since its length (38.459 m) exceeds the bridge length (25 m).

This is probably due to differences in how the load is distributed. These results show that axle configuration and spacing matter as much as the total load in affecting dynamic demand. This acknowledges with Liu et al. [5], who highlighted how load distribution impacts structural response.

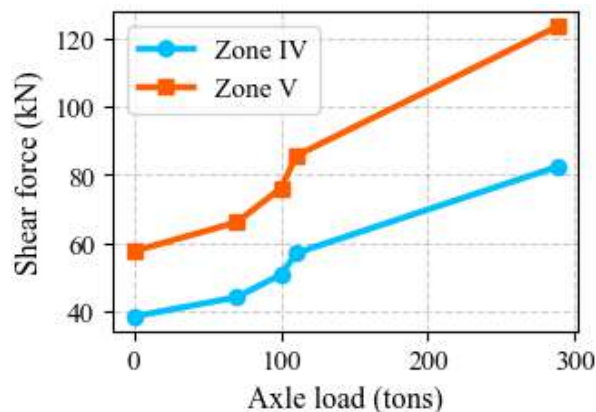


Fig. 12. Variation of shear force with axle load in seismic zones IV and V

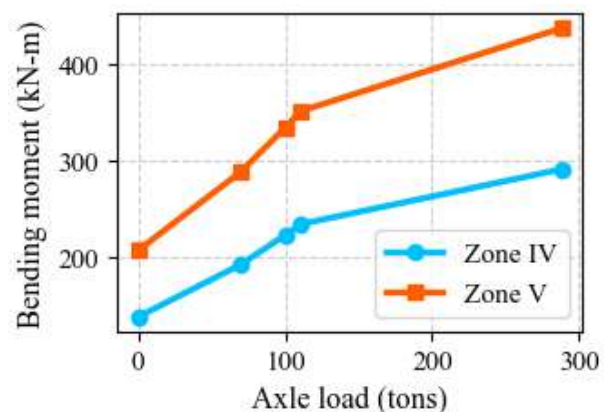


Fig. 13. Variation of bending moment with axle Load in seismic zones IV and V

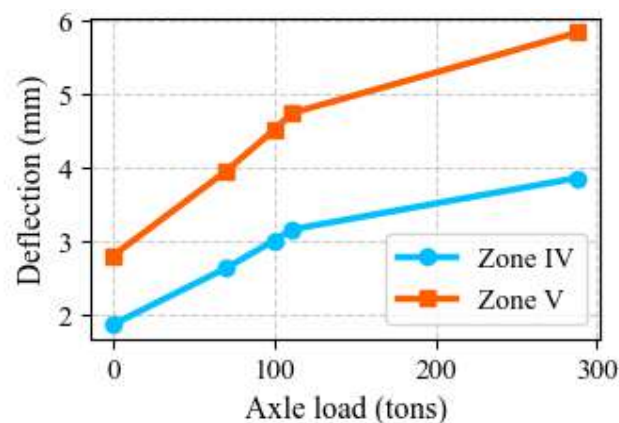


Fig.14. Variation of deflection with axle load in seismic zones IV and V

4.2 Time History Analysis Results

THA results (Figure 15-17) confirm the load-response relationship and also highlight how strongly earthquake record characteristics affect absolute demand values. The Chamoli earthquake, with its high PGA and close proximity, produced the highest responses across all load cases, which is 2 to 3 times higher than in the Bhuj or India-Burma border cases. This shows that short duration ground motions with high intensity can increase the negative impact of live loads. The results are in good agreement with the findings of Lin et al. [8] and Liu et al. [10] that near-fault motions with strong velocity pulses amplify structural responses more than far-field motions.

In the Bhuj earthquake case, the overall values were lower however the percentage increase caused by vehicles was still significant. For instance, the Special Vehicle increased bending moment from 119.9 kN-m (unloaded) to 216.5 kN-m which is about an 80% rise. In the India-Burma border case, some unexpected patterns appeared, such as the Special Vehicle producing lower deflection than lighter loads in certain situations. This likely happened because of complex interactions between vibration modes and timing differences between inertial forces and ground motion.

Figures 18-20 shows the displacement-time history plot of the different earthquakes and its variation in presence of loading conditions. The displacement-time history response for all loading conditions exhibits a fundamentally similar oscillatory behavior, characterized by a consistent harmonic motion around the neutral axis. However, the displacement amplitude varies significantly as per the loading conditions. This indicates that the natural frequency and damping properties of the structure itself control the dynamic response. While the presence of varying

vehicular loads alters the intensity of the oscillations, it does not significantly modify the fundamental pattern of structural vibration.

Table 13. Summary of time history analysis

Vehicle	Bhuj/Kachchh Earthquake			Chamoli Earthquake			India-Burma Border Earthquake		
	Max. SF (kN)	Max. BM (kN-m)	Max. Def. (mm)	Max. SF (kN)	Max. BM (kN-m)	Max. Def. (mm)	Max. SF (kN)	Max. BM (kN-m)	Max. Def. (mm)
No Vehicle	26.0	119.9	1.745	29.3	216.0	2.710	32.1	118.8	1.516
Class AA (Tracked)	33.7	144.0	2.273	74.3	533.8	8.189	75.9	229.3	3.680
Class 70R (Wheeled)	42.8	182.8	2.800	88.4	528.6	8.281	59.3	221.0	3.730
Dual Class A	44.3	176.4	2.793	92.0	623.4	8.932	59.1	288.9	4.049
Special Vehicle	55.9	216.5	2.836	96.1	592.9	9.016	80.6	199.5	2.777

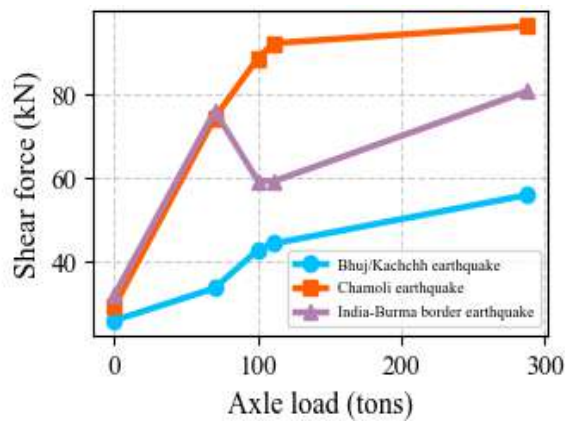


Fig. 15. Variation of shear force with axle load in different seismic events

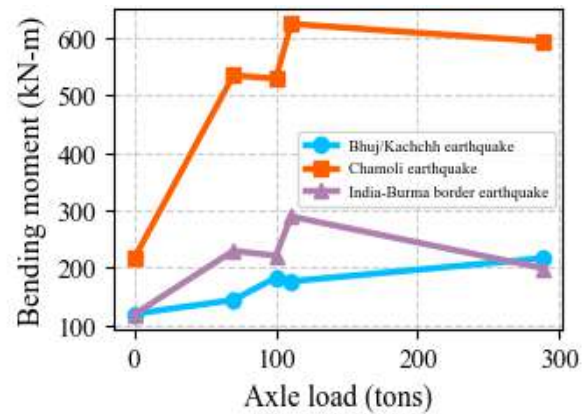


Fig. 16. Variation of bending moment with axle load in different seismic events

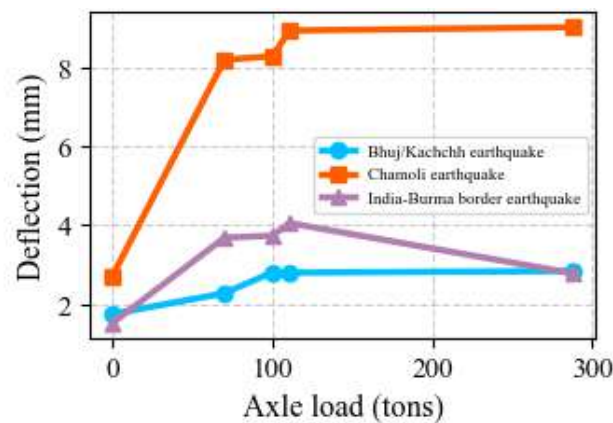


Fig. 17. Variation of deflection with axle load in different seismic events

In addition, the seismic performance of bridge structures is governed not only by the intensity of ground motion but also by inherent structural characteristics such as lateral strength, stiffness distribution, and energy dissipation capacity. Implementing seismic isolation systems can significantly reduce seismic demands by increasing the fundamental period and providing additional damping, which in turn limits the forces transmitted to the superstructure [21–23]. At

the same time, the seismic response is highly sensitive to material properties, reinforcement detailing as these factors influence stiffness degradation, ductility, and the progression of damage under cyclic loading [24]. Therefore, with a balanced combination of global response-control strategies and careful local detailing adequate seismic resilience can be achieved.

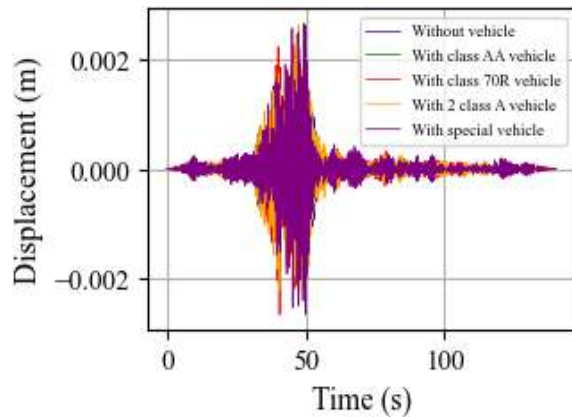


Fig. 18. Displacement time history of Bhuj/Kachchh earthquake

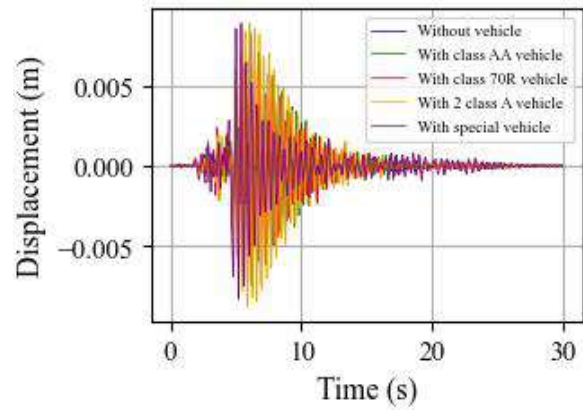


Fig. 19. Displacement time history of Chamoli earthquake

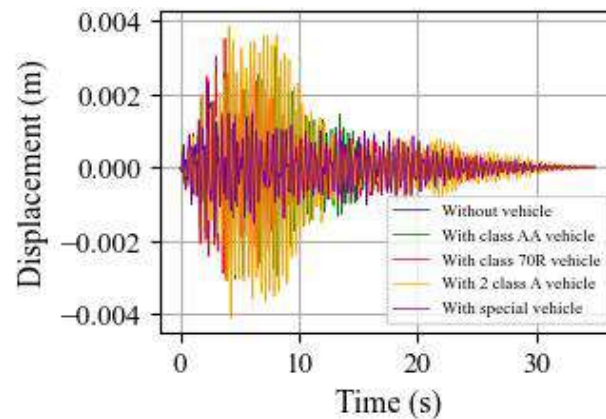


Fig.20. Displacement time history of India-Burma Border earthquake

5. Conclusions

This study analyzed the seismic response of a steel-concrete composite I-girder bridge in the presence of heavy vehicle utilizing both Response Spectrum Analysis (RSA) and Linear Time History Analysis (THA). RSA covers Indian seismic Zones IV and V. THA uses recorded ground motions from Bhuj Kachchh 2001, Chamoli 1999, and the India Burma Border 1995. The study evaluates several vehicular loading cases defined by the Indian Roads Congress. These include an unloaded bridge, Class AA Tracked, Class 70R Wheeled, dual Class A, and Special Vehicle configurations.

The results show that the presence of stationary vehicles during seismic events increase shear force, bending moment, and deflection compared to an unloaded bridge condition. Heavier axle loads and closer axle spacing generally lead to greater amplification, underscoring the importance of accounting for traffic conditions in seismic bridge analysis. In RSA, Zone V responses were about 50% higher than Zone IV for both loaded and unloaded conditions, showing the importance of region-specific seismic provisions.

THA results showed that the response depends strongly on the ground motion characteristics. The Chamoli earthquake, with its near-field effects, produced peak responses up to three times higher than the Bhuj/Kachchh and India-Burma border events, especially with heavy vehicles. This behavior emphasizes the critical role of near-field effects and frequency content in governing seismic demand. The Special Vehicle often produced the highest shear forces, yet depending on ground acceleration, dual Class A or Class 70R loads sometimes caused larger deflections or

bending moments. This highlights the complex interaction between vehicle configuration, bridge dynamics, and seismic input.

Both RSA and THA showed that vehicles increase seismic demands, but THA usually gave higher peaks and captured interaction effects missed by RSA, making it more useful for assessing performance for near field earthquakes. The correlation between axle load and seismic response was strong but non-linear, affected by axle spacing and load distribution.

Overall, the results show that ignoring the combined effects of traffic loads and earthquakes leads to unconservative bridge designs. In moderate to high seismic regions, design practice should include the simultaneous action of vehicles and seismic excitation. This approach improves structural safety, serviceability, and resilience of critical transportation infrastructure.

6. Limitations

This study considered only single-span composite I-girder bridge with stationary mid-span vehicles under three Indian ground motions and two seismic zones. Traffic movement, wider seismic datasets, nonlinear effects, soil-structure interaction, and bearing damage were not modeled.

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References

- [1] Ghosh J, Caprani CC, Padgett JE. Influence of traffic loading on the seismic reliability assessment of highway bridge structures. *Journal of Bridge Eng.* 2013;19(3):04013003. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000535](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000535)
- [2] Borjigin S, Kim CW, Chang KC, Sugiura K. Non-linear seismic response analysis of vehicle-bridge interactive systems. *Struct Control Health Monit.* 2015;22(9):1110–25. <https://doi.org/10.1002/stco.201510006>
- [3] Shaban N, Caner A, Yakut A, Askan A, Karimzadeh Naghshineh A, Domanic A, Can G. Vehicle effects on seismic response of a simple-span bridge during shake tests. *Earthquake Eng Struct Dyn.* 2015;44(5):889–905. <https://doi.org/10.1002/eqe.2491>
- [4] Kameshwar S, Padgett JE. Effect of vehicle bridge interaction on seismic response and fragility of bridges. *Earthquake Eng Struct Dyn.* 2017;47(3):713–35. <https://doi.org/10.1002/eqe.2986>
- [5] Liu L, Yang X, Yan B, Miao S. Dynamic responses of RC girder bridge under heavy truck and seismic loads combined. *Sustainability.* 2022;14(15):9263. <https://doi.org/10.3390/su14159263>
- [6] Fu G, Feng J, Dekelbab W, Moses F, Cohen H, Mertz D. Impact of commercial vehicle weight change on highway bridge infrastructure. *J Bridge Eng.* 2008;13(6):556–64. [https://doi.org/10.1061/\(ASCE\)1084-0702\(2008\)13:6\(556\)](https://doi.org/10.1061/(ASCE)1084-0702(2008)13:6(556))
- [7] Itani AM, Bruneau M, Carden L, Buckle IG. Seismic behavior of steel girder bridge superstructures. *J Bridge Eng.* 2004;9(3):243–9. [https://doi.org/10.1061/\(ASCE\)1084-0702\(2004\)9:3\(243\)](https://doi.org/10.1061/(ASCE)1084-0702(2004)9:3(243))
- [8] Lin Y, Bi K, Zong Z, Hao H, Lin J, Chen Y. Seismic performance of steel-concrete composite rigid-frame bridge: Shake table test and numerical simulation. *J Bridge Eng.* 2020;25(7):04020043. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0001558](https://doi.org/10.1061/(ASCE)BE.1943-5592.0001558)
- [9] Wang T, Xie X, Cheng C. Probabilistic seismic demand analysis for bridges based on earthquake scenarios. *Eng Struct.* 2025;323:119128. <https://doi.org/10.1016/j.engstruct.2024.119128>
- [10] Liu X, Li L, Liu Y, Chen S, An W. Study on the influence of near-fault vertical ground motion on the dynamic response of separation and collision of bridge. *Sci Rep.* 2025;15:36372. <https://doi.org/10.1038/s41598-025-20347-5>
- [11] Ramadan OMO, Mehanny SSF, Kotb AAM. Assessment of seismic vulnerability of continuous bridges considering soil-structure interaction and wave passage effects. *Eng Struct.* 2020;206:110161. <https://doi.org/10.1016/j.engstruct.2019.110161>
- [12] Indian Roads Congress. IRC 6-2017: Standard specifications and code of practice for road bridges—Section II: Loads and load combinations. 7th rev. New Delhi: Indian Roads Congress; 2017.

- [13] American Association of State Highway and Transportation Officials. AASHTO LRFD bridge design specifications. 8th ed. Washington, DC: AASHTO; 2017.
- [14] European Committee for Standardization. EN 1991-2: Eurocode 1: Actions on structures—Part 2: Traffic loads on bridges. Brussels: CEN; 2003.
- [15] American Association of State Highway and Transportation Officials. AASHTO guide specifications for LRFD seismic bridge design. 2nd ed. Washington, DC: AASHTO; 2011.
- [16] European Committee for Standardization. EN 1998-2: Eurocode 8: Design of structures for earthquake resistance—Part 2: Bridges. Brussels: CEN; 2005.
- [17] Indian Roads Congress. IRC 112-2011: Code of practice for concrete road bridges. New Delhi: Indian Roads Congress; 2011.
- [18] Indian Roads Congress. IRC 22-2015: Standard specifications and code of practice for concrete road bridges—Section VI: Composite construction (limit state design). New Delhi: Indian Roads Congress; 2015.
- [19] Indian Roads Congress. IRC SP:120-2018: Explanatory handbook to IRC:22-2015. New Delhi: Indian Roads Congress; 2018.
- [20] Center for Engineering Strong Motion Data (CESMD). Strong Motion Virtual Data Center. Available from: <http://www.strongmotioncenter.org> [Accessed 19 Jun 2025].
- [21] Soneji B B, Jangid R S. Effectiveness of seismic isolation for cable-stayed bridges. *Int. J. Struct. Stab. Dyn.* 2006;6(1):77–96. <https://doi.org/10.1142/S0219455406001836>
- [22] Patel D, Mourya VK, Pandey G, Kumar P. Advancements in base isolation for seismic mitigation: Perspectives on elastomeric and lead rubber bearings. *Res. Eng. Struct. Mater.*, 2024; 10(3): 1017-1049. <https://doi.org/10.17515/resm2024.15ma0927rv>
- [23] Jangid R S. Seismic performance assessment of clutching inerter damper for isolated bridges. *Pract. Period. Struct. Des. Constr.*, 2022;27(2):04021078. [https://doi.org/10.1061/\(ASCE\)SC.1943-5576.0000661](https://doi.org/10.1061/(ASCE)SC.1943-5576.0000661)
- [24] Ozmen H B, Inel M. Effect of concrete strength and detailing properties on seismic damage for RC structures. *Research and Design*. 2024;1(1):1–11. <https://doi.org/10.17515/rede2024-005en1124rs>

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A multi-response optimization of ternary geopolymer mortar using Taguchi–GRA integrated method

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Taguchi-GRA integrated method

Abstract

This research addresses a critical gap in sustainable binder development by integrating experimental optimization and multi-response analysis to assess the performance of ternary Red Mud–Ground Granulated Blast Furnace Slag–Metakaolin (RM–GGBS–MK) geopolymer mortars. Although geopolymers are recognized for their potential to reduce cement consumption and CO₂ emissions, limited studies have systematically optimized ternary waste-based binders considering both fresh and hardened properties. In this study, a hybrid Taguchi–Grey Relational Analysis (GRA) framework was employed to simultaneously optimize the compressive strength, flowability, and setting times of geopolymer mortar mixes under three criteria: balanced performance, strength, and workability. The experimental design followed an L16 orthogonal array comprising five parameters—ternary composition, NaOH molarity, activator-to-binder (AL/B) ratio, Na₂SiO₃/NaOH (SS/SH) ratio, and binder-to-sand (B/S) ratio—each at four levels. The GGBS-rich mixes attained the maximum compressive strength of 49.09 MPa at 28 days, whereas the RM and MK dominant mixes demonstrated diminished strengths comparatively. ANOVA results indicated the most significant factors affecting the strength and flowability as SS/SH and Al/B ratios, whereas the binder compositions regulated initial and final setting times. Across all three criteria, GRA identified 14 M NaOH and a 1:1.5 B/S ratio as common significant parameters.

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1. Introduction

Cement production contributes to around 8% of worldwide CO₂ emissions[1]. This global challenge has prompted research into alternative binders that can diminish the carbon footprint of cement production. By utilizing industrial wastes, geopolymers have emerged as sustainable replacements[2]. Aluminosilicate-rich industrial by-products are activated with alkaline solutions to form geopolymers[3]. Compared with ordinary Portland cement (OPC), these materials exhibit enhanced physical and mechanical performance.

Red Mud (RM), an alkaline by-product of the Bayer process used in alumina production, is generated in substantial amounts globally and poses significant environmental disposal challenges [4]. In India, the annual generation of red mud is approximately 9 to 12 million metric tons [5]. RM contains high levels of Fe₂O₃ and Al₂O₃, while showing a lower concentration of reactive SiO₂, thereby limiting its geopolymer reactivity. [6] When used alone as a binder, RM often requires thermal curing and shows minimal early strength development. RM stored in landfills may pose environmental risks due to high alkalinity, which can lead to soil and water contamination [7].

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Ground Granulated Blast Furnace Slag (GGBS) is an industrial by-product produced from blast furnaces utilized in iron manufacturing. This material is abundant in aluminosilicates and CaO, rendering it an appropriate source for geopolymer binders. It improves geopolymer characteristics by accelerating strength attainment and diminishing the necessity for thermal curing [8,9]. The annual production of GGBS in India is estimated at approximately 15 million tons[10]. Although majority of GGBS is used as supplementary cementitious material, unused GGBS creates significant pollution in the form of airborne particulate matter.

Proportions of oxides such as SiO₂, Al₂O₃, Fe₂O₃ and CaO strongly influence the performance of geopolymer binders [11]. Industrial by-products like RM and GGBS vary in chemical compositions, and their combined use may not always provide sufficient reactive silica and alumina for optimal geopolymerization. Metakaolin (MK), a highly reactive aluminosilicate obtained by calcining kaolinite clay at elevated temperatures, is incorporated to supplement these oxides. By enhancing reactive Si and Al, MK promotes the development of stronger and more durable geopolymer matrix[12,13].

Geopolymer mortar properties are affected by multiple factors such as molarity of the NaOH, alkaline activator to binder (Al/B) ratio, binder to aggregate proportions, and curing conditions [14]. Binary mixes comprising RM-GGBS and RM-MK have been studied previously in the literature. RM-GGBS blends benefit from calcium-induced strength gain and reduced curing temperature but can suffer from quick setting times and workability when RM content is high[15]. RM-MK blends improve the Si and Al content but lack calcium for rapid early strength [16]. Wang et.al[17] had demonstrated that mixtures containing 60–70% GGBS, with Red Mud as a secondary component, attained compressive strengths greater than 45 MPa, with enhanced thermal resistance and matrix densification. The use of Metakaolin at 60%, in conjunction with RM, has demonstrated a significant improvement in long-term strength and microstructural integrity[18,19]. In addition to these findings, RM-rich blends consisting of 60% Red Mud and 40% GGBS exhibited compressive strengths between 30 and 38 MPa [20,21]. Nevertheless, few studies have been done on the combined impact of all the three binders. The ternary mix presents a promising option for a sustainable substitute for cement [22].

The Taguchi approach along with Grey Relational Analysis (GRA), has been utilized to efficiently study and optimize multiple parameters in geopolymer systems, including strength, setting time, and workability [23,24]. Orthogonal arrays (OA) are employed in Taguchi method to identify critical factors and minimize the number of experimental trials[25–27]. The GRA enhances the Taguchi approach by facilitating multi-response optimization, aiding in the identification of parameter combinations that balance fresh state and hardened state properties [28]. The GRA consolidates various performance attributes into a singular Grey Relational Grade, hence easing the optimization procedure[29,30]. It facilitates the identification of optimal factor levels by ranking the Grey Relational Grades, consequently revealing the parameter configurations that yield superior overall performance.

This study investigates the influence of mix parameters on the performance characteristics of RM-GGBS-MK geopolymer mortar. The factors considered include the sodium silicate to sodium hydroxide (SS/SH) ratio, NaOH molarity, Al/B ratio, and binder to sand ratio, with the ternary mix composition (different proportions of binders) included as one of the factors due to its influence on aluminosilicate availability and geopolymerization. Taguchi design of experiments combined with GRA was employed to determine optimal parameter combinations under ambient curing conditions for multi-response optimization under three different optimization objectives corresponding to strength, workability, and balanced overall performance.

2. Materials

RM was obtained from NALCO, Damanjodi, Odisha. RM is predominantly composed of Fe₂O₃ and Al₂O₃ and has a characteristic red color. It was oven-dried at 80°C for a minimum of 24 hours to remove moisture, and the dried lumps were grounded and sieved through a 75-micron sieve. GGBS was sourced from Visakhapatnam. It is rich in calcium silicates and acts as a significant source of CaO, and has a greyish-white color. MK was sourced from Mumbai, India. It is a highly reactive

aluminosilicate material with good pozzolanic properties. It has an off-white colour. The chemical compositions of RM, GGBS and MK as determined by X-ray fluorescence (XRF) analysis are presented in Table 1, while their particle size distribution and XRD details are presented in Figure 1. XRD pattern of MK exhibits a large diffuse hump alongside identifiable crystalline peaks primarily attributed to quartz (Q). RM sample displays distinct diffraction peaks corresponding to gibbsite (G) and hematite (H) as the primary crystalline phases, alongside modest reflections of quartz (Q) and weak carbonate (C). The XRD pattern of GGBS is characterized by a large amorphous halo, usually found in the 2θ range of about $20\text{--}35^\circ$, indicating the prevalence of glassy amorphous phases in the slag.

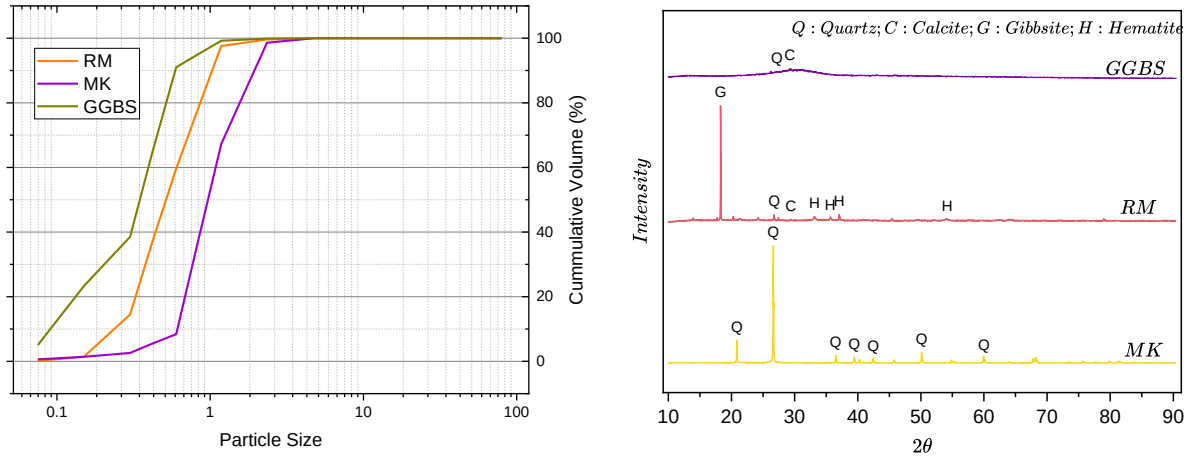


Fig. 1. Particle size distribution and XRD of the binder materials

Commercially available sodium silicate and sodium hydroxide solutions were used as alkaline activators in this study. SS/SH ratio was considered as a variable affecting the geopolymer mortar properties. Sodium hydroxide (NaOH) pellets with 99% purity were dissolved in water for 24 hours, prior to casting. Sodium silicate solution comprised of 26.5 wt% SiO_2 and 8.0 wt% Na_2O . River sand conforming to Grade 2 as per IS:383 [31] was used as fine aggregate. Binder-to-sand ratio was considered as one of the experimental factors, due to its influence on the strength of mortar.

Table 1. Chemical compositions of binder materials

Materials	Al_2O_3	CaO	SiO_2	MgO	Na_2O	K_2O	Fe_2O_3	TiO_2
RM	15.03	1.5	7.85	2.14	4.65	2.59	49.18	4.5
GGBS	10.39	38.12	32.41	8.96	0.35	0.5	0.69	1.23
MK	41.41	0.08	52.22	0.26	0.01	1.73	0.49	0.65

3. Methodology

The methodology employed for single response optimization utilizing Taguchi and multi-response optimization employing GRA is illustrated in Figure 2. In this study, five key factors influencing geopolymerization were selected based on prior literature and preliminary trials. The factors considered include the ternary mix proportions of RM, GGBS, and MK at specified ratios; NaOH at 8, 10, 12 and 14 molarity, Al/B ratio between 0.45 and 0.60, SS/SH ratio from 1.0 to 2.5 and B/S ratio from 1:1.5 to 1:3. Each parameter was chosen based on its impact on the geopolymerization process. The ternary blend ratio and B/S ratio regulates the proportions of reactive alumina, silica and calcium, whereas the molarity of NaOH influences precursor dissolution through the provision of OH^- and Na^+ ions[32–34]. The Al/B ratio ensures an adequate amount of activator in relation to the binder, while the SS/SH ratio controls the availability of soluble silica for polycondensation [22,32].

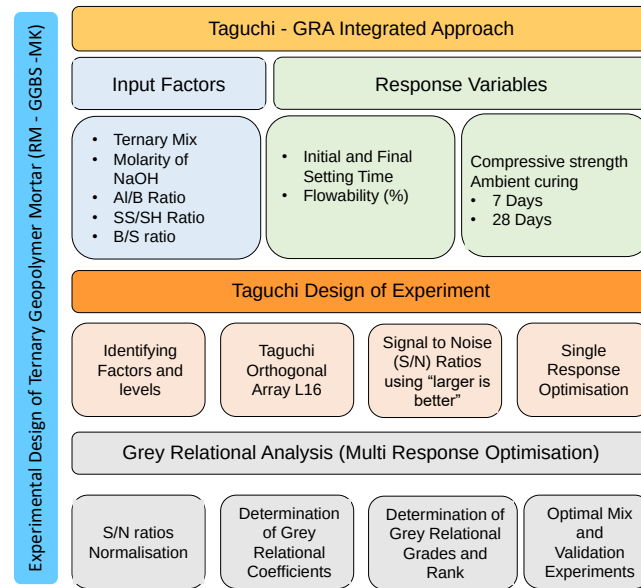


Fig. 2. Flowchart for optimisation methodology

The integrated Taguchi-GRA method is utilized to optimize various performance parameters of ternary RM-GGBS-MK geopolymer mortar. An L16 Taguchi OA is employed to investigate the main factor effects on geopolymer mortar properties. The full factorial design requires 1024 experimental combinations (4^5), making the process expensive and time-consuming. The Taguchi method enables systematic evaluation of multiple variables with a reduced number of trials through orthogonal arrays, whereas GRA facilitates multi-response optimization by consolidating different performance attributes into a single comparative index. Factors considered and the corresponding four levels are shown in Table 2. The factor assignments are presented in Table 3.

Table 2. Factors and levels for Taguchi method

Parameters	Level 1	Level 2	Level 3	Level 4
Ternary Mix	A1-20G60M20	A2-20G20M60	A3-60G20M20	A4-33G33M33
NaOH Molarity	8	10	12	14
AL/B Ratio	0.6	0.55	0.5	0.45
SS/SH Ratio	2.5	2	1.5	1.0
B/S Ratio	1:3	1:2.5	1:2	1:1.5

The Taguchi method evaluates robustness through signal-to-noise (S/N) ratios, which can be determined on the basis of "larger-the-better," "smaller-the-better," or "nominal-the-best" [36]. Since extended setting time, increased compressive strength and higher workability are targeted, the larger-the-better formulation was used for all responses as shown

$$y_{pq} = -10 \times \log_{10} \left[\frac{1}{n} \sum \frac{1}{v_{pi}^2} \right] \quad (1)$$

where, v_{pi} is the response of p^{th} trial mix for the i^{th} response and n is the number of observations, y_{pq} denotes the experiment's signal/noise ratio.

ANOVA was performed on S/N ratios to enumerate the effect of each factor. As the OA, had zero degrees of freedom. Hence, factor contributions were considered as the percentage of the sum of squares [37].

Table 3. Mix compositions and factor levels

Mix	Ternary Mix	NaOH Molarity	AL/B Ratio	SS/SH Ratio	B/S Ratio
M1	R20G60M20	8	0.6	2.5	1:3
M2	R20G60M20	10	0.55	2	1:2.5
M3	R20G60M20	12	0.5	1.5	1:2
M4	R20G60M20	14	0.45	1	1:1.5
M5	R20G20M60	8	0.55	1.5	1:1.5
M6	R20G20M60	10	0.6	1	1:2
M7	R20G20M60	12	0.45	2.5	1:2.5
M8	R20G20M60	14	0.5	2	1:3
M9	R60G20M20	8	0.5	1	1:2.5
M10	R60G20M20	10	0.45	1.5	1:3
M11	R60G20M20	12	0.6	2	1:1.5
M12	R60G20M20	14	0.55	2.5	1:2
M13	R33G33M33	8	0.45	2	1:2
M14	R33G33M33	10	0.5	2.5	1:1.5
M15	R33G33M33	12	0.55	1	1:3
M16	R33G33M33	14	0.6	1.5	1:2.5

The following equation was used to normalize the signal-to-noise ratio for each result;

$$Z_{pq} = \frac{y_{pq} - \min(y_{pq})}{\max(y_{pq}) - \min(y_{pq})}; p = 1, 2, \dots, r; q = 1, 2, \dots, n \quad (2)$$

Grey Relational Coefficient (GRC) was calculated using the following equation;

$$GRC_{opq} = \frac{\Delta_{min} + \zeta \Delta_{max}}{\Delta_{opq} + \zeta \Delta_{max}} \quad (3)$$

Δ_{opq} represents the discrepancy between the ideal and actual values of the normalized signal-to-noise ratio. The variable ζ denotes the identification or distinguishing coefficient, which ranges from 0 to 1, typically set at 0.5. [32]. Grey Relational Grade (GRG) was determined using;

$$GRG_{opq} = \frac{1}{n} \sum_{n=1}^m wGRC_{opq}; p = 1, 2, \dots, r \quad (4)$$

For various multi-response optimization objectives, suitable weighting factors were allocated to the response variables, and the Grey Relational Grade (GRG) was computed by the weighted aggregation of the respective Grey Relational Coefficients (GRCs).

3.1 Mix Procedure

The RM, GGBS, and MK were dry-mixed with fine aggregate for two minutes in a mortar mixer. The pre-prepared SS+SH alkaline solution was subsequently added, and the process was carried out for three minutes. The mortar was poured into 70.6 mm cube moulds as shown in Figure 3, compacted, and subjected to vibration for a duration of two minutes. 03 specimens for each mix were demoulded and stored at room temperature until testing at 7 and 28 days at an average temperature of 29°C. Flowability (%) for the mortar samples was determined according to IS 5512[39]. Tests for setting times were conducted as per IS 4031-Part 5[40]. Tests for compressive strength were carried out according to IS: 516[41]. The Bruker D8 Advance diffractometer is employed for X-ray diffraction investigation to examine crystalline structure and phase composition. Phase identification was achieved by correlating diffraction patterns with standard reference cards. The Bruker ALPHA-II ATR-FTIR spectrophotometer is employed for FTIR analysis.



Fig. 3. Geopolymer mortar specimens

4. Results and Discussion

S/N ratios and the experimental results for setting time, flowability, and compressive strength for all mixes are displayed in Table 4a and 4b. The results were evaluated by computing the average S/N ratio for each individual response across all the mixes with identical factors.

Table 4. (a) S/N ratios of the response variables

Mix Designation	Ternary Mix	NaOH Molarity	Al/B	SS/SH	B/S	S/N Ratios				
						Flow	IST	FST	CS-07 Days	CS -28 Days
M1	A1	8	0.6	2.5	1:3	41.41	32.67	35.99	27.74	31.37
M2	A1	10	0.55	2	1:2.5	38.59	34.96	40.91	27.02	21.42
M3	A1	12	0.5	1.5	1:2	34.81	38.59	46.24	20.87	28.63
M4	A1	14	0.45	1	1:1.5	43.52	34.32	39.09	33.25	33.82
M5	A2	8	0.55	1.5	1:1.5	43.52	47.24	50.93	24.35	28.80
M6	A2	10	0.6	1	1:2	39.65	45.80	48.13	23.33	27.95
M7	A2	12	0.45	2.5	1:2.5	43.52	36.26	41.94	28.28	30.29
M8	A2	14	0.5	2	1:3	27.96	50.32	52.42	15.04	18.57
M9	A3	8	0.5	1	1:2.5	33.06	45.58	49.13	22.15	24.88
M10	A3	10	0.45	1.5	1:3	31.36	45.58	45.34	16.22	26.82
M11	A3	12	0.6	2	1:1.5	41.18	52.67	55.71	16.83	22.29
M12	A3	14	0.55	2.5	1:2	43.52	46.44	50.26	23.89	21.18
M13	A4	8	0.45	2	1:2	34.85	33.98	40.91	30.23	29.07
M14	A4	10	0.5	2.5	1:1.5	43.52	40.83	43.81	24.06	28.74
M15	A4	12	0.55	1	1:3	43.52	39.55	47.78	26.22	27.18
M16	A4	14	0.6	1.5	1:2.5	43.52	38.79	45.85	17.01	28.82

Table 4. (b) Experimental results of response variables

Mix Designation	Setting Time		Flowability (%)	07 Days Compressive Strength			28 Days Compressive Strength		
	Initial, mins	Final, mins		Strength, MPa	Std. Deviation	Coefficient of Variation	Strength, MPa	Std. Deviation	Coefficient of Variation
M1	43	63	117.6	24.38	0.872	3.57%	37.02	1.758	4.75%
M2	56	111	85	22.45	2.227	9.92%	11.78	1.054	8.94%
M3	85	205	55	11.05	1.039	9.41%	27.02	1.453	5.38%
M4	52	90	150	45.96	2.242	4.88%	49.09	6.145	12.52%
M5	230	352	150	16.5	0.919	5.57%	27.54	2.539	9.22%
M6	195	255	96	14.67	0.544	3.71%	24.98	1.932	7.73%
M7	65	125	150	25.94	0.983	3.79%	32.71	3.903	11.93%
M8	328	418	25	5.65	0.344	6.09%	8.48	0.397	4.68%
M9	190	286	45	12.81	0.911	7.11%	17.53	1.236	7.05%
M10	190	185	37	6.47	0.13	2.01%	21.93	0.453	2.06%
M11	430	610	114.5	6.94	0.622	8.96%	13.02	1.073	8.24%
M12	210	326	150	15.65	0.283	1.81%	11.45	0.898	7.84%
M13	50	111	55.25	32.48	1.62	4.99%	28.4	1.803	6.35%
M14	110	155	150	15.95	0.64	4.01%	27.36	0.979	3.58%
M15	95	245	150	20.46	0.276	1.35%	22.86	3.055	13.36%
M16	87	196	150	7.09	0.212	2.99%	27.59	2.807	10.17%

4.2 Compressive Strength

The compressive strength test was carried out for each of the 16 mixes designed through the Taguchi method. Figure 4 shows the computed average compressive strength. At all the curing intervals, Mix M4 (60% GGBS and 20% RM & MK, Molarity: 14, Al/B ratio: 0.45, SS/SH ratio: 1, and B/S ratio: 1:1.5) demonstrated the highest compressive strength. Mix M8 (60% RM, 20% GGBS & MK, Molarity: 14, Al/B ratio: 0.55, SS/SH ratio: 2, B/S ratio: 1:3) exhibited the lowest compressive strength.

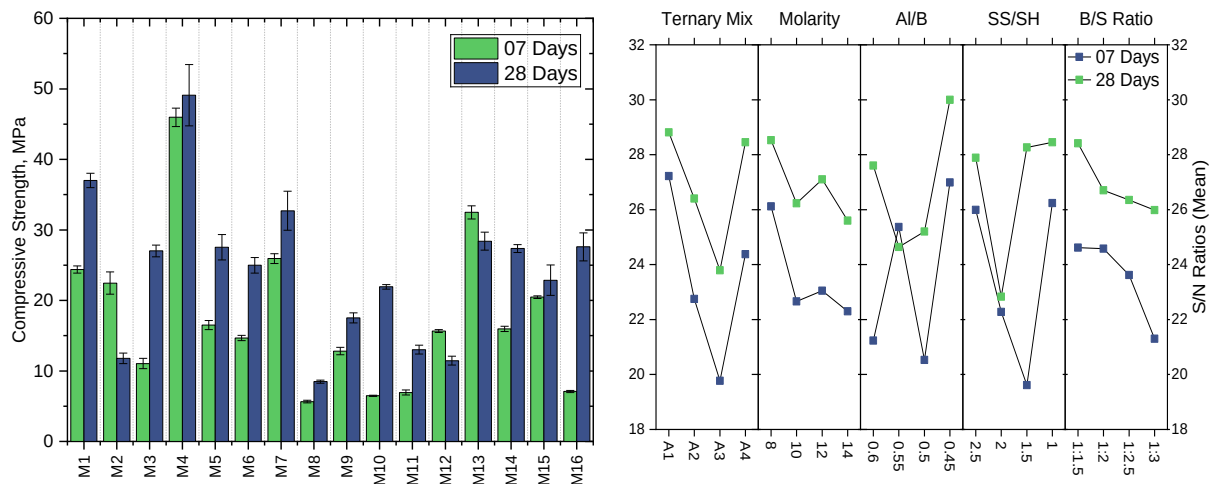


Fig. 4. Compressive strength and S/N ratios (mean)

From Figure 4, the ternary mixes with 60% GGBS had a pronounced effect on the compressive strength. Mixes comprising 60% RM and 60% MK had notable reduction in strength. As evident from Table 4, increasing the GGBS content from 20% to 60% resulted in improved compressive strength. The total calcium content of the mixes increased with increase in GGBS content. Thus, improving the solubility of raw materials which facilitates enhanced geopolymerization producing supplementary C-A-S-H, N-A-S-H and C-S-H gels [42–44].

From the previous literature, the relationship between molarity and strength is uncertain. Certain studies indicate that higher molarity of NaOH correlates with increased compressive strength whereas other studies has shown that elevated NaOH concentration may result in decreased compressive strength [45,46]. Source of aluminosilicate is a critical factor in determining the optimal NaOH concentration [47]. 8 Molar alkaline activators have exhibited higher compressive strength. In this study, increasing molarity resulted in decreased compressive strength, with the exception of 12 molar NaOH. Total amount of alkaline solution in the system is directly proportional to Al/B ratio. The maximum compressive strength was determined for Al/B ratio of 0.45. Increase in Al/B ratio results an inconsistent pattern in compressive strength. Excess alkaline activator contributed to an increased water content in the mixture, which inhibited geopolymerization[48]

Figure 4 shows the S/N ratio (mean) for various SS/SH ratios, the result doesn't follow a consistent pattern. Certain literature indicates that increased silicates due to increased SS/SH ratio leads to an increased strength whereas others indicate the opposite, reduction in strength [49,50]. In the present study, SS/SH ratio of 1 exhibited superior performance. From the mean S/N ratios for binder to sand ratio of 1: 1.5 has resulted in enhanced compressive strength. Increase in B/S ratio generally leads to decrease in the compressive strength which might be due to insufficient aluminosilicate [51] as its associated with a decrease in binder content and an increase in fine aggregate content.

Table 5 presents the results of the ANOVA for the compressive strength. The SS/SH, Al/B ratio, and binder materials together contributed to 84.41% and 87.07% of the strength at 7 and 28 days, respectively. The optimization of individual factors for compressive strength involves a geopolymer mortar mix which comprises 60% GGBS, 20% MK and RM, with 8 Molar NaOH, an Al/B ratio of 0.45, an SS/SH ratio of 1.5, and a B/S ratio of 1:1.5.

Table 5. ANOVA analysis for compressive strength

Factors	CS-07 Days				CS-28 Days			
	SS _m	SS _t	Mean SS	% Contribution	SS _m	SS _t	Mean SS	% Contribution
Ternary Mix	116.28	422.08	38.76	27.55	63.88	256.44	21.29	24.91
Molarity	36.84		12.28	8.73	19.32		6.44	7.53
Al/B	118.8		39.6	28.15	72.24		24.08	28.17
SS/SH	121.16		40.39	28.71	87.16		29.05	33.99
B/S	29		9.67	6.87	13.84		4.61	5.4
Total	422.08			100	256.44			100

4.3 Setting Time

The initial and final setting times of the 16 mixes are shown in Figure 5. Ternary Mixes (M9-M12) containing 60% RM have higher setting times and ternary mixes (M1-M4) with 60 % GGBS have the least setting times, which is in confirmatory with the literature [51]. Prolonged setting time can be attributed to an increase in the RM Content, which results in extended setting time[52] along with higher Al/B ratio[17]. An increase in the CaO owing to increased GGBS shortens the setting times[53] and higher B/S ratio which reduces the binder quantity.

S/N ratios (mean) for the initial and final setting times decreased with increase in the GGBS content (A3, A2 - A4 - A1) which is in line with previous studies[54,55]. The accessible calcium in GGBS provides heterogeneous nucleation sites, expediting the GPZ and leading to a reduced setting time. Consequently, GGBS engages with the alkaline solution more rapidly [53] [56]. The elevated alkalinity of Bayer's red mud may influence the reaction kinetics, potentially resulting in an extended setting time.[57]. From Figure 4, the setting times increased with increase in NaOH Molarity, except for the final setting time which slightly decreases after 12 Molar, which is in line with the previous studies[38] . An increase in Al/B ratio from 0.45 to 0.5 resulted in enlarged setting times[58]. However, this effect from 0.5 to 0.6 is not significant. Increase in the AL/B ratio results in dilution of the alkaline solution, potentially leading to a slowdown in the

geopolymerization and in delay of the setting time [59]. Increase in the SS/SH ratio results in increased setting times upto a threshold ratio of 2, after which a substantial decline is observed at the ratio of 2.5. An increased SS/SH and Al/B ratio enhances the soluble Na^+ ions, thereby facilitating geopolymerization and lower setting times [60]. Changing the B/S ratio from 1:2.5 to 1:1.5 led to increase in the setting times which can be due to the increased binder quantity.

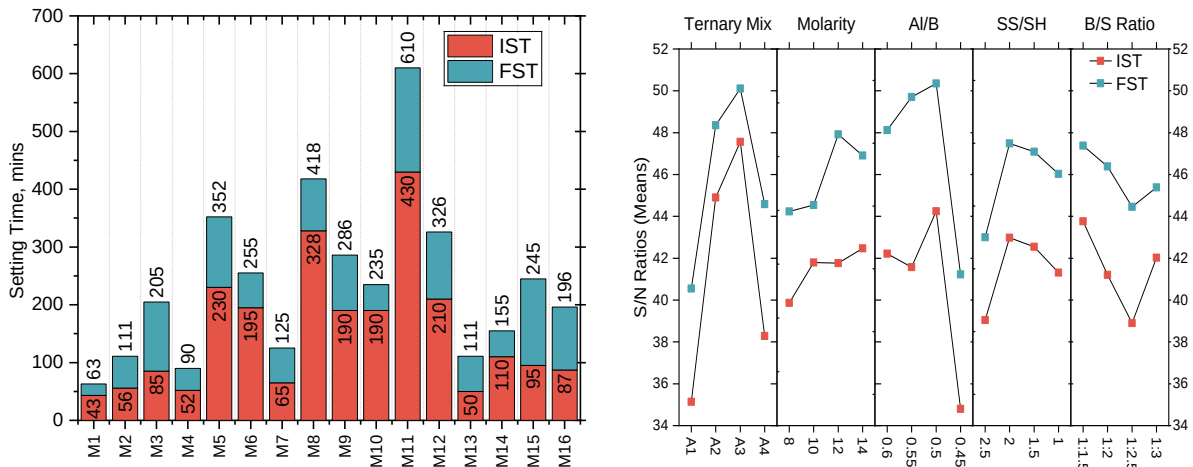


Fig. 5. Setting times of mortars and S/N ratios (means)

Table 6 demonstrates that the Ternary Binder materials is the primary factor influencing the setting times, contributing to 67.49% and 51.85% of the variation in the initial and final setting times, respectively. Al/B ratio is the second most important factor. The optimal mix for setting times is Mix: A3 - 60% RM and 20% GGBS and MK, Molarity: 12, Al/B ratio: 0.45, SS/SH: 2 and B/S ratio: 1:1.5.

Table 6. ANOVA analysis for initial and final setting times

Factors	Initial Setting Time				Final Setting Time			
	SS_m	SS_t	Mean SS	% Contribution	SS_m	SS_t	Mean SS	% Contribution
Ternary Mix	396.8	587.92	132.27	67.49	216.28	417.12	72.09	51.85
Molarity	15.08		5.03	2.56	38.68		12.89	9.27
Al/B	89.68		29.89	15.25	93.56		31.19	22.43
SS/SH	37.32		12.44	6.35	49.44		16.48	11.85
B/S	49.04		16.35	8.34	19.16		6.39	4.59
Total	587.92			100	417.12			100

4.4 Flowability

Flowability test was used to determine the workability of the geopolymer mortars. For mixes M4, M5, M7, M12, M14- M16 the spread diameter is in excess of the base plate, which is reported as 150%. The least flowability of 25% has been determined for Mix M8. The S/N ratios of flowability as shown in Figure 6 indicate ternary mix with equal proportions of all the binders exhibits the highest flow characteristics, whereas mixes with higher percentages of RM display the least flow. An increase in RM content leads to decreased workability, likely due to the high surface area of RM, which raises water demand [61]. The influence of molarity on the S/N ratio is to enhance the flow up to 12 Molar and then diminish at 14 Molar, although this change is not significant. An increase in NaOH molarity resulted in enhanced flow. An elevated molarity of NaOH results in increase of OH^- ions concentration, hence accelerating dissolution and increasing flow[62]. The S/N ratios for flow, plotted alongside the Al/B ratios, indicate a random distribution of performance. The flowability has not decreased with the decrease in the Al/B ratio, which is consistent with previous studies [63,64]. This is due to the complicated action of binders, which effect the flowability [65]. Mixes with Al/B ratio of 0.5 are stiffer compared to mixes with other Al/B ratios. The literature indicates an absence of identifiable trends regarding the influence of the SS/SH ratio. Research

indicates that flowability increases as the SS/SH ratio decreases, and conversely, decreases with an increase in the SS/SH ratio[66,67]. The current study indicates that a reduction in SS/SH from 2.5 to 2 results in a significant decrease in flow, while further reductions lead to an increase in flow. The increase in binder quantity led to an increase in flow, with a slight variation observed at a binder-to-sand ratio of 1:2. These variations can be attributed to the enhanced geopolymerization resulting from the increased availability of aluminosilicate materials[68]. Binder content influences inter-particle interactions and solidification, thereby impacting slump values.[38]

Table 7. ANOVA analysis for flowability

Factors	Flowability			
	SS _m	SS _t	Mean SS	% Contribution
Ternary Mix	35	403.76	11.67	8.67
Molarity	17.72		5.91	4.39
Al/B	137.44		45.81	34.04
SS/SH	113.56		37.85	28.13
B/S	100.04		33.35	24.78
Total	403.76			100

From Table 7, it can be concluded that the Al/B, SS/SH, and B/S ratios are the major factors influencing flowability. Al/B ratio is the primary factor influencing 34.04% of the variation. The optimal mix for setting times is Mix: A4 – 33.33% RM, GGBS and MK, Molarity: 12, Al/B ratio: 0.55, SS/SH: 2.5 and B/S ratio: 1:1.5.

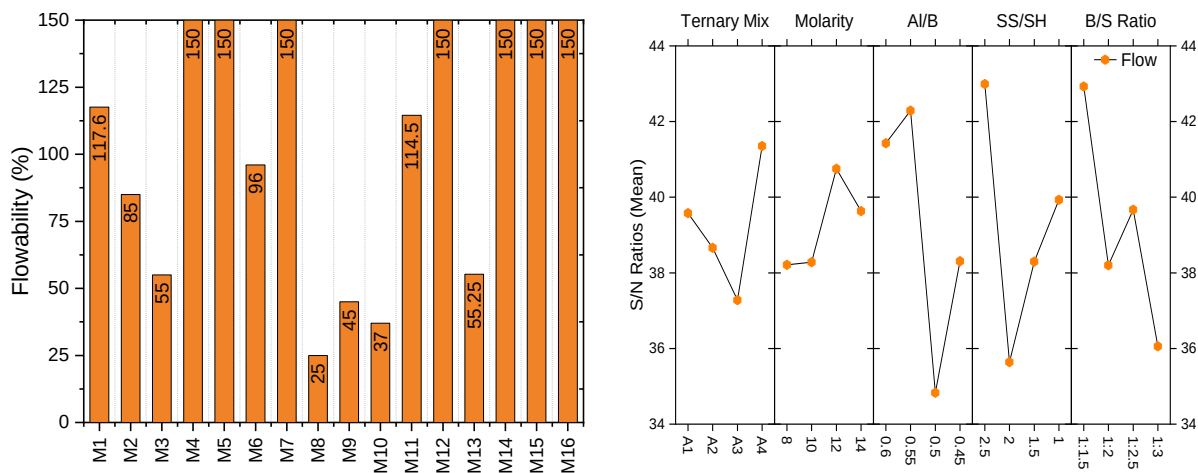


Fig. 6. Flowability of Mortars and S/N Ratios (Mean)

4.5 Multi Response Optimization by Taguchi Grey Relational Analysis

The optimal factor level combinations for each factor, that demonstrated superior performance for each of the response variables are identified on the basis of S/N ratios. Confirmatory experiments based on Taguchi Analysis were conducted for each individual response variable, and the results are shown in Table 8. The compressive strength of 51.94 MPa achieved after 28 days is 5.81% greater than the maximum strength determined using the L16 orthogonal array mixes. Additionally, this mix showed a shorter setting time of 25 minutes, which necessitates multi-factor optimization using the Taguchi-GRG method.

S/N ratios were normalized and transformed into Grey Relational Coefficients (GRCs) for multi-response comparison and the results are tabulated in Table 9 to identify optimal factor levels under different optimization objectives. Multi-response optimization was carried out using three different weights as shown in Table 10 to prioritize compressive strength, workability and balanced overall performance. Grey Relational Grades (GRGs) of the different optimization criteria are shown in Table 11.

Table 8. Taguchi - Optimum levels of each factor for response variables

Factors	Flowability	IST	FST	CS-07 Days	CS-28 Days
Ternary Mix	A4	A3	A3	A1	A1
Molarity	12	14	12	8	8
Al/B	0.55	0.5	0.5	0.45	0.45
SS/SH	2.5	2	2	1	1
B/S Ratio	1:1.5	1:1.5	1:1.5	1:1.5	1:1.5
Confirmatory Experiment	150 %	96 mins	158 mins	46.54 MPa	51.94 MPa

Table 9. Grey relational coefficients based on S/N ratios for the geopolymer mortar

Mix No	Normalized S/N Ratios						Delta				GRC				
	Flow	IST	FST	CS-07 Days	CS-28 Days	Flow	IST	FST	CS-07 Days	CS-28 Days	Flow	IST	FST	CS-07 Days	CS-28 Days
M1	0.86	0.00	0.00	0.70	0.84	0.14	1.00	1.00	0.30	0.16	0.79	0.33	0.33	0.62	0.76
M2	0.68	0.12	0.25	0.66	0.19	0.32	0.89	0.75	0.34	0.81	0.61	0.36	0.40	0.59	0.38
M3	0.44	0.30	0.52	0.32	0.66	0.56	0.70	0.48	0.68	0.34	0.47	0.42	0.51	0.42	0.60
M4	1.00	0.08	0.16	1.00	1.00	0.00	0.92	0.84	0.00	0.00	1.00	0.35	0.37	1.00	1.00
M5	1.00	0.73	0.76	0.51	0.67	0.00	0.27	0.24	0.49	0.33	1.00	0.65	0.67	0.51	0.60
M6	0.75	0.66	0.62	0.46	0.62	0.25	0.34	0.38	0.55	0.39	0.67	0.59	0.57	0.48	0.57
M7	1.00	0.18	0.30	0.73	0.77	0.00	0.82	0.70	0.27	0.23	1.00	0.38	0.42	0.65	0.68
M8	0.00	0.88	0.83	0.00	0.00	1.00	0.12	0.17	1.00	1.00	0.33	0.81	0.75	0.33	0.33
M9	0.33	0.65	0.67	0.39	0.41	0.67	0.36	0.33	0.61	0.59	0.43	0.59	0.60	0.45	0.46
M10	0.22	0.65	0.47	0.07	0.54	0.78	0.36	0.53	0.94	0.46	0.39	0.59	0.49	0.35	0.52
M11	0.85	1.00	1.00	0.10	0.24	0.15	0.00	0.00	0.90	0.76	0.77	1.00	1.00	0.36	0.40
M12	1.00	0.69	0.72	0.49	0.17	0.00	0.31	0.28	0.51	0.83	1.00	0.62	0.64	0.49	0.38
M13	0.44	0.07	0.25	0.83	0.69	0.56	0.93	0.75	0.17	0.31	0.47	0.35	0.40	0.75	0.62
M14	1.00	0.41	0.40	0.50	0.67	0.00	0.59	0.60	0.51	0.33	1.00	0.46	0.45	0.50	0.60
M15	1.00	0.34	0.60	0.61	0.57	0.00	0.66	0.40	0.39	0.44	1.00	0.43	0.55	0.56	0.54
M16	1.00	0.31	0.50	0.11	0.67	0.00	0.69	0.50	0.89	0.33	1.00	0.42	0.50	0.36	0.60

Table 10. Weighting factors for response variables under different optimization criteria's

Optimization Criteria	Flowability	IST	FST	CS-07 Days	CS-28 Days
Balanced Overall Performance	0.2	0.2	0.2	0.2	0.2
Compressive Strength	0.1667	0.1667	0.1666	0.15	0.35
Workability	0.5	0.125	0.125	0.125	0.125

Mix M4 has the highest GRG for all the three different optimizations considered. The Mix M4 has the highest reported compressive strength at the age of 7 and 28 days and flow value; nonetheless, it has quick initial and final setting times. Based on Table 12, the mean GRG analysis, distinct optimal combinations of mix parameters were identified for different optimization criteria. For balanced overall performance, the highest mean GRG was obtained at Ternary mix A2 - 60% MK, 20% GGBS, and 20%RM, 14 molar NaOH, Al/B ratio of 0.6, SS/SH ratio-1, and B/S ratio of 1:1.5, indicating a mix expected to provide balanced flowability, setting behavior, and strength

development. Under the compressive strength criteria, the optimal combination corresponded to Ternary mix– A2 - 60% MK, 20% GGBS, and 20%RM, 14 molar NaOH 14, Al/B ratio–0.45, SS/SH ratio–1, and B/S ratio of 1:1.5, suggesting enhanced compressive strength performance.

Table 11. GRG's under different optimization criteria's

Mix Designation	Balanced Overall Performance	Compressive Strength	Workability
M1	0.57	0.60	0.65
M2	0.47	0.45	0.52
M3	0.48	0.50	0.48
M4	0.75	0.79	0.84
M5	0.69	0.67	0.80
M6	0.57	0.57	0.61
M7	0.63	0.64	0.77
M8	0.51	0.48	0.44
M9	0.51	0.50	0.48
M10	0.47	0.48	0.44
M11	0.71	0.65	0.73
M12	0.63	0.58	0.77
M13	0.52	0.53	0.50
M14	0.60	0.60	0.75
M15	0.62	0.60	0.76
M16	0.58	0.59	0.74

Table 12. Mean GRG response values for different optimization criteria's

Optimization Criteria	Parameter	Level1	Level2	Level3	Level4
Balanced Overall Performance	Ternary Mix	0.57	0.60*	0.58	0.58
	Molarity	0.57	0.53	0.61	0.61*
	Al/B	0.61*	0.6	0.53	0.59
	SS/SH	0.6	0.55	0.55	0.61*
	B/S	0.68*	0.55	0.54	0.54
Strength	Ternary Mix	0.59	0.59*	0.55	0.58
	Molarity	0.58	0.53	0.60	0.61*
	Al/B	0.60	0.58	0.52	0.61*
	SS/SH	0.61	0.53	0.56	0.62*
	B/S	0.68*	0.55	0.54	0.54
Workability	Ternary Mix	0.62	0.66	0.60	0.69*
	Molarity	0.61	0.58	0.68	0.70*
	Al/B	0.68	0.71*	0.54	0.64
	SS/SH	0.73*	0.55	0.61	0.67
	B/S	0.78*	0.59	0.62	0.57

* Optimum Values

For the workability criteria, the highest mean GRG was associated with Ternary mix–A4-33.33% of RM, GGBS and MK, NaOH molarity–14, Al/B ratio–0.55, SS/SH ratio–2.5, and B/S ratio of 1:1.5, indicating improved workability characteristics. The higher mean GRG implies that the mix prepared with these levels of mix parameters would perform more favorably with respect to all the evaluated properties of geopolymer concrete. Despite variations in the optimal levels of several parameters across the three criteria, 14 M NaOH molarity and B/S ratio of 1:1.5 consistently emerged as common optimal parameters on the overall performance of the geopolymer mortar irrespective of the selected optimization objective.

6. XRD Analysis

Among the 16 Taguchi Mixes, M4 and M8, which demonstrated the highest and lowest compressive strength respectively, were analyzed using XRD spectra as shown in Figure 7. The XRD spectra revealed crystalline phases and an amorphous component. The predominant crystalline phases derived from the raw precursor materials, including quartz (SiO_2), hematite (Fe_2O_3), and minor gibbsite or mullite, remained evident in the geopolymer mortar mixes, signifying the retention of unreacted particles from the binder materials during alkali activation. Alongside these remaining phases, new diffraction characteristics linked to geopolymerization products were detected. A wide, diffuse amorphous peak ranging from roughly 20° to 35° 2θ was observed in all geopolymer mixtures, confirming the development of alkali-activated aluminosilicate gel phases. This indistinct area is usually attributed to the existence of C-(A)-S-H and/or N-A-S-H type gels generated during the dissolution and polycondensation of aluminosilicate species. Additionally, faint crystalline reflections associated with calcium-bearing phases, including calcite and calcium silicate hydrate-related phases, were seen, especially in mixtures with elevated GGBS content. The presence of remnant crystalline phases alongside newly generated amorphous aluminosilicate gels validates the heterogeneous characteristics of the geopolymerization process in the examined systems.

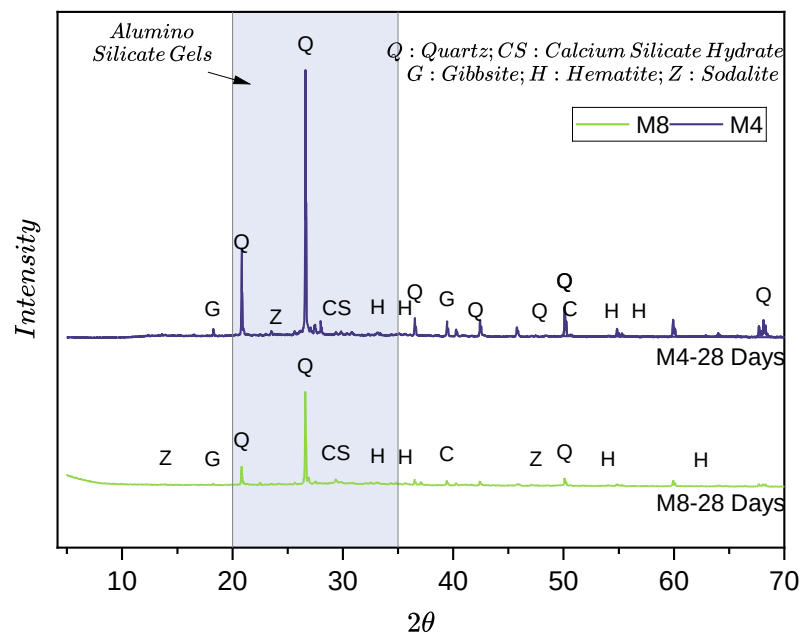


Fig. 7. XRD Spectra of GPM mixes at 28 days ambient curing

7. Conclusions

The integrated Taguchi and GRA methods were successfully used to develop an optimal geopolymer mortar mix that has superior performance in terms of compressive strength, flowability and setting times. This method aids in studying the effects of different factors and their corresponding levels on the desired response variables, and helps in identifying the optimal mix combinations which reduces the cost and time of experiments. The outcomes of the study are as follows:

- The ANOVA results of S/N ratios reveal SS/SH and Al/B ratio as significant factors affecting the flowability and compressive strength. The ternary mix combination of RM, GGBS and MK is the single most contributing factor for the setting times. From the Taguchi - GRA approach binder to sand ratio is the most influential factor when all the response variables are considered.
- The presence of GGBS accelerated the geopolymerization process, leading to reduced setting time and improved early-age compressive strength, while the inclusion of RM and MK played a crucial role in maintaining adequate workability and controlled setting behavior.

- The comparison of the XRD spectra of the binder materials RM, MK and GGBS with those of the ternary geopolymer mixes revealed the synthesis aluminosilicates which enhances compressive strength development. The heightened intensity of the amorphous halo in the ternary mixes with elevated GGBS concentration indicated increased gel formation, correlating with greater compressive strength at both early and later ages under ambient curing conditions.
- The Taguchi S/N ratio analysis revealed that the ternary mix containing 60% GGBS and 20% MK, 20% RM exhibited superior compressive strength performance. The Taguchi-GRG results for balanced overall performance and compressive strength identified the ternary mix comprising 60% MK and 20% GGBS, 20% RM as optimal, reflecting improved combined behavior in terms of workability, setting characteristics, and strength. This illustrates that the ideal proportions of ternary binders vary based on whether the performance objective emphasizes strength or overall balanced performance.
- The mean Grey Relational Grade (GRG) study indicated that various parameter combinations are ideal for achieving balanced performance, compressive strength, and workability. Nonetheless, a NaOH molarity of 14 M and a binder-to-sand ratio of 1:1.5 consistently proved to be ideal parameters across all optimization criteria.

The synthesized geopolymer mortar may serve as a substitute for conventional cement mortar and for precast applications. It also improves the visual appeal, attributed to its inherent red color.

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References

- [1] Ghanbari M, Hadian AM, Nourbakhsh AA, MacKenzie KJD. Modeling and optimization of compressive strength and bulk density of metakaolin-based geopolymer using central composite design: A numerical and experimental study. *Ceram Int* 2017;43:324–35. <https://doi.org/10.1016/j.ceramint.2016.09.159>.
- [2] Pobłocki K, Pawlak M, Drzeżdżon J, Gawdzik B, Jacewicz D. Clean production of geopolymers as an opportunity for sustainable development of the construction industry. *Science of The Total Environment* 2024;928:172579. <https://doi.org/10.1016/j.scitotenv.2024.172579>.
- [3] Matsimbe J, Dinka M, Olukanni D, Musonda I. Geopolymer: A Systematic Review of Methodologies. *Materials* 2022;15:6852. <https://doi.org/10.3390/ma15196852>.
- [4] He J, Jie Y, Zhang J, Yu Y, Zhang G. Synthesis and characterization of red mud and rice husk ash-based geopolymer composites. *Cem Concr Compos* 2013;37:108–18. <https://doi.org/10.1016/j.cemconcomp.2012.11.010>.
- [5] Central Pollution Control Board ND. Guidelines for Handling and Management of Red Mud Generated from Alumina Plants 2023.
- [6] Li Z, Dong H, Wang Y, Men J, Wang J, Zhao X, et al. Valorization of Alkali-Thermal Activated Red Mud for High-Performance Geopolymer: Performance Evaluation and Environmental Effects. *Buildings* 2025;15:2471. <https://doi.org/10.3390/buildings15142471>.
- [7] Mounika G, Ramakrishna U, Naresh Kumar Reddy G, Suresh Kumar K. A review on effect of red mud on properties of alkali activated materials (AAMs) and geopolymers (GPs). *Mater Today Proc* 2023. <https://doi.org/10.1016/j.matpr.2023.03.446>.
- [8] Anilkumar undefined, Sreeekeshava KS, Bhargavi C. Studies on Optimization of Fly Ash, GGBS and Precipitated Silica in Geopolymer Concrete. *Construction Materials* 2025;5:29. <https://doi.org/10.3390/constrmater5020029>.
- [9] Jin Z, Chen S, He X, Su Y, Qi H, Wang C, et al. Enhancing the mechanical properties of hemihydrate phosphogypsum-ground granulated blast-furnace slag composite binder by incorporating carbide slag as an alkaline modifier. *Mater Today Commun* 2025;48:113509. <https://doi.org/10.1016/j.mtcomm.2025.113509>.
- [10] Agnihotri A, Ramana P V. GGBS: Fly-Ash evaluation and mechanical properties within high strength concrete. *Mater Today Proc* 2022;50:2404–10. <https://doi.org/10.1016/j.matpr.2021.10.257>.

- [11] Reddy MS, Dinakar P, Rao BH. A review of the influence of source material's oxide composition on the compressive strength of geopolymer concrete. *Microporous and Mesoporous Materials* 2016;234:12–23. <https://doi.org/https://doi.org/10.1016/j.micromeso.2016.07.005>.
- [12] Nakashima H, Kurumisawa K. Reaction and microstructure of metakaolin-based geopolymer. *J Sustain Cem Based Mater* 2025;1–14. <https://doi.org/10.1080/21650373.2025.2531266>.
- [13] Zheng J, Li X, Bai C, Zheng K, Wang X, Sun G, et al. Rapid fabrication of porous metakaolin-based geopolymer via microwave foaming. *Appl Clay Sci* 2024;249:107238. <https://doi.org/10.1016/j.clay.2023.107238>.
- [14] H.M. T, Unnikrishnan S. Mechanical Strength and Microstructure of GGBS-SCBA based Geopolymer Concrete. *Journal of Materials Research and Technology* 2023;24:7816–31. <https://doi.org/10.1016/j.jmrt.2023.05.051>.
- [15] Ma Z, Dan H-C, Tan J, Li M, Li S. Optimization Design of MK-GGBS Based Geopolymer Repairing Mortar Based on Response Surface Methodology. *Materials* 2023;16:1889. <https://doi.org/10.3390/ma16051889>.
- [16] Wang L, Geddes DA, Walkley B, Provis JL, Mechtcherine V, Tsang DCW. The role of zinc in metakaolin-based geopolymers. *Cem Concr Res* 2020;136:106194. <https://doi.org/10.1016/j.cemconres.2020.106194>.
- [17] Jia K, Zhou Z, Singh SV, Wang C. A review of the engineered treatment of red mud: Construction materials, metal recovery, and soilization revegetation. *Results in Engineering* 2024;24:102927. <https://doi.org/https://doi.org/10.1016/j.rineng.2024.102927>.
- [18] Song W, Guo T, Han P, Wang X, Ma F, He B. Durability study and mechanism analysis of red mud-coal metakaolin geopolymer concrete under a sulfate environment. *Constr Build Mater* 2023;409:133990. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2023.133990>.
- [19] Liu J, Li X, Lu Y, Bai X. Effects of Na/Al ratio on mechanical properties and microstructure of red mud-coal metakaolin geopolymer. *Constr Build Mater* 2020;263:120653. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2020.120653>.
- [20] Li Z, Gao M, Lei Z, Tong L, Sun J, Wang Y, et al. Ternary cementless composite based on red mud, ultra-fine fly ash, and GGBS: Synergistic utilization and geopolymerization mechanism. *Case Studies in Construction Materials* 2023;19:e02410. <https://doi.org/https://doi.org/10.1016/j.cscm.2023.e02410>.
- [21] Du Z, Sheng S, Guo J. Effect of composite activators on mechanical properties, hydration activity and microstructure of red mud-based geopolymer. *Journal of Materials Research and Technology* 2023;24:8077–85. <https://doi.org/https://doi.org/10.1016/j.jmrt.2023.05.049>.
- [22] Li Z, Gao M, Lei Z, Tong L, Sun J, Wang Y, et al. Ternary cementless composite based on red mud, ultra-fine fly ash, and GGBS: Synergistic utilization and geopolymerization mechanism. *Case Studies in Construction Materials* 2023;19. <https://doi.org/10.1016/j.cscm.2023.e02410>.
- [23] Prusty JK, Pradhan B. Multi-response optimization using Taguchi-Grey relational analysis for composition of fly ash-ground granulated blast furnace slag based geopolymer concrete. *Constr Build Mater* 2020;241:118049. <https://doi.org/10.1016/j.conbuildmat.2020.118049>.
- [24] Hisam MW, Dar AA, Elrasheed MO, Khan MS, Gera R, Azad I. The Versatility of the Taguchi Method: Optimizing Experiments Across Diverse Disciplines. *Journal of Statistical Theory and Applications* 2024;23:365–89. <https://doi.org/10.1007/s44199-024-00093-9>.
- [25] Rajesh MN, Siddaraju C, Prabhu TR, Sachit TS, Suresh R. Machine learning and Taguchi optimization technique for wear rate analysis of Al 2024/ SiC/CNT based hybrid metal matrix composites. *Research on Engineering Structures and Materials* 2025;11:2559–79. <https://doi.org/10.17515/resm2025-810ma0308rs>.
- [26] Hadi MNS, Farhan NA, Sheikh MN. Design of geopolymer concrete with GGBFS at ambient curing condition using Taguchi method. *Constr Build Mater* 2017;140:424–31. <https://doi.org/10.1016/j.conbuildmat.2017.02.131>.
- [27] Sadri N, Baghernejad M, Ghasemi-Fasaei R, Moosavi AA, Hardie AG. Optimizing the adsorption conditions for cadmium and lead using various synthesized adsorbents with the Taguchi method. *International Journal of Environmental Science and Technology* 2026;23. <https://doi.org/10.1007/s13762-025-06921-4>.
- [28] Li Y, Huang L, Gao C, Mao Z, Qin M. Workability and mechanical properties of GGBS-RFBP-FA ternary composite geopolymer concrete with recycled aggregates containing recycled fireclay brick aggregates. *Constr Build Mater* 2023;392:131450. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2023.131450>.
- [29] Jain D, Adhikary S Das, Sarker PK. Novel one-part geopolymer concrete mix design using hybrid Taguchi GRA-PCA optimization approach. *Structures* 2025;75:108805. <https://doi.org/https://doi.org/10.1016/j.istruc.2025.108805>.

- [30] Kumar D, Singh S. Enhancing friction and wear performance in hybrid aluminum composites through grey relational analysis. *Research on Engineering Structures and Materials* 2024;10:943–56. <https://doi.org/10.17515/resm2024.05ma1012tn>.
- [31] Bureau of Indian Standards. Coarse and Fine Aggregate for Concrete-Specification (Third Revision). 2016.
- [32] Prusty JK, Pradhan B. Multi-response optimization using Taguchi-Grey relational analysis for composition of fly ash-ground granulated blast furnace slag based geopolymer concrete. *Constr Build Mater* 2020;241:118049. <https://doi.org/10.1016/j.conbuildmat.2020.118049>.
- [33] Aliabdo AA, Abd Elmoaty AEM, Salem HA. Effect of water addition, plasticizer and alkaline solution constitution on fly ash based geopolymer concrete performance. *Constr Build Mater* 2016;121:694–703. <https://doi.org/10.1016/j.conbuildmat.2016.06.062>.
- [34] Vora PR, Dave U V. Parametric Studies on Compressive Strength of Geopolymer Concrete. *Procedia Eng* 2013;51:210–9. <https://doi.org/10.1016/j.proeng.2013.01.030>.
- [35] Alnkaa AA, Danish A, Yaprak H, Memiş S, Gencel O, Amran M, et al. Performance evaluation and cost analysis of ternary blended geopolymers for sustainable built environment under different curing regimes. *Constr Build Mater* 2023;409:133842. <https://doi.org/10.1016/j.conbuildmat.2023.133842>.
- [36] BJ A, Parthasarathy P. Enhanced approach for geopolymer mix-design and performance evaluation: Integrating hybrid Taguchi-GRA-PCA for improved properties and behavioral insights. *Constr Build Mater* 2024;433:136701. <https://doi.org/10.1016/j.conbuildmat.2024.136701>.
- [37] Ferdous W, Manalo A, Aravinthan T. Bond behaviour of composite sandwich panel and epoxy polymer matrix: Taguchi design of experiments and theoretical predictions. *Constr Build Mater* 2017;145:76–87. <https://doi.org/10.1016/j.conbuildmat.2017.03.244>.
- [38] Ali A, uz Zaman Khan Q, Saqib Mehboob S, Tayyab A, Hayyat K, Khan D, et al. Enhancing multi-objective mix design for GGBS-based geopolymer concrete with natural mineral blends under ambient curing: A Taguchi-Grey relational optimization. *Ain Shams Engineering Journal* 2024;15. <https://doi.org/10.1016/j.asej.2024.102708>.
- [39] Bureau of Indian Standards. Fresh Concrete — Methods of Sampling, Testing and Analysis Part 2: Determination of Consistency of Fresh Concrete. 2018.
- [40] Bureau of Indian Standards. Methods of Sampling and Analysis of Concrete 1959.
- [41] Bureau of Indian Standards. Hardened concrete – Methods of test – Part 1: Testing of strength of hardened concrete – Section 1: Compressive, flexural and split tensile strength (First Revision). 2021.
- [42] Al-Majidi MH, Lampropoulos A, Cundy A, Meikle S. Development of geopolymer mortar under ambient temperature for in situ applications. *Constr Build Mater* 2016;120:198–211. <https://doi.org/10.1016/j.conbuildmat.2016.05.085>.
- [43] Guo S, Ma C, Long G, Xie Y. Cleaner one-part geopolymer prepared by introducing fly ash sinking spherical beads: Properties and geopolymerization mechanism. *J Clean Prod* 2019;219:686–97. <https://doi.org/10.1016/j.jclepro.2019.02.116>.
- [44] Al-Majidi MH, Lampropoulos A, Cundy AB. Steel fibre reinforced geopolymer concrete (SFRGC) with improved microstructure and enhanced fibre-matrix interfacial properties. *Constr Build Mater* 2017;139:286–307. <https://doi.org/10.1016/j.conbuildmat.2017.02.045>.
- [45] Wongpa J, Kiattikomol K, Jaturapitakkul C, Chindaprasirt P. Compressive strength, modulus of elasticity, and water permeability of inorganic polymer concrete. *Mater Des* 2010;31:4748–54. <https://doi.org/10.1016/j.matdes.2010.05.012>.
- [46] Nazari A, Bagheri A, Riahi S. Properties of geopolymer with seeded fly ash and rice husk bark ash. *Materials Science and Engineering: A* 2011;528:7395–401. <https://doi.org/10.1016/j.msea.2011.06.027>.
- [47] Riahi S, Nazari A, Zaarei D, Khalaj G, Bohlooli H, Kaykha MM. Compressive strength of ash-based geopolymers at early ages designed by Taguchi method. *Mater Des* 2012;37:443–9. <https://doi.org/10.1016/j.matdes.2012.01.030>.
- [48] Pavithra P, Srinivasula Reddy M, Dinakar P, Hanumantha Rao B, Satpathy BK, Mohanty AN. A mix design procedure for geopolymer concrete with fly ash. *J Clean Prod* 2016;133:117–25. <https://doi.org/10.1016/j.jclepro.2016.05.041>.
- [49] Dave S V, Bhogayata A. The strength oriented mix design for geopolymer concrete using Taguchi method and Indian concrete mix design code. *Constr Build Mater* 2020;262:120853. <https://doi.org/10.1016/j.conbuildmat.2020.120853>.
- [50] Nath P, Sarker PK. Effect of GGBFS on setting, workability and early strength properties of fly ash geopolymer concrete cured in ambient condition. *Constr Build Mater* 2014;66:163–71. <https://doi.org/10.1016/j.conbuildmat.2014.05.080>.
- [51] Zailani WWA, Abdullah MMAB, Zainol MRRMA, Razak RA, Tahir MFM. Compressive and bonding strength of fly ash based geopolymer mortar. *AIP Conf. Proc.*, vol. 1887, American Institute of Physics Inc.; 2017. <https://doi.org/10.1063/1.5003541>.

- [52] Singh S, Aswath MU, Ranganath R V. Effect of mechanical activation of red mud on the strength of geopolymer binder. Constr Build Mater 2018;177:91–101. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2018.05.096>.
- [53] Mallikarjuna Rao G, Gunneswara Rao TD. Final Setting Time and Compressive Strength of Fly Ash and GGBS-Based Geopolymer Paste and Mortar. Arab J Sci Eng 2015;40:3067–74. <https://doi.org/10.1007/s13369-015-1757-z>.
- [54] El-Hassan H, Ismail N. Effect of process parameters on the performance of fly ash/GGBS blended geopolymer composites. J Sustain Cem Based Mater 2018;7:122 – 140. <https://doi.org/10.1080/21650373.2017.1411296>.
- [55] Xie J, Wang J, Rao R, Wang C, Fang C. Effects of combined usage of GGBS and fly ash on workability and mechanical properties of alkali activated geopolymer concrete with recycled aggregate. Compos B Eng 2019;164:179–90. <https://doi.org/https://doi.org/10.1016/j.compositesb.2018.11.067>.
- [56] Lee WKW, van Deventer JSJ. The effect of ionic contaminants on the early-age properties of alkali-activated fly ash-based cements. Cem Concr Res 2002;32:577–84. [https://doi.org/https://doi.org/10.1016/S0008-8846\(01\)00724-4](https://doi.org/https://doi.org/10.1016/S0008-8846(01)00724-4).
- [57] Mohapatra A, Bose P, Pandit SS, Kumar S, Alex TC. Bulk Utilization of Red Mud in Geopolymer Based Products. Advances in Sciences and Engineering 2020;12:86–91. <https://doi.org/10.32732/ase.2020.12.2.86>.
- [58] Arjun Raj PK, Sarath D, Nagarajan P, Thomas BS. A Simplified Mix Design for GGBS–Dolomite Geopolymer Concrete Using the Taguchi Method. Iranian Journal of Science and Technology, Transactions of Civil Engineering 2024;48:3189–212. <https://doi.org/10.1007/s40996-024-01354-4>.
- [59] Shi C, Day RL. Some factors affecting early hydration of alkali-slag cements. Cem Concr Res 1996;26:439–47. [https://doi.org/https://doi.org/10.1016/S0008-8846\(96\)85031-9](https://doi.org/https://doi.org/10.1016/S0008-8846(96)85031-9).
- [60] Hou L, Li J, Lu Z. Effect of Na/Al on formation, structures and properties of metakaolin based Na-geopolymer. Constr Build Mater 2019;226:250–8. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2019.07.171>.
- [61] Bajpai R, Shrivastava A, Singh M. Properties of fly ash geopolymer modified with red mud and silica fume: a comparative study. SN Appl Sci 2020;2. <https://doi.org/10.1007/s42452-020-03665-3>.
- [62] Malkawi AB, Nuruddin MF, Fauzi A, Almattarneh H, Mohammed BS. Effects of Alkaline Solution on Properties of the HCFA Geopolymer Mortars. Procedia Eng 2016;148:710–7. <https://doi.org/https://doi.org/10.1016/j.proeng.2016.06.581>.
- [63] Shilar FA, Ganachari S V, Patil VB, Nisar KS. Evaluation of structural performances of metakaolin based geopolymer concrete. Journal of Materials Research and Technology 2022;20:3208–28. <https://doi.org/https://doi.org/10.1016/j.jmrt.2022.08.020>.
- [64] Shilar FA, Ganachari S V, Patil VB, Neelakanta Reddy I, Shim J. Preparation and validation of sustainable metakaolin based geopolymer concrete for structural application. Constr Build Mater 2023;371:130688. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2023.130688>.
- [65] Qiu J, Zhao Y, Xing J, Sun X. Fly Ash/Blast Furnace Slag-Based Geopolymer as a Potential Binder for Mine Backfilling: Effect of Binder Type and Activator Concentration. Advances in Materials Science and Engineering 2019;2019. <https://doi.org/10.1155/2019/2028109>.
- [66] Deb PS, Nath P, Sarker PK. The effects of ground granulated blast-furnace slag blending with fly ash and activator content on the workability and strength properties of geopolymer concrete cured at ambient temperature. Materials & Design (1980-2015) 2014;62:32–9. <https://doi.org/https://doi.org/10.1016/j.matdes.2014.05.001>.
- [67] Aliabdo AA, Abd Elmoaty AEM, Salem HA. Effect of water addition, plasticizer and alkaline solution constitution on fly ash based geopolymer concrete performance. Constr Build Mater 2016;121:694–703. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2016.06.062>.
- [68] Yaseri S, Hajiaghahi G, Mohammadi F, Mahdikhani M, Farokhzad R. The role of synthesis parameters on the workability, setting and strength properties of binary binder based geopolymer paste. Constr Build Mater 2017;157:534–45. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2017.09.102>.

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Experimental and numerical investigation of brick masonry behavior considering mortar joint thickness using an RVE approach

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Article Info	Abstract
Article History: Received 13 Oct 2025 Accepted 07 Jan 2026 Keywords: Masonry wall; Representative volume element; Mortar joint thickness; Concrete damage plasticity model; Uniaxial compression	In this study, the authors explore how the thickness of mortar joints has a significant impact on compressive behavior of the masonry of bricks in an integrated numerical modeling and experimental system. The joint thickness of masonry prism nine-course and joint thickness of 5mm, 10mm and 20mm were subjected to uniaxial compression tests. The results show that making the joint thicker i.e. 5mm to 20mm, compressive strength reduces by 34 % and the initial value of 3.5 MPa to 2.3 MPa, and initial elastic modulus decreases by 23 %. The middle area of the 9-course sample was verified as a Representative Volume Element (RVE), which was a successful way of eliminating border influences. Also, a micro modeling model of ABAQUS using the Concrete Damage Plasticity (CDP) model resulted in a high level of accuracy and the differences between experimental values of strength were limited to a range of 4.3 to 8.5%. These findings provide quantitative evidence supporting design code recommendations (e.g., ASTM C1314) to limit mortar bed joint thickness to 10 mm or less, thereby preserving the structural integrity and stiffness of masonry walls.

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1. Introduction

Modern high-rise structures and innovative building technologies require accurate methods for calculating the stress-strain state of structures. This is particularly important for masonry, which exhibits significant structural heterogeneity due to the varying geometric and mechanical properties of bricks and mortar joints. Although brickwork exhibits high compressive strength, its low tensile strength often leads to cracking under loading conditions. This behavior must be carefully considered in design, especially when masonry is regarded as a load-bearing element in complex structures [1, 2].

The mechanical behavior of masonry is significantly influenced by the properties of its components, with mortar playing a key role. Recent studies have established that the mortar ratio and joint thickness are key parameters that determine compressive strength and shear resistance. Optimal load distribution is associated with 15 mm thick joints [5]; However, increasing the joint thickness above 20 mm leads to a reduction in structural integrity as a result of stress concentration [9]. These findings are consistent with previous studies on the influence of mortar type [7], effects at the brick-mortar interface [8], and the critical role of mortar strength in seismic performance [3, 4]. Despite these valuable findings, a systematic quantitative analysis of the effect of joint thickness in a practical range (e.g., 5–20 mm) coupled with a rigorously verified numerical model remains a gap in the literature.

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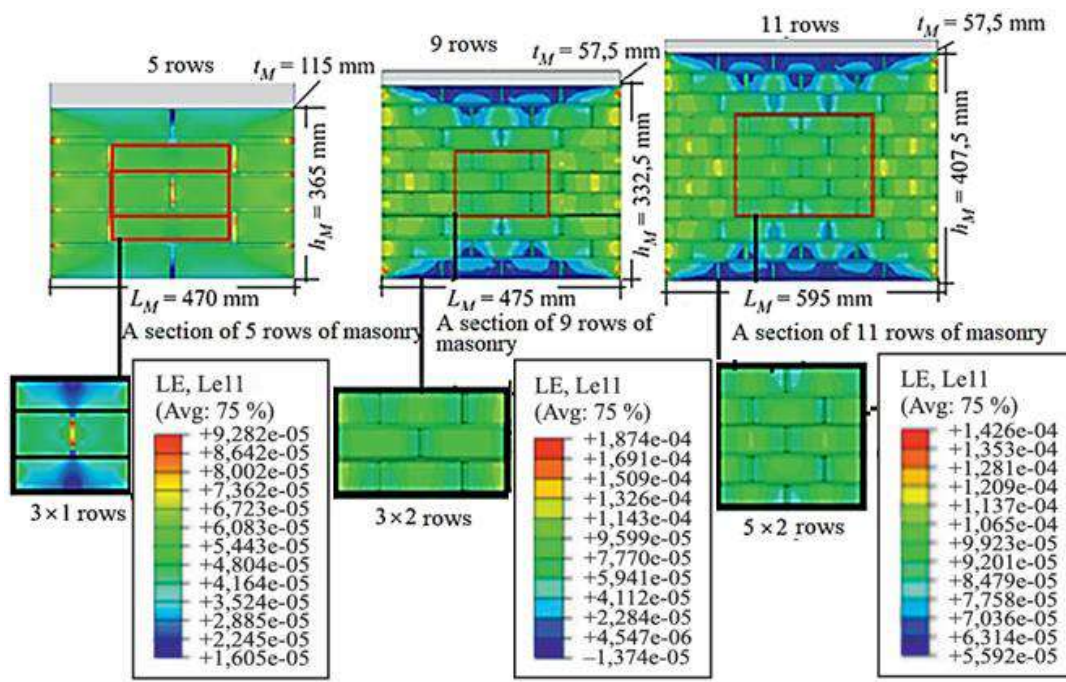


Fig. 1. Isofields of tensile strains in the horizontal direction for masonry of 5, 9, and 11 rows of bricks [1]

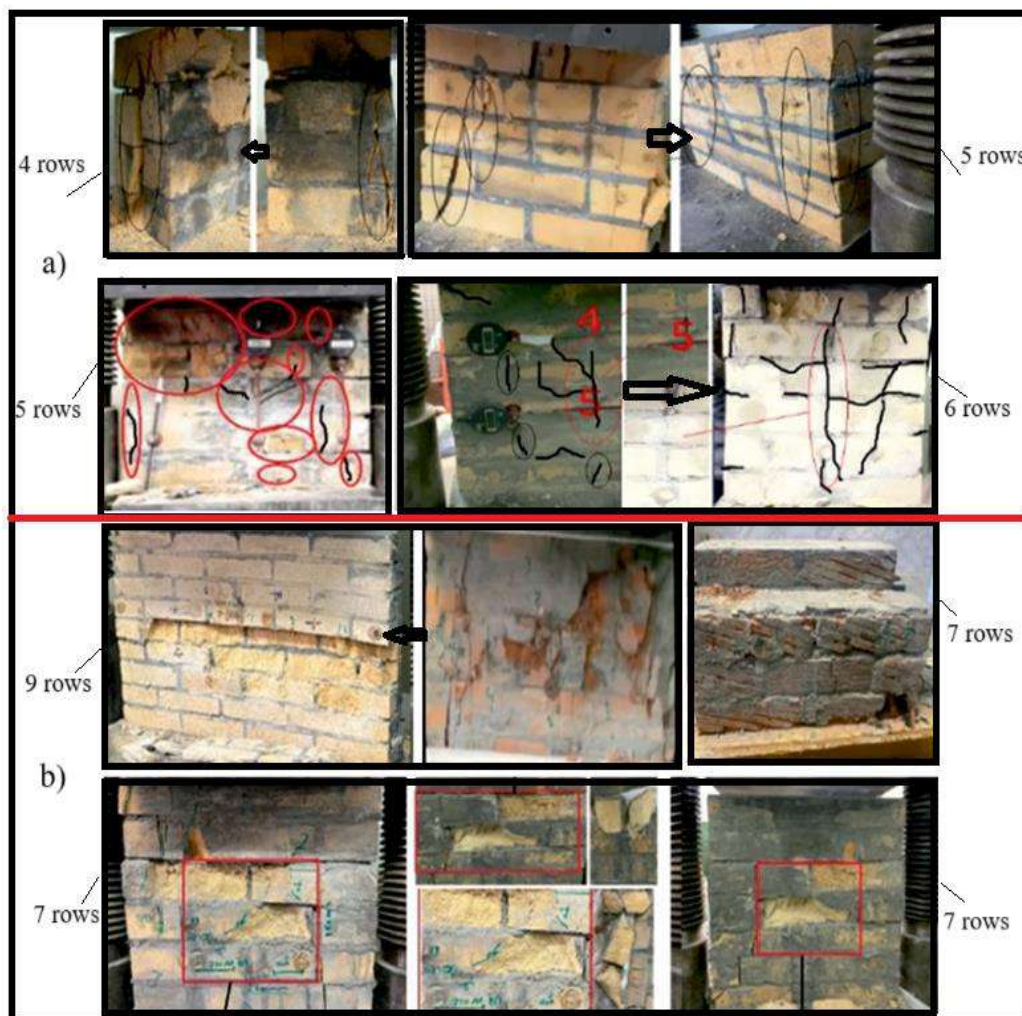


Fig. 2. Type of failure a- 4, 5, and 6 rows, b- 7 and 9 rows

A powerful approach to modeling such heterogeneous materials is multiscale homogenization based on the Representative Volume Element (RVE) [14]. RVE is the smallest volume of a material that effectively represents the overall macroscopic properties. For masonry, this means a fragment containing a sufficient number of bricks and mortar joints to capture the composite behavior while minimizing boundary effects. Consequently, a combined numerical-experimental study [1] justified the choice of a 9-course masonry fragment as a suitable RVE.

Finite element analysis of the strain fields under compression showed that the 9-course configuration minimizes boundary effects (in contrast to 5-course specimens) and ensures strain stabilization in the central zone (Fig. 1), which is confirmed by experimental strain measurements. Moreover, failure mechanism analysis [14, 15] confirmed that unreinforced 9-course masonry exhibits a representative failure pattern, including a combination of mortar damage and vertical cracking (Fig. 2), in contrast to smaller, non-representative specimens.

Independent experimental studies support the critical influence of specimen geometry on strain distribution, a key aspect of the RVE concept. It has been shown that lateral strains at mid-height of 1-meter-high walls are 3-4 times greater than at the boundaries, providing quantitative evidence of a significant strain gradient and the confinement effect exerted by loading platens [24]. The central zone is identified as the region representing true material behavior; accordingly, a 9-brick-high masonry fragment has been established as the minimum effective RVE in previous systematic studies [12, 13]. This size minimizes boundary effects, ensures strain stabilization in a central zone (approximately three bricks in height), and complies with code recommendations for deformation measurement bases. Thus, while smaller assemblies are used in practice, a 9-brick-high fragment is necessary to accurately characterize the macroscopic behavior of masonry under compression. Thus, despite existing research, there is a lack of studies that provide a systematic quantitative analysis of joint thickness effects across a practical range (5–20 mm) using an experimentally validated RVE-based micromodel. This study aims to fill this gap.

1.1 Novelty and Objectives of the Study

Based on these findings, this study fills the identified gap by providing a comprehensive experimental and numerical investigation of the effect of mortar joint thickness on the compressive behavior of masonry. The novelty of this work is threefold:

- **Experimental Validation of the Methodology:** Conducting a comprehensive experimental program on masonry fragments approved as RVE (9 courses) with systematically varied mortar joint thicknesses (5, 10, and 20 mm) to determine their compressive strength, elastic modulus, and deformation characteristics.
- **Development and Verification of the Numerical Model:** Developing a detailed micromodel in ABAQUS using the Concrete Damage Plasticity (CDP) model and rigorously validating it through direct comparison with the obtained experimental data for all joint thicknesses studied.
- **Quantitative Analysis and Practical Recommendations:** Conducting a direct quantitative comparative analysis of experimental and numerical results to assess the influence of mortar joint thickness and, based on this, formulating definitive, quantitative guidelines for optimal mortar joint thickness for design practice.

The goal of this study is to create a verified tool for numerical prediction and provide reliable, quantitatively substantiated recommendations for the design of high-performance masonry structures.

2. Materials and Methods

2.1 Materials

2.1.1. Brick Units

The clay bricks used in this study had standard Iraqi dimensions of 230 × 120 × 75 mm (length × width × height). Their physical and mechanical properties are summarized in Table 1.

2.1.2 Mortar

Ordinary Portland Cement (OPC) as a binder and local sand for the fine aggregate were used to prepare the mortar specimens. This mortar is notable for its strength, durability, and resistance to atmospheric and chemical deterioration. It is widely used for mortar and stucco work in stonework, block, and brick masonry. Mortar mix consisting of cement: sand proportions 1:3 was prepared manually. A total of nine specimens were fabricated, with a w/c ratio of 0.46 and mortar ratios: 1:3. The specimens were tested at 28 days. All specimens were cured in a controlled environment at a temperature of $22 \pm 2^\circ\text{C}$ and relative humidity above 95% until testing. The procedures outlined in Fig. 3 were consistently followed throughout the study. The selection of specimen numbers followed the specifications outlined in ASTM C109.[23]. All properties are shown in Table 1.

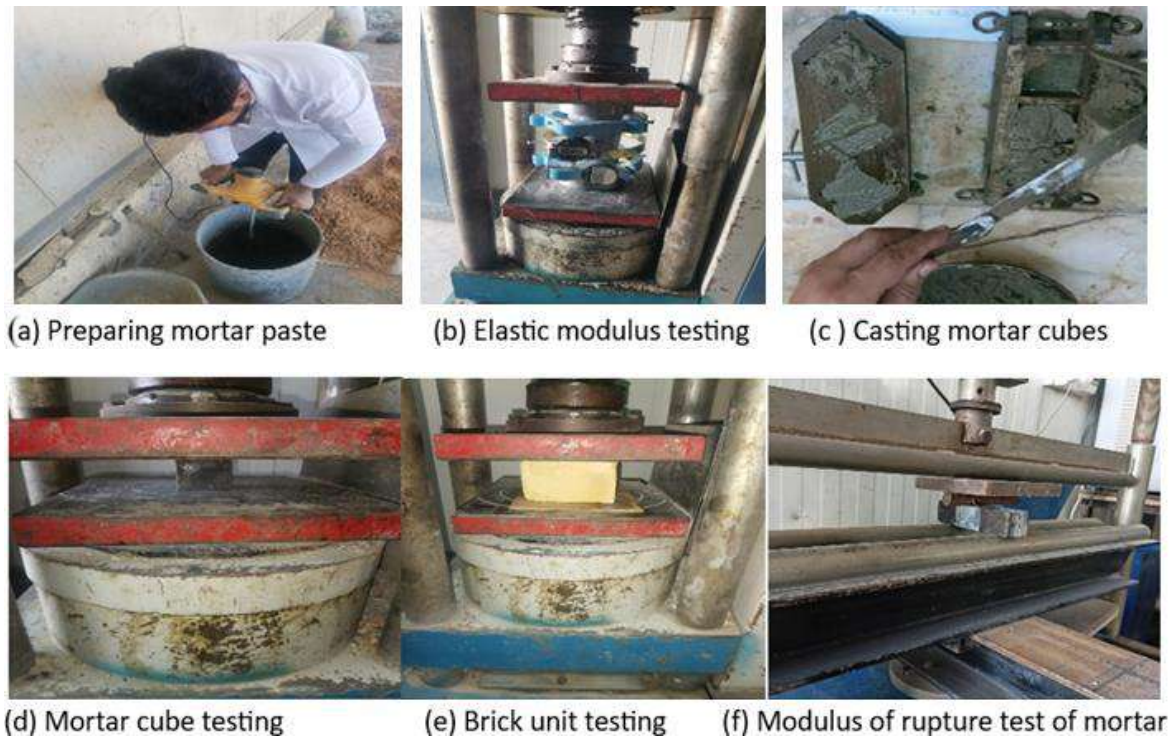


Fig. 3. Preparation and testing of mortar specimens (cube/cylinder)

Table 1. Physical and mechanical properties of constituent materials

Property	Mortar (1:3)	Masonry Unit (Brick)	Test Standard
Density (kg/m^3)	1850	1950	ASTM C642
Water Absorption (%)	8.0	8.5	ASTM C140
Compressive Strength (MPa)	5.0 ¹	6.6 ²	ASTM C109 / EN 772-1
Flexural/Tensile Strength (MPa)	1.1	1.9	ASTM C348 / ASTM C67
Elastic Modulus (MPa)	4500	3750	ASTM C469

2.2 Methods

2.2.1 Sample Preparation

Masonry specimens were fabricated with three distinct mortar joint thicknesses: 5 mm, 10 mm, and 20 mm, applied uniformly to both bed (horizontal) and head (vertical) joints. For each thickness, two masonry walls were constructed using nine rows of standard bricks. The cross-sectional plan dimensions remained constant at 950 mm $\times$ 120 mm, while the total height varied due to the cumulative mortar joint thickness: 715 mm (5 mm joints), 755 mm (10 mm joints), and 835 mm (20 mm joints). Joint thickness was meticulously controlled to ensure uniformity.

Before construction, bricks were immersed in water for 24 hours to minimize water absorption from fresh mortar during curing. After immersion, bricks were arranged on a flat surface, and mortar was applied to achieve specified joint thicknesses. A stretcher bond pattern was used for wall assembly. Specimens were cured under controlled conditions for 28 days in accordance with standard cementitious curing practices (Fig. 4). A recognized limitation of this experimental campaign is the use of two specimens per configuration, which restricts detailed statistical analysis. However, the primary focus was on identifying clear trends and capturing failure mechanisms for subsequent numerical validation, a common approach in exploratory micromechanical studies.



Fig. 4. Construction and setup of walls to test

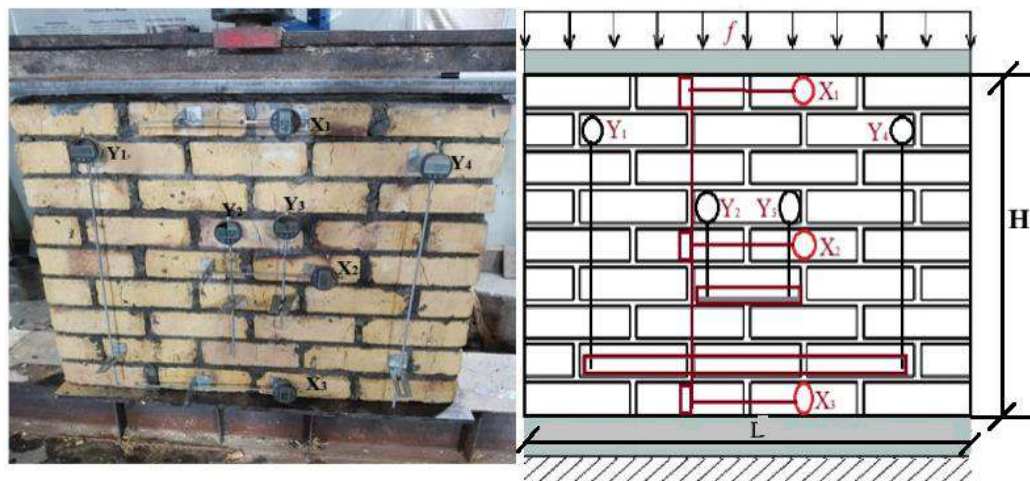


Fig. 5. General view of the compression test setup for masonry and the arrangement diagram of the instruments

2.2.2. Compressive Stress-Strain Test of The Specimen

Each specimen underwent uniaxial compression testing in a Universal Testing Machine (UTM) to determine its compressive strength and mechanical response of the masonry. The loads applied to the specimens were increased, and the strain at each level of 10 kN was measured with dial gauges with a measurement accuracy of 0.01 mm. The longitudinal and lateral strains were recorded at the same locations in the central parts of the masonry walls, which is equal to the representative volume element (RVE), and at the peripheries, which is equal to the boundary zones. Figure 5 gives

a general illustration of the experimental situation, giving the geometrical features of the brickwork samples and showing a schematic of the measuring device. Each of the given mortar joint thicknesses was tested on six specimens, two specimens in each thickness increment. All the specimens were tested on compressive strength through measuring the highest load before it failed. To define the mechanical response and failure mode of the masonry walls during axial loading, the resulting stress strain trajectories of the individual mortar joint thicknesses were discussed to explain the behavior of the masonry walls.

At each stage of loading, the average relative elongations (deformations) in vertical and horizontal directions are calculated from the readings of the gauges(sensors) according to the formulas

$$\varepsilon_x = \frac{\Delta_x}{b_x}, \varepsilon_y = \frac{\Delta_y}{b_y} \quad (1)$$

Analogous to the elastic constants of a homogeneous material, we introduce the initial deformation modulus of the masonry E_0 :

$$E_0 = \frac{F_y}{\varepsilon_y A} \quad (2)$$

Compressive strength of an individual tested sample f_k , was calculated using the formula:

$$f_k = \frac{f_{fai}}{A} \quad (3)$$

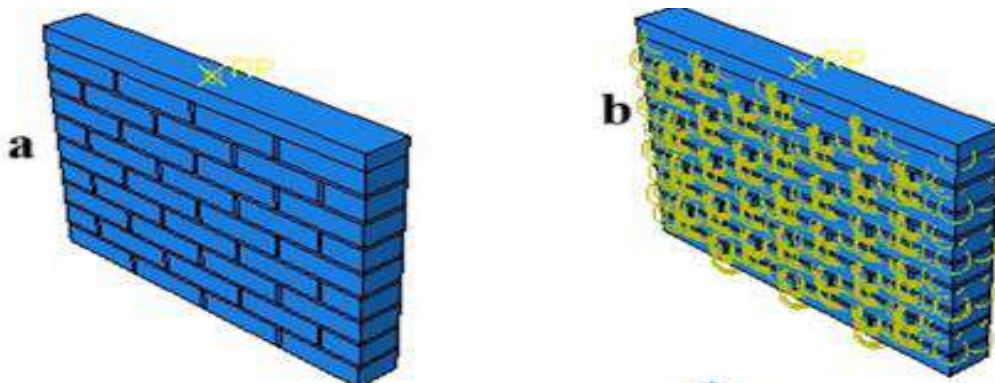
Table 3 presents the physical and mechanical properties obtained from the brick material tests.

2.3. Numerical Modeling Framework

To complement the experimental study, a detailed three-dimensional finite element (FE) micromodel was developed in ABAQUS/Explicit. The modeling strategy aimed to explicitly represent the heterogeneous structure of masonry, capturing the interaction between bricks and mortar joints.

2.3.1. Micro Modeling Approach and Geometry

The masonry walls were modeled brick-by-brick with explicit representation of bed (horizontal) and head (vertical) mortar joints. Bricks and mortar were discretized using 8-node linear brick elements with reduced integration (C3D8R). A mesh sensitivity study was conducted, leading to an optimal global element size of 10 mm, which ensured result convergence, particularly in the critical RVE zone, while maintaining computational efficiency. Convergence was considered achieved when the difference in predicted peak load between successive mesh refinements (e.g., 15 mm, 10 mm, 7 mm global size) was less than 2%. The interaction between brick and mortar was simulated using a surface-based cohesive contact formulation. A "hard" contact property governed the normal direction, preventing penetration.



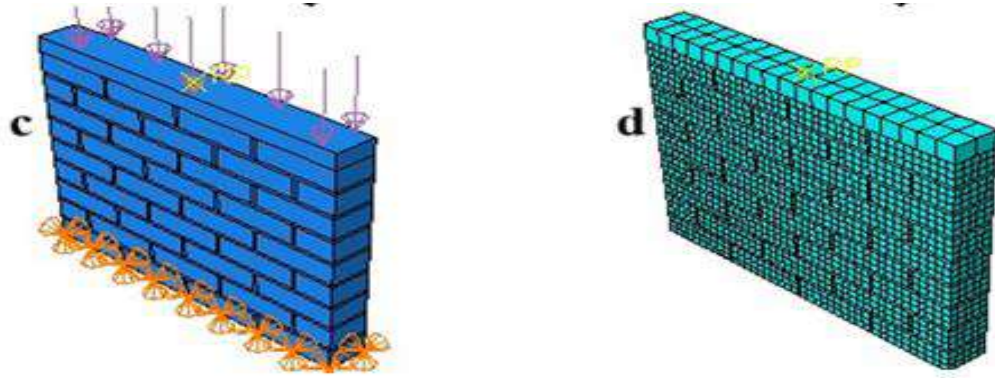


Fig. 6. Finite element methodology design for reinforced masonry: a) micromodel, b) behavior of base cohesive, c) masonry load, d) mesh

In the tangential direction, a Coulomb friction model with a coefficient of $\mu = 0.7$ was adopted to model shear transfer and potential sliding. [17, 18, 20]. Loading was applied via an analytical rigid plate in displacement control, replicating the experimental setup and ensuring uniform displacement at the top of the specimen (Fig. 6). The bottom surface of the model was constrained against all translations and rotations, simulating the fixed support condition of the experimental test setup.

2.3.2. Material Constitutive Model and Calibration

The nonlinear behavior of both brick and mortar was simulated using the Concrete Damage Plasticity (CDP) model in ABAQUS [18, 20, 21]. This model is suitable for quasi-brittle materials as it captures stiffness degradation under both tensile (cracking) and compressive (crushing) failure modes through damage variables d_t and d_c .

The core of the model implementation lies in defining the post-elastic stress-strain relationships. The uniaxial compressive stress is expressed as a function of inelastic strain ϵ_c^{in} and the compressive damage variable d_c :

$$f_c = (1 - d_c)E_0(\epsilon_c - \epsilon_c^{pl}) \quad (4)$$

where E_0 is the initial elastic modulus, and ϵ_c^{pl} is the plastic strain derived from the inelastic strain [19]:

$$\epsilon_c^{pl} = \epsilon_c^{in} - \frac{dc}{(1 - d_c)} \frac{f_c}{E_0} \quad (5)$$

The damage variable d_c evolves from 0 (undamaged) to <1 and is directly related to the stress in the softening branch:

$$d_c = 1 - \frac{f_c}{f_{cu}}, (0 \leq d_c < 1) \quad (6)$$

where f_{cu} is the ultimate (peak) compressive strength. Analogous formulations with tensile damage, d_t , govern behavior under tension [21].

Model calibration was performed in two steps. First, the CDP parameters for brick and mortar as standalone materials were calibrated by simulating uniaxial compression tests on their respective standard specimens (cubes/cylinders). The input parameters—compressive strength (f_c , f_{cu}), tensile strength (f_t), elastic modulus (E_0), and Poisson's ratio (ν)—were taken from the experimentally determined values in Table 1. The inelastic stress-strain curves defining the post-peak softening for each material were derived from these experimental data.

Second, the key plasticity parameters governing the yield surface and flow rule in the CDP model were selected based on established literature for masonry constituents [25, 26, 29] and are summarized in Table 2. The dilation angle ($\psi=10^\circ$) and eccentricity ($\epsilon=0.1$) were kept identical for both materials to ensure numerical stability. The default values for the biaxial-to-uniaxial

compressive strength ratio ($f_{b0}/f_{c0}=1.16$) and the shape parameter K_c (0.667 for mortar, 0.7 for brick) were adopted, as they provided the best fit to the experimental load-displacement response of the full masonry walls during initial model tuning.

Table 2. Plasticity parameters for brick and mortar at masonry failure

Plasticity Parameter	Materials for masonry	
	Brick	Mortar
Dilation Angle, (ψ)	10°	10°
Flow Potential Eccentricity (ϵ)	0.1	0.1
Biaxial/Uniaxial Comp. Ratio, (f_{b0} / f_{c0})	1.16	1.16
Yield Surface Shape Parameter, (k)	0.7	0.667
Viscosity Parameter, (ν)	0.0001	0.0001

3. Results and Discussion

3.1. Validation of Representative Volume Element (RVE) through Strain Analysis

The experimental strain data, recorded via seven indicators (four vertical: Y_1 - Y_4 ; three horizontals: X_1 - X_3), revealed the heterogeneous nature of masonry under compression. A clear strain gradient was observed from the specimen boundaries to its center. The readings from indicators Y_1 and Y_4 (edge zones) and X_1 and X_3 showed minimal and statistically insignificant variation, indicating constrained deformation near the loading platens due to the "platen restraint effect" [17, 22].

In contrast, indicators Y_2 , Y_3 (vertical), and X_2 (horizontal), positioned in the central section, recorded substantially higher strain values (Table 3). This central zone exhibited stabilized strains, relatively free from the direct influence of boundary conditions. This pattern confirms the formation of a region with a representative and uniform stress-strain state, justifying its designation as the Representative Volume Element (RVE) for homogenization analysis. These results are in full agreement with established findings that maximum deformations in masonry prisms occur at mid-height, with central strains being 3-4 times greater than those at the top and bottom due to end restraints [17, 24].

The successful isolation of this stabilized central zone within the 9-row specimen validates its sufficiency as an RVE. Smaller configurations (e.g., 5 rows) are dominated by boundary effects, preventing the development of a representative stress field, while the 9-row fragment provides the necessary height for stress redistribution and the establishment of a characteristic deformation kernel.

Table 3. Experimental values of transverse and longitudinal deformations

Masonry wall		$\epsilon_{y1,4}$	$\epsilon_{y2,3}$ RVE zone	$\epsilon_{x1,3}$	RVEon ϵ_{x2}
Type 1	1	-0.00016	-0.00028	0.000045	0.00021
	2	-0.00021	-0.00036	0.000051	0.00027
	Average Value	-0.00018	-0.00032	0.000048	0.00024
	Std. Dev.	0.000035	0.000057	0.000004	0.000042
Type 2	1	-0.000159	-0.00082	0.000092	0.00033
	2	-0.000360	-0.00086	0.000170	0.00039
	Average Value	-0.000260	-0.00084	0.000130	0.00036
	Std. Dev.	0.000142	0.000028	0.000055	0.000042
Type 3	1	-0.000192	-0.001400	0.000350	0.00100
	2	-0.000390	-0.001700	0.000580	0.00130
	Average Value	-0.000290	-0.001500	0.000460	0.00110
	Std. Dev.	0.000140	0.000212	0.000162	0.000212

Note: Std. Dev. = Standard Deviation, calculated from two specimens per joint thickness, indicating the scatter in measured values. Lower values suggest better reproducibility

3.2. Quantitative Influence of Mortar Joint Thickness on Mechanical Behavior

The compressive strength and deformation characteristics of masonry are critically dependent on mortar joint thickness, as summarized in Tables 3 and 4. The analysis reveals a pronounced trend: increasing joint thickness from 5 mm to 20 mm significantly degrades structural performance.

- **Compressive Strength:** The average compressive strength decreased by 20% (from 3.5 MPa to 2.8 MPa) when joint thickness increased from 5 mm to 10 mm, and by a further 18% (to 2.3 MPa) when increased to 20 mm, resulting in a total reduction of 34%. The ratio of cracking load to failure load ($\frac{F_{cr}}{F_{fr}}$) also decreased from 0.256 to 0.2, indicating that masonry with thicker joints enters the nonlinear, damaging phase at a lower proportion of its ultimate capacity.
- **Stiffness:** The initial elastic modulus (E_0) showed a consistent decline, dropping by 12% (from 11,128 MPa to 9,750 MPa) and then 13% (to 8,525 MPa) for the 10 mm and 20 mm joints, respectively. This 23% total reduction signifies a substantial loss in global stiffness, making the structure more susceptible to deformation under service loads.
- **Deformation Capacity:** Conversely, the deformation capacity, particularly in the transverse direction, increased dramatically. The average transverse strain in the RVE zone (ϵ_{x2}) amplified by approximately 50% (from 0.00024 to 0.00036) and then 300% (to 0.0011) for the 10 mm and 20 mm joints, respectively, compared to the 5 mm baseline.
- **Mechanistic Interpretation:** This behavior stems from the changing role of the mortar layer. In thin joints (5 mm), the mortar acts as a stiff, high-confinement binder, facilitating efficient load transfer and forcing a brittle, brick-dominated tensile failure. As thickness increases, the mortar volume creates a more compliant, weak interface. This layer undergoes greater shear and compressive plastic deformation, leading to stress concentration, reduced composite action, and a transition to a mortar-dominated, quasi-plastic failure mode. The non-linear relationship—where a 400% increase in transverse strain corresponds to a 34% strength loss—highlights this fundamental shift in the failure-controlling phase from brick to mortar.

The standard deviations (SD) presented in Table 4 indicate the scatter of the experimental data. The relatively low values of standard deviation for the mechanical properties (particularly for the compressive strength f_i and elastic modulus E_0) confirm good reproducibility of the results even with the limited number of specimens ($n=2$ per joint thickness). However, it should be noted that the small sample size restricts the possibility of conducting a comprehensive statistical analysis, and the obtained results primarily demonstrate clear qualitative trends of the joint thickness influence. This mechanistic shift is accurately captured by the CDP model in the numerical simulation, where the damage variable (d) develops more rapidly and extensively within the thicker mortar joints (see Section 4.3).

Table 4. Strength characteristics of masonry under uniaxial compression

Masonry wall		f_i (MPa)	F_{fr} (kN)	F_{cr} (kN)	$\frac{F_{cr}}{F_{fr}}$	E_0 (MPa)
Type1	1	3.7	431	120	0.278	11406.22
	2	3.3	466	110	0.236	10850.40
Average Value		3.5	448	115	0.256	11128.3
Standard deviation (SD)		0.283	24.75	7.07	0.030	393
Type2	1	2.9	350	80	0.228	9 500
	2	2.7	330	77	0.233	10000
Average Value		2.8	340	78.5	0.230	9750
Standard deviation (SD)		0.141	14.14	2.12	0.004	354
Type3	1	2.2	280	50	0.178	8700
	2	2.3	295	65	0.220	8350
Average Value		2.3	287	57.5	0.200	8525
Standard deviation (SD)		0.071	10.61	10.61	0.030	247

The standard deviation (SD) values indicate good reproducibility despite the limited sample size ($n=2$ per configuration). For validation, the numerical model showed deviations of 8.5%, 7.0%, and 4.3% for 5 mm, 10 mm, and 20 mm joints, respectively. The coefficient of determination (R^2) exceeded 0.98 for all cases, and the root mean square error (RMSE) was below 0.15 MPa, confirming high predictive accuracy.

3.3. Analysis of Stress-Strain Response and Failure Modes

The stress-strain curves for each joint thickness (Figs. 7-9) provide further insight into the evolving mechanical response.

- 5 mm Joint (Fig. 7): The response is predominantly linear-elastic until near peak load, with a sharp post-peak drop, characteristic of brittle fracture. The significant strain difference between the central RVE and edge zones underscores the effectiveness of the 9-row setup in mitigating boundary effects for homogenization.
- 10 mm Joint (Fig. 8): Increased nonlinearity is evident, especially beyond 50-60% of peak stress. This represents progressive damage accumulation within the mortar joints. The strain gradient between the center and edges persists but is less pronounced than in the 5 mm case, indicating more distributed damage.
- 20 mm Joint (Fig. 9): The curve exhibits marked nonlinearity from the onset of loading, signaling immediate inelastic compaction and microcracking in the thick mortar beds. The overall response is more ductile-like (gradual softening), with failure governed by the crushing and shear failure of the mortar.

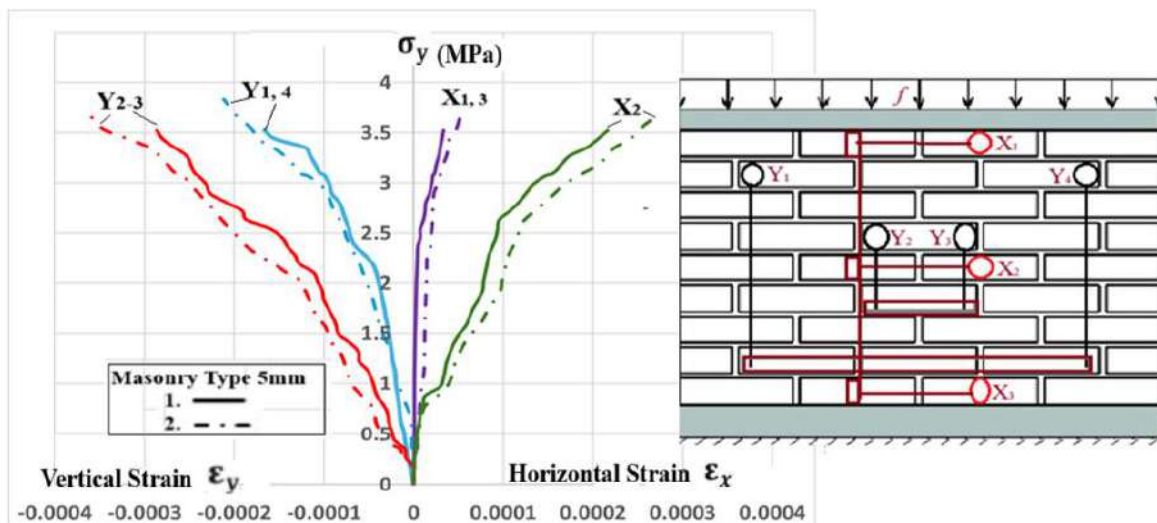


Fig. 7. Stress vs strain relationships for masonry with 5 mm mortar joints

3.4. Synthesis and Practical Implications

The integrated results demonstrate that mortar joint thickness is not merely a construction detail but a key design parameter controlling the composite behavior of masonry. The validated 9-row RVE provides a reliable basis for homogenized modeling. The quantitative relationships established here—linking joint thickness to strength, stiffness, and ductility offer valuable input for performance-based design.

From a practical standpoint, the sharp decline in performance beyond a 10 mm joint thickness is critical. For seismic-resistant design, where controlled energy dissipation is required, the limited benefit of increased deformation capacity with 20 mm joints is far outweighed by the severe penalty in strength and stiffness. Therefore, based on this evidence, limiting mortar bed joint thickness to 10 mm or less is recommended in design codes and specifications for load-bearing masonry, particularly in mid- to high-rise construction. This recommendation aligns with the upper limits suggested in best practice guides [22] and ensures optimal utilization of the brick masonry's

inherent compressive strength. Increased nonlinearity indicates progressive damage in mortars. The central RVE zone experiences significantly higher strains.

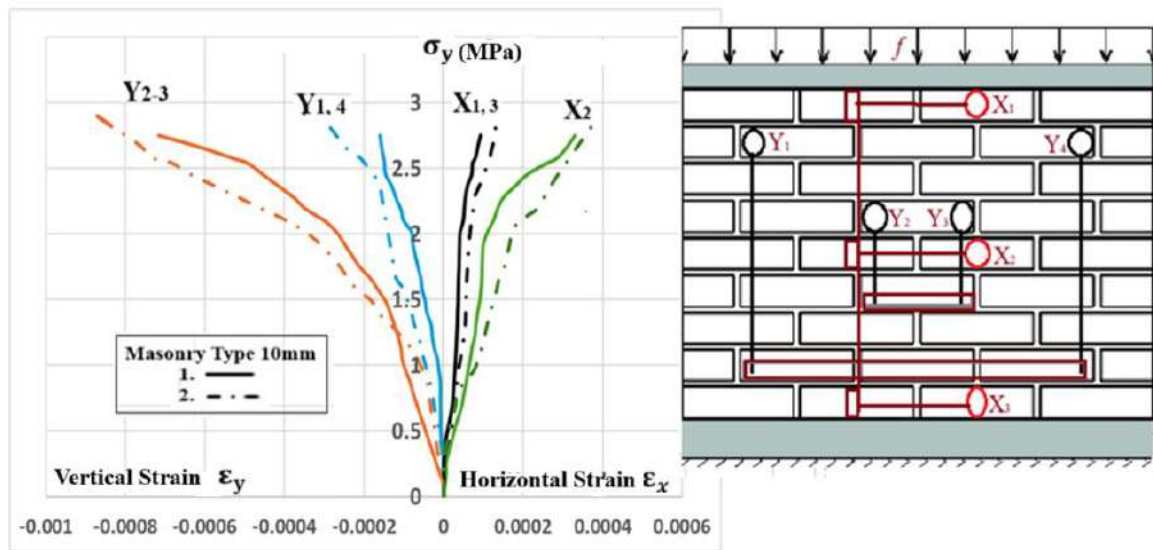


Fig. 8. Stress vs strain relationships for masonry with 10 mm mortar joints

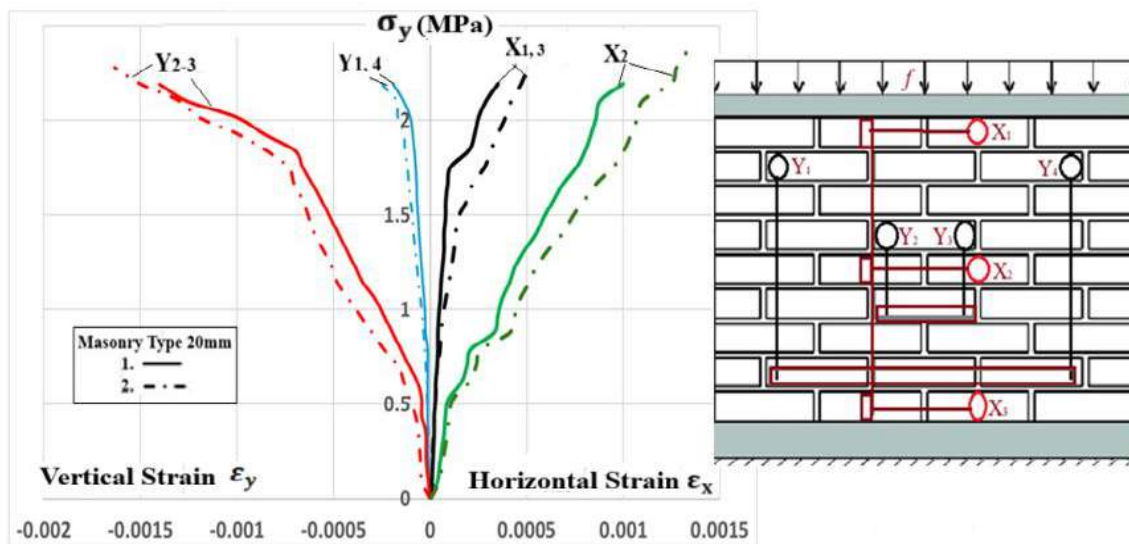


Fig. 9. Stress vs strain relationships for masonry with 20 mm mortar joints

4. Numerical Results and Validation

The validated numerical model successfully replicated the key experimental findings. Fig. 10 shows a direct comparison of the failure modes. For the 5 mm joint, both model and test exhibit vertical cracking in the central RVE. For the 20 mm joint, the model accurately predicts the shift to mortar-crushing failure.

Quantitative validation is presented in Fig. 11, comparing the experimental and numerical stress-strain curves. The model shows excellent agreement with the experimental compressive strength, with deviations of only 8.5%, 7.0%, and 4.3% for the 5 mm, 10 mm, and 20 mm joint thicknesses, respectively. The coefficient of determination (R^2) between the experimental and numerical stress-strain curves exceeded 0.98 for all cases. Furthermore, the model accurately captures the reduction in initial stiffness (E_0) and the increasing nonlinearity with thicker joints. The high degree of correlation (deviations < 9%, $R^2 > 0.98$) validates the chosen micro modeling approach and the calibration of the CDP parameters.

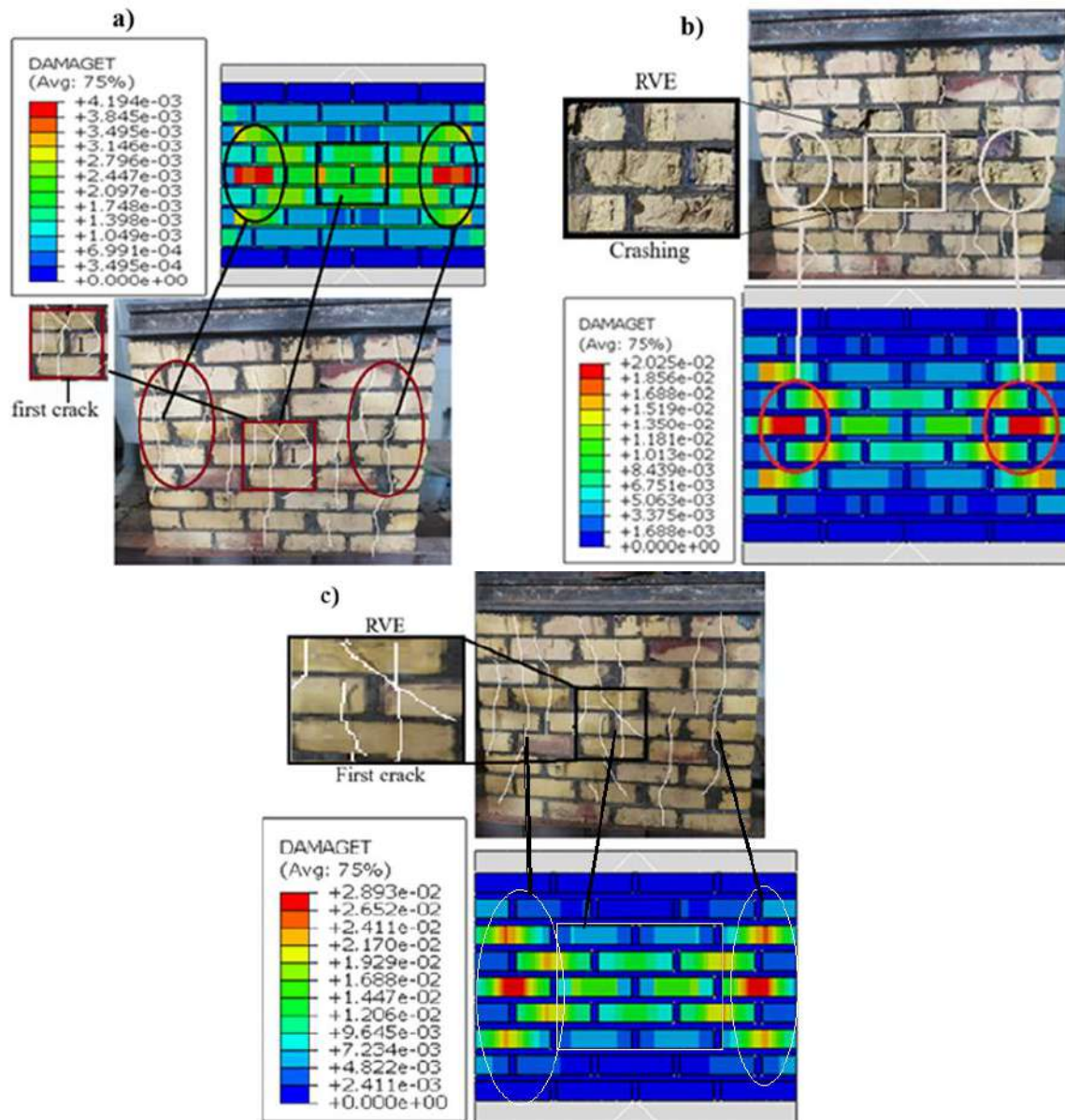
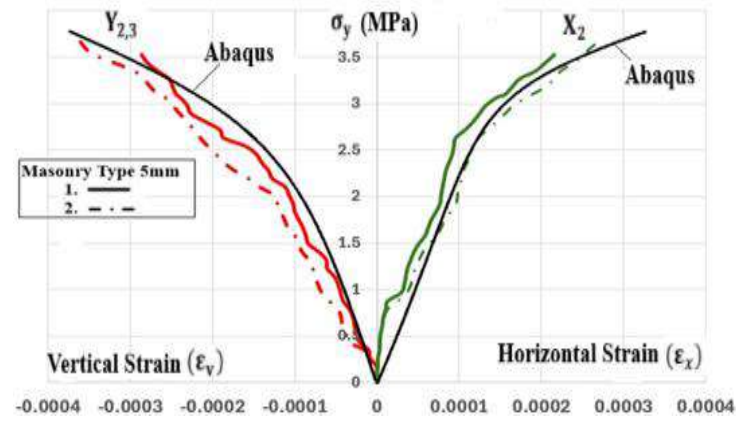


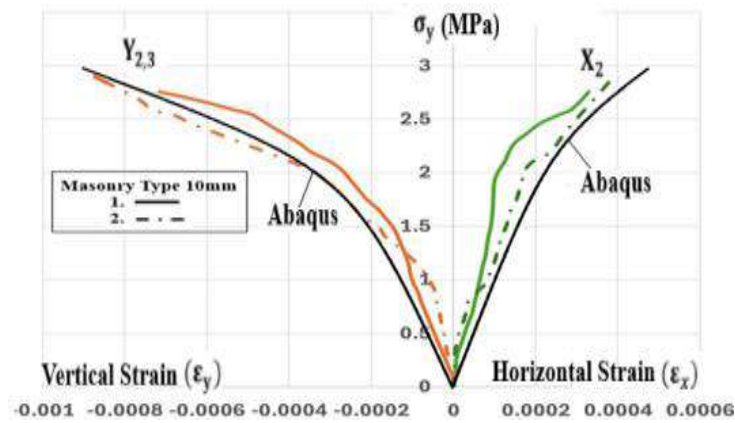
Fig. 10. Comparative analysis of masonry wall failure modes: Experimental vs. numerical simulation for (a) 5mm, (b) 10mm, and (c) 20mm mortar joint thicknesses

Quantitative validation is presented in Figure 11, comparing the experimental and numerical stress-strain curves. The model shows excellent agreement with the experimental compressive strength, with deviations of only 8.5%, 7.0%, and 4.3% for the 5 mm, 10 mm, and 20 mm joint thicknesses, respectively. Furthermore, the model accurately captures the reduction in initial stiffness (E_0) and the increasing nonlinearity with thicker joints.

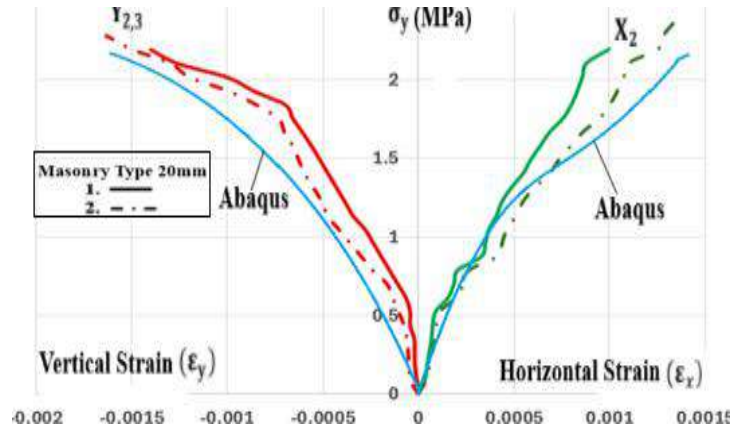
The high degree of correlation (deviations < 9%) validates the chosen micro modeling approach, the calibration of the CDP parameters, and the overall numerical framework. This model can now be considered a reliable tool for predicting the compressive behavior of the studied masonry type under the investigated range of joint thicknesses.



(a)



(b)



(c)

Fig. 11. Comparison and Validation of Numerical (ABAQUS) and Experimental Results for Mortar Joints with Thicknesses of (a) 5 mm, (b) 10 mm, and (c) 20 mm

5. Conclusions

This integrated experimental and numerical study investigated the influence of mortar joint thickness on the mechanical behavior of brick masonry. Based on the analysis of results, the following key conclusions can be drawn:

- Validation of RVE Configuration: The nine-row masonry fragment was rigorously validated as a Representative Volume Element (RVE) for the analysis of heterogeneous masonry. Experimental strain measurements confirmed that this configuration effectively minimizes

boundary effects and ensures strain stabilization in the central zone, thus providing a reliable basis for determining homogenized mechanical properties and macro-modeling.

- **Quantitative Impact of Mortar Joint Thickness:** Mortar joint thickness was identified as a critical parameter governing the macroscopic mechanical response. A clear and quantifiable detrimental relationship was established: increasing the joint thickness from 5 mm to 20 mm resulted in a significant 34% reduction in compressive strength (from an average of 3.5 MPa to 2.3 MPa) and a 23% decrease in the initial elastic modulus (E_0 from 11,128 MPa to 8,525 MPa). Concurrently, transverse deformations in the RVE zone amplified by approximately 400%.
- **Shift in Failure Mechanism:** The increase in joint thickness precipitated a fundamental shift in the failure mode. For thin joints (5 mm), failure was characterized by brittle vertical cracking localized in the brick units of the central RVE zone, driven by tensile stresses from Poisson's effect. In contrast, for thick joints (20 mm), the failure transformed into a quasi-plastic mode dominated by crushing and shear damage within the mortar joints themselves, leading to a loss of structural integrity at lower stress levels.
- **Reliability of Numerical Framework:** The micro modeling approach in ABAQUS, utilizing the Concrete Damage Plasticity (CDP) model with carefully calibrated parameters, demonstrated high predictive fidelity. The model accurately replicated the experimental stress-strain responses and failure patterns across all joint thicknesses, with deviations in compressive strength of less than 8.5%. This validates the numerical framework as a robust tool for the virtual analysis and design of similar masonry systems.
- **Practical Implications and Recommendations:** For optimal structural performance, particularly in contexts requiring high load-bearing capacity or seismic resistance (e.g., high-rise or critical infrastructure), the findings mandate a strict control of mortar joint thickness. Based on the sharp decline in performance beyond 10 mm, it is recommended that the mortar joint thickness in design and construction practice should not exceed 10 mm to preserve strength, stiffness, and a favorable brittle failure mode. This recommendation supports and quantifies the prudent limits often suggested in best practice guides and provides a technical basis for code specifications (e.g., the emphasis on thin joints in ASTM C1314 for prism testing).
- **Study Limitations and Future Work:** The conclusions are derived from a specific brick type and mortar mix, with a limited number of specimens per configuration. Future research should expand the statistical basis and investigate the interaction of joint thickness with other variables, such as mortar strength, brick geometry, and reinforcement, under more complex loading scenarios (e.g., cyclic, shear). The validated numerical model presented here provides a solid foundation for such parametric studies.

References

- [1] Adishchev VV, Shakarneh OMD. Identification of effective stiffness characteristics of masonry based on comparative analysis results of numerical and physical experiments. *News of Higher Educational Institutions. Construction*. 2024;8:5-21. <https://doi.org/10.32683/0536-1052-2023-771-3-5-21>
- [2] Adishchev VV, Shakarneh OMD. The effect of reinforcement on the physical and mechanical characteristics of masonry of ceramic blocks. *News of Higher Educational Institutions. Construction*. 2022;5:14-30. <https://doi.org/10.32683/0536-1052-2022-761-5-14-30>
- [3] El-Dakhkhni MA, El-Sheikh MA. Influence of mortar properties on the seismic behavior of masonry walls. *Journal of Building Engineering*. 2021;28:101724.
- [4] Haach VG, Vasconcelos G, Lourenço P, Mohamad G. Influence of the mortar on the compressive behavior of concrete masonry prisms. *Revista da Associação Portuguesa de Análise Experimental de Tensões*. 2010;18:79-84.
- [5] Jadhav S, Awardi G, Yeri S. Influence of mortar strength and thickness on the mechanical characteristics of masonry brickwork. *Journal of Systems Engineering and Electronics*. 2021;31(1):265-277.
- [6] Jagadish K, Shetty GK, Nayak GU. Effect of mortar mix on compressive strength of brickwork. *International Journal of Applied Engineering Research*. 2016;11(5):3129-3134.
- [7] Liang B, Zhang H, Liu Z, Hou J. The influence of different types of mortar on the compressive strength of masonry. *Journal of Building Engineering*. 2023;65:105635. <https://doi.org/10.1016/j.jobe.2022.105635>

- [8] Sarhosis V, Garrity SW, Sheng Y. Influence of brick-mortar interface on the mechanical behaviour of low bond strength masonry brickwork lintels. *Engineering Structures*. 2015;88:1-11. <https://doi.org/10.1016/j.engstruct.2014.12.014>
- [9] Usman M, Panwar R. The influence of mortar thickness and strength on the mechanical characteristics of masonry brickwork. *International Journal of Progressive Research in Engineering Management and Science*. 2024;4(5):1775-1783.
- [10] Zengin B. The effect of the bricks used in masonry walls on characteristic properties. *Sigma Journal of Engineering and Natural Sciences*. 2017;35(4):667-677.
- [11] Ghiassi B, Milani G. Numerical modeling of masonry and historical structures: From theory to application. London: Woodhead Publishing; 2019. (Woodhead Publishing Series in Civil and Structural Engineering).
- [12] Shakarneh OMD. Homogenization method for determining the matrix of effective stiffness coefficients of reinforcement masonry wall. *Trudy NGASU = Proceedings of the Novosibirsk State University of Architecture and Civil Engineering*. 2024;27(4).
- [13] Adishchev VV, Shakarneh OMD. Determination of the physical characteristics of masonry based on a structural approach. *News of Higher Educational Institutions. Construction*. 2025;4:32-44.
- [14] Shakarneh OMD. Main mechanisms of masonry failure as a structurally heterogeneous material. *News of Higher Educational Institutions. Construction*. 2025;6:31-43.
- [15] Adishchev VV, Shakarneh OMD. Processes of elastic and inelastic deformation and destruction of stone masonry. *News of Higher Educational Institutions. Construction*. 2025;2:5-18.
- [16] Hermoune H, Benabed B, Alshugaa M, Abousnina R, Guettala A. Strengthening of masonry walls with CFRP composite: Experiments and numerical modeling. *Épitoanyag - Journal of Silicate Based and Composite Materials*. 2020;72(1):2-11. <https://doi.org/10.14382/epitoanyag-jsbcm.2020.1>
- [17] Kaushik HB, Rai DC, Jain SK. Stress-strain characteristics of clay brick masonry under uniaxial compression. *Journal of Materials in Civil Engineering*. 2007;19(9):728-739. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2007\)19:9\(728\)](https://doi.org/10.1061/(ASCE)0899-1561(2007)19:9(728))
- [18] Lee J, Fenves G. Plastic-damage model for cyclic loading of concrete structures. *Journal of Engineering Mechanics*. 124(8):892-900. [https://doi.org/10.1061/\(ASCE\)0733-9399\(1998\)124:8\(892\)](https://doi.org/10.1061/(ASCE)0733-9399(1998)124:8(892))
- [19] Lourenco PB. Computational strategies for masonry structures [PhD thesis]. Delft: Delft University of Technology; 1996.
- [20] Lubliner J, Oliver J, Oller S, Onate E. A plastic-damage model for concrete. *International Journal of Solids and Structures*. 1989;25(3):299-326. [https://doi.org/10.1016/0020-7683\(89\)90050-4](https://doi.org/10.1016/0020-7683(89)90050-4)
- [21] Milani G, Valente M, Alessandri C. The narthex of the Church of the Nativity in Bethlehem: A non-linear finite elements approach to predict the structural damage. *Computers and Structures*. 2018;207:3-18. <https://doi.org/10.1016/j.compstruc.2017.03.010>
- [22] European Committee for Standardization. EN 1052-1:2009 - Methods of test for masonry. Part 1: Determination of compressive strength. Brussels: European Committee for Standardization; 2009.
- [23] ASTM International. ASTM C109-14: Standard test method for compressive strength of hydraulic cement mortars (using 2-in. or 50-mm cube specimens). West Conshohocken, PA: ASTM International; 2014.
- [24] Fonseca FS, Mohamad G, Roman HR, Vermeltfoort A, Rizzatti E. Deformation and failure mode of masonry. In: 12th North American Masonry Conference; 2015 May 17-20; Denver, Colorado. p. 1-12.
- [25] Mousavi R, Champiri MD, Willam KJ. Efficiency of damage-plasticity models in capturing compaction-expansion transition of concrete under different compression loading conditions. *Proceedings of the 7th European Congress on Computational Methods in Applied Sciences and Engineering*. 2016;172:1-12. <https://doi.org/10.7712/100016.1810.12018>
- [26] Genikomsou AS, Polak MA. Finite element analysis of punching shear of concrete slabs using damaged plasticity model in ABAQUS. *Engineering Structures*. 2015;98:38-48. <https://doi.org/10.1016/j.engstruct.2015.04.016>
- [27] Dassault Systèmes. ABAQUS user's guide (Version 6.13). Providence, RI: Dassault Systèmes Simulia Corp; 2013.
- [28] Wang SY, Sloan SW, Abbo AJ, Masia MJ, Tang CA. Numerical simulation of the failure process of unreinforced masonry walls due to concentrated static and dynamic loading. *International Journal of Solids and Structures*. 2012;49(2):377-394. <https://doi.org/10.1016/j.ijsolstr.2011.10.016>

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Numerical study of stiffness degradation and dissipated energy in early cyclic behavior of non-conforming RC beam-column joints

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Abstract

Beam-column joints are critical components influencing the seismic performance of reinforced concrete (RC) frames, especially in regions with moderate to high seismic activity. A significant proportion of Algeria's building stock was constructed before 1980, prior to the adoption of the Algerian seismic regulations, resulting in inadequate seismic detailing and making these joints especially vulnerable to earthquakes. This study examines the impact of initial cycles of lateral load on the seismic performance of non-conforming external reinforced concrete beam-column joints, focusing on early stiffness degradation and energy dissipation processes. A three-dimensional nonlinear finite element model was created in ANSYS and compared with experimental data from the literature, confirming the accuracy of the numerical method. The originality of this work resides in the infrequently studied correlation between stiffness loss and energy dissipation, illustrated by a normalized dual-curve representation. This approach helps the identification of a pivotal intersection point that signifies the threshold of post-elastic behavior. A hybrid index (D_{Hybrid}) is proposed, incorporating these two parameters into a singular synthetic indicator for the preliminary evaluation of seismic performance. A detailed parametric analysis was performed to investigate the impact of critical variables such as concrete compressive strength, beam longitudinal reinforcement ratio, and joint core confinement. The findings indicate that the early loading cycles are crucial in initiating stiffness degradation and cumulative energy dissipation, determining the beginning of nonlinear behavior, and substantially affecting the seismic response of nonconforming joints.

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1. Introduction

Algeria is one of the most seismically active countries in the Mediterranean due to the convergence between the African and Eurasian plates. Past earthquakes, such as the 1980 El-Asnam (moment magnitude, M_w 7.3) and the 2003 Boumerdès (M_w 6.8), have highlighted the high vulnerability of existing reinforced concrete frame buildings, particularly at beam-column joints [1]. This vulnerability is not limited to Algeria but is observed worldwide, where many RC buildings were built prior to the adoption of modern seismic codes and the capacity design philosophy. Such structures often exhibit inadequate detailing, which significantly compromises the seismic performance of beam-column joints, leading to rapid stiffness degradation and limited energy dissipation under seismic loading [2, 3]. Numerous catastrophic structural failures reported in past earthquakes have been attributed to the poor performance of RC beam-column joints. Post-

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earthquake reconnaissance missions consistently show that damage is frequently concentrated in these joints, often resulting in partial or total collapse of RC frame systems [1, 4, 5].

In view of these deficiencies, substantial research has been conducted in the past few decades to enhance the understanding of non-ductile reinforced concrete beam-column joints and to design appropriate assessment and retrofit methodologies. Pioneering research from the 1980s, by Durrani and Wight [6] and Ehsani and Wight [7], showed that the number of joint stirrups, joint shear stress, and the presence of transverse beams affect stiffness degradation, energy dissipation, and the overall seismic performance of reinforced concrete beam-column joints. Subsequent investigations of gravity-load-designed sub assemblages [8, 9] demonstrated flexure-dominated damage, insufficient ductility, and significant column failures, affirming the critical influence of lap splice location, joint reinforcement, and bar anchorage. Pampanin et al. [1] examined joints characteristic of 1950s–1970s construction, characterized by smooth bars, absence of transverse reinforcement, and insufficient anchorage, revealing panel-zone vulnerability, stiffness loss due to bar slip, and brittle "concrete wedge" failures in external joints. Clyde et al. [10] and Pantelides et al. [2] proved the seismic vulnerability of these joints, illustrating that insufficient anchoring led to bond-slip and pinching, while increased axial load somewhat improved shear strength but reduced ductility and accelerated stiffness degradation. These results led to an extensive amount of research on ways to retrofit. RC or Carbon Fiber Reinforced Polymer CFRP jacketing effectively converted brittle shear failures into ductile beam hinging, significantly improving energy dissipation [11, 12]. Kuang and Wong [13] illustrated how important beam bar anchorage is for joint shear strength and hysteretic response. Testing on real joints from a building that was damaged in the 2009 L'Aquila earthquake revealed that FRC jacketing stopped joint shear failure, modified the response to beam yielding, and significantly enhanced shear strength and energy dissipation [14].

Recent studies have concentrated on new retrofitting and design strategies for non-seismic reinforced concrete joints. Sonawane and Dubey [15] noted that rectangular spiral reinforcement significantly enhanced crack control, ductility, and energy dissipation in comparison to traditional stirrups. Shaaban and Seoud [16] demonstrated that the combination of ferrocement layers with steel angles significantly enhances stiffness retention, energy dissipation, and ductility under cyclic loading. Lim et al. [17] have shown that incorporating a CFRP grid into engineered cementitious composite (ECC) enhances crack control, ductility, and energy dissipation relative to high-strength mortar or retrofitted specimens. Araby et al. [18] augmented gravity-load-designed joints using ferrocement layers, demonstrating considerable enhancements in deformation capacity, stiffness, and energy dissipation, while exhibiting no notable change in ultimate strength. Following these trends, Helal et al. [19] used post-tensioned metal straps to reduce the loss of stiffness and increase strength and energy dissipation. Similarly, Golias and Karayannis [20] used X-shaped near-surface mounted NSM Carbon Fiber Reinforced Polymer C-FRP ropes to get strength gains of up to ~65% and energy dissipation gains of up to three times.

In addition to experimental research, various analytical formulations have been created to analyze the cyclic behavior of RC beam-column joints. Paulay and Priestley [21] developed significant formulations for joint shear strength and ductile detailing that became the basis for many models developed after them. Biddah and Ghobarah [22] and Hwang and Lee [23] established models that included joint shear deformation, bond-slip effects, and softened strut-and-tie mechanisms, and they were tested against large sets of experimental data. Pantelides et al. [2] tested six full-scale exterior joints with inadequate reinforcement details. They evaluated load capacity, drift, shear strength, and ductility under two levels of axial load. They proposed and approved limit-state and strut-and-tie models to forecast the failure mechanisms of joints designed for gravity loads. Lowes et al. [24] created a four-node joint element that was added to OpenSees. It modeled bar slip, panel shear, and interface shear degradation, and it was tested against a lot of experimental data. Mitra and Lowes [25] modified this model by incorporating diagonal strut shear formulation and an improved bar-slip representation, resulting in increased precision in illustrating stiffness and strength degradation under cyclic loading. Celik and Ellingwood [26] and Park and Mosalam [27] complemented these developments with calibrated models for joints without transverse reinforcement and for fragility assessment, whereas Le-Trung et al. [28] applied analytical

modeling to CFRP-strengthened connections, precisely predicting stiffness degradation and energy dissipation. LaFave and Kim [29] collected 341 joint tests and created Bayesian shear strength and deformation models that highlight the impact of joint aspect ratio, confinement, and concrete strength, outperforming code-based equations and enabling mechanism-based joint modeling.

Recent research has concentrated on the advancement of advanced macro-elements and retrofit-oriented modeling to enhance predictive accuracy and computational efficiency. Mosallam et al. [30] used analytical and nonlinear numerical modeling to evaluate FRP-retrofitted joints, showing substantial enhancements in strength, stiffness retention, and energy dissipation. Pantò et al. [31] reinforced this trend by introducing the "Joint with Hinges" macro-element, which can reproduce panel shear, bar slip, and plastic hinge formation in one framework. Simultaneously, Gombosuren and Maki [32] created simpler predictive equations for joint shear deformation indices. These equations are useful for finding inelastic joints without having to do a full nonlinear analysis. Grande et al. [33] developed a nonlinear macro-model derived from the "scissors model," calibrated in OpenSees, to simulate pinching and cyclic degradation phenomena. The latest works by Irreño Palomo et al. [34] and Kalogeropoulos et al. [35] integrated analytical and numerical methods for Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) UHPFRC-jacketed and deficient joints, respectively, precisely replicating stiffness degradation, energy dissipation, and failure mechanisms under cyclic loading.

In general, these analytical studies reveal how approaches have evolved, from local shear models to more comprehensive macro-models. This data shows how advanced this field of research has become. The first numerical methods used simplified macro-models to make frame-level simulations cheaper to run. Lowes and Altoontash [24] developed a joint element in OpenSees that combines a shear panel and bar-slip springs. This element shows how stiffness and strength change under cyclic loading. Mitra and Lowes [25] modified this formulation by incorporating a confined-concrete shear strut and refining hysteretic rules to more accurately simulate pinching and energy dissipation. These advancements formed the basis for subsequent finite element modeling, prompting a shift towards more detailed 3D simulations capable of representing localized cracking, bond-slip phenomena, and joint shear stress distributions.

Researchers gradually adopted detailed 3D nonlinear finite element method (FEM) simulations to enhance the depiction of joint mechanics. Goto and Joh [36] combined cyclic tests with DIANA analyses to observe shear cracking caused by eccentricity and a reduction in stiffness after the peak. Ibrahim and Elbadry [37] validated ATENA 3D models for joints reinforced with double-headed studs, replicating hysteretic loops and proving their ductile behavior. Eslami et al. [38] utilized ANSYS to illustrate that CFRP retrofitting can move plastic hinges to beams and establish a strong-column weak-beam mechanism, increasing the estimated seismic capacity from 0.37g to 0.45g. Baji et al. [39] utilized VecTor2 to forecast load-displacement responses and failure modes of FRP-strengthened joints beyond experimental thresholds. Further research by Venkatesan et al. [40] and Laseima et al. [41] used ANSYS simulations and cyclic tests together to show that ferrocement or FRP strengthening increases moment capacity and cumulative energy dissipation (up to 200%) and delays stiffness loss. Al-Rousan and Alkhawaldeh [42] extended this study by explicitly simulating FRP-concrete bond degradation in ANSYS, demonstrating that bond deterioration under 30% continues to enhance ductility and retard stiffness loss.

Recent studies have concentrated on advanced materials, hybrid methodologies, and machine-learning-enhanced predictions. Kytinou et al. [43] employed full-scale tests and complete Abaqus models to verify the seismic performance of CFRP-reinforced joints, precisely predicting bar slip and hysteretic loops. Fayaz et al. [44] numerically illustrated in Abaqus that the combination of UHPFRC and CFRP retrofitting results in a 2.6-fold increase in peak load and considerably postpones the deterioration of strength and stiffness. Marie et al. [45] established a machine learning framework [(Ordinary Least Squares (OLS), Support Vector Machines (SVM), K-Nearest Neighbour Regression (KNN), Multivariate Adaptive Regression Splines (MARS), Artificial Neural Networks (ANN), Kernel Regression with mixed data types (Kernel)] that reached an R^2 value of 0.9752 for predicting shear strength, providing a cost-effective substitute for complicated simulations. Zhuang et al. [46] employed OpenSees fiber models to examine prefabricated FRC

joints featuring grouted sleeves, suggesting optimal shear-span ratios ranging from 2.0 to 2.5. In 2025, Sayed Ahmed et al. [47] demonstrated the efficacy of steel-angle retrofits through 3D finite element modeling. Concurrently, Maulana et al. [48] and Sabbahfar et al. [49] confirmed the DIANA and ATENA 2D models on non-ductile interior joints, successfully capturing crack propagation, hysteresis loops, and stiffness degradation. Furthermore, Prakash and Parthasarathi [50] used both Complete Abaqus Environment (Abaqus/CAE) simulations and ANN regression to test FRP-rehabilitated joints at high temperatures. They found that CFRP laminates were the best for thermal and mechanical strength. The critical review of experimental, analytical, and numerical studies highlights significant advances in understanding the seismic behavior of RC beam-column joints, yet underscores a lack of focused research on the early degradation phase and on the correlation between stiffness loss and energy dissipation in such joints.

The current study is part of a larger research project aimed to investigate the seismic strength of existing RC buildings, with a focus on the exterior beam-column joints in Algerian buildings constructed before 1980. These joints often lack adequate seismic detailing, which makes them very likely to be damaged by earthquakes. More specifically, this work examines the early behavior under reversed lateral loading, which is a phase characterized by significant stiffness degradation that primarily affects the subsequent structural response. Unlike most existing studies that analyze beam-column joint behavior until failure, this research concentrates on the onset of degradation, a critical phase for forecasting structural vulnerability. Its originality resides in the establishment of a direct correlation between stiffness loss and cumulative energy dissipation via a normalized dual-curve representation, a topic rarely addressed concurrently in existing literature. This method shows a key point where post-elastic behavior starts. A hybrid index (D_{Hybrid}) is also suggested, which combines normalized residual stiffness and cumulative energy dissipation into one measurable indicator for early damage assessment.

The created three-dimensional nonlinear finite element model, which was validated against experimental data in ANSYS [51], allows for accurate tracking of stiffness degradation and energy dissipation at each cycle. This provides new insights into the vulnerability of RC joints to seismic activity. A detailed parametric analysis subsequently examines the influence of important factors such as concrete compressive strength, beam longitudinal reinforcement ratio, and joint core confinement on energy dissipation and stiffness degradation. The results reveal the factors influencing initial degradation under cyclic loads and offer a practical instrument for evaluating seismic vulnerability.

2. Materials and Modelling Procedure

2.1. Finite Element Modelling of Exterior RC Beam-Column Joints

Using ANSYS [51], a three-dimensional nonlinear finite element (FE) model was created to simulate the cyclic behavior of RC exterior beam-column joints. The modeled joint subassembly accurately reproduced the geometry and reinforcement details of the experimental specimen. The Solid65 element, an eight-node brick with three translational degrees of freedom per node, was used to model concrete. It could simulate both tensile cracking and compressive crushing. We used a multi-linear stress-strain curve, the Von Mises yield criterion, and the Willam-Warnke failure model to define how the concrete would behave. This work was done to show how it would respond and fail in a nonlinear way [52]. Shear transfer coefficient is taken 0.25 for open cracks and 0.99 for closed cracks [51, 53].

LINK8 truss elements, each with two nodes and three translational degrees of freedom, were used to show the steel reinforcement in the beams and columns. A multi-linear kinematic hardening model (MKIN) was used to reproduce the inelastic steel response. This model utilized parameters such as elastic modulus, yield strength, and tangent modulus [53]. Even though bond-slip between concrete and reinforcement significantly influences joint behavior, this study assumed that there was a perfect bond. To satisfy this assumption, reinforcement elements were connected to adjacent concrete solid elements by sharing common nodes, thus enabling full interaction between concrete and steel.

The finite element model was developed based on the test specimen RCNH1, selected from the literature (Figure 1). This specimen was part of an experimental program on exterior RC beam-column joints conducted by Gencoglu and Mobasher [54]. In the nonlinear analysis, the total load was applied incrementally, and the solution was obtained using the modified Newton-Raphson method [51]. Appropriate boundary conditions and loading were defined to replicate the experimental setup. Taking advantage of the symmetry in both geometry and loading, only half of the joint was modeled in the finite element analysis (FEA), which considerably reduced computational time and memory requirements. In the numerical model, Solid45 elements were used to simulate the steel loading plates. The nodes of these steel elements were connected to those of the adjacent concrete elements (Solid65) to satisfy the perfect bond assumption.

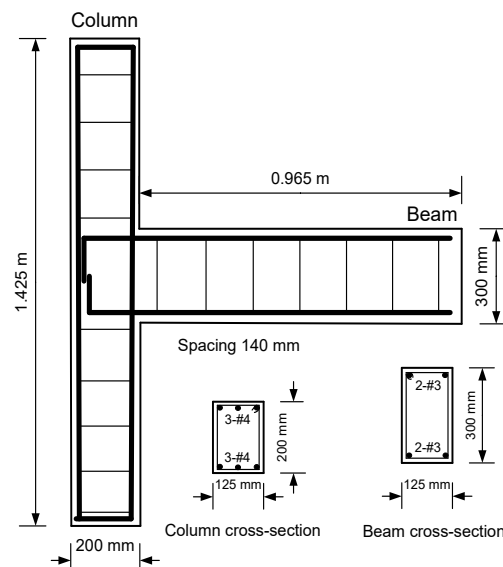


Fig. 1. Geometry and reinforcement details of exterior beam-column joint specimen RCNH1 [54]

The RCNH1 specimen was a half-scale exterior joint designed to replicate the characteristics of older RC structures with poor seismic detailing. The beam was 125 mm × 300 mm in cross-section, and the column was 125 mm × 200 mm. The longitudinal reinforcement of the beam penetrated 200 mm into the joint. The joint region didn't have any transverse reinforcement, which is a common problem seen in pre-code construction.

The specimen was constructed using concrete with a compressive strength of 30 MPa. The longitudinal reinforcement was made of Grade 60 steel bars (yield strength $\approx$ 525 MPa), and the stirrups were made of Grade 40 steel bars (yield strength $\approx$ 290 MPa). The column had a constant axial load of 90 kN on it, which is about 13% of its axial capacity ($0.13 A_g f'_c$). Reversed cyclic lateral loading was simulated in ANSYS under displacement control. Alternating positive and negative displacement cycles with progressively increasing amplitudes were applied at the beam free end using APDL tabular functions. This loading history reproduced the experimental protocol slowly imposed by a 250 kN Mechanical Testing Systems (MTS) actuator [54].

2.2. Experimental Validation of the Finite Element Model

To validate the accuracy of the developed numerical model, the experimental specimen RCNH1, tested at Arizona State University, was adopted as a benchmark reference. This specimen, reported by Gencoglu and Mobasher [54], represents a non-conforming exterior RC beam-column joint subjected to reversed cyclic loading. Since the original experimental data were not publicly available, representative points were extracted from the published hysteresis curves using a digitization tool to reproduce the overall cyclic response for model validation.

Figure 2 presents the cyclic load-displacement response at the beam tip, comparing the experimental data of specimen RCNH1 with the finite element (FE) simulation. The experimental

loops (in black) exhibit the typical pinching effect and progressive stiffness degradation resulting from bond-slip and shear damage in the unconfined joint core. Conversely, the simulated loops (in red) display a fuller hysteresetic shape with a less pronounced pinching behavior, especially at larger displacement amplitudes.

The numerical model constructed with ANSYS properly represents the seismic behavior of RC beam-column connections, particularly for the load-displacement envelope, peak strength, and stiffness progression. The initial loading cycles are captured with sufficient accuracy to enable a reliable analysis of stiffness deterioration and energy dissipation, which are the primary focuses of this study. The hysteresis loops generated by the simulation appear fuller and exhibit reduced pinching compared to those observed in experiments. This reaction aligns with conventional finite element modeling techniques that do not explicitly incorporate bond-slip effects and progressive local degradation. As previously highlighted by Ronagh and Baji [55], this modeling simplification does not significantly compromise the model's ability to provide reliable predictions for assessing the seismic vulnerability of existing joints during the early phases of cyclic loading.

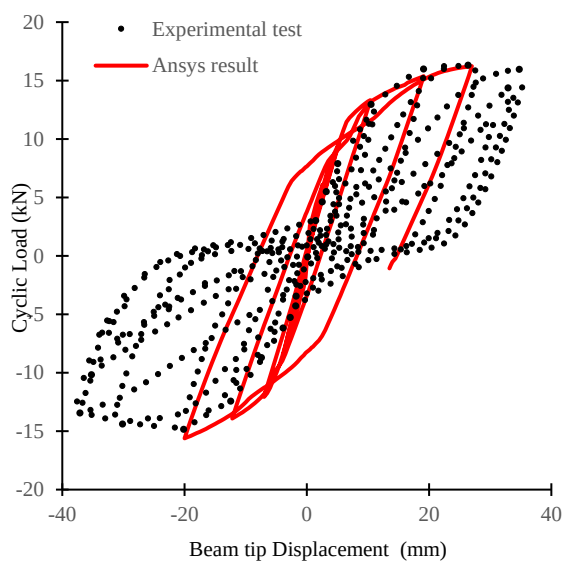


Fig. 2. Comparison of cyclic load-displacement responses between FE simulation (ANSYS) and experimental results for specimen RCNH1 [54]

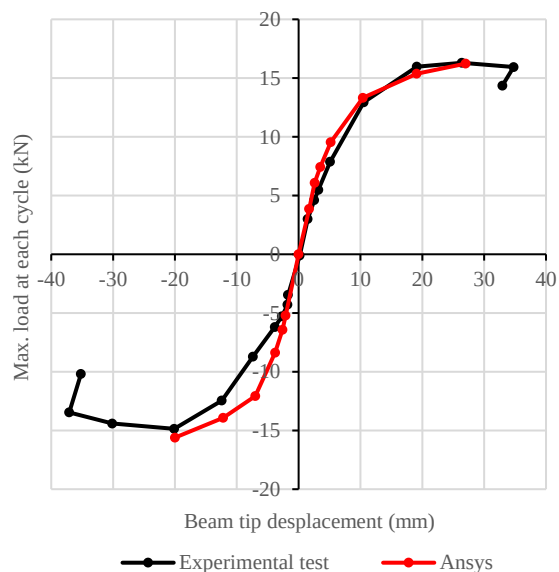


Fig. 3. Comparison of load-displacement backbone curves from the experimental test [54] and the FE simulation (Ansysis)

The curve in Figure 3 shows the maximum cyclic load attained by both the finite element (FE) model (ANSYS) and the experimental specimen RCNH1 at each displacement level. In general, the two responses are in good agreement. The general shape and trend of the load-displacement backbone curves are well reproduced, confirming the model's ability to accurately capture early stiffness degradation and the nonlinear cyclic behavior of RC joints.

2.3. Modeling of Pre-Code Non-Conforming RC Joints in Algerian Buildings

After validating the initial numerical framework, a finite element model was developed to analyze the seismic response of RC exterior beam-column joints representative of typical Algerian buildings constructed in 1971. These joints commonly exhibit inadequate detailing practices, including smooth longitudinal bars, inadequate anchorage, no transverse reinforcement in the joint core, and poor confinement conditions. All of these factors make them significantly more likely to fail when they are loaded laterally. This numerical model serves as the foundation for a parametric study designed to quantify the impact of critical design parameters, such as concrete compressive strength and longitudinal and transverse reinforcement ratios, on seismic behavior in the early stages. We put a lot of focus on assessing stiffness degradation and hysteretic energy dissipation during the first loading cycles. These are important signs of when structural damage starts and how it becomes worse.

Figure 4 illustrates the geometry and size (in cm) of the modeled specimen for the numerical analysis using ANSYS [51] to simulate the RC exterior beam-column joint. The appropriate element types and material properties used in the model are described below.

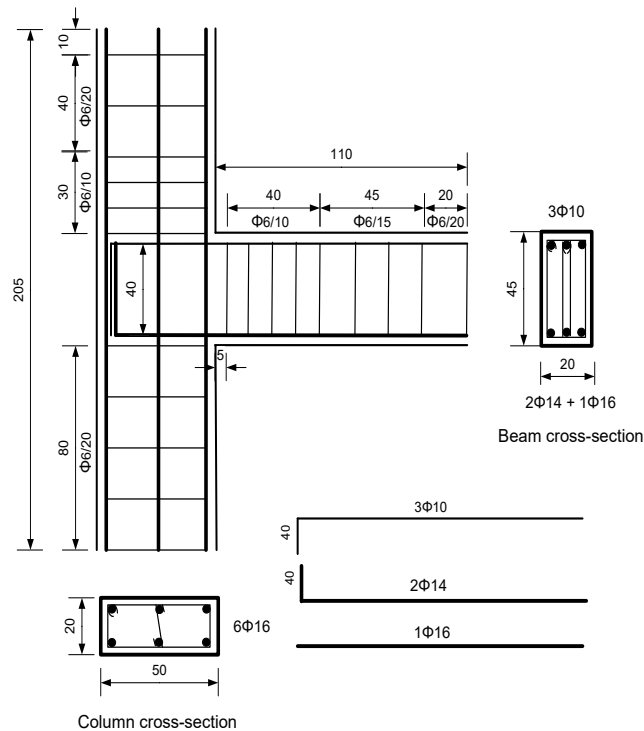


Fig. 4. Overall geometry and dimensions of the modeled RC beam-column joint (in cm)]

The material properties of concrete and reinforcement adopted for the numerical modeling of the beam-column joints are summarized in Tables 1 and 2.

Table 1. Concrete material properties

Compressive strength, f'_c (MPa)	Tensile strength, f_t (MPa)	Young's modulus E_s (MPa)
30	2.4	32164.19

Table 2. Reinforcement material Properties

Reinforcement	Yield strength, f_y (MPa)	Young's modulus, E_s (GPa)
Longitudinal reinforcement	400	200
Transverse reinforcement	240	200

The FE model using ANSYS [51] was analyzed under reversed cyclic lateral loading. Figure 5 shows the stress-strain relationship of concrete, modeled using the Multilinear Isotropic Hardening option, based on the approach proposed by Kachlakev et al. [53]. The stress-strain relationship for reinforcing steel is defined as a bilinear model, assumed to be elastic-perfectly plastic, as illustrated in Figure 6. A perfect bond between concrete and steel reinforcement was assumed. The steel reinforcement was connected to the nodes of each adjacent concrete solid element, so that both materials shared common nodes, ensuring full interaction.

The finite element mesh of the beam-column model is shown in Figure 7. A uniform element size of 25 mm was adopted, as it ensured a satisfactory compromise between numerical convergence stability and computation time. Due to symmetry in both geometry and loading, only half of the width of the model was represented in the finite element analysis (FEA). Cyclic bending was applied at the free end of the beam to simulate the effect of a severe earthquake (Figure 8). The load was applied at a distance of 5 cm from the free end of the beam, through steel plates.

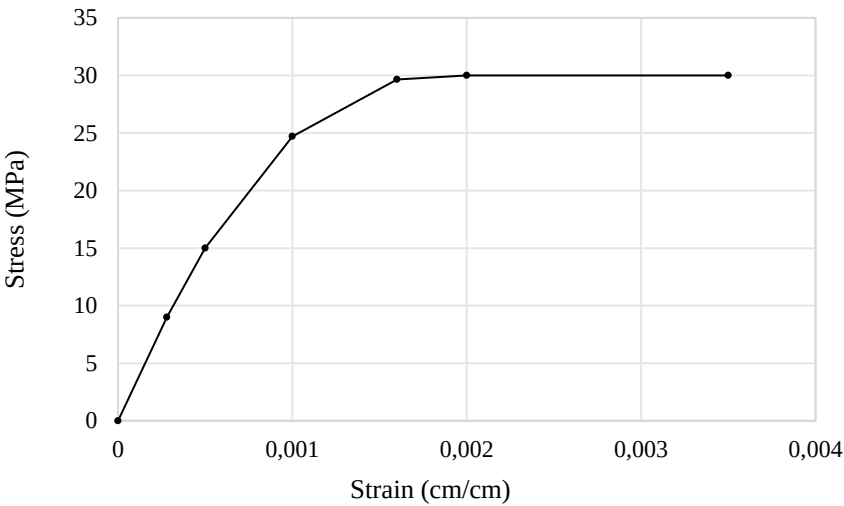


Fig. 5. Idealized stress-strain curve of concrete

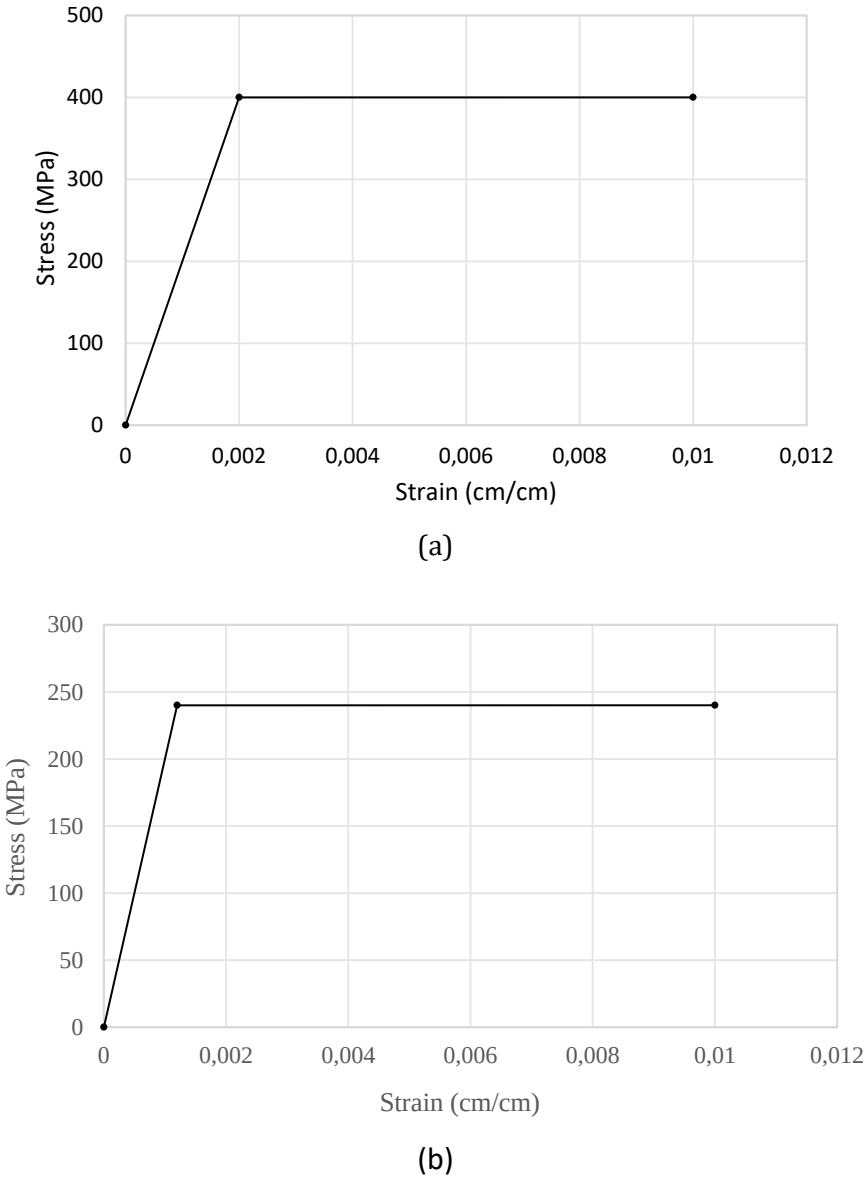


Fig. 6. Idealized stress-strain curves of reinforcing steel (a) Longitudinal reinforcement (b) Transverse reinforcement

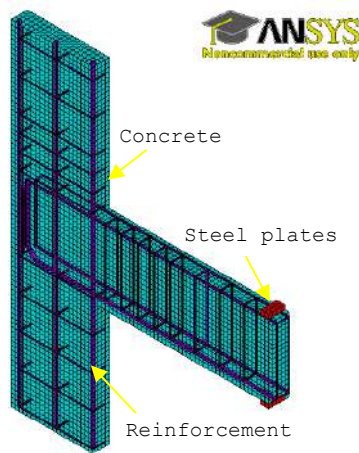


Fig. 7. Mesh of the concrete, steel plate, and steel support

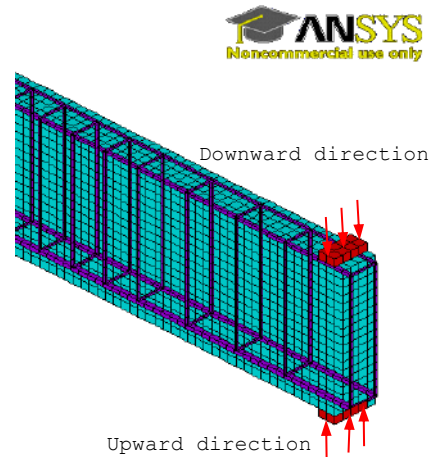


Fig. 8. Application of cyclic loading through steel plates

The boundary conditions were defined at the plane of symmetry and at the ends of the columns. The displacements of the nodes at the top of the columns were constrained to zero in all three directions. With respect to the plane of symmetry located in the XY plane, the Z-displacement of the nodes lying on this plane was also set to zero.

3. Results and Discussion

3.1 Hysteretic Moment–Rotation Response from FE Simulation

Figure 9 illustrates the moment–rotation hysteresis curve. It displays a steep initial elastic segment, then the stiffness decreases and the hysteresis loops become wider, signifying cumulative energy dissipation. There is a clear difference between the positive and negative branches, caused by the uneven distribution of longitudinal reinforcement ($3\varnothing 10$ at the top of the beam vs. $2\varnothing 14 + 1\varnothing 16$ at the bottom). The joint reached a maximum moment of 106 kN.m at a rotation of 0.0072 rad in the upward direction and 60.2 kN.m at 0.0085 rad in the downward direction, beyond which convergence failure occurred due to excessive cracking.

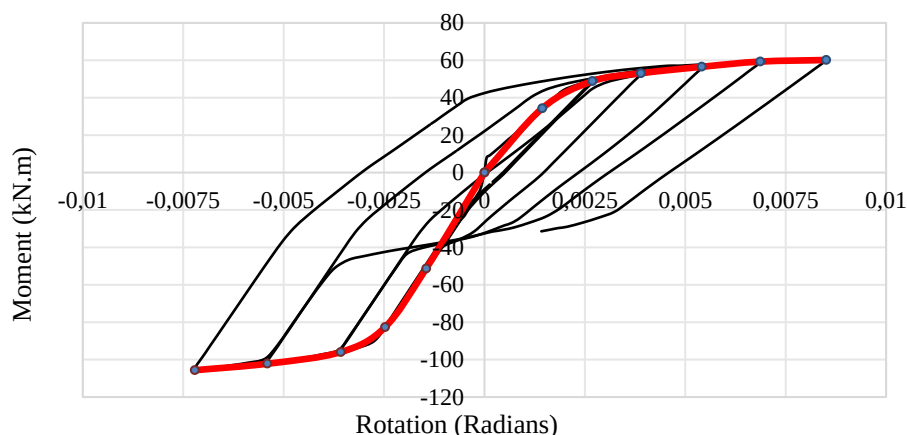


Fig. 9. Moment-rotation hysteresis curve of the RC joint obtained from Ansys

3.2 Stiffness and Energy Dissipation Characteristics

In seismic studies of reinforced concrete structures, two aspects are often examined to understand how beam–column joints behave under cyclic loading: the loss of stiffness and the ability to dissipate energy. The gradual reduction in stiffness reflects how the joint becomes less rigid as loading cycles progress, while energy dissipation describes its effectiveness in absorbing and releasing the input energy through the hysteretic response of the materials.

The degradation of stiffness was evaluated cycle by cycle through the peak-to-peak stiffness K_{pp} . This parameter corresponds to the slope of the secant line connecting the maximum and minimum moment points on the moment-rotation hysteresis loop for each cycle [35, 56]. The energy dissipated per cycle is quantified by the area enclosed within the moment-rotation hysteresis loop [17, 18]. The results summarized in Table 3 and illustrated in Figures 10 to 12 present the combined evolution of peak-to-peak secant stiffness and energy dissipation.

Table 3. Evolution of stiffness degradation and cumulative energy dissipation during early cyclic loading

Cycles	Peak to Peak Stiffness K_{pp} (kN.m/rad)	Normalised stiffness degradation (K_{norm})	Energy dissipation E_d (N.m)	Cumulative energy dissipation $E_{d,cum}$ (N.m)	Normalised cumulative Energy (E_{norm})
1	29632.82	1	9.81	9.81	0.0047
2	25474.38	0.86	34.40	44.21	0.0212
3	19948.88	0.67	145.00	189.21	0.0908
4	14674.85	0.49	359.00	548.21	0.2631
5	11717.11	0.39	602.00	1150.21	0.5520

From the first loading cycles, the stiffness starts to reduce: the peak-to-peak stiffness decreases from 29632.82 kN.m/rad in the first cycle to 25474.38 kN.m/rad in the second, corresponding to a 14% reduction. This degradation continues to occur at different levels of intensity until it reaches 11717.11 kN.m/rad by the fifth cycle. This decline corresponds to a total loss of more than 60%, as shown in Table 3, Figures 10 and 11.

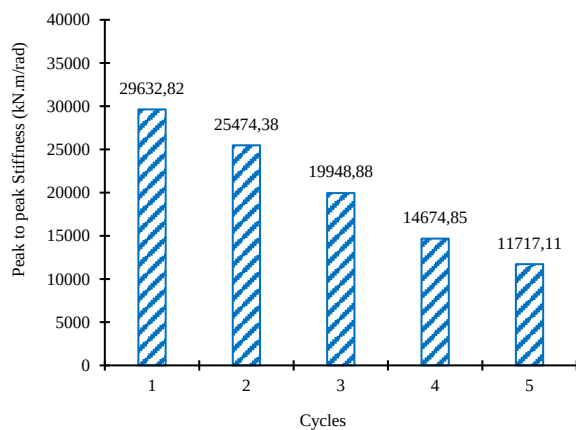


Fig. 10. Peak to peak stiffness degradation during early cyclic loading

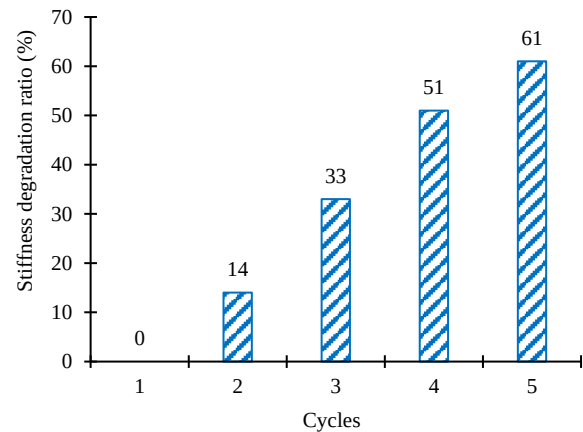


Fig. 11. Evolution of stiffness degradation ratio during early cyclic loading

At the same time, energy dissipation stays relatively small in the beginning, with only 9.81 N.m and 34.40 N.m dissipated in the first and second cycles, respectively. This indicates that the initial hysteretic energy absorption capacity is limited. From the third cycle on, there is a significant increase, and by the fifth cycle, it reaches 602 N.m, as shown in Table 3. This behavior shows that large inelastic deformations are starting to happen, marking the transition of the joint into a nonlinear response regime.

As illustrated in Table 3, the normalized stiffness degradation (K_{norm}) is defined as the ratio K_i/K_0 , where K_0 denotes the initial reference stiffness evaluated from the first loading cycle. In parallel, the normalized cumulative energy dissipation (E_{norm}) is given by the ratio $E_{cum,i}/E_{ref}$, with E_{ref} corresponding to the cumulative energy dissipated at the end of the analysis.

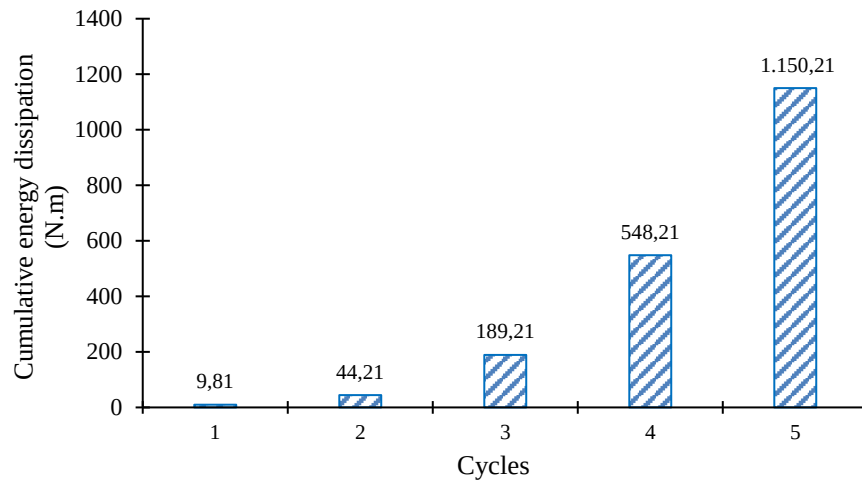


Fig. 12. Cumulative energy dissipation during early cyclic loading

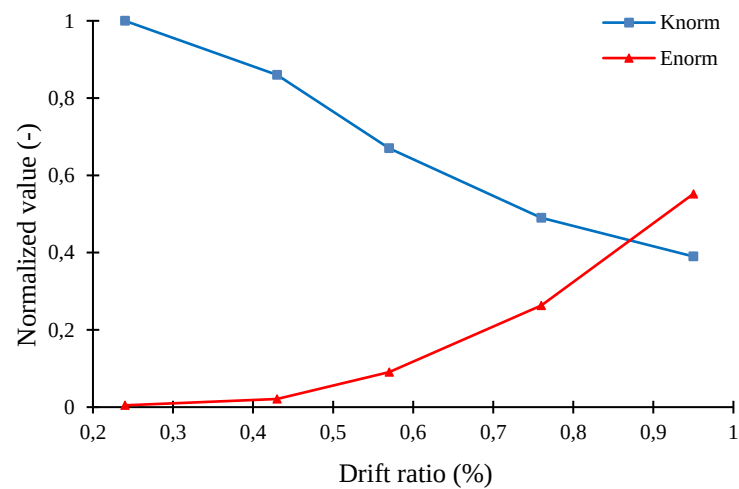


Fig. 13. Evolution of normalized stiffness degradation and cumulative energy dissipation during early cyclic loading

Figure 13 shows how normalized stiffness degradation (K_{norm}) and normalized cumulative energy dissipation (E_{norm}) change as drift (%) changes. There is a clear inverse relationship: as drift increases through the first five loading cycles, stiffness decreases and energy dissipation, which was low at first, rises sharply at higher drift levels.

The normalized stiffness degradation (K_{norm}) is equal to its reference value (1.00) at cycle 1, which corresponds to a drift of 0.24%. The normalized cumulative energy dissipation (E_{norm}), on the other hand, is almost zero, or close to 0%. At cycle 2 (0.43% drift), the normalized stiffness decreases to 0.86, reflecting a reduction of approximately 14%. Energy dissipation stays low, though, making up only about 2% of the total cumulative energy at the end of the analysis. At cycle 3, the drift reaches 0.57%, and the normalized stiffness exhibits a further reduction to 0.67, corresponding to a loss of about 33%. The normalized cumulative energy, on the other hand, increases to about 9%. At this point, inelastic mechanisms start to work, and the material moves out of the elastic range. The degradation grows worse at a drift of 0.75% (cycle 4) and 0.95% (cycle 5): the stiffness diminishes to 0.49 and 0.39, respectively, amounting to a total decrease of about 61 %. At the same time, normalized energy dissipation speeds up quickly, reaching 26% and 55% of the total cumulative energy.

This contrasting evolution underscores the gradual involvement of energy-dissipating mechanisms in a low-ductility RC joint. Previous research has frequently examined stiffness degradation and energy dissipation in isolation; however, the novelty of the current study resides in the direct correlation of these two mechanical responses via a dual normalized-curve representation. The

intersection of the two normalized curves, seen in the drift range of 0.75% - 0.95% (cycles 4 and 5) at a normalized value of about 0.45, may be a key point where energy dissipation becomes more important than stiffness, marking the shift from serviceability to damage. The combined normalized-curve approach is unique to this study; however, analogous mechanical behaviors have been documented in experimental research. Tsonos et al. [57] specifically noted that RC joints constructed per outdated code provisions demonstrate significant stiffness degradation, while their energy dissipation capacity is restricted during initial loading phases, unlike well-detailed joints that preserve a more consistent hysteretic response.

3.3 Hybrid Index Linking Stiffness Degradation and Energy Dissipation

In this section, we propose the D_{Hybrid} index, which combines the evolution of normalized stiffness loss (K_{norm}) and normalized energy dissipation (E_{norm}). It is inspired by the work of Park and Ang [58], who related ultimate deformation to hysteretic energy dissipation for the evaluation of seismic damage. Unlike this formulation, which depends on ultimate deformation unobservable during the early cycles, the present approach replaces it with normalized stiffness loss while preserving the contribution of cumulative energy dissipation. By construction, the index remains bounded between 0 (undamaged) and 1 (fully damaged), thereby providing a simple and directly applicable tool for assessing early-stage damage in RC beam-column joints subjected to cyclic seismic loading. At cycle i , the hybrid damage index is defined as:

$$D_{Hybrid}(i) = \alpha_i \cdot \left(1 - \frac{K_i}{K_0}\right) + (1 - \alpha_i) \cdot \left(\frac{E_{cum,i}}{E_{ref}}\right) \quad (1)$$

Where; K_i : the secant stiffness at cycle i ; K_0 : the reference stiffness (initial tangent or first-cycle secant); $E_{cum,i}$: the cumulative hysteretic energy up to cycle i ; E_{ref} : a common energy reference. The adaptive coefficient, denoted as α_i , is expressed as follows:

$$\alpha_i = \frac{\left(1 - \frac{K_i}{K_0}\right)}{\left(1 - \frac{K_i}{K_0}\right) + \left(\frac{E_{cum,i}}{E_{ref}}\right)} \quad (2)$$

The normalized stiffness degradation at the cycle " i " is represented by the value of K_i/K_0 , while the normalized cumulative hysteretic energy dissipation is represented by the value of $E_{cum,i}/E_{ref}$. Thus, the adaptive coefficient α_i evolves depending on whether stiffness degradation or energy dissipation predominates, ensuring a balanced contribution within the hybrid index.

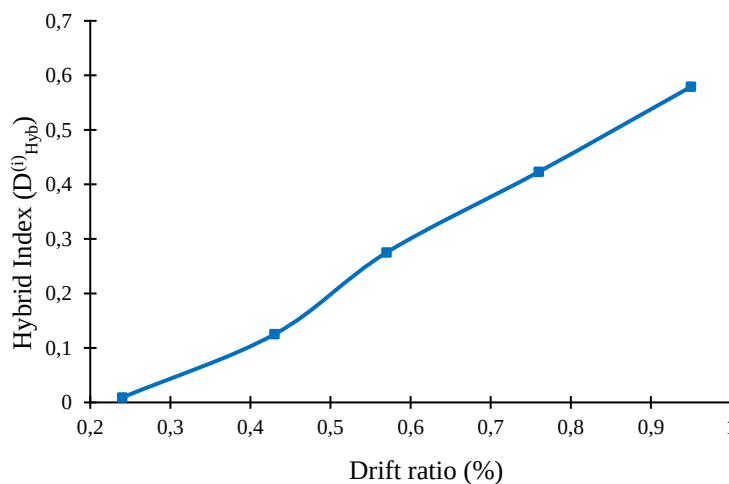


Fig. 14. Evolution of hybrid index (D_{Hybrid}) vs. drift ratio (%) during early cyclic loading

Figure 14 shows how the proposed hybrid index (D_{Hybrid}) changes over the first five loading cycles as the drift ratio changes. The proposed index exhibits a gradual rise with increasing drift, reflecting the combined effects of stiffness degradation and cumulative energy dissipation. During the first loading cycle, at a drift ratio of about 0.24%, its value grows only slightly from 0.009 to 0.125, indicating that the response remains essentially quasi-elastic with negligible hysteretic losses. The

index rises more noticeably between the second (0.43%) and third (0.57%) cycles, reaching almost 0.275. This phase is the beginning of inelastic behavior, which is marked by a clear drop in stiffness and an increase in energy dissipation. After the third cycle, the evolution becomes more stable and follows a more linear path. It goes from about 0.35 at the fourth cycle (0.76%) to about 0.58 at the fifth (0.95%). This pattern shows that the index can pick up on early-stage degradation and show when RC beam-column joints go from elastic to inelastic response. The drift range observed in this study, attaining a moderate value of 0.95%, highlights substantial early-cycle damage. The (D_{Hybrid}) index offers an objective assessment of degradation and serves as a foundation for the parametric analysis of critical parameters in their cyclic response.

4. Parametric Study

The studied parameters are the concrete compressive strength, the beam longitudinal reinforcement ratio, and the transverse reinforcement ratio within the joint core. A 3D nonlinear finite element analysis in ANSYS was conducted to assess their influence on joint performance in terms of stiffness degradation, cumulative energy dissipation, and the hybrid index (D_{Hybrid}). The analysis focuses on the first five loading cycles, which capture early inelastic mechanisms.

4.1 Effect of Concrete Compressive Strength: f'_c

This parametric study first examines the influence of concrete compressive strength (f'_c) on stiffness degradation and energy dissipation during the first few loading cycles. Three strength levels are considered: 30, 40, and 60 MPa.

Figure 15 shows how the stiffness degradation ratio ($1 - (K_i/K_0)$) increases in the first five loading cycles for all concrete strengths. Higher f'_c variants show a faster degradation in the early cycles. This is because the initial stiffness is higher and the unconfined joint core is brittle when it shears. But by cycle 5, the curves come together at about 61–63% degradation, which shows that the effect of f'_c disappears quickly.

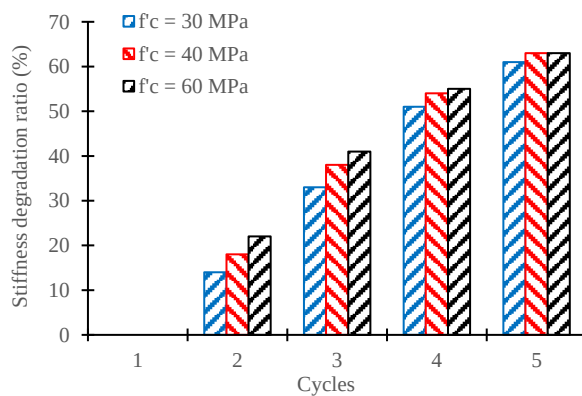


Fig. 15. Evolution of stiffness degradation ratio during early cyclic loading for various concrete strengths

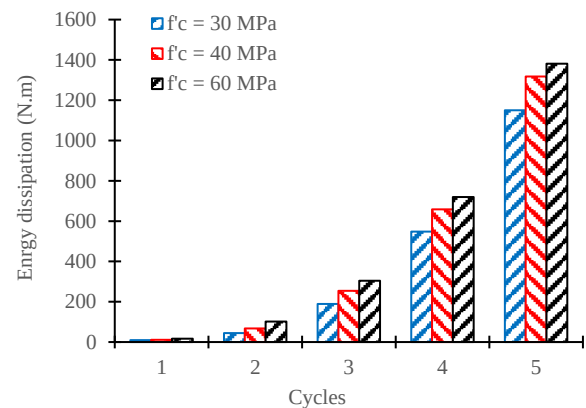


Fig. 16. Cumulative energy dissipation during early cyclic loading for various concrete strengths

These results confirm that in non-conforming joints, higher concrete strength may delay crack initiation but does not prevent significant stiffness degradation at early stages, consistent with findings reported in experimental studies.

Figure 16 reveals that cumulative energy dissipation (E_{cum}) increases slowly over the first five cycles, which is different from what Figure 15 shows, where stiffness rapidly decreases. Higher-strength concretes dissipate more energy in the first few cycles (≈ 16.6 N.m for $f'_c = 60$ MPa versus 9.8 N.m for $f'_c = 30$ MPa at cycle 1), and this difference stays the same through cycle 5 (≈ 1382 versus 1150 N.m). Consequently, although stiffness degradation tends to stabilize at comparable levels, energy dissipation continues to correlate with concrete strength, suggesting that the impact remains substantial on energy dissipation. The comparison between normalized stiffness

degradation (K_{norm}) and normalized cumulative energy dissipation (E_{norm}), conducted for the three considered concrete compressive strengths (f'_c), highlights the contrasting evolution of stiffness loss and energy dissipation, as shown in Figure 17.

While stiffness drops sharply from the very first cycles (drift $\leq 0.43\%$), energy dissipation remains nearly negligible, indicating an early loss of performance without significant absorption. At a drift of 0.57% (cycle 3), energy contribution becomes more pronounced, while stiffness continues to decline. The two curves intersect between 0.76% and 0.95% drift (cycles 4–5), which may be interpreted as a mechanical transition marking the progressive shift from quasi-elastic to inelastic response. This combined representation therefore highlights the complementary nature of the two parameters and emphasizes the importance of a coupled approach for a more refined characterization of early-stage damage.

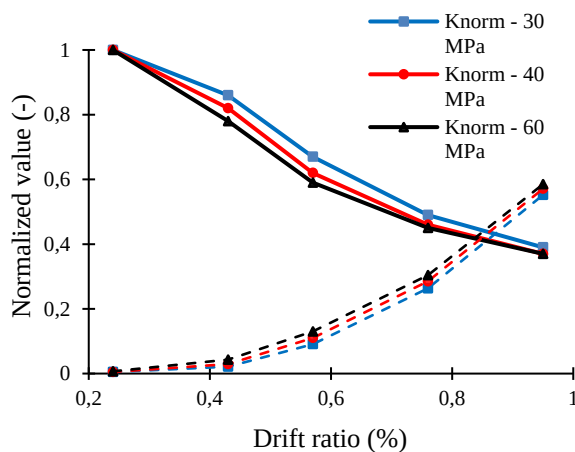


Fig. 17. Evolution of normalized stiffness degradation and cumulative energy dissipation during early cyclic loading for various concrete strengths

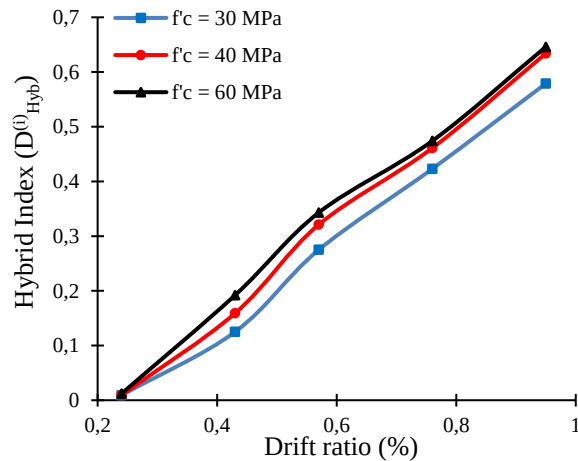


Fig. 18. Evolution of hybrid index (D_{Hybrid}) vs. drift ratio (%) during early cyclic loading for various concrete strengths

The hybrid indicator D_{Hybrid} exhibits a monotonic and steadily increasing trend from the very first cycles (Figure 18). In cycles 1 and 2 (drifts of 0.24% and 0.43%), values remain low ($\approx 0.01 - 0.19$ depending on f'_c), reflecting limited dissipation despite an already initiated stiffness reduction. At cycle 3 (drift $\approx 0.57\%$), the D_{Hybrid} index reaches 0.343 for $f'_c = 60$ MPa, compared to 0.275 for $f'_c = 30$ MPa. In cycles 4 and 5 (drifts of 0.76% and 0.95%), the indicator continues to rise and differences persist, with values ranging from 0.58 (30 MPa) to 0.65 (60 MPa). These results confirm that the effect of f'_c begins in the first cycles and continues steadily as D_{Hybrid} progresses. This means that stronger concretes are more effective at promoting energy dissipation.

This study, limited to the first five loading cycles, highlights the interplay between stiffness degradation, energy dissipation, and the hybrid indicator D_{Hybrid} . The indicator effectively captures these coupled trends and the influence of concrete compressive strength, confirming that high-strength concretes, while initially stiffer, tend to exhibit lower ductility under cyclic loading [59, 60].

4.2 Effect of Beam Longitudinal Reinforcement Ratio: ρ

After examining the influence of concrete compressive strength, the following subsection evaluates the effect of the beam longitudinal reinforcement ratio on joint behavior. This ratio, denoted as ρ , is defined according to the BAEL 91 revised 1999 provisions [61]. A parametric variation of ρ is conducted using values of 0.5% , 0.83% , 1.0% , and 1.3% . The objective is to assess how changes in longitudinal reinforcement influence stiffness degradation, hysteretic energy dissipation, and the evolution of the D_{Hybrid} index during the early loading cycles.

Figure 19 shows the stiffness degradation ratio ($1 - (K_i/K_0)$) over the first five cycles for different beam longitudinal reinforcement ratios. For all variants, stiffness loss occurs rapidly as early as the

second cycle, representing approximately 26% for $\rho = 0.5\%$ and only 8% for $\rho = 1.3\%$. In subsequent cycles, degradation continues: by the third cycle, stiffness loss reaches nearly 46% for the lowest-reinforced variant, compared with 19% for the most heavily reinforced one; by the fifth cycle, losses peak around 68% and 55% respectively. Increasing ρ therefore mitigates the initial stiffness degradation but does not prevent the overall loss of stiffness.

Figure 20 shows the evolution of cumulative energy dissipation (E_{cum}) for different beam longitudinal reinforcement ratios over the first five cycles. As early as the second cycle, clear differences emerge: the lightly reinforced variant $\rho = 0.5\%$ dissipates 71 N.m compared to only 25 N.m for $\rho = 1.3\%$. The result shows that the first variant enters the inelastic regime earlier. This pattern continues through the fifth cycle, where the cumulative energy dissipation for $\rho = 0.5\%$ reaches 1327 N.m and for $\rho = 1.3\%$ it is 1143 N.m. The $\rho = 1\%$ variant exhibits better performance than the 1.3% case at cycle 4 (594 vs. 555 N.m) and at cycle 5 (1249.25 vs. 1142.51 N.m). This shows that the energy dissipation efficiency does not increase linearly with the reinforcement ratio, and that additional reinforcement could limit the activation of dissipative mechanisms. These results confirm that lightly reinforced beams dissipate more energy because they bend more, while heavily reinforced beams delay the activation of dissipative mechanisms, which is consistent with Figure 19, where they kept their stiffness longer.

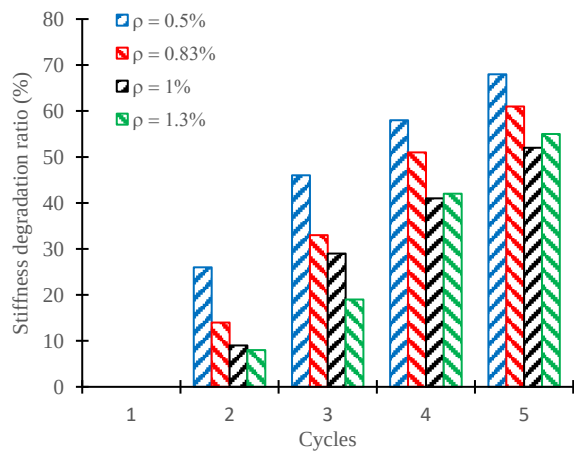


Fig. 19. Evolution of stiffness degradation ratio during early cyclic loading for various beam reinforcement ratios

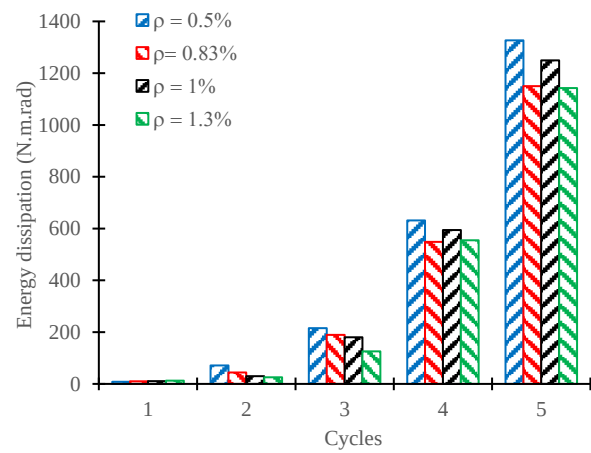


Fig. 20. Cumulative energy dissipation during early cyclic loading for various beam reinforcement ratios

The combined analysis of normalized stiffness degradation (K_{norm}) and normalized cumulative energy dissipation (E_{norm}), conducted for beam longitudinal reinforcement ratios (ρ) of 0.5%, 0.83%, 1.0%, and 1.3%, as illustrated in Figure 21, reveals two distinct behavioral phases over the first five cycles. Up to cycle 3, the response remains predominantly quasi-elastic: stiffness decreases rapidly, with a loss of about 26% for $\rho = 0.5\%$ compared to only 8% for $\rho = 1.3\%$, while energy dissipation remains very low (0.034 vs. 0.012). At Cycle 3, for a drift of 0.57%, energy dissipation begins to accelerate (0.103 for 0.5% vs. 0.060 for 1.3%) while stiffness degradation continues (46% vs. 19%). From cycle 4 (drift $\approx 0.76\%$), a clear transition to a post-elastic regime is observed, with energy dissipation becoming increasingly significant. At this point, the least reinforced variant has already lost considerable stiffness (about 58%) while exhibiting substantial normalized energy dissipation (about 0.30). This step is when the post-elastic behavior starts to become clear. The most reinforced variant, on the other hand, shows a higher residual stiffness (about 42%) and a lower energy dissipation (about 0.27).

This result means that dissipative mechanisms take longer to activate. The least reinforced variant loses 68% of its stiffness by cycle 5 (drift $\approx 0.95\%$), while the most reinforced variant loses 55%. The normalized energy dissipation values are 0.64 and 0.55, respectively. As stiffness approaches its residual plateau, this evolution shows that energy dissipation becomes increasingly significant

in the global response. These results support the idea that increasing ρ makes stiffness last longer and delays the activation of dissipative mechanisms, as illustrated in Figures 19 and 20.

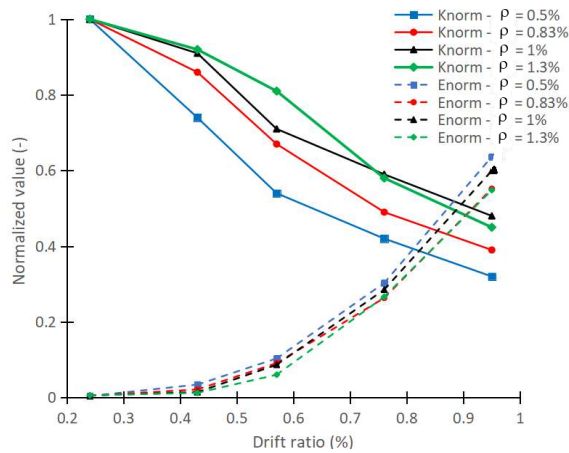


Fig. 21. Evolution of normalized stiffness degradation and cumulative energy dissipation during early cyclic loading for various beam reinforcement ratios

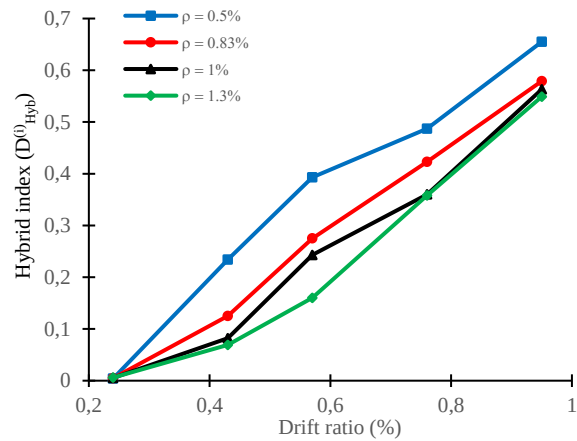


Fig. 22. Evolution of hybrid index (D_{Hybrid}) vs. drift ratio (%) during early cyclic loading for various beam reinforcement ratios

The hybrid index D_{Hybrid} increases monotonically over the first five cycles, reflecting a continuous progression of joint damage. The results illustrated in Figure 22 show that, for cycles 1 and 2, the values remain low (≈ 0.23 for $\rho = 0.5\%$ versus ≈ 0.07 for $\rho = 1.3\%$), indicating that the damage is still dominated by the initial stiffness loss. The index grows faster after cycle 3, increasing from 0.16 to 0.39 (drift $\approx 0.57\%$). This means that the joint is beginning to activate energy-dissipating mechanisms. The trend continues in cycles 4 and 5, with a steady gap (0.66 versus 0.55). By synthesizing stiffness and energy contributions into a single indicator, D_{Hybrid} highlights the transition from quasi-elastic to post-elastic behavior and enables a clear comparison between reinforcement configurations. It thus is shown to be a relevant synthetic indicator for comparing damage evolution at low drift levels.

The joint evolution of stiffness degradation and energy dissipation marks a clear transition from stiffness-controlled behavior in the early cycles to energy-dominated response thereafter. The D_{Hybrid} indicator confirms that lightly reinforced joints reach higher damage levels at moderate drift ratios ($\approx 1\%$). These results align with prior studies showing that higher reinforcement ratios enhance stiffness retention but require sufficient confinement to sustain ductility and energy dissipation [62, 63]. Similarly, Sabbahfar et al. [64] reported that deformed bars improve both stiffness stability and energy capacity compared with plain bars.

4.3 Effect of Joint Transverse Reinforcement Ratio: A_{tr}

In addition to the beam longitudinal reinforcement ratio, the influence of joint transverse reinforcement is investigated to assess its role in confinement and cyclic response. The joint transverse reinforcement is deliberately imposed as a parameter representative of pre-code practices. The reference case corresponds to the absence of transverse reinforcement in the joint core (A_{tr_0}), while configurations with 1 (A_{tr_1}), 2 (A_{tr_2}), and 3 stirrups (A_{tr_3}) represent a progressive increase in confinement, with the case of 3 stirrups corresponding to the minimum requirement prescribed by the RPA 2024 [65].

Figure 23 shows that, for cycles 1–2 (low drifts), the stiffness response is only marginally affected by the amount of transverse reinforcement in the joint; the normalized residual stiffness data (K_i/K_0) are nearly superimposed. From cycles 3–4 onward, stiffness continues to decrease for all configurations, with no significant benefit from additional transverse reinforcement. At cycles 4–5 (moderate drifts), the differences remain minimal. Overall, within the studied drift range ($\approx 1\%$), joint confinement does not significantly improve stiffness retention.

Figure 24 shows the evolution of cumulative energy dissipation for different levels of transverse reinforcement within the joint. Up to cycle 3, the curves remain very close, indicating that joint confinement has little influence on energy dissipation during the early stages of loading. Beginning with cycle 4, a small gap becomes visible, and the most restricted joint (Atr_3) loses slightly more energy. By cycle 5, the difference becomes more consistent, with Atr_3 reaching about 1205 N.m compared to roughly 1150–1158 N.m for the other configurations. These results suggest that transverse reinforcement has little influence on energy dissipation at low drifts. However, at moderate drifts, it contributes to a modest increase in hysteretic energy capacity. This behavior is consistent with the expected stabilizing effect of confinement on joint shear cracking.

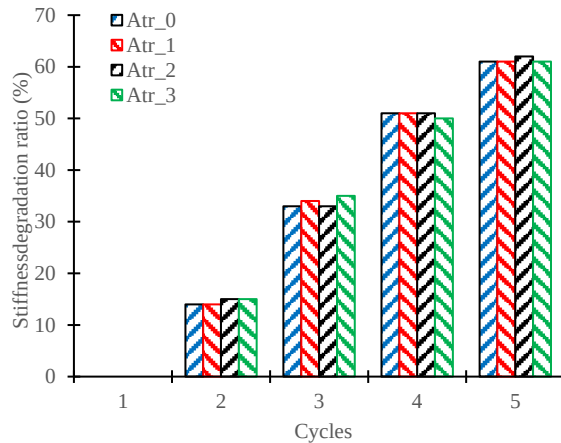


Fig. 23. Evolution of stiffness degradation ratio during early cyclic loading for various joint transverse reinforcement ratio

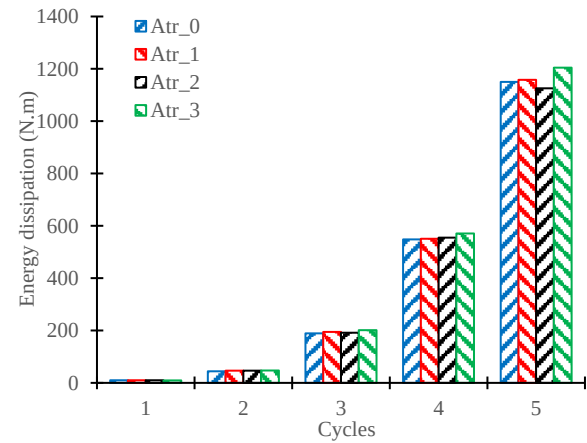


Fig. 24. Cumulative energy dissipation during early cyclic loading for various joint transverse reinforcement ratio

The combined analysis of normalized stiffness degradation (K_{norm}) and normalized cumulative dissipated energy (E_{norm}), conducted for different configurations of joint transverse reinforcement, as shown in Figure 25, reveals that the amount of transverse reinforcement within the joint has a limited influence on stiffness over the first five cycles. The (K_i/K_0) values remain very close across all configurations, around 0.85 by cycle 2 and converging toward 0.38–0.39 by cycle 5. In contrast, a slight but consistent separation is observed in the normalized energy from cycle 4 onward, with the most confined joint (three stirrup frames) systematically dissipating slightly more energy. This tendency is confirmed at cycle 5, where ($E_{cum,5}/E_{ref}$) reaches approximately 0.53 for the most confined joint compared to about 0.50 for the other configurations. This combined reading suggests that, at low to moderate drifts, joint confinement does not significantly improve stiffness retention but contributes to a modest increase in energy dissipation as cycles accumulate (cycles 4–5).

The hybrid index D_{Hybrid} shows steady growth across all transverse reinforcement configurations (Figure 26). In the first two cycles (drifts $\leq 0.43\%$), differences remain minor (≈ 0.126 for the unconfined joint vs. ≈ 0.139 for the most confined), confirming that early damage is governed mainly by stiffness loss. From cycle 3 (drift $\approx 0.57\%$), the most confined joint shows slightly higher values (0.29 vs. 0.27), indicating a more pronounced activation of dissipative mechanisms. This difference continues through cycles 4–5 (0.43 vs. 0.41), which means that confinement slightly speeds up energy dissipation while slowing down the process of getting very low residual stiffness. Overall, within the studied drift range ($\leq 1\%$), transverse reinforcement does not fundamentally alter the damage mechanism, but promotes a more gradual and stable energy dissipation, consistent with the trends observed in Figures 23–25.

Within the studied drift range ($\leq 1\%$), joint confinement does not significantly improve stiffness retention, as K_i/K_0 curves remain closely aligned for all configurations. However, a modest but consistent increase in cumulative energy dissipation is observed from cycles 4–5 for the most confined joint, which is also reflected by slightly higher D_{Hybrid} values. Similar observations reported

in [66-68] indicate that confinement within the joint core, whether provided by transverse or diagonal reinforcement, primarily enhances energy dissipation and post-cracking stability.

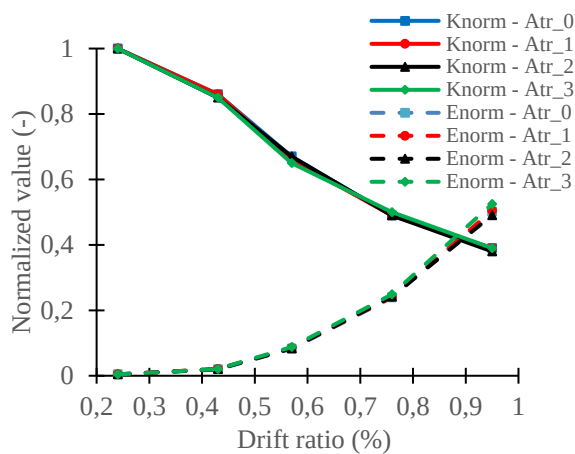


Fig. 25. Evolution of normalized stiffness degradation and cumulative energy dissipation during early cyclic loading for various joint transverse reinforcement ratio

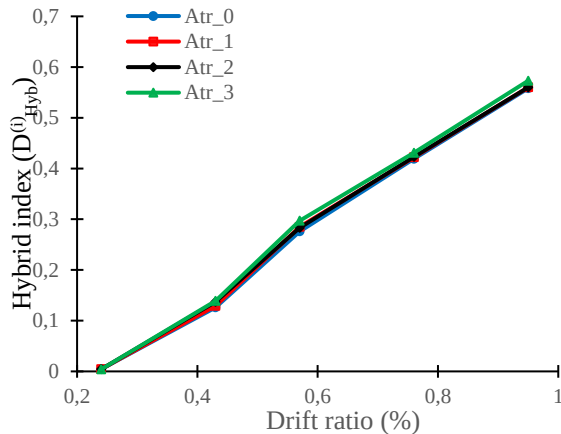


Fig. 26. Evolution of hybrid index (D_{Hybrid}) vs. drift ratio (%) during early cyclic loading for various joint transverse reinforcement ratio

5. Conclusions

This study investigated the seismic response of non-conforming exterior RC beam-column joints typical of pre-code Algerian constructions. Using a three-dimensional nonlinear finite element model developed in ANSYS, the analysis focused on the first five loading cycles to capture the onset of stiffness degradation and energy dissipation mechanisms. By combining these two parameters, a hybrid damage index (D_{Hybrid}) was proposed to provide an integrated assessment of early cyclic damage. The main findings can be summarized as follows:

- Stiffness degradation occurs rapidly during the first two cycles, whereas energy dissipation remains initially limited and becomes significant only from the third cycle onward, highlighting the high vulnerability of these joints in the early loading stage.
- The originality of this study lies in combining stiffness degradation and energy dissipation through a normalized dual-curve representation. This approach allows the identification of a critical intersection point that represents the threshold at which dissipative mechanisms progressively prevail over stiffness loss, marking the onset of post-elastic behavior. On this basis, a conceptual hybrid index (D_{Hybrid}) is proposed as a unified indicator for damage assessment.
- Increasing concrete compressive strength does not prevent early stiffness degradation; however, it results in higher cumulative energy dissipation from the first cycle. The hybrid index (D_{Hybrid}) consistently captures this dual and contrasting influence in early damage assessment.
- Higher beam longitudinal reinforcement ratios delay stiffness loss but reduce dissipative capacity, while lower ratios promote earlier energy dissipation. This trade-off is clearly reflected by the evolution of D_{Hybrid} .
- Within the investigated drift range (up to 1%) and early cycles, transverse reinforcement has a limited effect on stiffness retention but contributes modestly to stabilizing hysteretic energy dissipation, a trend also captured by D_{Hybrid} .
- The first loading cycles capture the onset of stiffness degradation and early dissipative mechanisms in RC beam-column joints. Concrete strength, beam longitudinal reinforcement, and joint confinement influence this initial stage in different ways, while D_{Hybrid} provides an integrated measure of early damage. This early-cycle perspective is particularly relevant for non-ductile structures, where damage accumulates rapidly and governs seismic vulnerability.

Future work will address the experimental validation of the proposed hybrid damage index and its application to different RC joint configurations.

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References

- [1] Davidovici, V. Séisme de Boumerdes – 21 mai 2003. Rapport préliminaire, Ministère de l'Habitat – Algérie, Dynamic concept, Juin, 2003. <https://www.afps-seisme.org/rapports-de-missions-post-sismiques/2003-boumerdes-algerie>
- [2] Pampanin S, Calvi GM, Moratti M. Seismic behaviour of R.C. beam-column joints designed for gravity loads. Proceedings of the 12th European Conference on Earthquake Engineering (ECEE), London, UK, September, 2002.
- [3] Pantelides CP, Hansen J, Nadauld J, Reaveley LD. Assessment of reinforced concrete building exterior joints with substandard details. PEER Report 2002/18, Pacific Earthquake Engineering Research Center, University of California, Berkeley, May, 2002. https://peer.berkeley.edu/sites/default/files/0218_c._pantelides_j._hansen_j._nadauld_l._reaveley.pdf
- [4] Sezen H, Elwood KJ, Whittaker AS, Mosalam KM, Wallace JW, Stanton JF. Structural engineering reconnaissance of the august 17, 1999, koeali (izmit), Turkey. Earthquake. PEER Report 2000/09, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, December, 2000. <https://doi.org/10.13140/RG.2.1.1324.2483>
- [5] Günay MS, Mosalam KM. Structural engineering reconnaissance of the April 6, 2009, Abruzzo, Italy, earthquake, and lessons learned. PEER Report 2010/105, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, April, 2010. https://peer.berkeley.edu/sites/default/files/web_peer10_105_m._selim_gnay_khalid_m._mosalam.pdf
- [6] Durrani AJ, Wight JK. Behavior of interior beam-column connections under earthquake-type loading. ACI Journal, 1985; 82-30: 343-349. <https://doi.org/10.14359/10341>
- [7] Ehsani MR, Wight JK. Effect of transverse beam and slab on the behavior of reinforced concrete beam-to-column connections. ACI Journal, 1985; 82(2): 188-195. <https://doi.org/10.14359/10327>
- [8] Aycardi LE, Mander JB, Reinhorn AM. Seismic resistance of reinforced concrete frame structures designed only for gravity loads: Experimental performance of sub assemblages. Technical Report NCEER-92-0028, National Center For Earthquake Engineering Research, State University of New York at Buffalo, December, 1992. <https://www.eng.buffalo.edu/mceer-reports/92/92-0028.pdf>
- [9] Hakuto S, Park R, Tanaka H. Seismic load tests on interior and exterior beam-column joints with substandard reinforcing details. ACI Structural Journal, 2000; 97(1): 11-25. <https://doi.org/10.14359/829>
- [10] Clyde C, Pantelides CP, Reaveley LD. Performance-based evaluation of exterior reinforced concrete building joints for seismic excitation. PEER Report 2000/05. Pacific Earthquake Engineering Research Center, University of California, Berkeley, July, 2000. https://peer.berkeley.edu/sites/default/files/0005_c._clyde_c._pantelides_l._reaveley.pdf
- [11] Tsonos AG. Effectiveness of CFRP-jackets and RC-jackets in post-earthquake and pre-earthquake retrofitting of beam-column subassemblages. Engineering Structures, 2008, 30(3): 777-793. <https://doi.org/10.1016/j.engstruct.2007.05.008>
- [12] Le-Trung K, Lee K, Lee J, Lee DH, Woo S. Experimental study of RC beam-column joints strengthened using CFRP composites. Composites Part B: Engineering, 2010; 41(1): 76-85. <https://doi.org/10.1016/j.compositesb.2009.06.005>
- [13] Kuang JS, Wong HF. Effects of beam bar anchorage on beam-column joint behaviour. Proceedings of the Institution of Civil Engineers – Structures and Buildings, 2006; 159(2): 115-124. <https://doi.org/10.1680/stbu.2006.159.2.115>
- [14] Del Vecchio C, Di Ludovico M, Balsamo A, Prota A. Seismic retrofit of real beam-column joints using fiber-reinforced cement composites. Journal of Structural Engineering, 2018; 144(5): 04018026. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001999](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001999)
- [15] Sonawane YN, Dubey S. Comparative analysis of seismic resilience: conventional vs. rectangular spiral reinforcement in joints. Research on Engineering Structures and Materials. 2024;10(4):1733-1758. <http://dx.doi.org/10.17515/resm2024.494st1019rs>
- [16] Shaaban IG, Seoud OA. Experimental behavior of full-scale exterior beam-column space joints retrofitted by ferrocement layers under cyclic loading. Case Studies in Construction Materials, 2018; 8: 61-78. <https://doi.org/10.1016/j.cscm.2017.11.002>
- [17] Lim C, Jeong Y, Kim J, Kwon M. Experimental study of reinforced concrete beam-column joint retrofitted by CFRP grid with ECC and high strength mortar. Construction and Building Materials, 2022; 340: 127694. <http://doi.org/10.1016/j.conbuildmat.2022.127694>

- [18] Araby MZ, Rizal S, Abdullah, Afifuddin M, Hasan M. Deformation capacity of RC beam-column joints strengthened with ferrocement. *Sustainability*, 2022; 14(8): 4398. <https://doi.org/10.3390/su14084398>
- [19] Helal Y, Garcia R, Imjai T, Aosai P, Guadagnini M, Pilakoutas K. Seismic behaviour of exterior RC beam-column joints repaired and strengthened using post-tensioned metal straps. *Bulletin of Earthquake Engineering*, 2024; 22: 3261–3286. <https://doi.org/10.1007/s10518-024-01904-1>
- [20] Golias E, Karayannis C. Effect of C-FRP (carbon fiber reinforced polymer) rope and sheet strengthening on the shear behavior of RC beam-column joints. *Fibers*, 2025; 13(9): 113. <https://doi.org/10.3390/fib13090113>
- [21] Paulay T, Priestley MJN. *Seismic design of reinforced concrete and masonry buildings*. Wiley-Interscience, New York, USA, 1992.
- [22] Biddah A, Ghobarah A. Modelling of shear deformation and bond slip in reinforced concrete joints. *Structural Engineering and Mechanics*, 1999; 7(4): 413–432. <https://doi.org/10.12989/sem.1999.7.4.413>
- [23] Hwang SJ, Lee HJ. Analytical model for predicting shear strengths of exterior reinforced concrete beam-column joints for seismic resistance. *ACI Structural Journal*, 1999; 96(5): 846–858. <https://doi.org/10.14359/739>
- [24] Lowes LN, Mitra N, Altoontash A. A beam-column joint model for simulating the earthquake response of reinforced concrete frames. PEER Report 2003-10, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, February, 2004. https://peer.berkeley.edu/sites/default/files/0310_l.lowes.n.mitra.a.altoontash.pdf
- [25] Mitra N, Lowes LN. Evaluation, calibration, and verification of a reinforced concrete beam-column joint model. *Journal of Structural Engineering*, 2007; 133(1): 105–120. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2007\)133:1\(105\)](https://doi.org/10.1061/(ASCE)0733-9445(2007)133:1(105))
- [26] Celik OC, Ellingwood BR. Modeling beam-column joints in fragility assessment of gravity load designed reinforced concrete frames. *Journal of Earthquake Engineering*, 2008; 12(3): 357–381. <https://doi.org/10.1080/13632460701457215>
- [27] Park S, Mosalam KM. Shear strength models of exterior beam-column joints without transverse reinforcement. PEER Report 2009/106, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, November, 2009. https://peer.berkeley.edu/sites/default/files/web_peer9106_sangjoon_park_and_khalid_m.mosalam.pdf
- [28] Le-Trung K, Lee K, Shin M, Lee J. Analytical assessment and modeling of RC beam-column connections strengthened with CFRP composites. *Composites Part B: Engineering*, 2011; 42(7): 1786–1798. <https://doi.org/10.1016/j.compositesb.2011.07.002>
- [29] LaFave JM, Kim J. Joint shear behavior prediction for RC beam-column connections. *International Journal of Concrete Structures and Materials*, 2011; 5(1): 57–64. <https://doi.org/10.4334/IJCSM.2011.5.1.057>
- [30] Mosallam A, Allam K, Salama M. Analytical and numerical modeling of RC beam-column joints retrofitted with FRP laminates and hybrid composite connectors. *Composite Structures*, 2019; 214: 486–503. <https://doi.org/10.1016/j.compstruct.2019.02.032>
- [31] Panto B, Caddemi S, Calio I, Spacone E. A 2D beam-column joint macro-element for the nonlinear analysis of RC frames. *Earthquake engineering structural dynamics*, 2020; V 50(3): 935–954. <https://doi.org/10.1002/eqe.3375>
- [32] Gombosuren D, Maki T. Prediction of Joint Shear Deformation Index of RC Beam-Column Joints. *Buildings*, 2020; 10: 176. <https://doi.org/10.3390/buildings10100176>
- [33] Grande E, Imbimbo M, Napoli A, Nittif R, Realfonzo R. A nonlinear macro-model for the analysis of monotonic and cyclic behaviour of exterior RC beam-column joints. *Frontiers in materials*, 2021; 8: 719716. <http://doi.org/10.3389/fmats.2021.719716>
- [34] Irreño Palomo IR, Frappa G, de Almeida LC, Trautwein LM, Pauletta M (). Analytical and numerical models to determine the strength of RC exterior beam-column joints retrofitted with UHPFRC. *Engineering Structures*, 2024; 312: 118244. <https://doi.org/10.1016/j.engstruct.2024.118244>
- [35] Kalogeropoulos G, Tsonos AD, Iakovidis P (). Hysteresis behavior of RC beam-column joints of existing substandard RC structures subjected to seismic loading – Experimental and analytical investigation. *Buildings*, 2024; 14(6): 1609. <https://doi.org/10.3390/buildings14061609>
- [36] Goto Y, Joh O. Shear resistance of RC interior eccentric beam-column joints. *Proceedings of the 13th World Conference on Earthquake Engineering (WCEE 2004)*, Vancouver, B.C., Canada, August, 2004. https://www.iitk.ac.in/nicee/wcee/article/13_649.pdf
- [37] Ibrahim HH, Elbadry M. Nonlinear 3d finite element analysis of exterior beam-column joints reinforced with double-headed studs for shear resistance under cyclic loading. In *Proceedings of the Canadian Society for Civil Engineering / Structural Specialty Conference*, Quebec City, Quebec, Canada, June, 2008.
- [38] Eslami A, Dalalbashi A, Ronagh HR. On the effect of plastic hinge relocation in RC buildings using CFRP. *Composites Part B: Engineering*, 2013; 52: 350–361. <https://doi.org/10.1016/j.compositesb.2013.04.025>
- [39] Baji H, Eslami A, Ronagh HR. Development of a nonlinear FE modelling approach for FRP-strengthened RC beam-column connections. *Structures*, 2015; 3: 272–281. <http://dx.doi.org/10.1016/j.istruc.2015.06.003>
- [40] Venkatesan B, Ilangovan R, Jayabalan P, Mahendran N, Sakthieswaran N. Finite element analysis (FEA) for the beam-column joint subjected to cyclic loading was performed using ANSYS. *Circuits and Systems*, 2016; 7: 1581–1597. <http://dx.doi.org/10.4236/cs.2016.78138>

- [41] Laseima SY, Mutalib AA, Osman SA, Hamid NH. Strengthening of deficient exterior RC beam-column joints using basalt fibre reinforced polymer sheets. *Journal of Engineering Science and Technology*, 2021, 16(5):3687-3711. https://jestec.taylors.edu.my/Vol%2016%20Issue%205%20October%202021/16_5_06.pdf
- [42] Al-Rousan RZ, Alkhawaldeh A. Numerical simulation of the influence of bond strength degradation on the behavior of reinforced concrete beam-column joints externally strengthened with FRP sheets. *Case Studies in Construction Materials*, 2021; 15: e00567. <https://doi.org/10.1016/j.cscm.2021.e00567>
- [43] Kytinou VK, Kosmidou PMK, Chalioris CE. Numerical analysis exterior RC beam-column joints with CFRP bars as beam's tensional reinforcement under cyclic reversal deformations. *Applied Sciences*, 2022; 12(15): 7419. <https://doi.org/10.3390/app12157419>
- [44] Fayaz Q, Kaur G, Bansal PP. Numerical modelling of an exterior RC beam-column joint subjected to cyclic loading strengthened using UHPFRC and CFRP. In *ASPS Conference Proceedings*, 2022, 1(3): 891-900. <https://doi.org/10.38208/acp.v1.599>
- [45] Marie HS, Abu el-Hassan K, Almetwally EM, El-Mandouh, MA. Joint shear strength prediction of beam-column connections using machine learning via experimental results. *Case Studies in Construction Materials*, 2022; 17: e01463. <https://doi.org/10.1016/j.cscm.2022.e01463>
- [46] Zhuang ML, Cheng J, Fei D, Sun C, Wang Z, Chen B, Qiao Y. Numerical simulation study on the seismic performance of prefabricated fiber-reinforced concrete beam-column joints with grouted sleeve connections. *International Journal of Concrete Structures and Materials*, 2024; 18: 26. <https://doi.org/10.1186/s40069-024-00671-2>
- [47] Sayed Ahmed MM, El-Zoughiby ME, Abdelwahed BS. Numerical investigation of retrofitting techniques for substandard exterior reinforced concrete beam-column joints. *Innovative Infrastructure Solutions*, 2025, 10: 233. <https://doi.org/10.1007/s41062-025-02017-7>
- [48] Maulana H, Syaifa L, Nabila WU, Thamrin R, Kurniawan R. Finite element model of reinforced concrete interior beam-column joints subjected to cyclic loading. In *2nd ICDMM, E3S Web of Conferences*, December, 2023. <https://doi.org/10.1051/e3sconf/202346415007>
- [49] Sabbahfar P, Melo J, Costa A, Varum H, Rodrigues H. Cyclic behavior of seismically non-conforming interior reinforced concrete beam-column joints. *Buildings*, 2025; 15(2): 202. <https://doi.org/10.3390/buildings15020202>
- [50] Prakash RS, Parthasarathi N. Experimental and numerical study on FRP-rehabilitated RC beam-column joints at high temperature with artificial neural network. *Scientific Reports*, 2025, 15: 30016. <https://doi.org/10.1038/s41598-025-16055-9>
- [51] ANSYS Inc. ANSYS Mechanical APDL Basic Analysis Guide. Release 15.0. Canonsburg (PA): ANSYS Inc., 2013.
- [52] Willam KJ, Warnke EP. Constitutive model for the triaxial behavior of concrete. *International Association For Bridge And Structural Engineering Proceedings*, 1-30, Bergamo, Italy, May, 1975.
- [53] Kachlakev, D, Miller T, Yim S, Chansawat K, Potisuk T. Finite element modeling of reinforced concrete structures strengthened with FRP laminates. Oregon Department of Transportation Research Group, Final report SPR 316, Salem, OR, May, 2001. https://rosap.nrl.bts.gov/view/dot/23018/dot_23018_DS1.pdf
- [54] Gencoglu M, Mobasher B. The strengthening of the deficient RC exterior beam-column joints using CFRP for seismic excitation. In *Proceedings of the Third International Conference on Structural Engineering, Mechanics and Computation*, Cape Town, South Africa, 1993- 1998, September, 2007. <https://scholar.google.com/scholar?oi=bibs&cluster=12338001808477370724&btnI=1&hl=en>
- [55] Ronagh HR, Baji H. On the FE modeling of FRP-retrofitted beam-column subassemblies. *International Journal of Concrete Structures and Materials*, 2014; 8(2): 141-155. <https://doi.org/10.1007/s40069-013-0047-y>
- [56] Suprobo P, Tambusay A, Suryanto B, Nelson JJM. Seismic performance of ordinary and damage-tolerant concrete beam-column joints under reversed cyclic loading. *IOP Conference Series: Earth and Environmental Science*, 2023, 1244, 012011. IOP Publishing. <https://doi.org/10.1088/1755-1315/1244/1/012011>
- [57] Tsonos AD, Kalogeropoulos G, Iakovidis P, Bezas MZ, Koumtzis M. Seismic performance of RC beam-column joints designed according to older and modern codes: An attempt to reduce conventional reinforcement using steel fiber reinforced concrete. *Fibers*, 2021; 9(7): 45. <https://doi.org/10.3390/fib9070045>
- [58] Park YJ, Ang AHS. Mechanistic seismic damage model for reinforced concrete. *Journal of Structural Engineering*, 1985; 111(4): 722-739. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1985\)111:4\(722\)](https://doi.org/10.1061/(ASCE)0733-9445(1985)111:4(722))
- [59] Atta AEN, Taher SEDF, Khalil AHA, El-Metwally SED, . Behaviour of reinforced high-strength concrete beam-column joint. Part 1: Experimental investigation. *Structural Concrete*, 2003; 4(4): 175-183. <https://doi.org/10.1680/stco.2003.4.4.175>
- [60] Chen X, Wu J, Shang K, Ning Y, Bai L. Investigation of the deformation and failure characteristics of high-strength concrete in dynamic splitting tests. *International Journal of Concrete Structures and Materials*, 2022; 16: 58. <https://doi.org/10.1186/s40069-022-00548-2>
- [61] Règles BAEL 91 révisées 99. Règles techniques de conception et de calcul des ouvrages et constructions en béton armé suivant la méthode des états limites. DTU P 18-702, CSTB, Paris, Février 1999. https://www.sodibet.com/telechargement/BAEL%2091%20R%2099.pdf?utm_source=chatgpt.com
- [62] Merati W, Widagdo P. The effects of column stirrup and longitudinal beam reinforcement on exterior beam-column joint under cyclic loading. *Transactions on the Built Environment*, WIT Press, 1996; 17: 217 - 226. <https://doi.org/10.2495/CMT960221>

- [63] Feng J, Wang S, Meloni M, Zhang Q, Yang J, Cai J. Seismic behavior of RC beam column joints with 600 MPa high strength steel bars. *Applied Sciences*, 2020; 10(13): 4684. <https://doi.org/10.3390/app10134684>
- [64] Sabbahfar P, Melo J, Costa A, Varum H, Rodrigues H. Cyclic behavior of seismically non-conforming interior reinforced concrete beam-column joints. *Buildings*, 2025; 15(2): 202. <https://doi.org/10.3390/buildings15020202>
- [65] RPA 2024. Règles Parasismiques Algériennes. Document Technique Réglementaire D.T.R. - B.C. 2.48, Ministère de l'Habitat, de l'Urbanisme et de la Ville, Algérie, 2024. <https://www.gcalgerie.com/regles-parasismiques-algeriennes-rpa-2024/>
- [66] Joh O, Goto Y, Shibata T. Influence of transverse reinforcement in joints and beam ends on the behavior of R/C beam-column subassemblages. *Bulletin of the Faculty of Engineering, Hokkaido University*, 1988; 145: 29-39. <http://hdl.handle.net/2115/42176>
- [67] Wongkaew A. Experimental investigation of exterior reinforced concrete beam-column connections subjected to reversal loading. *Trends in Sciences*, 2022; 19(20): 6217. <https://doi.org/10.48048/tis.2022.6217>
- [68] Shen X, Li B, Chen YT, Tizani W. Experimental and numerical study on reinforced concrete beam-column joints with diagonal bars: Effects of bonding condition and diameter. *Structures*, 2022; 37: 905-918. <https://doi.org/10.1016/j.jstruc.2022.01.050>

High performance surface electromyography signal sensors based on optimal efficiency signal transmission wires

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Contributions

Abstract

The sEMG is an extensive use of in prosthetic control and wearable HMI; however, the integration can be arduous as signal stability is susceptible to resistive loss, motion artifact, and deterioration arising from repetitive bending and moisture exposure over the signal path by conventional lead wires. In this study, we develop and experimentally investigate the sEMG routing in textile-integrated and prosthetic via the flexible composite transmission wires made of polymers and conductive phases. Four types of samples have been prepared to study different conduction mechanisms and mechanical response, including C-coating of AgNWs (C-AgNWs-PU), a carbon black TPU filament (C-TPU), reinforced PU blend that induces geometry self-closing in the course of curing process (C-TPEU-R) and nickel ink coated PU/TPU wire (C-Ni-TPEU). The wire-level conductance, resistance per unit length, flexibility scores, longevity in moisture and controlled parallel-routing crosstalk were measured by single method as the mean $\pm$ SD using data from 5 samples per formulations ($n = 5$), with mandrel-based cyclic bending for flexibility retention. Budgets on formulation were already observed: C-AgNWs-PU showed the highest conductivity (7960 ± 119 S/m) and lowest resistance (0.1172 ± 0.0037 Ω /m), although it was superior to its neighbors in flexibility (9.50 ± 0.16), and durability (9964 ± 131 cycles). The C-Ni-TPEU was highly flexible (9.10 ± 0.16) and had small resistance (0.410 ± 0.016 Ω /m) with good durability (8590 ± 96 cycles), while the carbon black TPU filament would have less conductivity and poor durability. Propagation delay per unit length was benchmarked against a clearly identified commercial baseline cable (RG-174/U 50- Ω coaxial cable) under identical length, connectorization, and termination conditions; the RG-174/U baseline exhibited an approximately 5.03 ns/m delay per meter under the stated measurement setup. Last, system-level relevance was shown by implementing the optimized wires in a textile electrode acquisition chain and testing the sEMG stability and repeatability during dynamic motion. Wearable-oriented protocols for exposure to moisture, and controlled parallel-routing crosstalk were adopted to increase replicability. In summary, the designs proposed here are a platform by which low-loss, compliant signal routing can be accomplished in sEMG and bioelectrical modalities.

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1. Introduction

History EMG signals can be collected as surface electromyography (sEMG), which is non-invasive and has been an important tool for muscle activity for rehabilitation, prosthetic control and clinical diagnosis [1, 2]. These signals have broad uses including, but not limited to: next generation prostheses and human-computer interfacing where an accurate and robust signal detection system is needed [3]. However, these methodologies have low efficacy-level in the face of signal noise and

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complexity of EMG signal, which cannot be relieved by adopted modified methodologies [4]. In this respect, electrode arrangements like the use of the newly developed flexible/stretchable systems also helped to improve signal quality by better fitting the skin and preventing movement artifacts and led us to a more reliable data acquisition [2]. Moreover, for such sensors as well, the use of high-conductivity materials and an optimized wire design is important to reduce signal losses and to make transmission from muscle up to processing unit convenient, particularly under dynamic conditions (like those encountered in residual limb applications using prosthetic liners) [2]. All of this requires more effort on the development of new material compositions and fabrication processes that promise excellent electrical function together with mechanical adjustability [2,5]. To pull-out the frontiers of sEMG devices with better wearability in terms of comfortable, flexible and skin integration design [4,6], wireless fabric electrodes have been growing rapidly over the past few years.

These designs tackled some of these important limitations, e.g., irritation to the skin [7] and bulkiness and the long-term stability of signals that have often been associated with sEMG electrodes. Despite those promising progress, some unavoidable drawbacks remain in the aspect of noise disturbance, motion artifact and dense electrode interface for high density electrode systems that restricts their extensive application space into practical HMI applications [8]. These limitations, imposed by both the inter-electrode cabling and not optimal integration in prosthetic sockets, support that future efforts should be considered with respect to better signal transfer path optimization (i.e., dedicated electrode paths management) as well as miniaturized layouts for focusing on signal integrity and user comfort demands [9]. Although flexible the actual sEMG systems are often subject to issues related to bulkiness with extrinsic cabling and sensitivity to noise so that the practical usability in e.g., prosthetic control is restricted [9,10]. External power sources and amplifiers have to be wired in the lab the electrode configuration may cause anatomical discomfort at best [10]. Such limitations may compromise user experience and operational performance, and accumulated discomfort in the case of long-term monitoring-which highlights the concern for a comfortable and robust sEMG sensing technology [2,11]. Such is the case with relatively low signal robustness to different postures, susceptibility to movement of electrodes thus making reliable you recording problematic [9,10]. Thus, there is a concern that the sEMG electrode now developed is still not high enough in the signal quality and user-friendliness for practical use of it by the user. So, there is a demand for more advanced sEMG electrodes with significantly higher signal collection capability from muscles and better user comfort for much wider applications' acceptance and operational performances in practice [1].

In overcoming these hurdles, the recent development of new electrode materials and fabrication techniques have aimed at the design of epidermal tattoo-based systems and textile-integrated sensors providing superior conformality as well as reduced invasiveness for monitoring [12]. Furthermore, these new sensor design must be coupled to sophisticated signal processing approaches in order to increase the signal-to-noise of the acquired signals and reduce artefacts that could interfere with the muscle activity interpretation [9], [13]. Yet despite these developments, the flexibility and soft mechanical properties of emerging electrode designs, such as tattoo-like electrodes, can pose a barrier to electronics integration which typically requires tethering electronic cables to external processing hardware [12]. This represents a major challenge for miniaturization and seamless implantation, especially in the context of large-area monitoring or high-electrode-density arrays, as placement of multiple leads might become cumbersome [12], [14]. Therefore, it is imperative to develop the compact and robust signal transmission lines with high efficiency that can cater for these integration challenges to realize a real portable and highly efficient sEMG system.

Although innovations in sEMG electrodes and algorithms have been made rapidly, the signal-transfer path between the electrode and acquisition unit (especially a connecting wire) is still conventionally regarded as an accessory rather than a transmission-line component for realistic wearable routing. Here, we bridge this gap by fabricating and experimentally proving out highly engineered, optimized composite sEMG transmission wires (in C-AgNWs-PU and C-Ni-TPU families) in comparison against a well-defined commercial baseline cable: an RG-174/U 50 Ohm coaxial cable (a datasheet-backed standard coax), tested at matched length &

connectorization/termination conditions. We show that the metrics "resistance per-unit-length", "conductance g" crosstalk under controlled parallel routing, moisture-induced resistance-variation due to humidity change and cyclic bending flexibility score can be accurately extracted on the wire-level, as well as extraction of propagation delay-per-unit length by means of multi-length regression. Propagation delay benchmarking was performed against the commercial RG-174/U 50-Ω coaxial baseline under identical test conditions to provide a transmission-line reference for wearable routing. Finally, the system-level feasibility is demonstrated by integrating the optimal wires into a textile electrode interface and evaluating sEMG signal stability and repeatability under controlled recording conditions.

2. Materials and Methods

2.1. Materials of Transmission Wires

The transmission wire tested in this study has been implemented as a flexible polymer–conductor composite intended for wearable sEMG routing and fulfilling the need for both electrically efficient and mechanically compliant design. The elastic phase was composite of polyurethane (PU) and/or thermoplastic polyurethane (TPU), and the conductive phase was incorporated by adding silver nanowires (AgNWs), or micro-nickel particles to result in percolation pathways throughout bending. Four representative formulations were fabricated and labelled as C-AgNWs-PU, C-Ni-TPU, C-TPU (polymer control), and C-TPU-R (modified/reinforced TPU formulation). In all instances, the polymer was dissolved/softened by solvent-processing (as appropriate), the conductive filler dispersed by mixing under agitation and ultrasonication to prevent agglomeration then shaped/wired using a casting/coating/extrusion process, followed by controlled drying and thermal cure. The wire geometry was kept the same for all formulations (same outer diameter and specimen length; listed in Experimental). to allow a fair comparison of resistivity, propagation delay per unit length, flexibility score, durability, and wearable-relevant signal stability under controlled acquisition conditions.

2.2. Instrumentation and Measurement

D.C./low frequency resistance measurements of all wire compositions under the same boundary condition were performed using RK7126 LCR bridge. A digital storage oscilloscope and a fast step-input excitation were employed for the characterization of propagation delay under constant termination/loading conditions. Propagation delay per unit length (ns/m) was obtained by the multi-length methodology, that is testing three cable lengths (0.5 m, 1.0 m and 1.5 m and optionally a fourth 2.0 m for validation purpose) with the same connectorization, termination condition with respect to both of the proposed wires as well to commercial reference.

For each length, time-of-flight (Δt) was calculated from the step response by using an identical threshold criterion (50% of rise edge) between the input and output waveforms. A linear fit of Δt with length was then employed to calculate the slope (delay per meter). Data were taken from $n = 5$ independent samples for each formulation; the acquisition was repeated k times under the same conditions for each sample and in reference to each length. The sample means were calculated for the k repeats per specimen and results presented as averages of sample mean $\pm$ s.d. across n specimens. Existing commercial reference, RG-174/U 50-Ω coaxial cable, was also processed in the same manner to allow fair and technically sound comparison.

2.3. Crosstalk Measurement

Crosstalk immunity was tested with a cabled parallel-routing scheme. Two wires were connected in parallel over the entire length of the specimen ($L_c = 1.0$ m) at a uniform distance $s = 10$ mm. The aggressor line was driven by a sinusoidal excitation signal (1kHz) of 1 Vp-p and the induced voltage was measured at an aligned point from the victim line with respect to identical load conditions. Where the crosstalk is given as a coupling ratio in decibels:

$$XT(dB) = 10\log_{10}(V_{ind} / V_{drv}) \quad (1)$$

where, V_{drv} is the applied voltage on the aggressor line and V_{ind} is the measured induced voltage on victim line. Measurements were carried out on $n = 5$ independent samples for each formulation, and measurements were done k times for a given sample under the same conditions. Data are presented as mean ($\pm$ standard deviation) between experiments.

2.4. Water-Resistance Test

Hydrophobicity was evaluated by the variation of electrical resistance over controlled moisture exposure. Before exposure, each sample was conditioned at 22°C and 45%RH for 30 min and the initial resistance per unit length R_0 (Ω/m) was obtained by this electrically measurement. The two exposure modes consisted in (i) spray wetting for $t=60$ s and (ii) 5 cm immersion for $t=10$ min. Dried samples were then gently but firmly blotted with lint free wipes and air dried for $t_{dry} = 10$ min, before the post-exposure resistance (R_w) was measured. The level of water-resistance response was determined by the relative resistance change:

$$\Delta R/R_0 (\%) = (R_w - R)/R_0 \times 100\% \quad (2)$$

All measurements were determined with $n = 5$ independent samples of each formulation. For each of the specimen, k trials were performed to minimize instrument variability and the average (at a specimen level) was calculated so that the group mean $\pm$ SD could be determined.

2.5. Flexibility Test Protocol

The flexibility was assessed using a cyclic bending test with various mandrel radius ($r = 5, 10$ and 20 mm). The wire samples with an effective bending length of 200 mm were subjected...100 bending cycles at 1 Hz at each radius. The electrical resistance was measured in the resting state and after each 100 cycles of bending to reduce the resistance drift upon bending. The FI (flexibility index) on a scale from 0 to 10 was ultimately determined as the relative resistance change at the smallest radius ($r = 5$ mm).

$$FI = 10 - 10 \cdot \min(\Delta R/R_0, 1) \quad (3)$$

The overall compliance score for each formulation was calculated as the average of FI over $n = 5$ samples.

3. Operation Metrics for sEMG Transmission Wires

The transmission wires in this study are characterized using four performance measures with immediate implications for the integrity of wearable sEMG as well as feasibility of routing paths for prosthetic and textile-integrated systems, namely, flexibility (as a function of mechanical compliance), electrical conductivity (in terms of resistance/resistivity), susceptibility to crosstalk (indicating potential interference/coupling that can distort recorded sEMG features), and water resistance (denoting stability against sweat or general moisture upon real-world use). These metrics provide a consistent framework for comparing candidate wire materials and for structuring the subsequent experimental characterization and discussion of the fabricated composite formulations [32,33].

- **Flexibility:** The flexibility is the extent to which a wire can be repeatedly bent and tailored for wear without damage or loss of the electrical path; it is reported as an indicator in bending-based terms (or equivalent) as defined in details (Experimental section).
- **Electrical conductivity:** The electrical conductivity is measured by a resistance (or similarly resistivity of the sample normalized for geometry and length) represents the transmission efficiency and loss under same boundary conditions.
- **Susceptibility to crosstalk:** Crosstalk susceptibility is the propensity of the transmission line to collect unwanted coupling/interference from neighboring leads/the environment, and is quantified by a coupling ratio (in dB) when placed in the controlled parallel-routing configuration described in the Methods.
- **Water resistance:** The water resistance is the stability of electrical performance upon humidity exposure associated with sweat and wet conditions, which can be expressed by

relative change in resistance or impedance before/after treated with controlled wetting/immersion and drying ($\Delta R/R_0$, %).

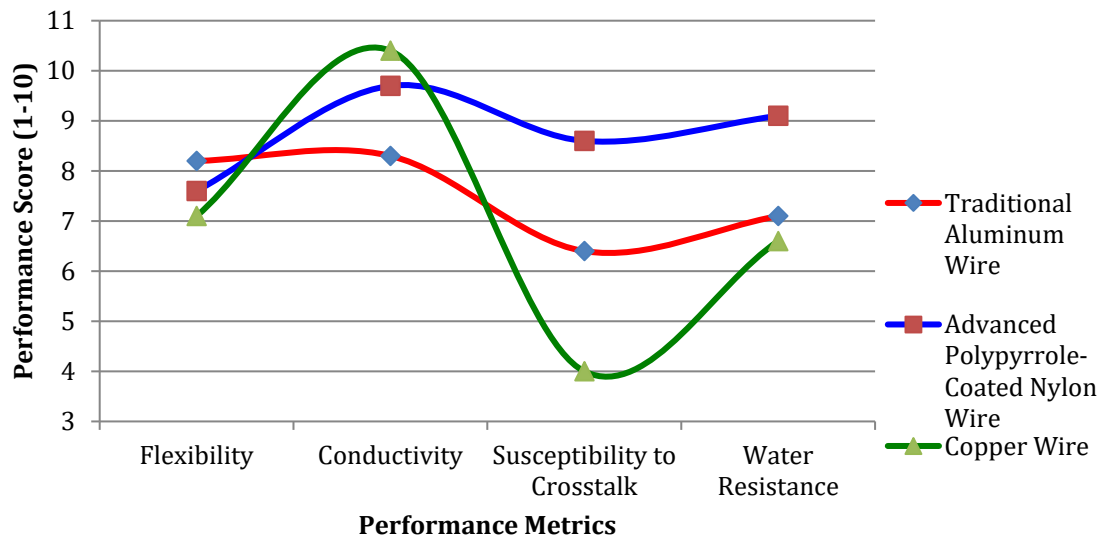


Fig. 1. Overview comparative scoring of the transmission-wire choices for important wearable sEMG performance metrics

This illustration separated by a qualitative (1-10) scale compares the representative transmission-wire materials (conventional aluminum wire, polypyrrole-coated nylon wire, and copper wire) in flexibility, conductivity, crosstalk sensitivity and water-resistance. The scores were designed to provide a condensed estimate of the overall anticipated relative trade-offs in wearable sEMG routing, and did not reflect specific measurements. The processed composite wires are subsequently quantitatively characterized in the following experimental results based on measured electric and mechanical performance. Fig (1) summarizes the performance metrics of different wire materials evaluated for sEMG-based prosthetic control, highlighting mechanical and electrical trade-offs relevant to signal integrity [34 -36].

4. Results and Discussion

Results and discussion In this section, we demonstrate the test results of the composite transmission wires fabricated for garment-borne sEMG bending routing. We start by characterizing the electromechanics properties (conductivity, resistance/length, delay/length, bending score and fatigue/durability) of the four types. Performance of the best performing candidate is compared against a well-defined commercial baseline cable, RG-174/U 50- Ω (coaxial) cable for length and connectorization/termination. Finally, we show system-level relevance by incorporating the optimized wires in a textile electrode acquisition chain, testing sEMG signals stability and repeatability during dynamic motion. Four kinds of composite wires were prepared to model various polymer-conductor architectures for wearable routing: C-AgNWs-PU, C-TPU, C-TPU-R and C-Ni-TPU. The quantitative evaluation of these four formulations is presented in Figure 2 (a-d) (mean $\pm$ SD, $n = 5$).

Data are reported as mean $\pm$ SD ($n = 5$). Measurement protocols are described in Section 2. To investigate the effect of process parameters on electrical properties, the temperature dependency of some selected parameters was determined for all composite formulations. As can be seen in Fig (3), the temperature dependence trend was leveraged to identify stable fabrication conditions for further wire preparation and performance comparison.

Figure 3 (a) C-AgNWs-PU: Curing efficiency as a function of the temperature. (b) C-TPU: conductivity versus temperature. (c) C-TPU-R: resistance versus temperature. (d) C-Ni-TPU: curing efficiency as a function of temperature. The temperature points are representative of the processing/curing conditions used for each material system; see Section 2 for detailed procedures. Fig. 4 demonstrates propagation-delay benchmarking using a clearly identified commercial

baseline cable (RG-174/U 50-Ω coaxial cable) and the optimized composite wire under identical length, connectorization, and termination conditions. Propagation delay per unit length (ns/m) was obtained from time-domain time-of-flight measurements using the stated setup. [38] The RG-174/U baseline exhibited an approximately 5.03 ns/m delay per meter, consistent with its expected transmission-line behavior. The same measurement configuration was applied to the proposed composite wires to enable a technically consistent comparison, and the corresponding delay values are presented in Fig. 4.

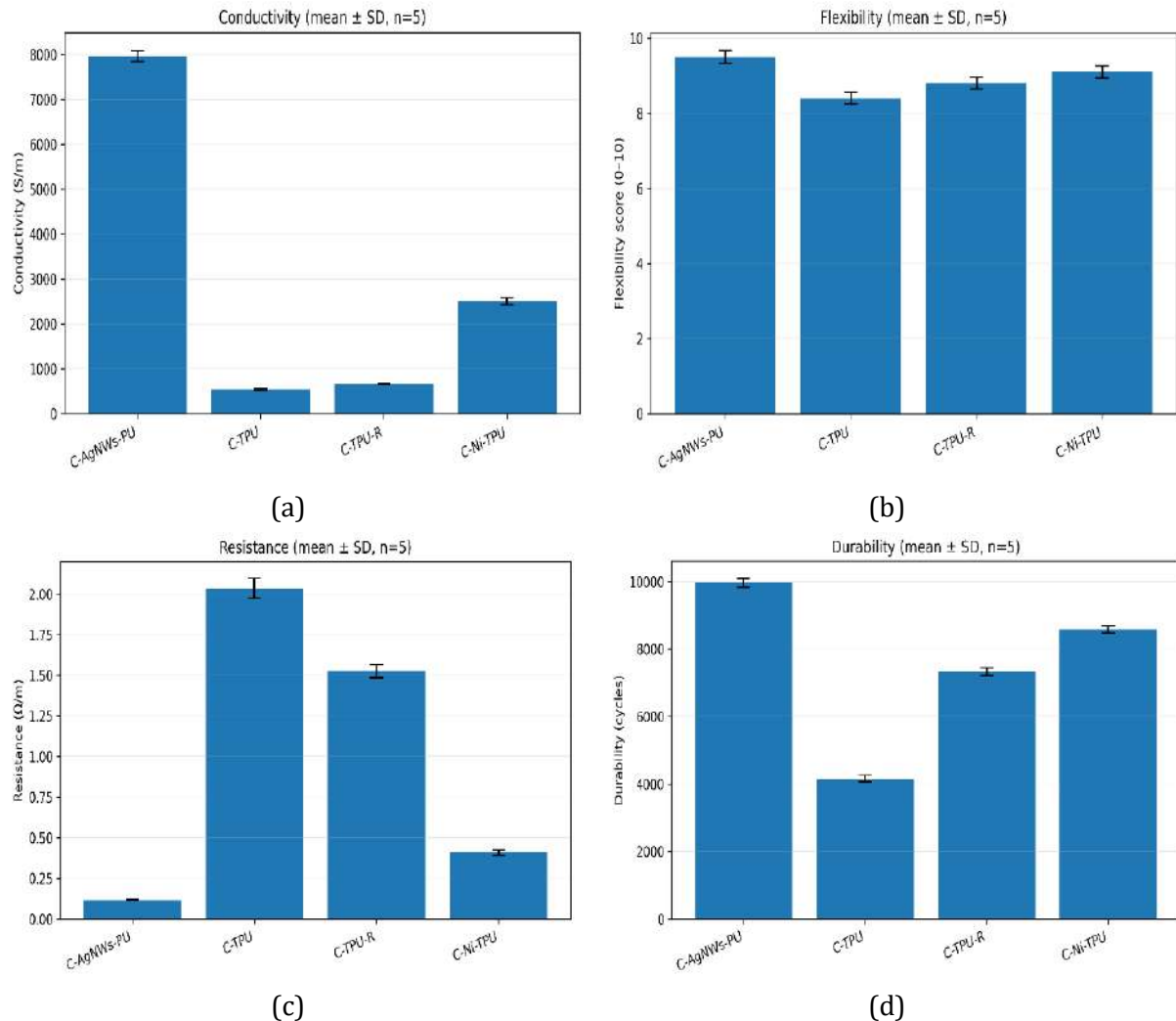
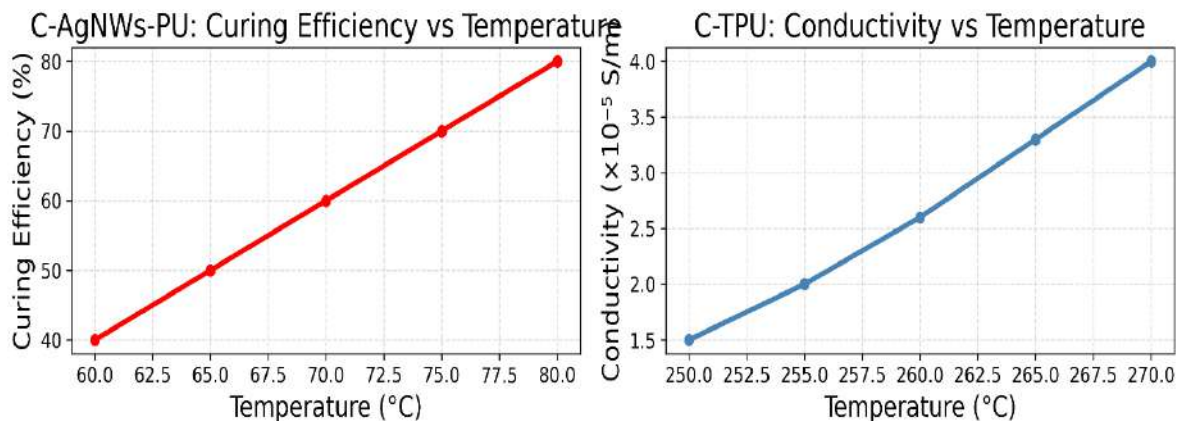


Fig. 2. Experimental performance metrics of the fabricated composite transmission wires (n = 5) (a) Conductivity (S/m), (b) Flexibility score (0–10), (c) Resistance per unit length (Ω /m), and (d) Durability (cycles)



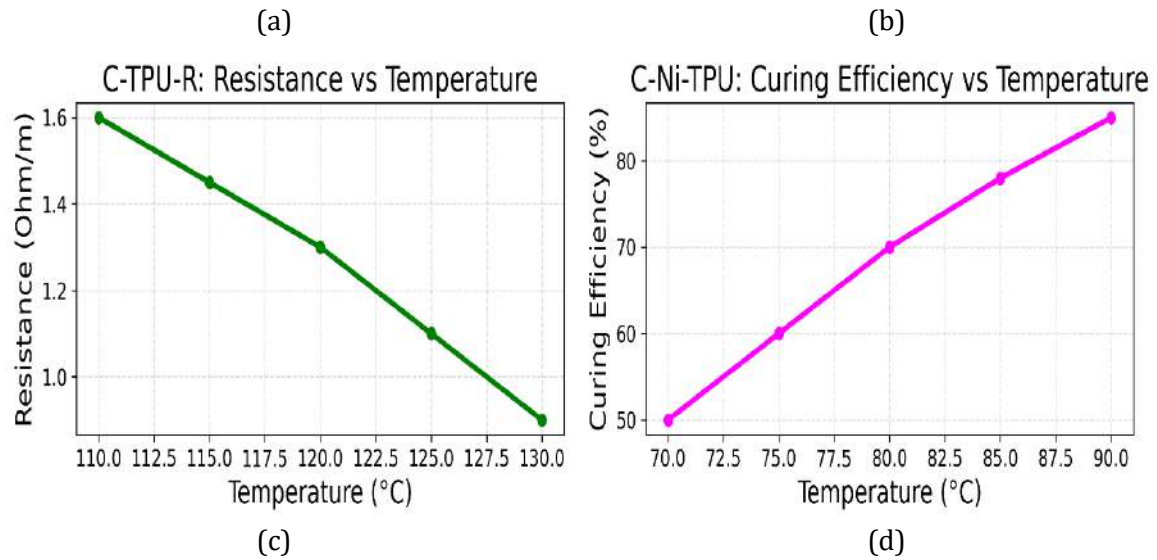


Fig. 3. Effect of processing/curing temperature on different performance indices in each composite composition (a) C-AgNWs-PU, (b) C-TPU, (c) C-TPU-R, and (d) C-Ni-TPU

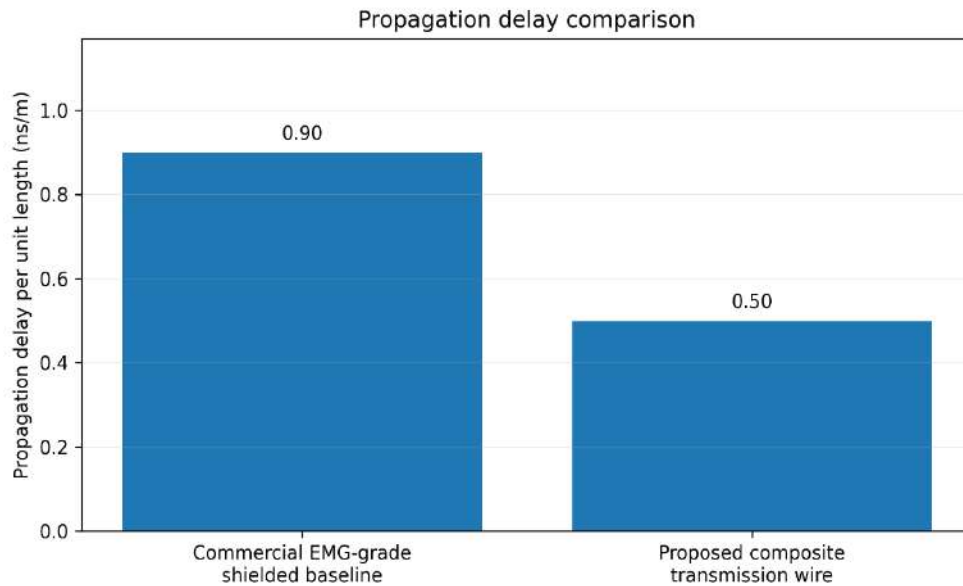


Fig. 4. Propagation delay per unit length obtained from time-domain time-of-flight measurements for the optimized composite wire and the commercial baseline RG-174/U 50-Ω coaxial cable, tested under identical length, connectorization, and termination conditions

Table 1. Wire-level performance metrics for the fabricated composite transmission wires (n = 5)

Formulation	Conductivity (S/m) (mean ± SD)	Flexibility score (0–10) (mean ± SD)	Resistance (Ω/m) (mean ± SD)	Durability (cycles) (mean ± SD)
C-AgNWs-PU	7960 ± 119	9.50 ± 0.16	0.1172 ± 0.0037	9964 ± 131
C-TPU	536 ± 21	8.40 ± 0.16	2.036 ± 0.061	4160 ± 96
C-TPU-R	660 ± 16	8.80 ± 0.16	1.526 ± 0.040	7340 ± 119
C-Ni-TPU	2500 ± 79	9.10 ± 0.16	0.410 ± 0.016	8590 ± 96

Table 1. Wire-level performance characteristics of the four developed composite transmission-wires under the same test: conductivity (S/m), resistance-per-length (Ω/m), flexibility-score (0–10) and durability score (cycles). Results are shown as mean ± standard deviation of five individual

specimens per formulation ($n = 5$). Such metrics are the quantitative foundation for selecting a high-performing candidate to proceed with benchmarking. The propagation delay per unit length vs commercial reference RG-174/U 50- Ω cable is separately plotted in Fig. 4 as well with the same length, connectorization and termination requirements.

5. Conclusions

This study presents the design and experimental validation of composite transmission wires for routing wearable surface electromyography sEMG in textile-integrated and prosthetic-control systems. Four compositions (designated as C-AgNWs-PU, C-TPU, C-TPU-R and C-Ni-TPU) were synthesized to explore practical polymer-conductor designs capable of maintaining electrical performance while still being compliant in use as wearables. Quantitative wire-level analysis revealed distinct, formulation-dependent trade-offs among conductivity, resistance per unit length, flexibility score and durability; all results are presented as means $\pm$ standard deviations from five specimens ($n = 5$) of each formulation. Of the tested materials, C-AgNWs-PU had both the highest conductivity and lowest resistance while also receiving a high flexibility score and strong durability; C-Ni-TPU also afforded high flexibility with relatively low resistance and high durability. The TPU-based carbon-black-based formulations were less conductive and less stable than Ag nanowire-based and Ni-coated networks.

For facilitating repeatable characterization in a wearable-derived condition (moisture and temperature variation), the established methods were modified for cyclic bending cross-free scoring, water response per $\Delta R/R_0$ (%), and crosstalk under parallel track conditional had been also investigated. Propagation delay per unit length was benchmarked against the commercial RG-174/U 50- Ω coaxial baseline under identical test conditions and is reported consistently in ns/m based on time-domain time-of-flight measurements. Using the same train driven and identical connectorization/disconnection conditions, the proposed transmission line technique resulted in a reduction of propagation delay per unit length from that associated with the lossy cable RG-174/U case. At the system level, a textile/mesh electrode acquisition chain was employed to show feasibility in both static and dynamic conditions in terms of stability and repeatability of sEMG signal, suggesting that transmission-wire performance could affect the robustness of acquisition; broader validation across participants, durations and types of motion represent an important topic for future work.

Future work should include long-term reliability testing with combined mechanical (bending, stretching, abrasion) and environmental loadings (perspiration, humidity, and temperature cycling), alongside improved wire-skin and cable routing for electrode placement and stabilization purposes to minimize motion artifacts during daily use. The inclusion of more compact wireless modules could additionally relax the constraints on external cabling, ...while incorporating broader motion scenarios, longer wear-time testing, and user variability analysis may further improve robustness for real-world prosthetic and wearable monitoring. Lastly, the wire-level evaluation framework developed here may be useful for other bioelectric modalities (e.g., EEG and ECG) where robust, low-loss signal routing is essential.

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References

- [1] Cheng L, Li J, Guo A, Zhang J. Recent advances in flexible noninvasive electrodes for surface electromyography acquisition. *npj Flex Electron.* 2023;7(1). <https://doi.org/10.1038/s41528-023-00273-0>

- [2] Park J, et al. Imperceptible and reusable dermal surface EMG for lower extremity neuro-prosthetic control and clinical assessment. *npj Flex Electron*. 2023;7(1). <https://doi.org/10.1038/s41528-023-00282-z>
- [3] Aghchehli E, Kyranou I, Dyson M, Nazarpour K. Digital Sensing Systems for Electromyography. *IEEE Trans Neural Syst Rehabil Eng*. 2024;32:2826. <https://doi.org/10.1109/TNSRE.2024.3435740>
- [4] Liang Z, et al. A Wireless, High-Quality, Soft and Portable Wrist-Worn System for sEMG Signal Detection. *Micromachines*. 2023;14(5):1085. <https://doi.org/10.3390/mi14051085>
- [5] Lee H, et al. Stretchable array electromyography sensor with graph neural network for static and dynamic gestures recognition system. *npj Flex Electron*. 2023;7(1). <https://doi.org/10.1038/s41528-023-00246-3>
- [6] Kim H, Rho S, Lim D, Jeong W. Characterization of embroidered textile-based electrode for EMG smart wear according to stitch technique. *Fashion and Textiles*. 2023;10(1). <https://doi.org/10.1186/s40691-023-00351-x>
- [7] Martins J, et al. Integrating sEMG and IMU Sensors in an e-Textile Smart Vest for Forward Posture Monitoring: First Steps. *Sensors*. 2024;24(14):4717. <https://doi.org/10.3390/s24144717>
- [8] Varghese RJ, Pizzi M, Kundu A, Grison A, Burdet E, Farina D. Design, Fabrication and Evaluation of a Stretchable High-Density Electromyography Array. *Sensors*. 2024;24(6):1810. <https://doi.org/10.3390/s24061810>
- [9] Quadrelli D, Canepa M, Domenico DD, Boccardo N, Chiappalone M, Laffranchi M. Advances in HD-EMG interfaces and spatial algorithms for upper limb prosthetic control. *Front Neurosci*. 2025;19. <https://doi.org/10.3389/fnins.2025.1655257>
- [10] Kaifosh P, Reardon TR. A generic non-invasive neuromotor interface for human-computer interaction. *Nature*. 2025;645(8081):702. <https://doi.org/10.1038/s41586-025-09255-w>
- [11] Ohiri KA, et al. E-textile based modular sEMG suit for large area level of effort analysis. *Sci Rep*. 2022;12(1):9650. <https://doi.org/10.1038/s41598-022-13701-4>
- [12] Zheng M, Crouch MS, Eggleston MS. Surface Electromyography as a Natural Human-Machine Interface: A Review. *IEEE Sens J*. 2022;22(10):9198. <https://doi.org/10.1109/JSEN.2022.3165988>
- [13] Tacca N, et al. Wearable high-density EMG sleeve for complex hand gesture classification and continuous joint angle estimation. *Sci Rep*. 2024;14(1):18564. <https://doi.org/10.1038/s41598-024-64458-x>
- [14] Rafeedi T, Abdal A, Polat B, Hutcheson KA, Shinn EH, Lipomi DJ. Wearable, epidermal devices for assessment of swallowing function. *npj Flex Electron*. 2023;7(1). <https://doi.org/10.1038/s41528-023-00286-9>
- [15] Al Mashhadany YI. Design and analysis of 7-DOF human-link manipulator based on hybrid intelligent controller. *SPIIRAS Proceedings*. 2020;19(4). <https://doi.org/10.15622/sp.2020.19.4.3>
- [16] Awsaj MK, Al Mashhadany Y, Fourati LC. Real-Time Healthcare Monitoring and Treatment System Based Microcontroller with IoT. In: *Proceedings - International Conference on Developments in eSystems Engineering (DeSE)*; 2023. <https://doi.org/10.1109/DeSE58274.2023.10099758>
- [17] Silvestre L, Matthies J, Boswell L, Stephens J. Nanosecond Breakdown Characteristics of C4F7N and Various Mixtures at Pressures Above 1 Atmosphere in Comparison with SF6. *Appl Sci*. 2024. <https://doi.org/10.3390/app142311268>
- [18] Awsaj MK, Al Mashhadany YI, Fourati LC. An analytical review for cloud computing in healthcare system based biosensors. *Int J Health Sci (Qassim)*. 2022. <https://doi.org/10.53730/ijhs.v6nS9.12297>
- [19] Xiang H, Yuan Y, Zhu T, Dai X, Zhang C. Anti-icing mechanism for a novel slippery aluminum stranded conductor. *Applied Materials*. 2023. <https://doi.org/10.1021/acsami.3c04797>
- [20] Al Mashhadany YI. Optimal Results Presentation Style for Engineering Research Article. In: *AIP Conference Proceedings*; 2022. <https://doi.org/10.1063/5.0112145>
- [21] Taha BA, et al. Next-generation nanophotonic-enabled biosensors for intelligent diagnosis of SARS-CoV-2 variants. *Sci Total Environ*. 2023;880. <https://doi.org/10.1016/j.scitotenv.2023.163333>
- [22] Alrawi AAA, Al-Ani A, Mashhadany YA, Algburi S. Performance and Latency Analysis of EEG Signal for Control System of Robotic Arm Movements Based on Sliding Mode Controller. In: *2025 IEEE 22nd International Multi-Conference on Systems, Signals & Devices (SSD)*; 2025. 1123-1128. <https://doi.org/10.1109/SSD64182.2025.10989963>
- [23] Dong H, Sun Z, Li J, Li Y, et al. Study on removal of aluminum alloy oxide film by continuous-nanosecond combined laser with different pulse delays. *Opt Commun*. 2024. <https://doi.org/10.1016/j.optcom.2023.130112>
- [24] Kosmatskiy YI, ALKhuzaie AS, ALJumaili MJ. Experimental research of the frictional condition in the pipes extrusion process. *IOP Conf Ser: Mater Sci Eng*. 2018;454(1):012008. <https://doi.org/10.1088/1757-899X/454/1/012008>
- [25] Clark R, Mounho M, Brooks W, Hopkins M. Spectroscopic investigation of early light emission from anode-initiated surface flashover in vacuum. *Phys Plasmas*. 2024. <https://doi.org/10.1063/5.0190819>

- [26] Gaeid KS, Homod RZ, Al Mashhadany Y, Al Smadi T, Ahmed MS, Abbas AE. Describing Function Approach with PID Controller to Reduce Nonlinear Action. *Int J Electr Electron Res.* 2022;10(4). <https://doi.org/10.37391/ijeer.100437>
- [27] Unanyan NN, Belov AA. Design of upper limb prosthesis using real-time motion detection method based on EMG signal processing. *Biomed Signal Process Control.* 2021. <https://doi.org/10.1016/j.bspc.2021.103062>
- [28] Kosmatskiy YI, AL Jumaili MJ, AL Khuzaie AS. Experimental Research of the View of the Metal Flow in the Pipes Side-Extrusion Process. *IOP Conf Ser: Mater Sci Eng.* 2018;454(1):012009. <https://doi.org/10.1088/1757-899X/454/1/012009>
- [29] Glencoe J, Deemyad T. Development of an EMG-Controlled Wire Feeder Mechanism for Hand Amputees in TIG Welding. *ASME.* 2024. <https://doi.org/10.1115/IMECE2024-147363>
- [30] Al-Jumaili MJ, Vydrin AV, Shkuratov YA. Elaboration of a digital model for estimation of power parameters of a rolling process in a continuous rolling mill. In: *AIP Conference Proceedings.* 2020;2213(1):020066. <https://doi.org/10.1063/5.0000302>
- [31] Makhkamov B. Development of an Approach to Analysis and Classification of EMG Signals for Prosthesis Control. In: *International IOT, Electronics and Mechatronics.* Springer; 2024. https://doi.org/10.1007/978-981-97-4784-9_22
- [32] Abbas AK, Al Mashhadany Y, Hameed MJ, Algburi S. Review of Intelligent Control Systems with Robotics. *Indones J Electr Eng Inform.* 2022;10(4). <https://doi.org/10.52549/ijeei.v10i4.3628>
- [33] Gao S, Gong J, Chen B, Zhang B, Luo F. Use of advanced materials and artificial intelligence in electromyography signal detection and interpretation. In: *Intelligent Systems;* 2022. <https://doi.org/10.1002/aisy.202200063>
- [34] Al Mashhadany YIM, Abbas AK, Algburi SS. Modeling and analysis of brushless DC motor system based on intelligent controllers. *Bull Electr Eng Inform.* 2022;11(6). <https://doi.org/10.11591/eei.v11i6.4365>
- [35] Pérez Obiols E. Analysis of electrode arrays for multichannel surface electromyography. [Thesis/Report]; 2019.
- [36] Al Mashhadany YI, Jasim WM. Real time modified programmable universal machine for assembly (PUMA) 560 with intelligent controller. *Indones J Electr Eng Comput Sci.* 2020;20(3):1194-1202. <https://doi.org/10.11591/ijeecs.v20.i3.pp1194-1202>
- [37] Vidhya CM, Maithani Y, Singh JP. Recent advances and challenges in textile electrodes for wearable biopotential signal monitoring: A comprehensive review. *Biosensors.* 2023. <https://doi.org/10.3390/bios13070679>
- [38] HUBER+SUHNER. RG_174_/U Coaxial Cable-Datasheet. HUBER+SUHNER AG; 2026 [Internet]. Available from: https://www.mouser.com/datasheet/2/829/HUBER_2bSUHNER_RG_174_U_DataSheet-1489489.pdf

Dynamic analysis of piled raft foundations: Influence of superstructure and geometric parameters

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Abstract

Designers opt for piled raft foundations in response to the rise in demand for high-rise structures and significant infrastructure in metropolitan regions with low soil-bearing capacity. However, analytical and numerical studies that consider both the foundation and the superstructure are scarce in the literature. Considering the superstructure, even in its simplest form, is decisive for assessing the combined behavior of the piled raft. This study aims to examine how variations in several significant factors affect the dynamic behavior of a piled raft foundation structure system in loose sand. The combined piled raft system (CPRF) is modelled using the three-dimensional finite element software Plaxis 3D. An extensive parameter-based investigation is performed to appraise the impact of crucial factors such as raft thickness, pile spacing-to-diameter ratios, and varied pile lengths on the dynamic and seismic behavior of the combined piled raft foundation structural system. It is noted that the fundamental time period of the piled raft structure system is increased substantially when compared to a piled raft foundation without a structure and with reduced raft thickness. Additionally, there is a noticeable decrease in base shear at the head of the pile.

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1. Introduction

Over the past few decades, the accelerated pace of urbanization and the demand for vertical expansion have resulted in the proliferation of high-rise structures across seismic and non-seismic regions worldwide. In the early stages of development, many tall buildings were founded on shallow footings; however, these systems often encountered significant differential settlements, especially in soft or heterogeneous soil profiles, exceeding permissible serviceability limits [1]. To mitigate these issues, deep foundations—primarily in the form of pile groups—were introduced, offering improved load transfer to deeper, more competent strata. Nevertheless, the high construction costs and logistical challenges associated with installing a large number of piles posed economic constraints. To overcome these limitations, the Combined Piled Raft Foundation (CPRF) configuration emerged as a technically efficient and economically viable solution that leverages the load-sharing capability of both the raft and piles [2]. By optimizing the interaction between these elements and the supporting soil, CPRF systems significantly enhance bearing capacity, reduce settlement, and improve seismic performance. As a result, they have gained wide acceptance in geotechnical engineering practice for use in critical infrastructure projects, including high-rise buildings, towers, nuclear facilities, bridges, and large-diameter storage tanks—especially in soft to medium clay and sandy soils where both vertical and lateral demands are considerable.

During seismic events, these structures are experiencing dynamic loads with a wide frequency range, making the seismic behavior of the foundation system a subject of prime importance. A critical concern arises when the natural frequency of the soil-structure system aligns with the

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dominant frequency content of seismic events, potentially causing resonance and structural failure. Traditionally, the design of CPRF under static loading focuses on optimizing raft stiffness and soil bearing capacity, with piles primarily designed to control settlements by carrying gravity loads [3]. However, the conventional seismic design of such systems often provided a fixed-base condition and neglects the role of soil-structure interaction (SSI). While this assumption may be valid for lightweight structures on moderately stiff soils, it becomes inaccurate for heavy superstructures resting on soft to medium soils. Numerous case studies, including the 1995 Kobe earthquakes, 2001 Bhuj and 2011 Christchurch earthquake, have demonstrated that neglecting SSI can lead to unsafe designs and severe structural damage.

Soil introduces flexibility at the foundation level, thereby altering the dynamic properties of the system, such as the natural period and modal behavior. In piled raft systems, the significant mass of the raft further increases inertial forces, amplifying the impact of SSI during seismic excitation [4]. Therefore, incorporating SSI into the seismic analysis of structures founded on CPRF systems is essential for achieving realistic and safe designs. A recent study [5] conducted a comprehensive three-dimensional finite element analysis of CPRFs with both long and short piles under static and seismic loading. Their results showed that replacing long piles with short piles slightly reduces vertical load-sharing but significantly enhances lateral resistance under earthquake excitation. Furthermore, short piles were found to experience higher bending moments and shear forces at the pile head, while also contributing to substantial material savings (up to 32%) with only marginal increases in settlement. Their findings confirm that CPRF is an optimized and economical foundation system for mid- and high-rise buildings under both static and seismic loading conditions. Author [6] developed a MATLAB-based educational platform for slope stability analysis using Bishop and OMS methods, offering an intuitive GUI, transparent computational framework, and reliability validated against Slide with only 5–6% error, thereby supporting improved conceptual learning in geotechnical engineering.

In addition, researchers performed experimental investigations showing the impact of several key design parameters, such as the raft-soil relative stiffness, pile length, spacing-to-diameter (S/d) ratio, number of piles in a group, and the type of pile-raft connection significantly influence the static response of CPRF systems, considering SSI. Although recognized as critical for design optimization, these parameters have not been comprehensively investigated in the context of seismic loading, changing the behavior of the superstructure in previous studies [7-10]. This study addresses the effect of some of these parameters on the behavior of the superstructure in response to seismic load.

The study is conducted in three parts. In Part I, the numerical model considered for the current analysis is validated through the previously conducted study by Kumar et al. [10]. In Part II, the 3D numerical analysis is performed for the CPRF, including and excluding superstructure, and its impact on the natural radial frequency of the whole system is examined. Moreover, parametric research is conducted to look at how changes in the geometrical and stiffness characteristics of the CPRF system affect the natural frequency or time period of the whole system, (i) include the superstructure in the model, (ii) ignoring the superstructure. In Part III, the study aims to comprehend the seismic behavior of piled raft foundations by analyzing the intricate relationship between the soil, piled-raft foundation, and structure. This includes examining various parametric variations that are crucial design factors. The dynamic analysis is conducted using numerical modelling based on finite elements. Plaxis 3D [11] is used to model the soil-piled raft-structure system. In conclusion, the outcome from the present work project provides a better understanding into the problems and also seismic design guidance for structures constructed on CPRF systems in loose sand.

2. Finite Element Simulation of Combined Pile-Raft-Structure Systems

2.1. Idealization of the Superstructure

To capture a broad spectrum of structural behavior under dynamic loading, the analysis includes structures with four characteristic natural periods with a fully restrained base: 0.30 sec, 0.50 sec, 1.5 sec, and 3.0 sec. These values are selected to span the range from short-period to long-period

structures, corresponding approximately to three-storey, five-storey, fifteen-storey, and thirty-storey buildings, respectively. Fig. 1 shows the superstructure combined with CPRF having fixed base time periods starting from 3 sec, 1.5 sec, 0.5 sec, and ending at 0.3 sec. Along with this, the figure shows the pile configuration for a three-story superstructure. These multi-degrees-of-freedom systems are analyzed through modelling first in ETABS [12] in order to fix the dimensions of the structural element, then again modelled in Plaxis 3D along with the soil domain and CPRF for further studies. The overall dimensions of the superstructure presented in Table 1 are determined based on an assumed bay size of 8.5 m × 8.5 m, with a storey height of 3.5 m at each level. The mass of the superstructure, considering dead load and live load, is calculated to be approximately 1755 kN, 2930 kN, 8780 kN, and 17550 kN for natural periods of 0.30 s, 0.50 s, 1.5 s, and 3.0 s, respectively. The material properties of the superstructure are considered elastic and presented in Table 2. For the case of comparative study of piled raft foundation with and without structure, the structure having a fixed base time period of 1.5s is considered, while for the piled raft foundation without superstructure, equivalent uniform loading of 175kPa is applied on the raft.

Table 1. Dimension of superstructure

Parameters	Unit	$T_{fixed} = 0.3\text{ s}$			$T_{fixed} = 0.5\text{ s}$		
		Beam	Column	Slab	Beam	Column	Slab
Length/ Height	m	4.25	3.5	4.25	4.25	3.5	4.25
Width	m	0.3	0.4	4.25	0.32	0.44	4.25
Depth	m	0.3	0.4	0.1	0.32	0.44	0.1

Parameters	Unit	$T_{fixed} = 1.5\text{ s}$			$T_{fixed} = 3.0\text{ s}$		
		Beam	Column	Slab	Beam	Column	Slab
Length/ Height	m	4.25	3.5	4.25	4.25	3.5	4.25
Width	m	0.4	0.48	4.25	0.45	0.82	4.25
Depth	m	0.4	0.48	0.1	0.45	0.82	0.1

2.2. Modelling of the CPRF System

A square raft foundation measuring 10 m in width and 1 m in thickness was modelled using plate elements in PLAXIS 3D. While its safety against shear failure is verified, the settlement under vertical loads corresponding to 3-, 5-, 15-, and 30-story buildings exceeded the permissible limit of 75 mm for sandy soils, as recommended by Bowles [13]. To mitigate this excessive settlement, a combined piled raft foundation (CPRF) system is implemented. Initially, fixed-head piles—each 20 m long and 0.5 m in diameter are modelled in different configurations for different heights of superstructure using the embedded pile element feature in PLAXIS 3D [11]. A schematic diagram (Fig. 1) illustrates the CPRF configuration.

Several ways suggested by researchers [14,15] to model the soil domain are Winkler, macro modelling and continuum approach. In this study FEM method as part of continuum approach is adopted. The soil domain, modelled with 10-node tetrahedral elements, measured 65 m × 65 m × 45 m. A commonly used linear elastic-perfectly plastic Mohr-Coulomb model available in PLAXIS 3D [11] library is utilized to model loose sand in this study. The Mohr-Coulomb model was adopted due to its computational efficiency and suitability for large-scale SSI simulations where capturing first-order seismic response trends is the primary objective. While it does not explicitly model stiffness degradation and strain-dependent damping, similar simplifications are widely adopted in parametric SSI studies [16-18].

The modulus of elasticity of soil are taken as 5000 kN/m² considered to be constant throughout the depth to avoid computational complexity taking reference of other studies [19-20]. The value of the angle of frictional and dilation angles is taken as 27 degrees and zero, respectively. Interface set as strength determination manual with R_{inter} value 0.67 [21]. Standard boundary conditions are employed, restraining lateral movements of the sides and fully fixing the base.

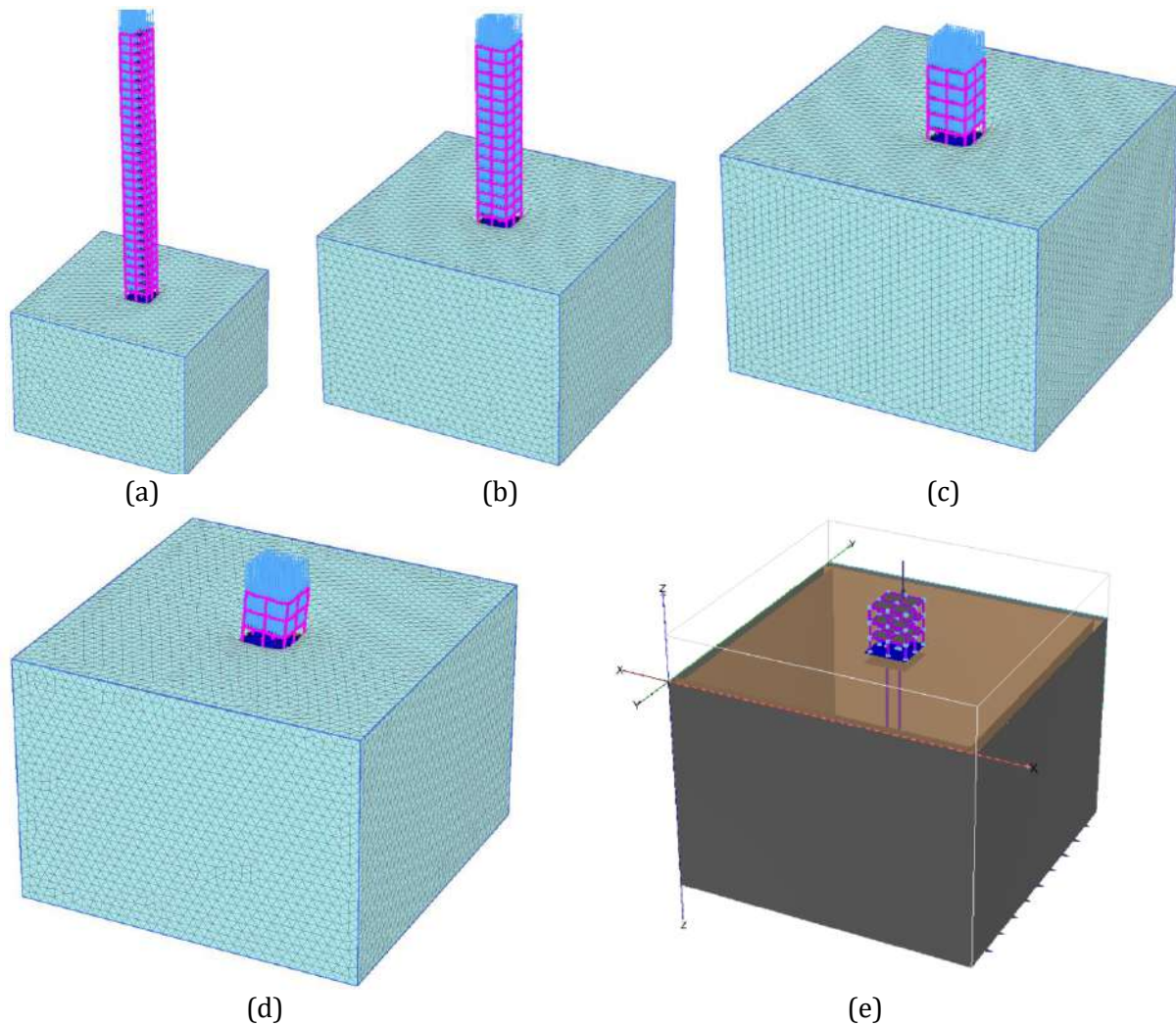


Fig. 1. Superstructure configurations (a) 30-story, (b) 15-story, (c) 5-story, (d) 3-story, (e) Pile Configuration for 3-story structure

A medium mesh was generated, consisting of 86,942 soil elements and 98,986 nodes, with an average element size of about 90 cm. In dynamic analysis, mesh distortion can affect wave propagation; to avoid this, the element size was selected based on the suggestion by researchers [15,22], which recommends it to be smaller than one-eighth to one-tenth of the shortest wavelength of the input motion. Accordingly, the mesh was refined to ensure accurate simulation of seismic wave propagation.

Table 2. Input engineering properties of soil, pile, raft, and structural elements

Parameter	Unit	Raft	Pile	Structure	Soil
Material	-	Concrete	Concrete	Concrete	
Length	m	10	20	-	65
Width	m	10	-	-	65
Thickness/height/depth	m	1	-	3.5	45
Diameter	m	-	0.6	-	-
Modulus of Elasticity	kN/m ²	41×10 ⁶	41×10 ⁶	41×10 ⁶	5000
Unit Weight	kN/m ³	24	24	24	15
Poisson's Ratio	-	0.2	0.2	0.2	0.3
Friction Angle	Degree	-	-	-	27

The chosen mesh size ensured convergence of results, alignment with experimental test simulations, and computational efficiency. Material damping—due to intrinsic soil properties such as viscosity, friction, and plasticity—is represented using Rayleigh damping, with a damping ratio

of 5%, typical for geological materials [23]. Damping coefficients are calculated directly by Plaxis 3D based on the first and second modal frequencies of the soil-CPRF system for different configuration.

To simulate an infinite domain and minimize artificial wave reflections, PLAXIS 3D [11] employs viscous boundaries with normal and shear dampers. These boundaries absorb outgoing waves, mimicking energy dissipation in an unbounded medium and accurately capturing dynamic response. Table 2 shows the adopted material features for the Soil, Piled Raft, and Superstructure components for the current analysis.

2.3. Ground Motions

An earthquake ground motion of the 1994 Northridge seismic event [Magnitude (M_w) = 6.7], with a peak ground acceleration of 0.57 g, is considered for the present study, taken from the PEER strong ground motion database [24]. Station with record sequence number (RSN) 963, Epicenter 34.21°N and 118.5°W. Fig. 2 illustrates the acceleration time histories corresponding to the selected input ground motions.

3. Validation of Numerical Model

The developed CPRF model in PLAXIS 3D [11] was rigorously validated by reproducing the results of Kumar et al. [10], whose numerical simulations were themselves benchmarked against a comprehensive series of centrifuge shaking table experiments on piled raft foundations in sand [25]. Thus, the numerical model of the present study indirectly inherits validation against high-quality experimental data, ensuring reliability under static, pseudo-static, and dynamic loading conditions. The material and geometric properties used for the soil, piles, raft, and superstructure are taken from the referenced study and are summarized in Table 3.

Table 3 Detailed material characteristics of the soil, pile, raft and superstructure adopted for the validation study [10]

Parameters	Unit	Soil	Raft	Pile
Material	-	Toyoura sand	Aluminum	Concrete
Unit weight	(kg/m^3)	1662	2752	2446
Poisson's Ratio (μ)	-	0.3	0.35	0.2
Young's Modulus	(MPa)	40	41700	70000
Cohesion	(kN/m^2)	10	-	-
Dilation Angle	-	1	-	-
Friction Angle (Φ)	-	31	-	-

The soil domain is modeled as a 28 m $\times$ 28 m $\times$ 16 m block of Toyoura Sand using 10-noded tetrahedral finite elements, and the Mohr–Coulomb model is employed to simulate soil behavior. Standard fixities were applied at the boundaries, with lateral constraints on the vertical faces and full fixity at the base. The raft foundation is modelled as a square plate element, 4 m wide and 1 m thick. The foundation system includes four fixed-head piles, each 9 m in length and 0.5 m in diameter, spaced 2 m c/c, and modelled using the embedded pile option available in PLAXIS 3D [11]. Under static loading, a vertical pressure equivalent to a total load of 5,862.5 kN is applied over the raft area. Fig. 3 shows the model predicted a maximum settlement of 17.45 mm, which showed acceptable conformity with the 21 mm settlement stated by Kumar et al. [10] in their study.

In the pseudo-static loading analysis, lateral loads are applied by multiplying the vertical load with different seismic coefficients to simulate earthquake-induced effects. Maximum horizontal loads are applied to CPRF are 621.4kN, 1178.4kN, 1635.6kN and 2520.8kN, corresponding to Bhuj 2001 (PGA=0.106g), Sikkim 2011 (PGA=0.201g), Loma Prieta 1989 (PGA=0.279g) and El-Centro 1979 (PGA=0.43g) ground motion, respectively. Bending moment profiles shown in Fig. 4 indicated that the maximum moments occurred near the pile heads, and their magnitudes increased with higher seismic acceleration values, consistent with the trends observed in the referenced study [10].

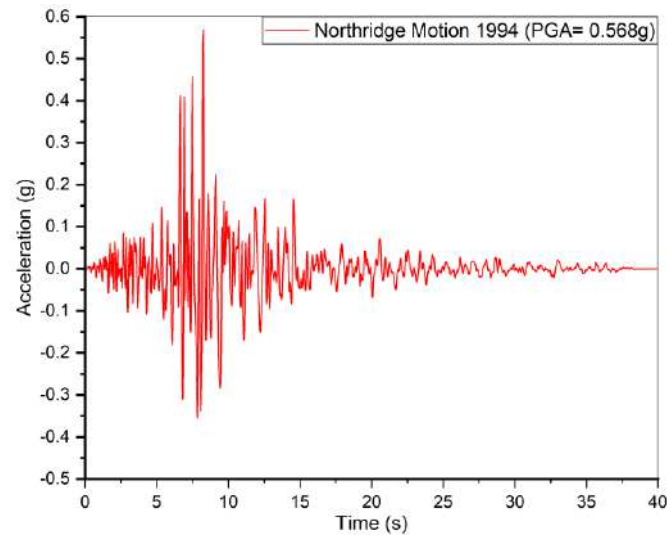


Fig. 2. Time history input motion

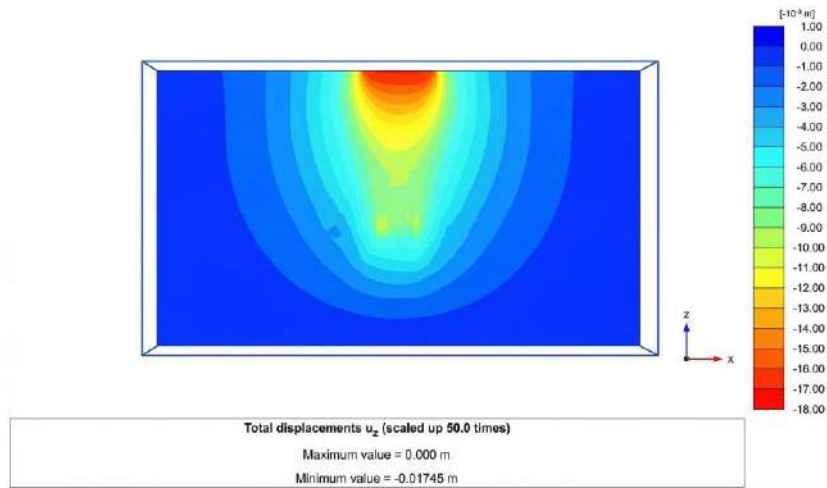


Fig. 3. Settlement of CPRF system due to applied vertical load

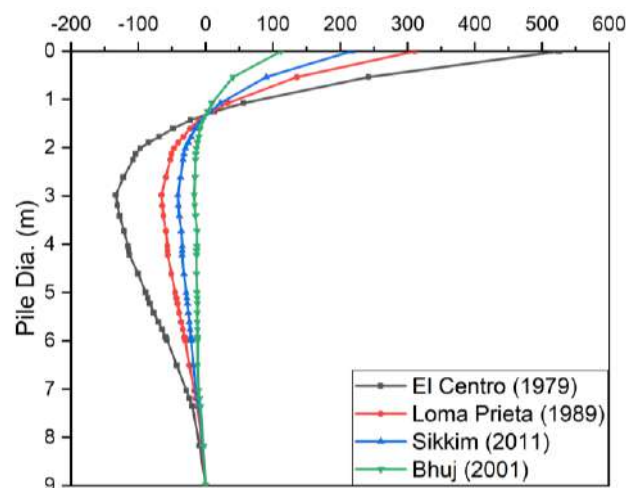


Fig. 4. Bending moment profile under different pseudo-static loading horizontally applied seismic load

In the dynamic analysis, a sinusoidal base acceleration with an amplitude of 1 m/s^2 and a frequency of 1 Hz was applied to simulate ground shaking. The maximum output acceleration noted at the uppermost location of the raft was 2.1 m/s^2 can be noted from Fig. 5, which showed reasonable

alignment with the value reported by Kumar et al [10]. These comparisons across all loading conditions support the accuracy and robustness of the analyzed CPRF model in PLAXIS 3D [11].

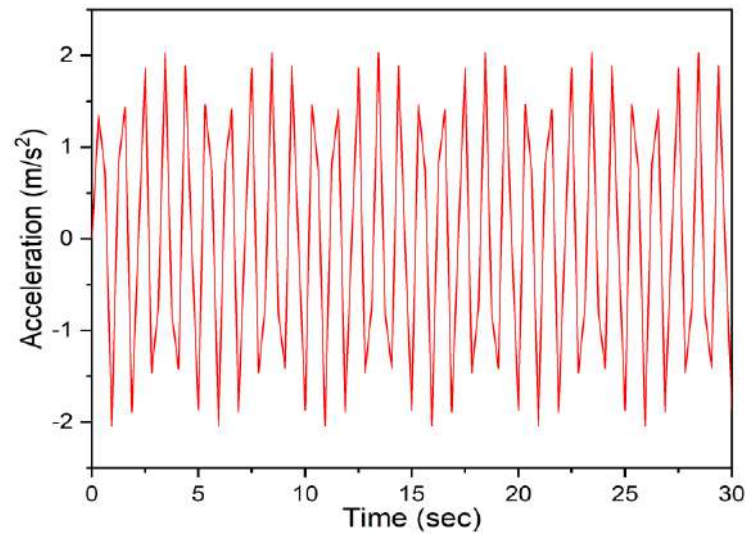


Fig. 5. Output Acceleration

4. Vibrational Behavior of Piled Raft Foundations Including and Excluding the Superstructure

A comparative analysis is performed on a 3×3 combined piled raft foundation system comprising 20 m long piles spaced at four times their diameter, embedded in loose sand, both with and without the inclusion of superstructure stiffness. A fifteen-story superstructure CPRF model has been considered. Also, the equivalent load of a fifteen-story structure is applied when the superstructure is neglected. All geometrical and material properties are given in Table 1. Fig. 6 represents the 3D FEM model for the CPRF system without and with superstructure.

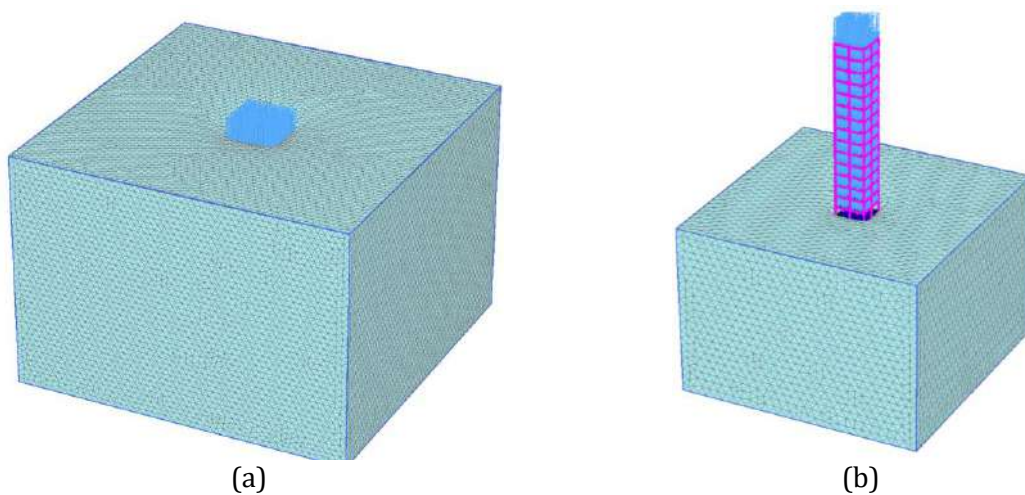


Fig. 6. CPRF (a) without and (b) with the superstructure

The results of a comparative analysis on combined piled raft foundations with and without the presence of a superstructure are shown in Fig. 7. The addition of superstructure stiffness increases the overall flexibility of the system, resulting in a decrease in its natural radial frequency. The percentage difference in natural frequency (ω) between the cases with and without superstructure reduces from 10% to 16% as the pile diameter increases from 0.5 m to 1.0 m, primarily due to the corresponding increase in foundation mass. These findings indicate that the presence of the superstructure in the model significantly affects the dynamic attribute of the system. Therefore, it is essential to account for the superstructure in the analysis of CPRF.

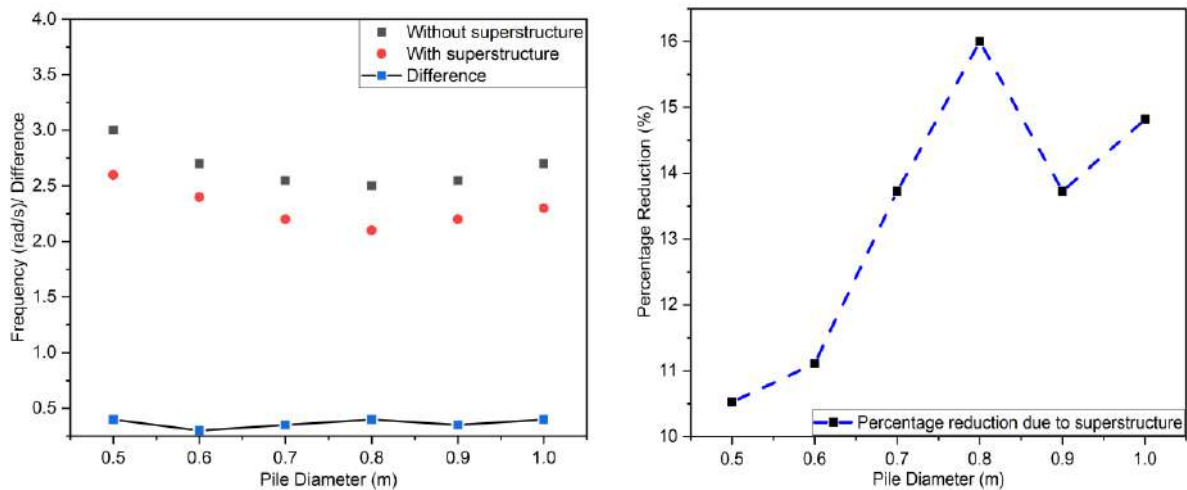


Fig. 7. Comparison between CPRF with and without superstructure

5. Impact of Governing Parameters on the Natural Circular Frequency of CPRF without Superstructure

To investigate the influence of different geometric parameters on the natural radial frequency of piled raft foundations, a comprehensive parametric analysis is performed by changing pile lengths, diameters and spacing. A uniformly distributed load of 175 kPa was considered to represent the superstructure load on the raft. The material and geotechnical characteristics used in the analysis are listed in Table 2.

5.1 Impact of Pile Length and Pile Diameter on Natural Frequency

As illustrated in Fig. 8, the natural circular frequency (ω) of the piled raft system diminishes with increasing pile length. This behavior is attributed to a decrease in the overall rigidity of the combined foundation system as pile length increases. Additionally, ω initially decreases with increasing pile diameter up to an optimum point, beyond which it begins to rise. This trend is explained by the dominance of the increase in mass over the gain in stiffness at smaller diameters, while at larger diameters, stiffness begins to govern the response, leading to an increase in ω .

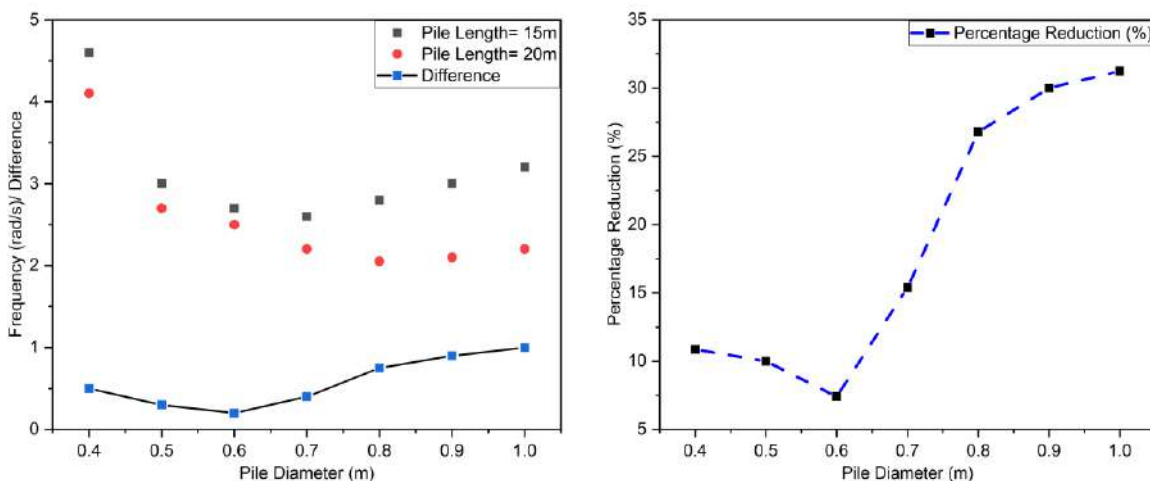


Fig. 8. Change in natural circular frequency with pile diameter for varying pile lengths

5.2 Impact of Variation on Pile Spacing-To-Diameter on Natural Frequency

In this part of the analysis, the pile spacing-to-diameter ratio (S/d) is varying from 2.5 to 8 for piles placed in the soil deposit. Initially, the diameter of the pile is held fixed at 0.6 m, while the pile

length ranges from 15 m to 20 m. As illustrated in Fig. 9, these changes in pile geometry (i.e spacing) have no substantial impact on the natural radial frequency of the piled raft.

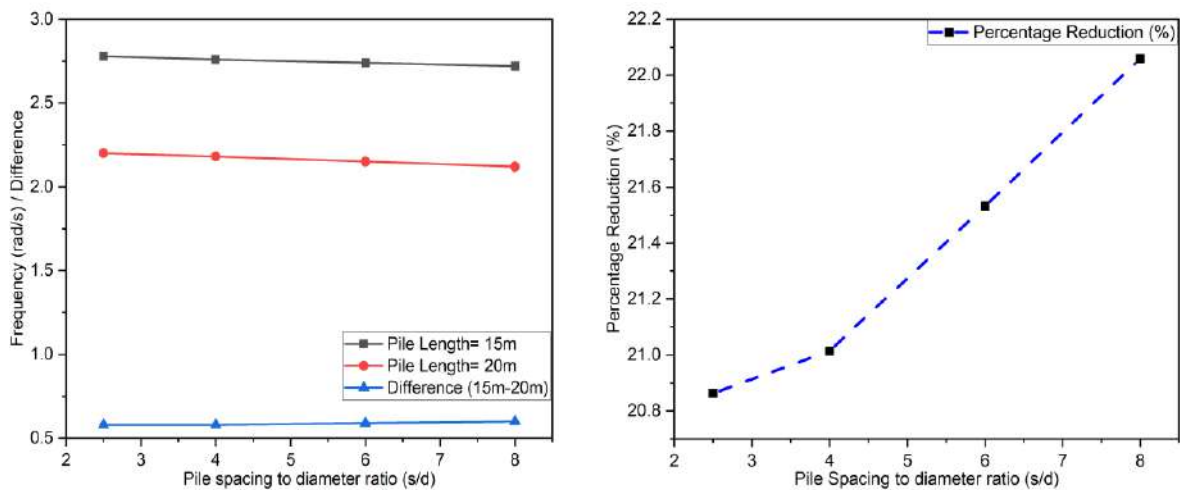


Fig. 9. Variation in natural circular frequency with change in pile spacing for different pile lengths

6. Parametric Study of Piled Raft Foundation Incorporating Superstructure

This part describes the range of parametric cases considered in the current analysis. These cases are developed based on the design of CPRF subjected to gravity loads, tailored considering four typical superstructures characterized by fundamental fixed base time periods of 0.3 s, 0.5 s, 1.5 s, and 3 s, as outlined earlier in subsection 2.1. Initially, a raft foundation of dimensions 10 m × 10 m with a thickness of 1 m is assumed, corresponding to the structural configurations of the selected buildings. The pile and raft are both modelled, considering concrete as a material with a modulus of elasticity $E_{\text{concrete}} = 41 \times 10^6 \text{ kN/m}^2$ and a mass density of 2400 kg/m^3 . Each pile is modelled as a solid circular section of 20 m long, a dia. of 0.6 m, and a spacing-to-diameter ratio (S/d) varied according to the values in Table 4. The pile is assumed to be founded in the soil, with an overall depth of soil of 45 m.

Once the appropriate pile group configurations are determined for each case, further analyses are conducted to study how variations in the spacing of piles and raft thickness influence the time period and seismic (namely, base shear) behavior of the combined piled raft system. Two levels of raft-soil relative stiffness (k_{rs}), approximately 3.3 and 51.5, are considered by choosing two values of the raft thickness, which are 1 m and 2.5 m, to capture both flexible and rigid raft behavior on application Northridge ground motion. The k_{rs} is calculated based on the approach proposed by Clancy and Randolph [26], given in Eq (1).

$$k_{rs} = \frac{4E_r B_r t_r^3 (1 - \nu_s^2)}{3\pi E_s L_r^4 (1 - \nu_r^2)} \quad (1)$$

where, E_r and E_s are Young's modulus of Raft and soil, B_r , L_r and t_r are the width, length and thickness of the raft respectively, ν_s and ν_r are the Poisson's ratio of soil and raft. Moreover, the impact of different pile spacing-to-diameter ratios is examined by considering (S/d) values of 2.5 and 8 for the two most suitable superstructure cases with fixed base periods of 0.3 s and 0.5 s. A detailed summary of all these parametric variations is provided in Table 4.

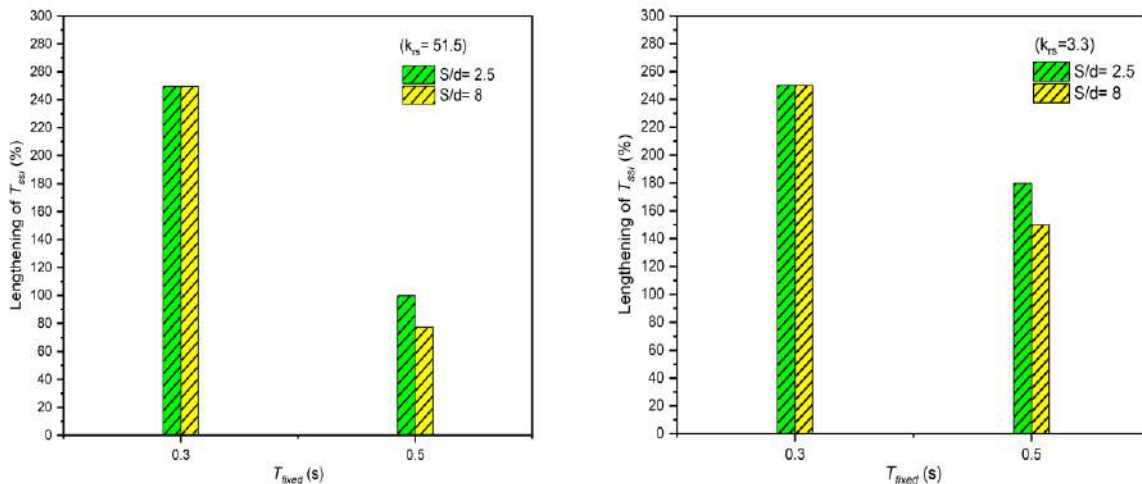
Table 4. Outline of the parametric study framework

Structure with Fixed base Time Period (T_{fixed})	Thickness of raft (m)	Raft-soil relative stiffness (k_{rs})	S/d	Pile Configuration
0.3	2.5	51.5 (Rigid)	2.5	1×2
	1	3.3 (Flexible)	8	
0.5	2.5	51.5 (Rigid)	2.5	2×2
	1	3.3 (Flexible)	8	
1.5	2.5	51.5 (Rigid)	2.5	3×3
	1	3.3 (Flexible)	8	
3	2.5	51.5 (Rigid)	2.5	4×4
	1	3.3 (Flexible)	8	

7. Results and Discussion

7.1 Impact of (S/d) Ratio on The Time Period Of The Structure

At $T_{\text{fixed}} = 0.3$ s, the change in the (S/d) ratio has little to no impact on period lengthening can be observed from Fig. 10, regardless of the k_{rs} value (3.3 or 51.5). The absence of period lengthening at $T_{\text{fixed}}=0.3$ may be attributed to the fact that increasing the (S/d) ratio in a 1×2 pile group has only a minimal influence on the overall foundation stiffness, resulting in negligible impact on the overall system time period. However, for a slightly more flexible structure with $T_{\text{fixed}} = 0.5$ s, the period lengthening shows a noticeable decrease—from 100% to 78%—as the (S/d) ratio increases from 2.5 to 8 for $k_{rs} = 51.5$. A similar declining trend is observed for $k_{rs} = 3.3$, where the period lengthening reduces from 180% to 150% over the same (S/d) range, which shows with the increase in number of pile (2×2) stiffness plays an important role in altering the time period of system.

Fig. 10. Impact of S/d ratio on time period for different raft soil stiffness

7.2 Impact of Stiffness of Raft on Time Period of The Structure

From Fig. 11, it is evident, for $T_{\text{fixed}} = 0.5$ s, the period of the superstructure constructed on a CPRF increases from 100% to 180% as k_{rs} decreases from 51.5 to 3.3. A similar trend is observed for longer periods: for $T_{\text{fixed}} = 1.0$ s, the period lengthening varies from 75% to 92%, and for $T_{\text{fixed}} = 3.0$ s, it ranges from 7.8% to 37% over the same reduction in k_{rs} . These results indicate that increasing raft flexibility significantly influences the extent of period lengthening.

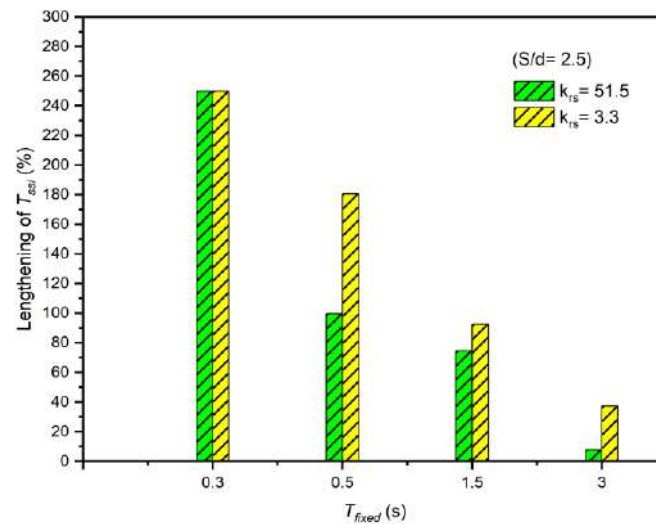


Fig. 11. Impact of raft soil stiffness on time period

7.3 Dynamic Response of Piled Raft Foundation Systems Under Seismic Loading

Under the elastic deformation range, base shear at the column head, as well as at the pile head, serves as a key factor for the evaluation, design and optimization of piled raft-supported structural systems. The column head shear ($V_{B,column}$) and pile head shear ($V_{B,pile}$) are normalised with reference to the base shear from the fixed-base condition of the superstructure ($V_{B,fixed}$), and expressed as ($V_{B,column}/V_{B,fixed}$) and ($V_{B,pile}/V_{B,fixed}$) respectively. These normalized shear values provide valuable insight into how the inclusion of a dynamic interaction factor between soil, pile, raft and superstructure affects the seismic force distribution between the superstructure and foundation elements.

7.3.1 Impact of S/d Ratio

Fig. 12 and Fig. 13 depict the impact of the S/d ratio on the normalized shear at the column and pile heads under the input ground motion from the 1994 Northridge earthquake, for $k_{rs} = 51.5$ and 3.3, respectively. For $k_{rs} = 51.5$, the variation in S/d shows negligible impact on the normalized column head shear ($V_{B,column}/V_{B,fixed}$). However, for $k_{rs} = 3.3$, a slight decrease in ($V_{B,column}/V_{B,fixed}$) is observed upon increasing S/d ratio. In contrast, the shear transmitted to the pile head exhibits its most significant variation at $T_{fixed} = 0.5$ s for $k_{rs} = 51.5$. Overall, it can be concluded that the normalized pile head shear tends to reduce as the S/d ratio increases.

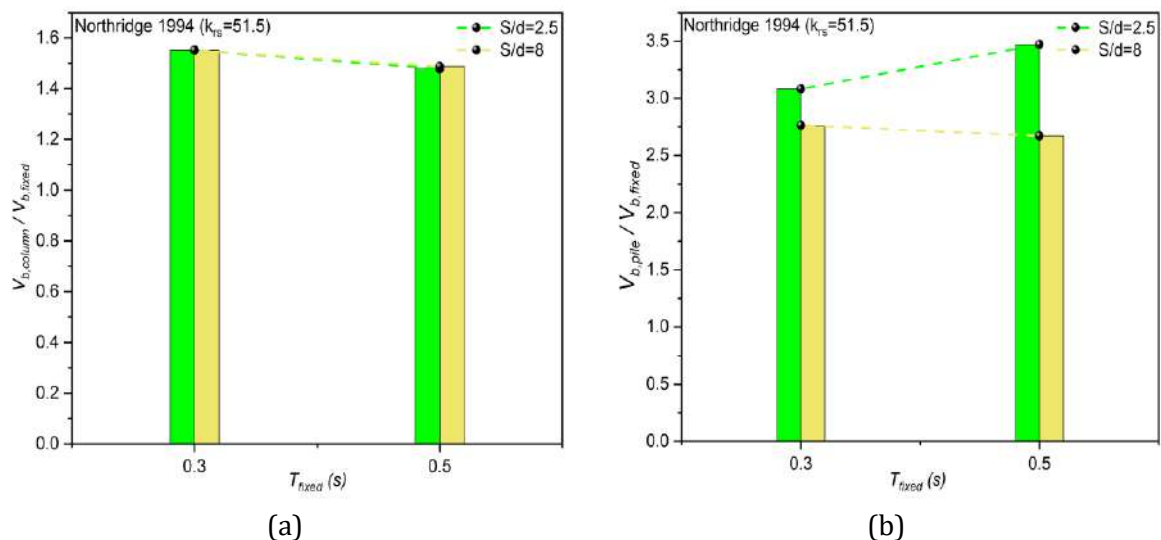


Fig. 12. Impact of S/d ratio on Normalized base shear (a) For, const. $k_{rs} = 51.5$; varying S/d ratio and (b) For, const. $k_{rs} = 3.3$; varying S/d ratio

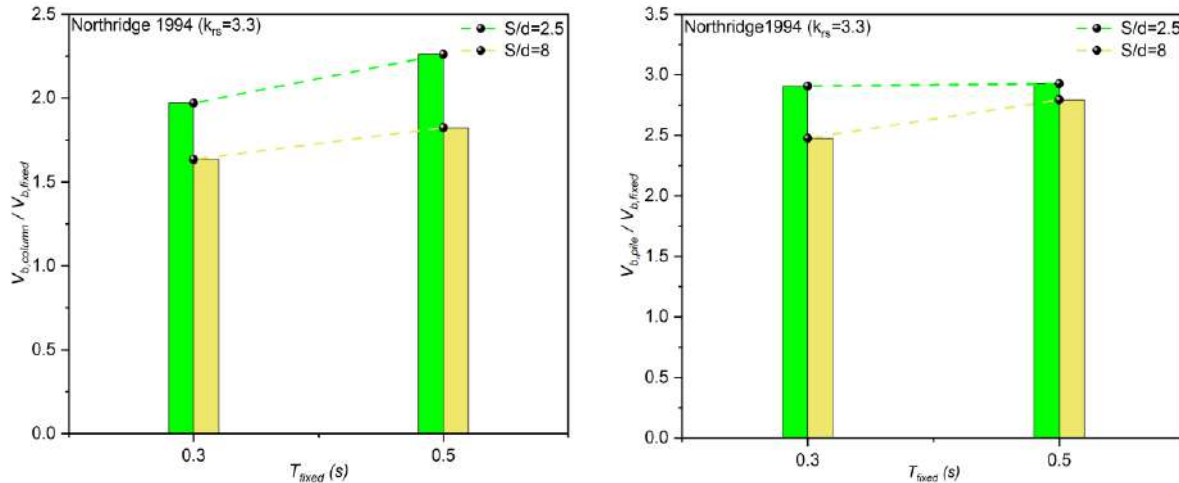


Fig. 13. Impact of S/d ratio on normalised base shear

7.3.2 Impact of Flexibility of The Raft

Under the Northridge 1994 ground motion, a significant alteration in both column and pile head shear is noticed primarily for the structure with $T_{fixed} = 0.5$ s, which is considered to be moderately stiff. As shown in Fig. 14, the column head shear increases by 57%, while the pile head shear decreases by 20% with increasing raft flexibility. However, the effect of raft flexibility on pile head shear is comparatively less pronounced for this ground motion. The non-monotonic variation of normalized base shear with period arises from frequency-dependent load sharing between piles, columns, and the raft. At short periods, the system behaves rigidly and piles attract higher shear, especially for higher raft-soil stiffness (k_{rs}). Around intermediate periods (1.5 s), foundation flexibility reduces the transmitted shear to both piles and columns. At longer periods, soil-foundation interaction amplifies displacements, causing renewed increases in pile and column shear. Thus, the observed trend reflects the combined influence of raft-soil stiffness, pile-soil interaction, and the dynamic characteristics of the superstructure.

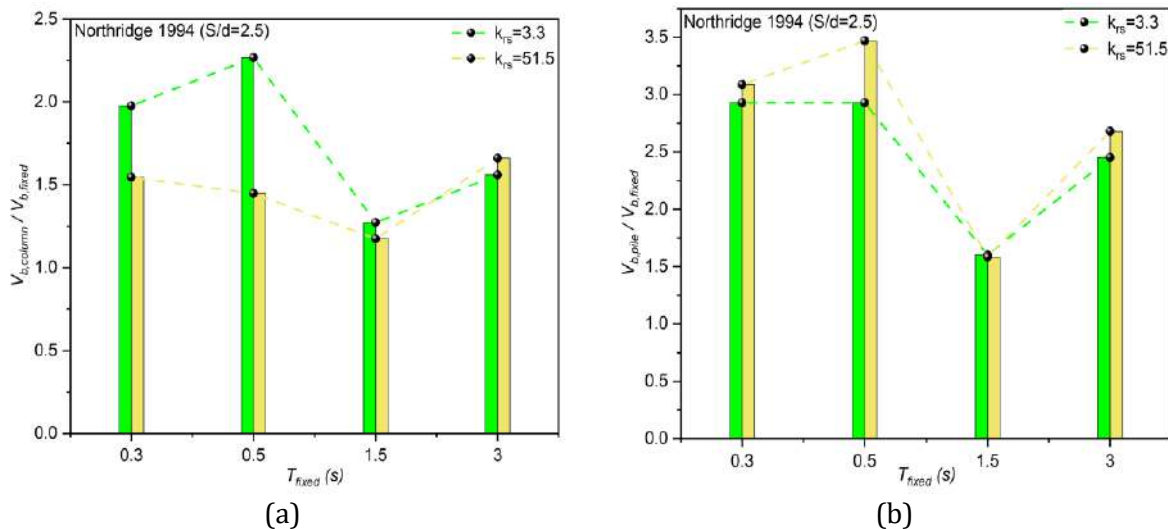


Fig. 14. Impact of raft soil stiffness on Normalised base shear (a) Const. $S/d = 2.5$; Varying k_{rs} ; column head shear, (b) Const. $S/d = 2.5$; Varying k_{rs} ; pile head shear

8. Conclusion

The dynamic behavior of CPRF under seismic loading is governed by complex interactions among soil, piles, raft, and superstructure. This study presents a comprehensive assessment of key parameters influencing the seismic response, such as pile length, superstructure mass, soil flexibility, S/d ratio, and raft stiffness. Observations include shifts in natural frequency, period

lengthening, and reductions in pile head shear, all strongly affected by soil–structure interaction. These findings highlight the critical need for integrated analysis and design approaches to ensure safety, resilience, and performance efficiency of CPRF systems in seismic regions.

- Decrease in natural circular frequency with increasing pile length: As the pile length increases, the natural circular frequency of the foundation system reduces due to a reduction in overall stiffness. This reduction is primarily attributed to the increased flexibility and diminished lateral stiffness of longer piles, which lowers the effective stiffness of the entire foundation system.
- Impact of superstructure on natural radial frequency: The natural circular frequency of a CPRF, when incorporating superstructure, lessens by 10–30% compared to a piled raft foundation that excludes superstructure. This reduction is due to the additional mass and flexibility introduced by the superstructure, which affects the free vibration behavior of the foundation system.
- Lengthening of fundamental period due to soil flexibility: The inclusion of soil flexibility into the soil–piled raft–superstructure system leads to a substantial increase in the fundamental period, with maximum lengthening observed as 250%, 180%, 92%, and 37% for superstructures having fixed-base fundamental time periods (T_{fixed}) of 0.30 s, 0.50 s, 1.5 s, and 3.0 s, respectively. This effect is particularly pronounced in CPRF systems with relatively low raft–soil stiffness. These findings highlight the significant impact of soil–foundation–structure interaction on the free vibration response of the overall system, especially for short-period structures, where the interaction causes more pronounced shifts in the fundamental period.
- Effect of S/d ratio and raft-soil relative stiffness on period lengthening: For structures with moderate fundamental periods (e.g., $T_{\text{fixed}} = 0.5$ s), an increase in the pile spacing-to-diameter ratio (S/d) and the relative stiffness between the raft and soil can reduce the extent of period lengthening. However, minimal or no variation is noted in the extent of period lengthening for short-period structures (e.g., $T_{\text{fixed}} = 0.3$ s) with variations in both the S/d ratio and raft flexibility. This suggests that for shorter period structures, the influence of these parameters is less significant.
- Effect on base shear and pile head shear due to S/d ratio and raft flexibility: The impact of the S/d ratio and flexibility of the raft on base shear at the column and pile head is almost the same. Additionally, the incorporation of raft flexibility significantly alleviates the pile head shear response under seismic ground motions, such as the Northridge ground motion. This reduction in pile head shear can be attributed to the increased energy dissipation and damping provided by the flexible raft, which helps to mitigate the forces transmitted to the pile heads during seismic events.
- Dynamic interaction between soil, piles, raft, and superstructure: The dynamic interaction between the soil, piles, raft, and superstructure significantly influences the overall seismic behavior of the foundation system. The interaction between soil, structure and piled raft can lead to changes in the natural frequencies, damping ratios of soil or structure, and different mode shapes of the system, affecting its performance during seismic events.
- Importance of accurate modelling and analysis: Accurate modelling and analysis of the soil–pile–raft–structure system are essential for predicting its seismic behavior. Factors such as soil nonlinearity, pile-soil interaction, and the dynamic characteristics of the superstructure must be considered to ensure realistic assessments of the foundation's performance.
- Design implications for piled raft foundations: The findings emphasize the importance of considering soil flexibility and superstructure stiffness in the design of combined piled raft foundations. Design strategies should account for the potential lengthening of the fundamental time period and the reduction in shear at the pile head to enhance the seismic resilience of the foundation system.

8.1 Recommendations for Future Research

Further research is recommended to examine the effects of layered soil, pile configurations, and superstructure designs on the dynamic behavior of the CPRF. Experimental studies and advanced

numerical simulations can offer a more detailed understanding of the complex interactions within the system.

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References

- [1] Poulos HG, Davis EH. Pile foundation analysis and design. 1980.
- [2] Brown PT, Poulos HG, Wiesner TJ. Piled raft foundation design. University of Sydney, School of Civil Engineering, Civil Engineering Laboratories; 1976.
- [3] Burland JB, Broms BB, De Mello V. Behaviour of foundations and structures. Proceedings of the 9th International Conference on Soil Mechanics, Tokyo, 495-546, 1977.
- [4] Bazar MH, Rafiee F, Azizkandi AS, Lee CJ. Effect of super-structure frequency on the seismic behavior of pile-raft foundation using physical modeling. *Soil Dynamics and Earthquake Engineering*. 2018 Jan 1; 104:196-209. <https://doi.org/10.1016/j.soildyn.2017.09.028>
- [5] Akbari A, Eslami A. Performance of Composite Piled Raft Foundations with Long and Short Piles Under Static and Seismic Loading. *Geotechnical and Geological Engineering*. 2025 Jun;43(5):188. <https://doi.org/10.1007/s10706-025-03136-9>
- [6] Afrazi, M., Razavi, M., Monjezi, M., Bhatawdekar, R., Mohamad, E. T. Development and Evaluation of a Computer-Aided Educational Platform for Advancing Understanding of Slope Stability Analysis. *Civil Engineering Infrastructures Journal*, 2024; 57(2): 371-382.
- [7] Pandey G, Mourya VK, Patel D, Kumar R, Kumar S. Load sharing behaviour in piled-raft foundations over sand and clay: An experimental investigation. *Res. Eng. Struct. Mater.* 2024;10(3):917-42. <http://dx.doi.org/10.17515/resm2023.41me0714rs>
- [8] Deb P, Debnath B, Reang RB, Pal SK. Structural analysis of piled raft foundation in soft soil: An experimental simulation and parametric study with numerical method. *Ocean Engineering*. 2022 Oct 1; 261:112139. <https://doi.org/10.1016/j.oceaneng.2022.112139>
- [9] Nguyen DD, Kim DS, Jo SB. Parametric study for optimal design of large piled raft foundations on sand. *Computers and Geotechnics*. 2014 Jan 1; 55:14-26. <https://doi.org/10.1016/j.compgeo.2013.07.014>
- [10] Kumar A, Choudhury D, Katzenbach R. Effect of earthquake on combined pile-raft foundation. *International journal of geomechanics*. 2016 Oct 1;16(5):04016013. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0000637](https://doi.org/10.1061/(ASCE)GM.1943-5622.0000637)
- [11] PLAXIS 3D Ultimate [Computer software]. PLAXIS BV, Netherlands. 2025.1.
- [12] ETABS Integrated Building Design Software, Version 19, Computers and Structures, Inc., 2019.
- [13] Bowles, J.E.: Foundation Analysis and Design. TheMcGraw-Hill Companies Inc, New York (1997)
- [14] Pandey G, Patel D, Mourya VK, Kumar R, Kumar S. A review on soil-foundation-interaction models. *Journal of Rehabilitation in Civil Engineering*. 2023 Aug 1;11(3):158-79.
- [15] Mourya VK, Pandey G, Patel D, Kumar R. Approaches considering non-linearity in soil-foundation-interaction: A State-of-the-Art Review. *Res. Eng. Struct. Mater.* 2023;9(3):989-1013. <http://dx.doi.org/10.17515/resm2023.646me0117>
- [16] Bhaduri A, Rao VD, Choudhury D. The behaviour of pile group and combined piled-raft foundation in liquefiable soil under seismic conditions. *Geotechnical Engineering Journal of the SEAGS & AGSSEA*. 2020;51(2):130-8.
- [17] Saadatinezhad M, Lakirouhani A, Jabini Asli S. Seismic response of non-connected piled raft foundations. *International Journal of Geotechnical Engineering*. 2021 Jan 2;15(1):66-80. <https://doi.org/10.1080/19386362.2019.1565392>
- [18] Patil G, Choudhury D, Mondal A. Nonlinear dynamic soil-foundation-superstructure interaction analysis for a reactor building supported on a combined piled-raft system. *International Journal of Geomechanics*. 2023 Apr 1;23(4):04023009. <https://doi.org/10.1061/IJGNALGMENG-8096>
- [19] Gazetas G. Seismic response of end-bearing single piles. *International Journal of Soil Dynamics and Earthquake Engineering*. 1984 Apr 1;3(2):82-93. [https://doi.org/10.1016/0261-7277\(84\)90003-2](https://doi.org/10.1016/0261-7277(84)90003-2)
- [20] Chaudhuri CH, Choudhury D. Buried pipeline subjected to seismic landslide: A simplified analytical solution. *Soil Dynamics and Earthquake Engineering*. 2020 Jul 1;134:106155. <https://doi.org/10.1016/j.soildyn.2020.106155>
- [21] Bentley KJ, Naggar MH. Numerical analysis of kinematic response of single piles. *Canadian Geotechnical Journal*. 2000 Dec 1;37(6):1368-82. <https://doi.org/10.1139/t00-066>
- [22] Kuhlemeyer, R.L. and Lysmer. J. (1973) Finite Element Method Accuracy for Wave Propagation Problems. *Journal of the Soil Dynamics Division*, 99, 421-427. <https://doi.org/10.1061/JSEFAQ.0001885>

- [23] Kramer, S. L. (1996). Geotechnical earthquake engineering, Prentice-Hall, New Jersey
- [24] Pacific Earthquake Engineering Research Center (PEER). PEER Ground Motion Database. Berkeley (CA): University of California, Berkeley. <https://ngawest2.berkeley.edu/>
- [25] Horikoshi K, Matsumoto T, Hashizume Y, Watanabe T. Performance of piled raft foundations subjected to dynamic loading. International Journal of Physical Modelling in Geotechnics. 2003 Jun;3(2):51-62. <https://doi.org/10.1680/ijpmg.2003.030205>
- [26] Clancy P, Randolph MF. Simple design tools for piled raft foundations. Geotechnique. 1996 Jun;46(2):313-28. <https://doi.org/10.1680/geot.1996.46.2.313>

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Review of improving the shear capacity of reinforced concrete beams augmented with fiber-reinforced polymer bars

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Abstract

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The use of fiber-reinforced polymer (FRP) bars represents a promising technique for enhancing the shear capacity of reinforced concrete (RC) members. Epoxy may be utilized to affix FRP bars into vertical holes drilled in concrete to enhance the shear strength of reinforced concrete (RC) beams. This paper presents a comprehensive review of the Embedded Through-Section (ETS) technique using FRP bars to enhance the shear strength of RC beams. From the literature review, the slanted ETS bars were significantly more efficient than the vertical ones, owing to the greater potential resistive bond length provided by the former shape because the directions of the bars will be perpendicular to the hypothetical directions of the shear cracks. For equivalent shear strengthening ratios, the ETS technique demonstrated superior efficacy compared to externally bonded (EB) and near-surface-mounted (NSM) methods. The remarkable efficacy of the ETS approach is attributable to the superior bonding conditions afforded by the core concrete encasing these bars.

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1. Introduction

Fiber Reinforced Polymer (FRP) composites used in reinforced concrete members have become one of the most promising technologies for repairing aging infrastructures during the last three decades. Reinforcement using Carbon Fiber Reinforced Polymer (CFRP) may be used in a variety of contexts, including both brand-new construction and renovation projects [1,2,3]. The many advantages of using FRP especially CFRP as an outside reinforcement for reinforced concrete (RC) beams have led to extensive research into their effectiveness. Most studies have focused on shear strengthening, which involves using CFRP products. While some studies have looked at the overall response of CFRP-reinforced beams, others have focused on the bond interface to see how it acts locally. Bond behavior at the interface, the source of many early failures, has been studied in an effort to pinpoint its causes [4,5]. FRP bars offer advantages like corrosion resistance, a superior strength-to-weight ratio, electrical non-conductivity, and durability [6,7].

There are many FRP forms used to strengthen reinforced concrete beams in shear such as: sheets, laminates, meshes, and bars [8,9]. The ETS method offers numerous benefits over existing methods, like externally bonded FRP sheets (EB FRP) and near-surface mounted FRP rods (NSM FRP). Unlike EB and NSM methods where the FRP relies on the concrete cover of RC beams, in the ETS method, the FRP relies on the concrete core of the RC beam, which offers a greater confinement and hence improves bonding performance [10].

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Ashour [11] presented the test findings of twelve concrete beams reinforced with glass fiber-reinforced polymer (GFRP) bars, which were subjected to a four-point loading method. All test specimens lacked transverse shear and compression reinforcement, and were categorized into two categories based on concrete compressive strength. The primary parameters examined in each group were the beam depth and the quantity of GFRP reinforcement. Two mechanisms of failure were identified: flexural and shear. Flexural failure primarily occurs due to tensile rupture of GFRP bars, either in the mid-span area or beneath the applied point stress. Shear failure is triggered by a significant diagonal crack in the beam's shear span. This diagonal crack extended horizontally at the level of the GFRP bars, signifying bond failure.

El Refai et al. [12] examined the shear effectiveness of concrete beams constructed using basalt fiber-reinforced concrete (BFRC) and reinforced with basalt fiber-reinforced polymer (BFRP) bars. Fourteen hybrid (BFRC-BFRP) beams devoid of stirrups were subjected to failure testing using a four-point loading configuration. The parameters examined were the volume fraction, V_f , of the incorporated fibers (0.75% and 1.5%), the reinforcement ratios of the BFRP bars (0.31%, 0.48%, 0.69%, 1.05%, and 1.52%), and the shear span-to-depth ratios, a/d , of the beams (3.3 for slim beams and 2.5 for short beams). The test findings indicated that the incorporation of 0.75% basalt macrofibres (BMF) enhanced the shear capacity of thin and short beams by 46% and 43%, respectively, in contrast to 81% and 82% when 1.5% BMF was utilized. The influence of including the BMF on the shear capacity of the beams decreased with increasing longitudinal reinforcement ratios.

In this study, a review was developed to deeply provide insight into improving the shear strength of reinforced concrete beams augmented with FRP bars this is because there are still gaps in our knowledge of long-term behavior under complicated loading scenarios and environmental circumstances, notwithstanding progress. In order to overcome these obstacles and guarantee the sustainability and dependability of RC beams strengthened in shear with FRP bars in a variety of applications, research is still concentrated on novel materials and hybrid bonding processes.

2. Review Methodology

In this study, a review of research published in reputable journals for the period from 2007 to 2025 was prepared, as this research encompassed a diverse range of experimental, analytical, and numerical studies. However, most of it was of the experimental type. The focus was on the effect of the spacing of steel stirrups, the strengthening technique, the embedment length of the FRP bar, and its angle of inclination on improving the shear resistance in reinforced concrete beams strengthened with FRP bars. Then, the necessary comparisons are made to reach useful conclusions.

3. Shear Failure Mechanism in Concrete Beams

A multitude of experimental and theoretical studies into the shear bearing capacity of reinforced concrete (RC) members had been undertaken. In concrete beams subjected to bending, which are longitudinally reinforced but lack stirrups, the shear failure mechanism exhibits considerable variation, particularly impacted by the percentage of main reinforcement, strength of concrete, member scale, and the shear span-to-depth ratio. This ratio considers the relationship between the moment of bending M and the shear force V acting concurrently in the same cross section, expressed as $M/(Vd)$. For three- and four-points bending tests, it can be calculated as a/d , where a represents the distance between the applied load and the supports, and d denotes the effective depth of the cross section. Experiments documented in scholarly articles indicate that characteristics like concrete strength and reinforcement ratio exert predominantly quantitative influence on shear failure mode rather than qualitative ones [13]. Shear failure could be classified according to three types based on the ratio of shear span to effective depth:

- Diagonal tensile failure
- Diagonal compressive failure
- Splitting or true shear failure

3.1 Diagonal Tension Failure

Diagonal tensile failure happens when the shear span (a) to effective depth (d) ratio ($a/d > 2.5$). Diagonal tension failure may arise when the load and reaction are considerably separated, as seen in Fig. 1. The diagonal fracture arises from previous flexural cracking and gradually inclines under shear strain. As the diagonal fracture advances into the compressive zone, it meets resistance. The tensile fracture progressively expands at a moderate slope as the load increases, ultimately resulting in an abrupt failure at point 2. Beam failure is an alternate term for this category of failure. [14].

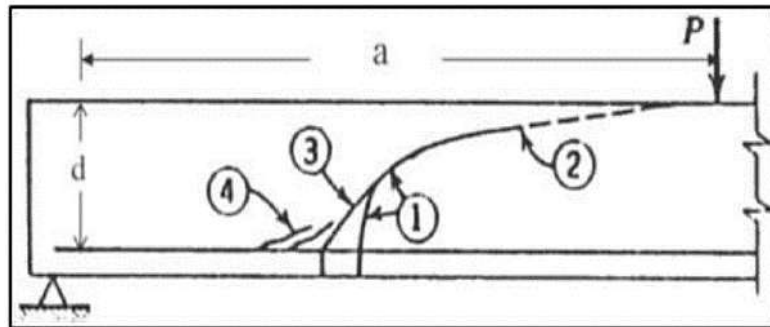


Fig. 1. Diagonal tensile cracks [15]

3.2 Diagonal Compressive Failure

Diagonal compressive failure transpires when the shear span (a) to effective depth (d) ratio is between 1 and 2.5, as seen in Fig. 2. In this failure mode, the specimen is capable of supporting an extra load subsequent to the emergence of the initial fully formed inclined tension fracture. The inclined cracking and bonding separation alongside the steel were constrained by vertical compression stresses throughout the response, and elevated shear over a brief span may trigger a 45° fracture (bending-shear cracking) the crack starts in the bending zone and progresses toward the compression zone (loading point). Such fissures impede the transmission of shear resistance to a lesser un-cracked depth, hence augmenting shear stress. Consequently, the beam functions as an Arch Action. The compression zone diminishes as the angled fracture propagates. Adjacent to the load point, it becomes insufficient to withstand the compressive force, resulting in crushing (shaded region depicted in Fig. 2) [16].

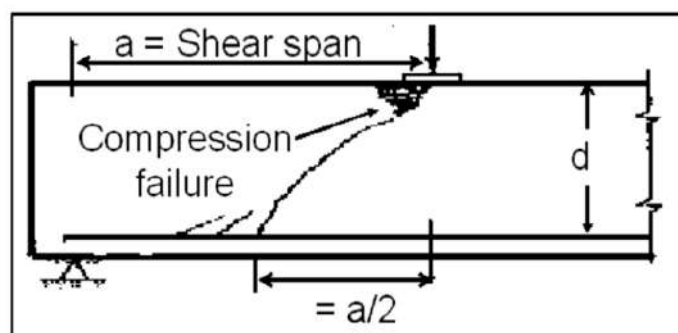


Fig. 2. Shear compressive failure [16]

3.3 Shear Failure in Deep Beams ($a/d < 1$)

Splitting or full shear failure happens in deep beams if the shear span is narrow, namely when $a/d < 1$ as shown in Fig. 3. Conventional diagonal tension concepts are negated in this situation because shear is transferred as an angled thrust between the load and the reaction. In such cases, the shear strength is greatly increased. Splitting is the ultimate failure, albeit it might also fail during compression at the reaction, [16].

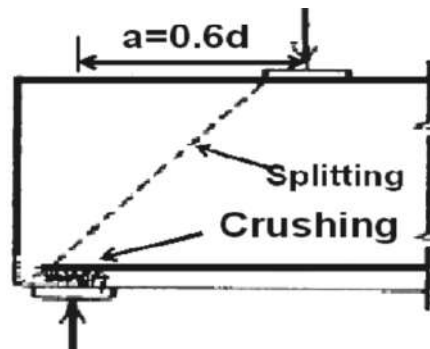


Fig. 3. True shear failure [16]

3.4 Factors Influencing the Shear Strength of Concrete

Numerous primary elements impact the shear strength of RC beams like [16], [17]:

- Concrete compressive Strength (f'_c)
- Tensile steel ratio (ρ)
- Beam size
- Shear span to effective depth ratio (a/d)
- Shear reinforcement ratio (ρ_v)

4. Previous Studies on Embedded Through Section Technique (ETS)

To make RC beams stronger in shear, epoxy can be used to bond FRP rods to vertical holes drilled into concrete. So far, there haven't been many experiments done on how embedded through section (ETS) FRP rods may be used to make RC beams stronger against shear. An overview of experimental research on shear strengthening RC beams using ETS FRP rods is provided below:

4.1 Effect of Steel Stirrups

Chaallal et al. [18] reported the findings of a study that compared the efficacy of the ETS approach with that of the externally bonded (EB) and near-surface-mounted (NSM) techniques. T-beams with a length of 4520 mm are subjected to a total of 12 tests. (1) The effectiveness of the ETS approach relative to the EB FRP sheets and NSM FRP bar techniques; (2) the existence of inside steel; and (3) the ratio of internal stirrups is examined. The test findings verify the effectiveness of the ETS technique and illustrate that the shear behavior of beams reinforced with this method significantly surpasses that of samples strengthened using the EB and NSM methods. Table 1 shows the experimental program details, revealing a mean concrete strength of 25 MPa for the So and S1 set, and 35 MPa for the S3 group. The inside of flexural steel possesses a nominal yield strength of 471 MPa for the So and S1 series, and 651 MPa for the S3 groups. The shear reinforcement possesses a yield capacity of 540 MPa for So and S1, and 650 MPa for the S3 groups. Table 2 presents the outcomes of the experiment. For the samples strengthened using the EB and NSM techniques, the inclusion of transverse steel reduced the contribution of FRP to the shear resistance. In the specimens strengthened using the ETS method, the addition of transverse steel reinforcement did not considerably reduce the contribution of FRP.

Table 1. Experimental layout [18]

Beam ID/set	So set	S1 set	S3 set
Reference beam	So-CO	S1-CO	S3-CO
EB FRP method	So-EB	S1-EB	S3-EB
NSM FRP method	So-NSM	S1-NSM	S3-NSM
ETS FRP method	So-ETS	S1-ETS	S3-ETS

Barros et al. [19] evaluated the effectiveness of the ETS approach. An extensive experimental program involving 14 reinforced concrete beams was conducted, yielding the following results: (i) inclined ETS strengthening bars demonstrated superior effectiveness compared to vertical ETS

bars, with shear capacity increasing as the spacing of bars decreased; (ii) shear-failure was transformed into flexural-failure; and (iii) shear capacity of the beams improved with reduced spacing between bars. The ACI 318-(08) and Eurocode-2nd-(2004) requirements for shear resistance were examined for potential applicability.

Table 2. Experimental outcomes [18]

Strength method	Series	Specimen	Load at Rupture (kN)	Total shear resistance (kN)	Resistance of concrete (kN)	Resistance attributable to steel (kN)	Failure Mode
Reference	So	SO-CO	122.7	81.3	81.3	0.0	Sh-F
	S1	S1-CO	350.6	232.2	81.3	150.9	Sh-F
	S3	S3-CO	294.0	194.7	96.2	98.5	Sh-F
EB	So	SO-EB	181.2	120.0	81.3	0.0	Sh-F
	S1	S1-EB	378.5	250.7	81.3	150.9	Sh-F
	S3	S3-EB	335.2	222.0	96.2	98.5	Sh-F
NSM	So	SO-NSM	198.0	131.1	81.3	0.0	Sh-F
	S1	S1-NSM	365.0	241.7	81.3	150.9	Sh-F
	S3	S3-NSM	380.0	251.6	96.2	98.5	Sh-F
ETS	So	SO-ETS	273.0	180.8	81.3	0.0	Sh-F
	S1	S1-ETS	397.0	262.9	81.3	150.9	F-F
	S3	S3-ETS	425.5	281.8	96.2	98.5	F-F

Sh-F = shear failure, F-F = flexural failure.

The experimental and analytical results exhibited a strong correlation. The experimental program consists of two groups, A and B, including beams with cross-sections of 15 cm by 30 cm and 30 cm by 30 cm, respectively, each with a full length of 245 cm and a 90 cm span length. Table 3 presents the general data regarding the beams in the two groups, where ρ_{sl} denotes the main steel reinforcement ratio ($\rho_{sl} = A_{s1} / d b_w$), with A_{s1} representing the area of cross-sectional of the main steel bars, b_w indicating the width of web, and d signifying the effective depth. ρ_{sw} denotes the shear reinforcement ratio ($\rho_{sw} = A_{sw} / s_w b_w$), where A_{sw} represents the cross-sectional area of the two legs of the stirrup, and s_w indicates the spacing between stirrups. The ρ_{fw} presented in Table 1 represents the ETS strengthening percentage, defined by the equation $\rho_{fw} = A_f / (s_f b_w \sin \theta_f)$, where A_f denotes the ETS shear strengthening bar area, s_f signifies the spacing of bars, and θ_f indicates the angle of inclination of the strengthening bars relative to the beam's longitudinal axis. Figure 4 illustrates the ETS reinforcement method utilized in this reference, whereas Figure 5 depicts the correlation between the load and the deflection of the section for groups A and B.

Table 3. General information on the beams [19]

Beams ID	150 × 300 mm ²			Batch	300 × 300 mm ²			Batch
	ρ_{sl} (%)	ρ_{sw} (%)	ρ_{fw} (%)		ρ_{sl} (%)	ρ_{sw} (%)	ρ_{fw} (%)	
Reference	2.50	0.00	0.00	1	1.88	0.00	0.00	1
S300.90	2.50	0.13	0.00	1	1.88	0.06	0.00	1
E300.90	2.50	0.00	0.17	1	1.88	0.00	0.11	1
E300.45	2.50	0.00	0.25	2	1.88	0.00	0.16	2
S300.90/E300.90	2.50	0.13	0.17	1	1.88	0.06	0.11	1
S300.90/E300.45	2.50	0.13	0.25	2	1.88	0.06	0.16	2
S225-90	2.50	0.17	0.00	2				
S225-90/E225.90	2.50	0.17	0.23	2				

RC T-cross section beams reinforced in shear with ETS steel rebars and ETS CFRP bars were utilized in an experiment. The primary objective of the research in [20] is to evaluate the efficacy of the

ETS on samples with varying levels of internal stirrups ($\rho_{sw} = 0$, $\rho_{sw} = 1$, and $\rho_{sw} = 0.17\%$), where ρ_{sw} denotes the shear reinforcement ratio ($\rho_{sw} = A_{sw}/s_w b_w$), with s_w indicating the spacing between stirrups. The efficacy of various ETS strengthening setups was also examined. The strong adhesion between the reinforcing ETS bars and the surrounding concrete enabled the ETS steel bars to begin yielding and allowed the ETS CFRP rods to attain elevated tensile stresses. This resulted in a substantial enhancement of shear strength, which was significantly affected by the angle of the ETS rebars and the quantity of stirrups. Based on the results, it can be said that the proposed method raised the load-carrying capacity of ETS bars in general, no matter what kind of material they were made of. Between 5% and 68%, more weight could be carried with the help of vertical ETS bars. Since the ultimate load went up from 53% to 136%, the angled ETS bars made sure that the strengthening worked better. The angled ETS bars were also better at making sure that the beams could bend more when they broke. The reason why inclined ETS bars work better is that they have a longer bonding length and are better positioned in relation to the angle of the shear failure cracking [20].



Fig. 4. ETS strengthening technique [19]

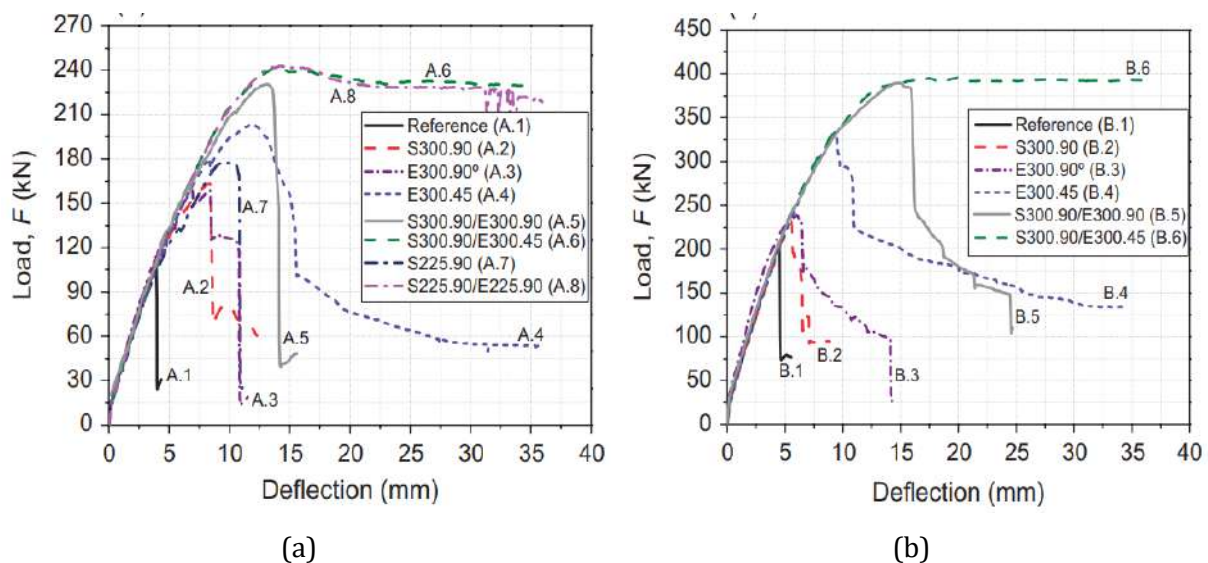


Fig. 5. Correlation between the applied load and the deflection of the loaded part for series: (a) A and (b) B [19]

Chalioris et al. [21] are conducting studies to evaluate the efficacy of a novel retrofitting technique aimed at enhancing the structural performance of RC deep beams devoid of steel stirrups, utilizing CFRP bars as the only stirrups. Continuous loading experiments are conducted on five shear-critical beams with T-shaped and rectangular cross sections. The reinforcement strategy consists of (a) two normal U-shaped 2-link ropes attached within the flanged beam's web's perimeter utilizing a near-surface-mounted (NSM) approach, and (b) a vertical and a diagonal 1-link CFRP rope installed internally with the web of the RC beam via an ETS approach. The CFRP ropes' free lengths were securely fastened in both cases utilizing epoxy-bonded lap splices as NSM at (a) the upper and lower portions of the rectangular beam's web and (b) the upper portion of the T-beam's slab. The

proposed reinforcing method improved strength and converted shear failure into flexural failure, signifying a favorable result. The findings from the experiments of this research were employed to validate the precision of an analytical method for predicting the capacity of shear-strengthened deep beams reinforced employing FRP ropes as transversal links.

Van et al. [22] reported the results of an experiment on RC beams augmented in shear using embedded-through-section (ETS) steel bars and (FRP) rebars, with and without an engineered anchoring method. The response of the beams, encompassing the relationship between load and deflection, fracture patterns, and failure mechanisms, are examined. The shear behavior of different strengthening methods is analyzed through contrasting ETS scenarios with traditional reinforcing techniques. The impact of various forms of ETS strengthening is examined using differing quantities of steel stirrups. The shear effect provided by the reinforcement bars is examined to evaluate the effectiveness of current models. A mean strain formula for ETS-FRP rebars is formulated from the original model. Comparing the findings of this article with existing literature demonstrates that the end-anchored significantly enhances the efficacy of shear strengthening. The following are the principal conclusions of this reference: The shear resisting force of the unanchored vertical ETS steel bars was less to that of the interior vertical steel stirrups, which employed closed form configurations. Nevertheless, the shear contribution of the unanchored diagonal ETS steel system was marginally more than that of the internal diagonal steel shear reinforcement. The data indicate that premature debonding of the ETS bars transpired due to inadequate bond length, leading to diminished shear strengthening efficacy.

4.2 Effect of the Embedment Length of FRP bar

Godat et al. [23] developed two-dimensional finite element models to examine the interaction between the CFRP bar and concrete at the interface during the pull-out tests. In the model, separate truss members are positioned beside the CFRP bar to link it to both sides of the concrete. The behavior of bonds is shown using the Cosenza–Manfredi–Realfonzo (CMR) bonding stress–slip model. Numerical simulations indicate that the interfacial profiles of CFRP bars and concrete may be utilized to find the optimal lengths for the CFRP rods. The work had been extended to test FRP shear-strengthened specimens utilizing the ETS approach through the creation of a 3-D finite element model. The appropriate constitutive examples have been employed to estimate the nonlinear behavior of steel bars, plain concrete, and CFRP rods. The results are presented regarding the maximum load-carrying capacity of the bars, the correlations between load and deflection, and the axial stresses in the CFRP bars. The finite element results are checked with experimental results, and they align closely.

Bui et al. [24] conducted tests, analytical assessments, and numerical simulations to study the bonding of (ETS) FRP rods to concrete and its impact on the structural behavior of beams reinforced in shear using ETS rods. A series of pullout experiments were done to determine the adhesion between ETS bars and concrete, including variables like embedment length and ETS stiffness. An exhaustive analytical technique is presented for delineating the ETS–concrete interface profile, using historical data. The suggested bonding model is utilized to develop a novel technique for finding the ETS shear capacity and determining the energy absorption capacity of reinforced beams. The findings show that the established bonding model effectively simulates the interface between ETS and concrete, and the approach for determining the shear involvement of ETS FRP bars via a bond-based mechanism performs favorably in comparison to alternative shear models. The interfacial ductility index and interfacial energy significantly influence the performance of well-strengthened components. Finite element (FE) calculations were conducted on beams reinforced with ETS bars, utilizing the suggested bonding approach. The comparison of FE modeling findings and experimental data demonstrates that FE simulation with the suggested bond model effectively analyzes shear-strengthened beams constructed with ETS bars.

4.3 Effect of Angle of FRP Bar

Breveglieri et al. [25] ran an experiment using T-RC beams that were strengthened with steel bars to see how well the ETS method could work for shear reinforcing of RC beams. The research looked at how the angle and shear reinforcement ratio of ETS forms affect the efficiency of shear strengthening. It also looked at how ETS bars and currently available steel stirrups work together.

The beams shear capacities, both with and without internal stirrups was increased by 109% and 136%, respectively. Shear resistance went up more with angled ETS bars than with vertical ones.

Al-Kaabi and Al-Fartosi [26] showed the experimental investigation results from reinforced concrete beams that were reinforced with a modified fiber-reinforced polymer (ETS) (FRP) technique. The ETS FRP rod technique could be used to enhance the load capacity of RC beams and increase their shear strength. Since ETS reinforcement is a new way to keep concrete structures from failing, this strategy allows for more confinement and better results. Seven rectangular RC beams with a span of 1500 mm are used in the tests. The impact of the angle of the CFRP rebars was taken into account when setting the parameters for this study. This study shows the findings of seven tests done on reinforced RC beams using the shear ETS method as part of an experimental study. The ETS method is used to make RC members better in terms of FRP. Using the results of this study, the suggested strategy was demonstrated to be accurate. Suggested that there is a good match between the experimental data and the model. Also, the most important results of this study are:

- The ETS FRP strengthening method has been shown to enhance the shear resistance of RC beams. For samples B200-2E10-V, B200-2E10-V, and B200-3E10-V, the shear increase was 23.96%, 23.96%, and 7.43%, respectively.
- It has been shown that putting CFRP rods inside RC beams makes the cross-section more rigid. This is because EST FRP-reinforced beams can handle higher loads than unreinforced beams.
- It was shown that implementing inclined CFRP rods is more effective than implementing vertical CFRP rods, because the shear increase for sample B200-E10-I was 32.2%. This is because the direction of the inclined rods is the opposite of the direction of shear.

4.4 Effect of the Strengthening Techniques

The interaction of ETS bars with preexisting steel stirrups, in addition to the effect of ETS's strengthening shear percentage and inclination with respect to shear strengthening effectiveness, were all evaluated by [16]. In this study, two mathematical methods for determining the impact of ETS on shear resistance were provided. The first model is based on observations and experiments, whereas the second model incorporates the method's underlying physical and mechanical concepts (mechanically-based approach). Both models' prediction abilities are compared using published experimental findings [27].

Moradi et al. [28] showed a new way to use FRP materials to improve the shear resistance of RC beams. This approach entails drilling openings across the beam web, introduce FRP reinforcement in these openings, and then fill them with grout, rather of gluing FRP strips to the outside of the element. Technical literature indicates that externally bonding and NSM FRP reinforcements effectively improve the shear capacity of RC components. Nevertheless, they are inefficient in situations when the concrete cover exhibits inadequate bond strength, as debonding may result in beam collapse before the whole capacity is utilized. In constructions where the beam is secured to the flooring, the application of FRP wrapping requires the drilling of holes in the floor for reinforcement. This makes the area where the floor and beam touch more vulnerable to shear. Experiments show that samples reinforced with the proposed method work as well as or better than beams with FRP reinforcements that are bonded on the outside. Furthermore, a comparison of the Applied Strengthening Material Index (ASMI) for samples indicates that the suggested approach exhibits a superior ratio of FRP shear involvement relative to the quantity of FRP utilized, in contrast to external bonding techniques.

Al-Mahmoud et al. [29] looked into whether the NSM technique could be utilized to strengthen in-shear RC elements with Carbon Fiber-Reinforced Polymer (CFRP) rods. Seven reinforced concrete beams with or without stirrups were tested in flexure. Rods made of (CFRP) with a diameter of 6 mm were used. The performance of a mortar and a resin is compared to see if it could be used as a filler substance with a surface-preconditioned CFRP rod. The results show that the NSM method is a very good way to make a reinforced concrete beam stronger in shear. Shear strengthening still works better when resin is used as the filler material rather than mortar, although the difference is

not very noticeable. The tests' outcomes are contrasted with previously developed analytical models. The NSM method of strengthening reinforced concrete beams in shear with CFRP rods works very well. The shear strength of a single surface-conditioned CFRP rod was approximately 43.6% over that of beams strengthened only with stirrups. The epoxy resin served as the filling medium, resulting in a 34.6% increase compared to the usage of mortar. However, as noted by other writers, augmenting the quantity of NSM CFRP rods within the shear zone significantly enhances the shear capacity of the beams. The data indicate that epoxy resin outperforms cement, however the disparity is not substantial. The interface between the mortar and the surface-conditioned CFRP rod functions effectively, since the failure occurred at the junction of the mortar and the concrete substrate. To enhance the efficacy of the reinforcement, consideration might be given to enhance the texture of the groove surface in the concrete.

5. Shear Behavior of Concrete Beams with Internal FRP Reinforcement

Hamid et al. [30] presented the test outcomes of simply supported RC beams strengthened with steel or GFRP. Sixteen substantial concrete beams reinforced with steel stirrups were constructed and subjected to testing until failure occurred under consistent pressure applied at four locations. Fifty percent of the samples were reinforced longitudinally by GFRP rebars, whereas the remaining fifty percent were longitudinally reinforced with steel bars as control examples. The primary factors examined in the research were longitudinal reinforcement ratios, shear span-to-effective depth ratios, and stirrup ratios. These were utilized to examine the shear performance of GFRP reinforced concrete (RC) beams. Two forms of failure were observed: flexural and shear. Because FRP bars have a lower modulus of elasticity than steel bars, concrete beams reinforced with GFRP bars have a lower shear capacity. The shear span-to- depth and main reinforcement ratios have a major impact on the force's distribution within GFRP reinforced concrete samples. The test findings agreed closely with the projected values supplied by conventional codes and design standards, except for GFRP reinforced concrete beams that encountered shear failure.

Said et al. [31] performed an investigation on the shear performance of beams reinforced with laboratory-manufactured GFRP rebars and stirrups. In the laboratory, the rebars and stirrups are produced from locally available raw materials and a bipartite die mold. Ten beams, each measuring 12 cm in width, 30 cm in depth, and 155 cm in length, were manufactured and subjected to four-point loading until collapse ensued. The primary criteria were the GFRP web reinforcement percentage and the compressive capacity of the concrete, quantified by the quantity of GFRP stirrups ($\varnothing 8$ at 215mm, 150, and 100mm). The deflection at mid-span, the load on the inclined crack, and the stresses in the GFRP reinforcing stirrups and bars were quantified and analyzed for each examined beam. The test findings indicated that the shear capacity of a stirrup-less beam improved by 41% and 82%, respectively, when web GFRP reinforcement of 8 @ 215 and 8 @ 100 was employed. As the concrete compressive strength increased from 25 MPa to 45 MPa and subsequently to 70 MPa, the shear strength augmented by 49% and 104%, respectively. The highest strain observed in GFRP stirrups was 0.0095. A unique approach for assessing the shear strength of FRP stirrups was introduced and corroborated by a comparison of the experimental results from this study with those of prior studies. The shear capacities of the examined materials were assessed utilizing nonlinear finite element analysis (NLFEA) and strut and tie models (STM). The experimental shear strength ratio to anticipated shear capacity ($V_{\text{exp}}/V_{\text{pred}}$) was 1.2 for NLFEA and 0.9 for STM.

Issa et al. [32] examined the shear capacity as well as behavior of RC beams strengthened with basalt fiber-reinforced polymer (BFRP) bars, as well as without stirrups. Six concrete beams measuring 200 mm × 300 mm (8 in × 12 in) and 300 mm × 200 mm (12 in × 8 in) were fabricated with and without BFRP stirrups. The main reinforcement ratios (ρ_f) for samples without shear reinforced (NSR) beams varied from 2.690 to 14.80 times the percentage of balance (ρ_{fb}). For shear-reinforced (SR) samples, the range varied from 1.690 to 6.880 times ρ_{fb} . For the NSR beams, two distinct shear span-to-depth (a/d) ratios (5.650 and 7.00) and three varying a/d ratios (1.50, 2.50, and 3.50) were examined. The experimental data are presented regarding fracture patterns, modes of material failure, load deflection, load-strain characteristics, and shear strength. For both SR- and NSR beams, the shear strength increased with the augmentation of BFRP reinforcement

area, despite the a/d ratio being constant. Conversely, when the a/d ratio increased, the shear strength diminished. The studies assessed predictive models and code equations for their efficacy in forecasting the shear capacity of BFRP reinforced concrete beams. Standard conditions were employed to produce both conservative and non-conservative projections. The predictions depend on the variables. Table 4 delineates the beam categories and reinforcing parameters for BFRP RC-Beams, indicating that the NSR group beams possess dimensions of 300mm in width, 200mm in whole depth, and 2440mm in span, whilst the SR group beams exhibit dimensions of 200mm in width, 300mm in total depth, and 3050mm in span. Figure 6 illustrates the BFRP flexural bar and shear reinforcement type, while Figure 7 depicts the fracture pattern in the BFRP reinforced concrete beams. Figures 8 and 9 illustrate the load-deflection behavior of non-shear reinforced BFRP RC beams.

Table 4. Beam labels and reinforcement specifications for BFRP RC-beams [32]

Type	Beam number	Flexural reinforcement of BFRP				Shear reinforcement		
		Bar diameter (mm)	Number of bars	Depth d (mm)	ρ_f/ρ_{fb}	10 mm BFRP	#3 steel	Shear span to depth (a/d)
NSR	5-10N5	10	5	170	2.69	No	No	5.65
	5-13N5	13	5	170	4.49	No	No	
	5-16N5	16	5	170	7.15	No	No	
	6-16N7	16	6	170	8.58	No	No	7.0
	3-25N7	25	3	165	11.10	No	No	
	4-25N7	25	4	165	14.80	No	No	
SR	2-13SB1	13	2	275	1.69	Yes	Yes	1.5
	3-13SB1	13	3	275	2.54	Yes	Yes	1.5
	3-16SB1	16	3	270	4.03	Yes	Yes	1.5
	2-25B1	25	2	265	6.88	Yes	No	1.5
	3-16B2	16	3	270	4.03	Yes	No	2.5
	3-16B3	16	3	270.0	4.03	Yes.	No.	3.5

Note: No = without stirrups; S = steel stir.; B = basalt stir.; i.e., Steel and basalt stir. (SB) and $a/d = 1.5$ (1). NSR = No stirrups, SR = with stirrups.

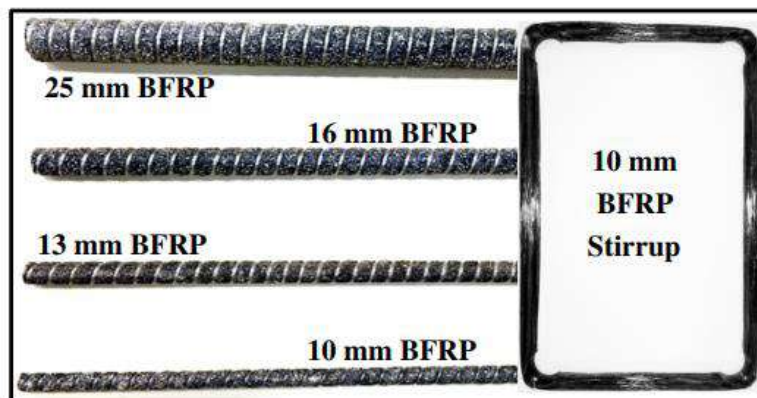
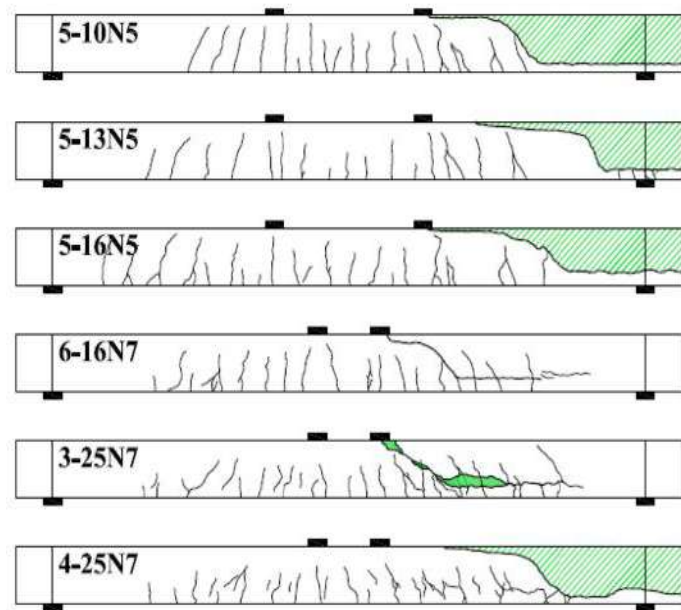


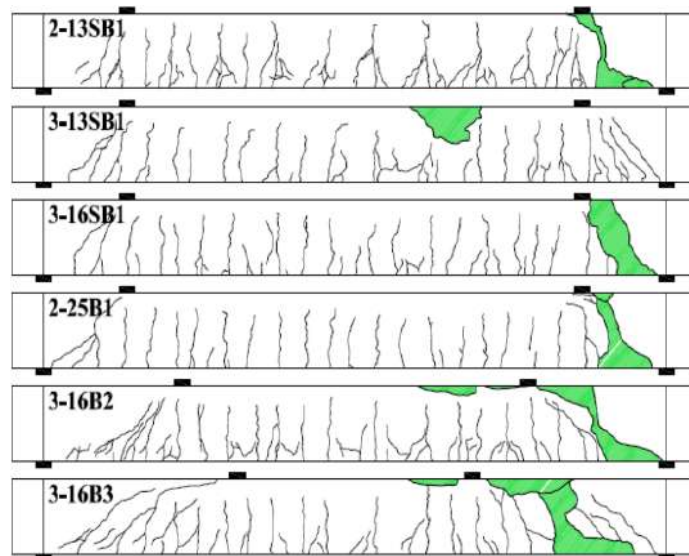
Fig. 6. BFRP flexure bar and shear reinforcement type [32]

Karayannis et al. [33] conducted experimental investigations on the behavior of seven RC beams have CFRP bars subjected to progressively increasing static loads. The load capacities, deflections, stiffness, fast local strength reductions, failure processes, and crack propagation are shown and analyzed. The bond properties of the anchoring distances of CFRP bars have garnered significant interest. The implementation of regional detention circumstances at the anchoring distances of CFRP bars appears to influence their cracking behavior in specific specimens. Furthermore, experimental data for CFRP beams and GFRP beams from contemporary literature are compared and analyzed. Herein are also comparisons between the experimental findings and the expectations based on ACI 440.1R-15 and CSA S806-12. The results of this study showed that: A generalization

derived from the deflection curves is that each time a fracture emerges in the CFRP beams, a sharp reduction in load occurs immediately.



(a)



(b)

Fig. 7. Crack formation in the BFRP reinforced concrete beams: (a) beams without shear reinforcement; (b) beams with shear reinforcement [32]

Huang et al. [34] examined the ambient-cured Geopolymer Concrete GPC beams reinforced with basalt fiber reinforced polymer BFRP bars that were developed, cast, and subjected to static and impact stresses. The mechanism of damage and quantitative findings, including mid-span deflection, impact and response forces, were documented and examined. In comparison to typical rectangular stirrups, spiral stirrups improved the behavior of beams under the effect of impact and static loads, according to test data. Using the improved GPC material model, numerical models were created and calibrated using the impact test findings. The effect of GPC substance strength and the proportions of main and stirrup reinforcement on the behavior of impact-loaded beams was examined by parametric analysis. The utilization of steel bars as compression reinforcements was shown to enhance performance, as BFRP bars were prone to breaking under shear and compression when subjected to impact loads.

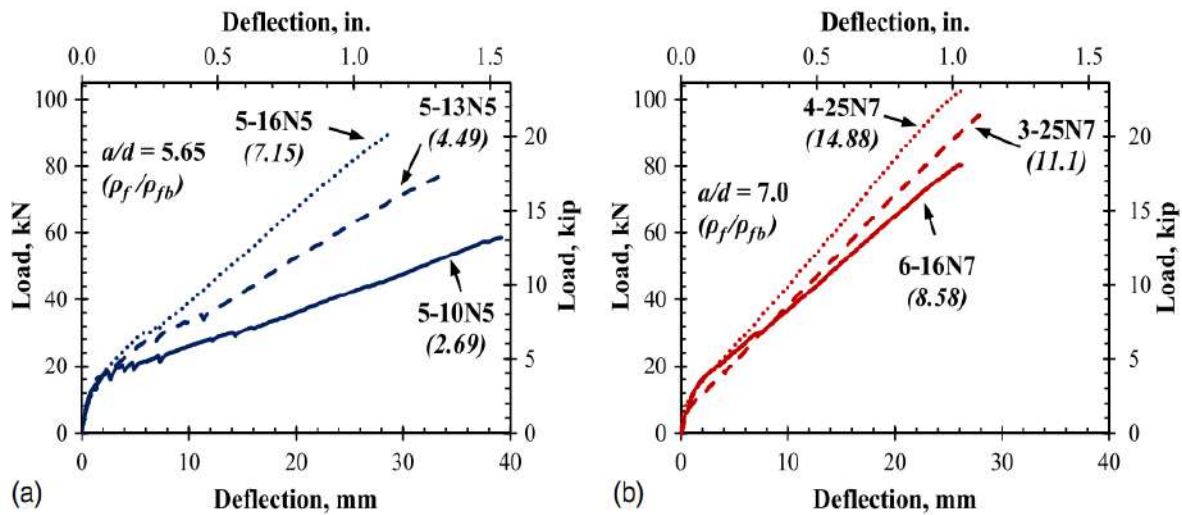


Fig. 8. Load deflection characteristics for non-shear reinforced member BFRP RC-beams: (a) $a/d = 5.65$; (b) $a/d = 7.0$ [32]

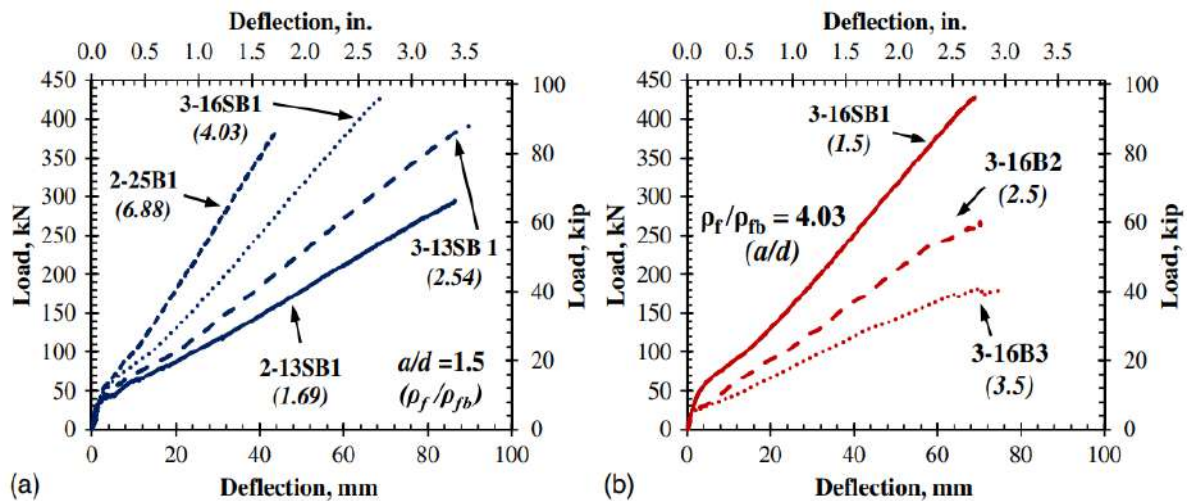


Fig. 9. Load deflection characteristics for non-shear reinforced member BFRP RC-beams: (a) $a/d = 1.5$; (b) $\rho_f/\rho_{fb} = 4.03$ [32]

Fan et al. [35] conducted an investigation of the shear behavior of inorganic polymer concrete (IPC) samples strengthened with stirrups and BFRP bars, considering the influence of stirrup spacing ($S = 8, 10$, and 15 cm) and ratios of shear span to depth ($1.50, 2.00$, and 2.50). The results indicate that all BFRP-IPC beams experience shear failure due to the rupture of BFRP stirrups, leading to failures in both compression and shear. Ratio of shear span to depth exerts a greater influence on the shear behavior of BFRP-reinforced IPC samples than S , decreasing the shear capacity by up to 29.39% . The outcomes of the simulation align closely with the test data; however, the current design guidelines for FRP-reinforced concrete do not include this, resulting in forecasts that deviate from the trials by over 30% . Then, modified equations were made. These equations were used to estimate the shear strength of BFRP-reinforced beams, which matches experimental data well with an average difference of only about 5% .

Hassan and Yousif [36] demonstrated the shear behavior of tapering beams reinforced with BFRP bars via experiments and studies. Eight samples were evaluated to investigate the effects of varying inclination degrees and the incorporation of BFRP stirrups. The beams were categorized into two sets, each including four beams: reference straight specimen and 3 tapered specimens with varying taper angles. The initial batch lacked shear reinforcement, whereas the subsequent group was augmented with BFRP stirrups. The findings indicated that augmenting tapered angles diminished

the shear capacity of BFRP tapered specimens devoid of stirrups, while enhancing it in beams equipped with BFRP stirrups.

Moussa et al. [37] examined the shear behavior of high-strength concrete (HSC) beams strengthened with CFRP bars. Thirteen HSC samples were exposed to static loading to failure, examining parameters including the shear span-to-depth (a/d) percentage, transverse reinforcement percentage, main reinforcement percentage, and the kind of main reinforcement (CFRP, hybrid CFRP-steel, and steel). Research suggests that shear failures in CFRP-reinforced high-strength concrete beams might manifest abruptly due to the brittle characteristics of both FRP and high-strength concrete. The incorporation of main steel bars with CFRP bars markedly improved the shear strength while maintaining a balance between ductility and durability.

6. Summary of Studies on Shear Strengthening of RC Beams

Table 5 illustrates a comparison among the research deals with RC beams enhanced in shear with FRP bars through number of samples, concrete type, FRP bars type and arrangement strengthening method, failure mode type, and percentage of enhancement in failure load.

Table 5. Summary of studies on shear strengthening of RC beams

Ref. No.	No. of samples	Concrete type	FRP bars type	Failure mode type	% of enhancement of shear capacity	Strengthening method
14	12	high strength	CFRP	Flexural for ETS and shear for the rest	19.23 - 61.5	EB *
20	19	Normal	CFRP	Shear	44.3-66.5	ETS
27	15	Normal	FRP	Shear	60-136	ETS
21	5	Normal	CFRP	Flexural or shear	6.5-29.8	ETS or NSM
22	11	Normal	GFRP	Flexural or shear	47-164	ETS
28	8	Normal	CFRP	Shear or failure of bonding	16.8-50.1	ETS or EB
24	9	Normal	GFRP	Premature or pullout	14.3-47.9	ETS
26	7	Self-compacting concrete	CFRP	Flexural or shear	1-13.2	ETS
29	7	Normal	CFRP	Flexural or shear	70-145	NSM

*ETS = embedded through section, NSM = near-surface-mounted; EB = externally bonded

7. Conclusions

Reviewing the literature on increasing the shear capacity of reinforced concrete beams enhanced with fiber-reinforced polymer bars is the main goal of this work. The reviews' key conclusions are provided below:

- Because the bars' directions will be perpendicular to the hypothetical directions of the shear cracks, the slanted ETS bars have a higher potential resistive bond length than the vertical ones, making them noticeably more efficient.
- Because early debonding occurs in NSM strengthened members, ETS bars continuously gave the reinforced member better shear resistance than NSM retrofitted bars.
- Brittle abrupt shear failure modes may be changed to ductile flexural failure modes by using FRP bars for shear strengthening.
- The ETS approach outperformed the EB and NSM approaches in terms of effectiveness for equal shear strengthening ratios. The improved bonding conditions given by the core concrete encasing these bars are responsible for the ETS approach's exceptional effectiveness.

- Most researchers have used CFRP as a prominent FRP product because CFRP has high strength, stiffness, and weight savings.
- In general, GFRP-reinforced concrete beams exhibited linear behavior until failure, attributable to the minimal flexibility of the reinforcing bars. Furthermore, the failure occurred at significant displacements relative to steel reinforced concrete beams.
- CFRP and GFRP bars serve as non-corrosive substitutes for steel, with CFRP exhibiting much superior tensile strength and stiffness relative to GFRP. CFRP is optimal for high-load and long-span applications, but GFRP is more economical for common structural purposes, providing superior bonding capacity with concrete.

As recommendations for future research work:

- Studying the behavior of strengthened reinforced concrete beams with FRP bars under fatigue and cyclic loading.
- Effect of environmental conditions (moisture, temperature, chemical exposure) on concrete beams with FRP bars.
- Studying the behavior of strengthened reinforced concrete beams with FRP bars under the effect of fire.

References

- [1] Dasgupta A, et al. Retrofitting of concrete structure with fiber reinforced polymer. *International Journal for Innovative Research in Science & Technology*. 2018;4(9):42-49.
- [2] Mohammed AS, Abdulkareem BF. A Numerical Study of the Behavior of Concrete Columns constructed with Recycled Aggregate reinforced with GFRP Rebars. *Engineering, Technology & Applied Science Research*. 2025;15(1):20175-20184. <https://doi.org/10.48084/etasr.9600>
- [3] Jumaa OW, Noamanb AT, Mahmoudc AS. Behavior of rubberized concrete deep beams reinforced with GFRP bars. *Res. Eng. Struct. Mat*. 2025. <http://dx.doi.org/10.17515/resm2025-1228ic1010rs>
- [4] Mhanna HH, Hawileh RA, Abdalla JA. Shear strengthening of reinforced concrete beams using CFRP wraps. *Procedia Structural Integrity*. 2019;17:214-221. <https://doi.org/10.1016/j.prostr.2019.08.029>
- [5] Abdulkareem B, Izzet AF. Serviceability of post-fire RC rafters with openings of different sizes and shapes. *Journal of Engineering*. 2022;28(1):19-32. <https://doi.org/10.31026/j.eng.2022.01.02>
- [6] Abbas SR, et al. Assessing the Impact of Using (FRP) Material for Strengthening the Holes in (RC) Beams. In: *Annales de Chimie Science des Matériaux*. 2024. <https://doi.org/10.18280/acsm.480407>
- [7] Rasheed MR, Mohammed SD. Structural behavior of one-way slabs reinforced by a combination of GFRP and steel bars: An experimental and numerical investigation. *Journal of the Mechanical Behavior of Materials*. 2024;33(1):20240002. <https://doi.org/10.1515/jmbm-2024-0002>
- [8] Ali AS, Al-Baghdadi HA. Behavior of RC Beams Strengthened with NSM-CFRP Strips Exposure to Fire. In: *IOP Conference Series: Earth and Environmental Science*. IOP Publishing; 2021. p. 012035. <https://doi.org/10.1088/1755-1315/856/1/012035>
- [9] Oukaili N, et al. On the effectiveness of shear reinforcement type in GFRP-reinforced concrete beams: Experimental study. *Structural Concrete*. 2025.
- [10] Alhilli HH, Al-Farttoosi MH. Shear performance of reinforced concrete t beams strengthened by carbon fiber-reinforced polymer bars. *Civil Engineering Journal*. 2023;9(10):2411-2429. <https://doi.org/10.28991/CEJ-2023-09-10-04>
- [11] Ashour AF. Flexural and shear capacities of concrete beams reinforced with GFRP bars. *Construction and Building Materials*. 2006;20(10):1005-1015. <https://doi.org/10.1016/j.conbuildmat.2005.06.023>
- [12] El Refai A, et al. Shear performance of basalt fiber-reinforced concrete beams reinforced with BFRP bars. *Composite Structures*. 2022;288:115443. <https://doi.org/10.1016/j.compstruct.2022.115443>
- [13] Słowik M. Experimental study of shear failure mechanism in concrete beams. In: *Brittle Matrix Composites 10*. Woodhead Publishing; 2012. p. 345-354. <https://doi.org/10.1533/9780857099891.345>
- [14] You YM, Kang WH. Diagonal Tension Failure Model for RC Slender Beams without Shear Reinforcement Based on Kinematical Conditions (I)-Development. *Journal of Ocean Engineering and Technology*. 2007;21(6):7-15.
- [15] Park R, Paulay T. Reinforced concrete structures. John Wiley & Sons; 1991.
- [16] Chen H, et al. Shear Strength of Reinforced Concrete Simple and Continuous Deep Beams. *ACI Structural Journal*. 2019;116(6). <https://doi.org/10.14359/51718003>
- [17] Ghadhbhan HN. Effect of beam size on shear strength of reinforced concrete normal beams. *Journal of Engineering and Sustainable Development*. 2007;11(1):152-168.

- [18] Chaallal O, et al. Embedded through-section FRP rod method for shear strengthening of RC beams: Performance and comparison with existing techniques. *Journal of Composites for Construction*. 2011;15(3):374-383. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000174](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000174)
- [19] Barros JAO, Dalfré GM. Assessment of the effectiveness of the embedded through-section technique for the shear strengthening of reinforced concrete beams. *Strain*. 2013;49(1):75-93. <https://doi.org/10.1111/str.12016>
- [20] Breveglieri M, Aprile A, Barros JAO. Embedded through-section shear strengthening technique using steel and CFRP bars in RC beams of different percentage of existing stirrups. *Composite Structures*. 2015;126:101-113. <https://doi.org/10.1016/j.compstruct.2015.02.025>
- [21] Chalioris CE, Kosmidou PK, Papadopoulos NA. Investigation of a new strengthening technique for RC deep beams using carbon FRP ropes as transverse reinforcements. *Fibers*. 2018;6(3):52. <https://doi.org/10.3390/fib6030052>
- [22] Van Hong Bui L, Stitmannathum B, Ueda T. Experimental investigation of concrete beams strengthened with embedded through-section steel and FRP bars. *Journal of Composites for Construction*. 2020;24(5):04020052. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0001055](https://doi.org/10.1061/(ASCE)CC.1943-5614.0001055)
- [23] Godat A, Chaallal O, Neale KW. Nonlinear finite element models for the embedded through-section FRP shear-strengthening method. *Computers & Structures*. 2013;119:12-22. <https://doi.org/10.1016/j.compstruc.2012.12.016>
- [24] Bui LVH, Stitmannathum B, Jongvivatsakul P. Comprehensive investigation on bond mechanism of embedded through-section fiber-reinforced polymer bars to concrete for structural analysis. *Journal of Building Engineering*. 2020;29:101180. <https://doi.org/10.1016/j.jobbe.2020.101180>
- [25] Breveglieri M, Aprile A, Barros JAO. Shear strengthening of reinforced concrete beams strengthened using embedded through section steel bars. *Engineering Structures*. 2014;81:76-87. <https://doi.org/10.1016/j.engstruct.2014.09.026>
- [26] Al-Kaabi AH, Al-Fartosi MH. Shear behavior of RC beams strengthened with CFRP Rod using Embedded Through-Section technique EST. *International Journal of Mechanical Engineering*. 2022;7(1):1901-1910.
- [27] Breveglieri M, Aprile A, Barros JAO. RC beams strengthened in shear using the Embedded Through-Section technique: Experimental results and analytical formulation. *Composites Part B: Engineering*. 2016;89:266-281. <https://doi.org/10.1016/j.compositesb.2015.11.023>
- [28] Moradi E, Naderpour H, Kheyroddin A. An experimental approach for shear strengthening of RC beams using a proposed technique by embedded through-section FRP sheets. *Composite Structures*. 2020;238:111988. <https://doi.org/10.1016/j.compstruct.2020.111988>
- [29] Al-Mahmoud F, et al. Reinforced concrete beams strengthened with NSM CFRP rods in shear. *Advances in Structural Engineering*. 2015;18(10):1563-1574. <https://doi.org/10.1260/1369-4332.18.10.1563>
- [30] Hamid NAA, Thamrin R, Ibrahim A. Shear capacity of non-metallic (FRP) reinforced concrete beams with stirrups. *International Journal of Engineering and Technology*. 2013;5(5):593. <https://doi.org/10.7763/IJET.2013.V5.624>
- [31] Said M, et al. Experimental and analytical shear evaluation of concrete beams reinforced with glass fiber reinforced polymers bars. *Construction and Building Materials*. 2016;102:574-591. <https://doi.org/10.1016/j.conbuildmat.2015.10.185>
- [32] Issa MA, Ovitigala T, Ibrahim M. Shear behavior of basalt fiber reinforced concrete beams with and without basalt FRP stirrups. *Journal of Composites for Construction*. 2016;20(4):04015083. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000638](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000638)
- [33] Karayannis CG, Kosmidou PK, Chalioris CE. Reinforced concrete beams with carbon-fiber-reinforced polymer bars-Experimental study. *Fibers*. 2018;6(4):99. <https://doi.org/10.3390/fib6040099>
- [34] Huang Z, et al. Shear behaviour of ambient cured geopolymers concrete beams reinforced with BFRP bars under static and impact loads. *Engineering Structures*. 2021;231:111730. <https://doi.org/10.1016/j.engstruct.2020.111730>
- [35] Fan X, et al. Shear behaviour of inorganic polymer concrete beams reinforced with basalt FRP bars and stirrups. *Composite Structures*. 2021;255:112901. <https://doi.org/10.1016/j.compstruct.2020.112901>
- [36] Hassan BR, Yousif AR. Experimental and Analytical Investigation of Shear Behavior and Strength of Haunched Beams Reinforced with Basalt Fiber-Reinforced Polymer Rebars. *Journal of Bridge Engineering*. 2024;29(12):04024098. <https://doi.org/10.1061/JBENF2.BEENG-6723>
- [37] Moussa AMA, et al. Shear behavior of high-strength concrete beams reinforced with carbon fiber-reinforced polymer bars. *Engineering Structures*. 2025;325:119411. <https://doi.org/10.1016/j.engstruct.2024.119411>

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Enhancing photovoltaic pumping systems through machine learning-based modeling and optimization

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Abstract

This paper deals with the enhancement of photovoltaic water pumping systems through machine learning-based modeling and optimization. First, parameter identification of the target PV module KC200GT is carried out using three algorithms: Newton-Raphson, fsolve, and particle swarm optimization (PSO). Among these methods, PSO shows the best agreement with the manufacturer's data, achieving best performance ($R^2 = 0.9901$ and $RMSE = 0.0358$). In addition, manufacturer data of the centrifugal pump, generally given for a unique nominal speed, are used to develop a model capable of predicting the pump flow rate, total head, and load torque at different rotational speeds. Furthermore, four machine-learning models - artificial neural networks (ANN), adaptive neuro-fuzzy inference system (ANFIS), support vector machine (SVM), and Gaussian process regression (GPR) - are evaluated for predicting the maximum power point voltage. Comparative analysis indicates that ANFIS achieves the highest prediction accuracy, followed by GPR, ANN, and SVM, outperforming conventional algorithms. Consequently, ANFIS combined with a PI regulator is adopted as the control strategy for tracking the maximum power point. Finally, the dynamic behavior of the photovoltaic water pumping system is examined under variations in irradiance and temperature. The results demonstrate that the proposed control strategy effectively maintains the PV voltage close to its optimal value and responds rapidly to changing climatic conditions, confirming its suitability for PV water pumping applications.

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1. Introduction

A photovoltaic water pumping system (PVWPS) is a self-sustained system powered by only solar energy. The lack of drinking and irrigation water was always a problem, especially in isolated locations. For this purpose, PVWPS offers an effective solution by supplying water whenever sufficient sunlight is available. Moreover, by incorporating storage tanks, this system can ensure a continuous water supply and meet demand even during nighttime or low-sunlight periods.

The effectiveness of the PV array in the PVWPS may be affected by changes in climate conditions and pumping load. These challenges can be addressed using maximum power point tracking (MPPT) techniques, which can extract the maximum power from the PV generator. Among these techniques, the conventional methods perturb and observe (P&O) and incremental conductance (INC) are the most widely used due to their simplicity.

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In recent years, artificial intelligence (AI)-based methods have been increasingly utilized across various research areas and industries to enhance services and address challenges that traditional methods cannot solve. These methods can be utilized to tackle the challenges of MPPT. Haddad et al. [1] presented several ANN-based methods for predicting water flow rates in a PVWPS. Their findings demonstrated accurate predictions based on measured temperature and irradiance. In [2], the authors present a Matlab-implemented SVM model for predicting photovoltaic energy production using irradiance and ambient temperature as inputs. Similarly, another method involving SVM is presented in [3]. A multi-input SVM model was evaluated using three input combinations: solar power alone, solar power with solar irradiance, and solar power with temperature and irradiance. In another work, the authors in [4] propose a SVM-based MPPT approach to estimate the maximum power point (MPP) of a PV panel using irradiance and temperature inputs. The estimated MPP voltage is then used to compute the duty cycle of the boost converter. Simulation results demonstrate effective MPP tracking with an efficiency exceeding 94%.

Chandra et al. [5] have introduced a novel photovoltaic power system topology incorporating an RBF neural network to enhance system efficiency. It is demonstrated that this MPPT method delivers greater efficiency than the conventional techniques. In [6], the authors present an ANN methodology for MPPT applied to a symmetrical multilevel boost converter in a solar-powered system. This innovative approach improves the output power under varying solar conditions. The results indicate enhanced efficiency by 99.99% and significant reductions in settling and rising times. The authors in [7] studied the impact of increasing the number of hidden layers in a neural network on predicting photovoltaic power. Several network structures were designed and trained using module temperature and solar radiation as inputs, and PV power as the output. Their results show that increasing the number of hidden layers significantly improves prediction accuracy. In [8], an ANFIS integrated with a proportional PI controller is presented for MPPT in a solar-powered brushless DC motor used for water pumping applications. Simulation studies conducted under Matlab confirm the controller's effectiveness, and experimental verification demonstrates its practicality. Sarkar et al. [9] introduced a novel hybrid model by integrating a Bat algorithm for training to enhance the ANFIS. The results demonstrate that this metaheuristic approach provides superior training efficiency in comparison with standard backpropagation. A hybrid method was proposed in [10] by employing the Crowded Plant Height Optimization algorithm for ANFIS training resulted in improved tracking speed and efficiency relative to the standard ANFIS model. Ahmed et al. [11] developed and implemented a soft-computing-based MPPT controller to predict the optimal converter duty cycle under varying climatic conditions and total static heads. A comparative study of the proposed soft-computing algorithms was conducted to evaluate and validate their accuracy. In [12], a hybrid FL-INC optimization algorithm is presented for PVWPS utilizing a SEPIC converter to enhance performance. This method combines fuzzy-logic with INC to maximize power extraction and improve PVWPS efficiency. In [13], the authors proposed a method utilizing a Genetic Algorithm (GA) to optimize gains for a backstepping controller (BSC) in MPPT for solar photovoltaic systems. This approach enhances tracking efficiency and stability, achieving MPP in 0.2 seconds, significantly outperforming conventional methods.

MPPT techniques based on ML techniques require significant real-time computation and therefore rely on powerful microcontrollers or DSPs. Integrating MPPT controllers with power system controllers (PSCs) requires careful coordination between power electronics, microcontrollers, and sensors to ensure stable and efficient operation under real-world conditions, such as changing irradiance and temperature [14]. Variations in solar irradiance caused by shading and clouds, along with the thermal degradation of PSCs, require fast and adaptive MPPT methods [15]. In practical conditions, sensors must remain accurate over a wide temperature range. In addition, real-world MPPT systems must be resistant to environmental factors such as dust and moisture.

This study investigates a PVWPS comprising a PV generator, DC/AC inverter, squirrel-cage induction motor, MPPT controller, centrifugal pump, and hydraulic circuit. Accurate modeling of such a system is essential for predicting behavior and optimizing control. A key aspect of this modeling involves the identification of critical components, such as the PV module. Where its parameters are identified using three methods: particle swarm optimization (PSO), Newton-

Raphson, and fsolve. In addition, the dynamic behavior of the entire PVWPS must be considered to develop an effective control strategy. To address these challenges, this study incorporates ML techniques such as ANN, ANFIS, SVM, and GPR for MPPT, with the goal of improving system adaptability under varying weather conditions. However, few studies combine parameter identification, ML-based MPPT, and dynamic modeling of the PVWPS within a unified framework. This work aims to fill that gap by providing a comprehensive evaluation to support the development of reliable and high-performance PVWPS. The remainder of the paper is organized into five sections. In section 2, the mathematical modeling of the various components of the PVWPS will be presented. In section 3, a comparative analysis between four ML models will be conducted. The simulation results are discussed in section 4. Furthermore, the parameters of the target PV module and centrifugal pump are presented. The last section summarizes the results of this work and concludes.

2. PVWPS Modeling

The investigated PVWPS is composed of different elements: PV generator, DC/AC inverter, squirrel-cage induction motor, MPPT controller, centrifugal pump, and hydraulic circuit including the pipes and storage tank.

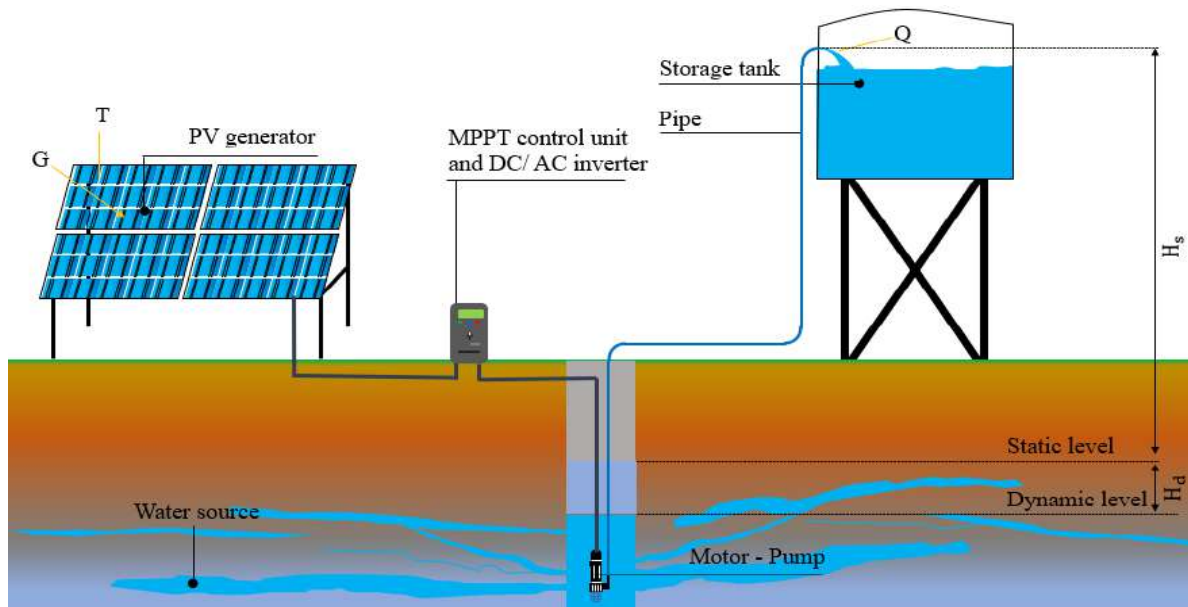


Fig. 1. Schematic of PV water pumping system

2.1. PV Generator

The PV generator is a group of modules connected in series/parallel to obtain the desired power. The basic element in a PV module is the solar cell; there are several mathematical models that describe its behavior [16]. This work employs the one-diode model, where the output current of a solar cell is described by:

$$I = \frac{G}{G_{ref}} \left(I_{ph_{ref}} + \mu_{sc} \cdot (T_c - T_{c_{ref}}) \right) - I_0 \cdot \left(e^{\frac{q \cdot (V + R_s \cdot I)}{A \cdot k \cdot T_c}} - 1 \right) - \left(\frac{V + R_s \cdot I}{R_{sh}} \right) \quad (1)$$

where, G : Solar irradiance intensity, G_{ref} : Solar irradiance intensity at STC, $I_{ph_{ref}}$: Photocurrent at STC, $T_{c_{ref}}$: Cell temperature at STC, T_c : Cell temperature, μ_{sc} : Coefficient temperature of short circuit current (A/K), I_0 : Reverse saturation current of the diode, q : Electron charge, R_{sh} : PV module's shunt resistance, R_s : PV module's series resistance, k : Boltzmann constant, A : Diode ideality factor.

While the single-diode model is simpler with five unknown parameters (R_s , R_{sh} , A , I_0 , I_{ph}), it may not capture the complexities of the PV cell's behavior as accurately as the double-diode model, particularly under diverse operating conditions; its simplicity, efficiency, and sufficient accuracy make it suitable for many processes optimization and system design applications.

Irradiance absorbed by the solar cell leads to a continuous increase in temperature and a growing temperature gradient between the solar cell and the environment. This temperature differential drives an escalation in the rate of heat dissipation by the solar cell. At some point, an equilibrium state is reached where the rate of heat loss matches the rate of solar energy absorption; thus, the following equation is obtained:

$$\varepsilon \cdot S \cdot G = h \cdot S \cdot (T_c - T_{amb}) \quad (2)$$

Solving the last equation, T_c is expressed by the following equation:

$$T_c = T_{amb} + \frac{\varepsilon}{h} \cdot G = T_{amb} + \alpha \cdot G \quad (3)$$

The parameter (α) can be obtained through interpolation of the measured data; however, it is commonly accepted to be equal to 0.03 [17]. This value corresponds to a temperature increase of 30 K for a flux of 1000 W/m².

The generator is composed of multiple PV modules interconnected in series and/or parallel configurations to obtain specific voltage and current requirements. The overall output current and voltage of the generator can be determined as follows:

$$\begin{cases} I_{gen} = N_p \cdot I \\ V_{gen} = N_s \cdot V \end{cases} \quad (4)$$

2.2. DC/AC Inverter

The main function of the DC/AC inverter is to transform the DC voltage produced by the solar generator into a three-phase voltage with variable frequency. The output voltages of a three-phase inverter can be computed by the following equation:

$$\begin{bmatrix} V_a \\ V_b \\ V_c \end{bmatrix} = \frac{V_{dc}}{3} \begin{bmatrix} 2 & -1 & -1 \\ -1 & 2 & -1 \\ -1 & -1 & 2 \end{bmatrix} \cdot \begin{bmatrix} S_1 \\ S_2 \\ S_3 \end{bmatrix} \quad (5)$$

With S_i is the state of the switch (IGBT), $S_i = 0$ if it is open and $S_i = 1$ if it is closed.

2.3. Squirrel-Cage Induction Motor

The voltage of the three-phase squirrel-cage induction motor can be computed in a d-q frame by the following equation [18]:

$$\begin{aligned} \begin{bmatrix} V_{ds} \\ V_{qs} \end{bmatrix} &= R_s \begin{bmatrix} I_{ds} \\ I_{qs} \end{bmatrix} + \frac{d}{dt} \begin{bmatrix} \varphi_{ds} \\ \varphi_{qs} \end{bmatrix} + \omega_s \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{bmatrix} \varphi_{ds} \\ \varphi_{qs} \end{bmatrix} \\ \begin{bmatrix} V_{dr} \\ V_{qr} \end{bmatrix} &= R_r \begin{bmatrix} I_{dr} \\ I_{qr} \end{bmatrix} + \frac{d}{dt} \begin{bmatrix} \varphi_{dr} \\ \varphi_{qr} \end{bmatrix} + \omega_r \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{bmatrix} \varphi_{dr} \\ \varphi_{qr} \end{bmatrix} \end{aligned} \quad (6)$$

For the d-axis, V_{ds} , I_{ds} , φ_{ds} , V_{dr} , I_{dr} and φ_{dr} referred to stator and rotor voltage, current, and flux linkage, respectively. For the q-axis, the last variables are referred V_{qs} , I_{qs} , φ_{qs} , V_{qr} , I_{qr} and φ_{qr} . R_s and R_r are stator and rotor winding resistance. ω_s and ω_r are stator and rotor frequency. For this study, V_{dr} and I_{dr} are set to zero. The currents of the asynchronous motor in the d-q frame are given as follows:

$$\begin{aligned} \begin{bmatrix} I_{ds} \\ I_{qs} \end{bmatrix} &= \frac{L_r}{L_s \cdot L_r - M_{sr}^2} \begin{bmatrix} \varphi_{ds} \\ \varphi_{qs} \end{bmatrix} - \frac{M_{sr}^2}{L_s \cdot L_r - M_{sr}^2} \begin{bmatrix} \varphi_{dr} \\ \varphi_{qr} \end{bmatrix} \\ \begin{bmatrix} I_{dr} \\ I_{qr} \end{bmatrix} &= \frac{L_r}{L_s \cdot L_r - M_{sr}^2} \begin{bmatrix} \varphi_{dr} \\ \varphi_{qr} \end{bmatrix} - \frac{M_{sr}^2}{L_s \cdot L_r - M_{sr}^2} \begin{bmatrix} \varphi_{ds} \\ \varphi_{qs} \end{bmatrix} \end{aligned} \quad (7)$$

where, L_s : Stator inductance, L_r : Rotor inductance, M_{sr} : Mutual inductance. The electromagnetic torque of the motor can be computed as follows [19]:

$$T_{em} = 1.5 \cdot p \cdot (\varphi_{ds} \cdot I_{qs} - \varphi_{qs} \cdot I_{ds}) \quad (8)$$

where, p : Pair pole number. The motor speed is calculated using the mechanical dynamic equation:

$$T_{em} = \frac{J}{p} \frac{d\omega_r}{dt} + f\omega_r + T_r \quad (9)$$

where, T_r : Load torque, f : Viscous torque constant, J : Inertia moment of the rotor.

2.4 Pump and Hydraulic Circuit

The hydraulic power needed to pump water at a volumetric rate (Q) and total head (H) is given by the following equation:

$$P_h = g \cdot \rho \cdot H \cdot Q \quad (10)$$

The pump efficiency (η) is given by the following equation:

$$\eta = \frac{P_h}{\omega_r \cdot T_r} \quad (11)$$

The load torque can be expressed as follows:

$$T_r = \frac{g \cdot \rho \cdot H \cdot Q}{\omega_r \cdot \eta} \quad (12)$$

In general, pumps are described by their $H(Q)$ characteristic that defines the total head as a function of flow rate for a single nominal speed (N_n); the equation that links the total head with the flow rate for any rotational speed (N) is given as follows:

$$H(Q, N^*) = h_0 \cdot (N^*)^2 - h_1 \cdot N^* \cdot Q - h_2 \cdot Q^2 \quad (13)$$

where, $N^* = \frac{N}{N_n}$: Normalized speed. The load torque caused by flow through the hydraulic part of the pump takes a similar form to the total head equation, and can be written as:

$$T_r(Q, N^*) = c_0 \cdot (N^*)^2 - c_1 \cdot N^* \cdot Q - c_2 \cdot Q^2 \quad (14)$$

The coefficients h_0 , h_1 , h_2 , c_0 , c_1 and c_2 are calculated from the data provided by the manufacturer.

The mathematical formulation governing the hydraulic circuit is provided below. The pump must work against the total head, which includes static head H_s , dynamic head H_d , and total head loss in the pipe. The total head is done by the following equation:

$$HMT = H_s + H_d + \left(\lambda \cdot \frac{L}{d} + K \right) \frac{8Q^2}{g\pi^2 d^4} \quad (15)$$

where, λ : Friction factor, K : Total fitting loss coefficient.

The head loss in pipe due to frictional effects can be determined from the Darcy-Weisbach equation. For laminar flow, λ is equal to $64/Re$. For turbulent flow of incompressible fluids, this factor can be determined from the Moody diagram. To avoid the reading error, λ is determined from the Colebrook equation.

2.5 Maximum Power Point Tracking (MPPT)

To enhance the energy efficiency of PV systems, it is essential to design an effective MPPT technique. Traditionally, the MPP has been tracked using conventional algorithms such as P&O and INC. The P&O method is widely used due to its simplicity; it works by slightly perturbing the operating voltage and observing changes in output power to approach the MPP. The INC algorithm is based on the slope of the power-voltage (P-V) curve, which equals zero at the MPP. While these classical techniques are effective under steady conditions, their performance often degrades under rapidly changing environmental conditions such as partial shading or sudden irradiance variations. To address these limitations, recent research has turned to Machine Learning (ML) techniques. By integrating AI into MPPT strategies, PV systems can achieve faster convergence, improved accuracy, and better adaptability, making these techniques a promising direction for future advancements in solar energy technologies. For this purpose, a comparative analysis of ML techniques will be conducted in this study.

3. Comparative Analysis of ML Techniques

This section presents a comparative analysis of ML techniques applied to predict the optimal voltage used as a reference signal of the PI controller. The observed data are derived through a numerical compilation based on measurements provided by the manufacturer. The mathematical formulations of ML techniques are reported in Appendix A.

The dataset is segregated into two subsets: training and testing; 70% of the dataset is used for the training, while the remainder is utilized for the testing phase. In addition, the following considerations are adopted:

- ANN structure: A feedforward network is considered with three layers: an input layer, a hidden layer, and an output layer. The inputs, for each ML model, are the irradiance and temperature; the output is the MPP voltage. The ANN structure is 2-x-1. The number of hidden neurons (x) is taken equal to "3"; this value is selected via different simulations to maximize R and to minimize MSE. For activation or transfer functions, layers 2 and 3 are concerned. A linear activation function for the output (layer 3) and a hyperbolic tangent sigmoid one for the single hidden layer (layer 2) are selected. In addition, Bayesian Regularization is opted as a training algorithm.
- Regarding ANFIS, a Sugeno FIS is adopted with a "4 3" structure by using the "Neuro-Fuzzy Designer" application under MATLAB. A Gaussian membership function is taken for the inputs, and a constant one for the output. In addition, a grid partition and hybrid learning method are considered during simulation limited to a maximum of 30 epochs. Notice that different types of membership functions (triangular ...) are tested, and we selected the one that offers the best predictions (Gaussian).
- The SVM model is trained using a quadratic kernel function. MATLAB provides six kernel functions (linear, quadratic, cubic, fine-Gaussian, medium-Gaussian, and coarse-Gaussian). After several simulations, the quadratic kernel yielded the best performance, with default following hyper-parameters: Box-constraint "auto mode", Epsilon "auto mode", Kernel-scale "auto mode", Standardize "yes".
- The selected GPR technique employed a Matern 5/2 kernel function. This last is selected from a range of tested kernels, including Rational Quadratic, Squared Exponential, and Exponential. Default settings are adopted under MATLAB for the hyper-parameters.

The training of these techniques is performed offline based on weather conditions. During simulation, the ML-MPPT receives the input information and provides predictions instantaneously, which is analogous to a testing phase. The simulation outcomes obtained during the training and testing phases using ANN, ANFIS, SVM, and GPR are presented in Table 1.

Table 1. Comparison between the observed and predicted values of V_{MPP}

Data	Inputs		Observed V_{MPP} (V)	Predicted V_{MPP} (V)			
	T (°C)	G (kW/m ²)		ANN	ANFIS	SVM	GPR
Training	20	1	382.34	381.69	382.06	385.98	381.99
	20	0.6	414.54	416.28	414.53	413.91	415.67
	20	0.4	428.40	427.52	429.03	424.74	427.95
	25	1	368.48	369.37	369.27	371.89	369.11
	25	0.8	386.96	386.29	386.47	386.77	386.58
	25	0.4	414.54	413.68	414.05	410.27	413.52
	25	0.2	419.16	418.20	418.63	418.89	418.41
	30	0.8	373.10	371.92	372.47	372.63	372.41
	30	0.6	386.96	387.89	388.06	385.30	387.57
	30	0.2	400.68	402.96	400.93	404.36	402.40
	35	1	340.90	341.48	340.86	343.94	341.04
	35	0.6	373.10	372.31	372.04	371.11	372.32
	35	0.4	382.34	382.93	382.77	381.55	382.82
	40	1	327.04	326.86	326.66	330.08	326.49
	40	0.8	340.90	341.42	341.82	344.58	341.93
	40	0.4	368.48	367.49	368.22	367.31	367.99
	40	0.2	373.10	372.25	373.14	375.54	372.70
	50	0.8	313.18	312.66	313.15	316.84	313.14
	50	0.6	327.04	328.21	327.21	328.99	327.79
	50	0.4	340.90	340.02	340.39	339.06	339.80
	50	0.2	345.38	346.12	345.73	347.03	345.88
Testing	20	0.8	400.68	399.36	398.84	400.99	399.65
	20	0.2	433.02	432.34	434.88	433.49	433.82
	25	0.6	400.68	402.74	402.15	399.57	402.16
	30	1	354.62	355.82	354.98	357.88	355.39
	30	0.4	396.06	398.59	397.53	395.87	398.16
	35	0.8	354.62	356.75	357.22	358.57	357.33
	35	0.2	386.96	387.37	386.85	389.91	387.05
	40	0.6	354.62	356.73	355.90	356.99	356.94
	50	1	299.32	298.88	298.14	302.60	299.13

To evaluate the performance of predictive models, RMSE, R-squared, and MAPE between observed and predicted values are used; their expressions are given as follows:

$$RMSE = \sqrt{\frac{\sum_{k=1}^n (V_{MPP\,obs}(k) - V_{MPP\,pred}(k))^2}{n}} \quad (16)$$

$$R^2 = 1 - \frac{\sum_{k=1}^n (V_{MPP\,obs}(k) - V_{MPP\,pred}(k))^2}{\sum_{k=1}^n (V_{MPP\,obs}(k) - \bar{V}_{MPP\,obs}(k))^2} \quad (17)$$

$$MAPE\ (%) = 100 \times \frac{1}{n} \sum_{k=1}^n \frac{|V_{MPP\,obs}(k) - V_{MPP\,pred}(k)|}{V_{MPP\,obs}(k)} \quad (18)$$

Table 2 presents a comparative analysis of the performance indicators for the ML models. The results reveal that all models exhibit strong performance, characterized by high R-squared (0.9935–0.9997), low RMSE (0.5472–2.5902), and low MAPE (0.1194 %–0.6143 %). Notably, the ANFIS model demonstrates high accuracy during both training and testing, followed by GPR and ANN. The SVM model, although exhibiting reasonable performance during both training and testing. Considering the results across all data points, the ANFIS demonstrates superior predictive accuracy with the lowest overall RMSE and MAPE, and a higher R-squared.

Table 2. Performance indicators for the ML models

		ANN	ANFIS	SVM	GPR
Training data	RMSE	0.9971	0.5472	2.5902	0.7613
	R-Squared	0.9990	0.9997	0.9935	0.9994
	MAPE (%)	0.2386	0.1194	0.6143	0.1775
Testing data	RMSE	1.6206	1.5305	2.4242	1.5504
	R-Squared	0.9981	0.9983	0.9957	0.9982
	MAPE (%)	0.3808	0.3606	0.5633	0.3402
All data	RMSE	1.2181	0.9552	2.5415	1.0615
	R-Squared	0.9987	0.9992	0.9943	0.9990
	MAPE (%)	0.2813	0.1918	0.5990	0.2263

Under MATLAB environment, three validation schemes are available for the Regression Learner application. First, the ML models were compared using the resubstitution validation by considering 70%–30% training–testing split. About the overfitting risks, k-fold cross-validation ($k = 3$) and holdout validation were employed as additional assessments for the SVM and GPR models. Despite the difference in performance during the learning phase, the results obtained during the testing phase indicate consistent performance for both algorithms. The extended validation results for the SVM and GPR models are summarized in Table 3.

Table 3. Performance of SVM and GPR under different validation schemes

Phase	Validation scheme	SVM		GPR	
		RMSE	R-Squared	RMSE	R-Squared
Training	Resubstitution	2.5902	0.9935	0.7613	0.9994
	3-fold cross	5.1716	0.9754	3.563	0.983
	Holdout	3.9106	0.9907	3.3287	0.9933
Testing	Resubstitution	2.4204	0.9957	1.5504	0.9962
	3-fold cross	2.5636	0.9952	1.5504	0.9982
	Holdout	2.93	0.9897	1.2653	0.9981

Figs. 2-3 show the comparison between observed and predicted results for the training and testing phases. A comparison of the performances between the predictive models based on ML techniques and those using conventional algorithms is depicted in Fig. 4. Step changes in irradiance level were applied at 2 s and 3 s. Irradiance level is changed from 0.5 kW/m² to 0.640 kW/m², then from 0.640 kW/m² to 0.47 kW/m².

Fig. 4 indicates that the ML techniques exhibit superior prediction performance compared to other conventional methods, as evidenced by a faster rise time of 0.033 s in contrast to the 0.2 and 0.4 s rise times observed in INC and P&O methods, respectively. In addition, to simulate the PVWPS under realistic operating conditions and to assess the robustness of the control strategy, noise measurement is added to the input signals, with variances of 50 W/m² for irradiance and 0.04 °C for temperature. Based on the outcomes of this comparative analysis, ANFIS with a PI regulator is employed for performance enhancement of the system control strategy due to its higher accuracy compared to the other three ML based MPPT, as well as conventional P&O and INC methods.

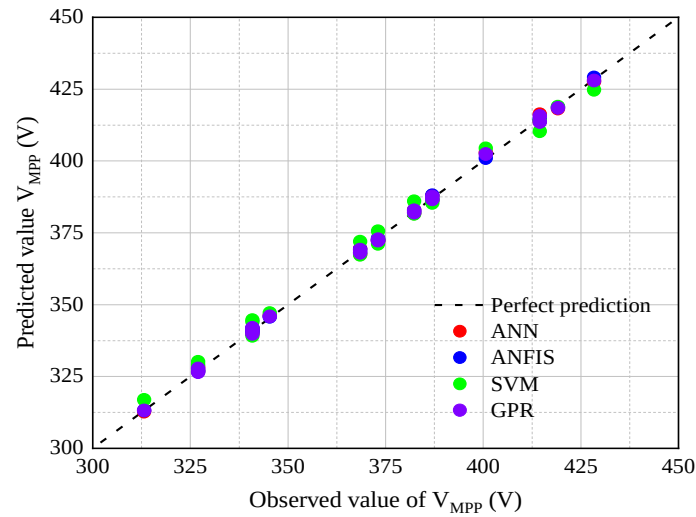


Fig. 1. Comparison between observed and predicted results for the training phase

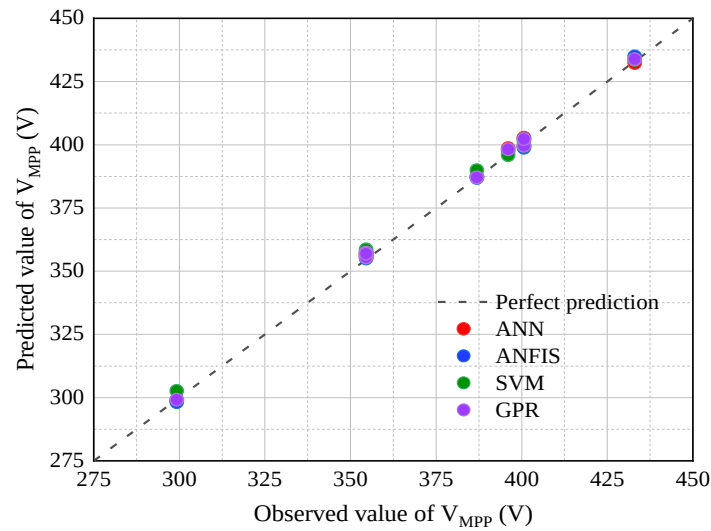


Fig. 2. Comparison between observed and predicted results for the testing phase

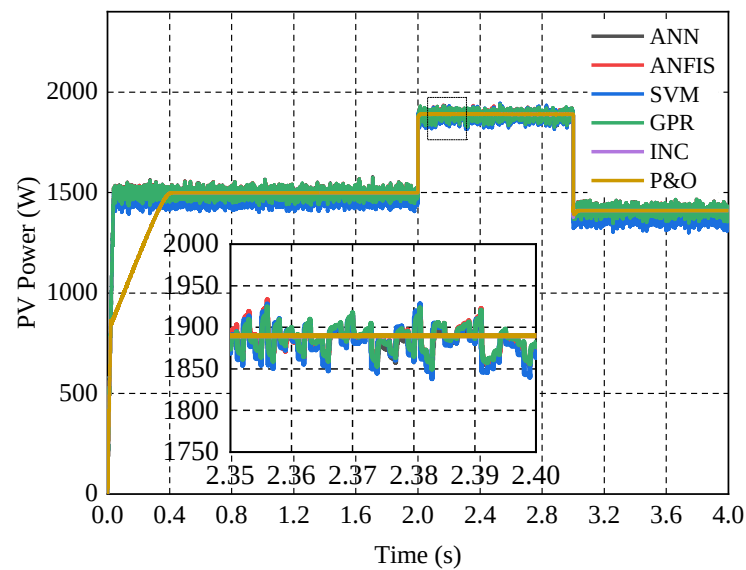


Fig. 3. PV power for a change in Irradiance levels

4. Results and Discussion

In this study, each component of the PVWPS has been programmed under the Matlab/Simulink environment. The system parameters used in the simulation are given in Appendix B. The pumping system has variable inputs, which are the weather conditions (solar irradiance and temperature), and has the flow rate as an output. The main objective is to find an operating point to manage the interactions between the solar irradiance, temperature, and flow rate. As can be seen from Fig. 5, the entire system model consists mainly of four interactive parts. The first part makes the connection between the voltage at the PV generator terminals and the current required by the inverter. The second part presents the link between the rotational speed of the squirrel cage motor and the load torque exerted by the pump on its shaft. The third part made the connection between the supply voltage at the inverter terminals and the current absorbed by the motor. The fourth connection is between the flow rate and the total head.

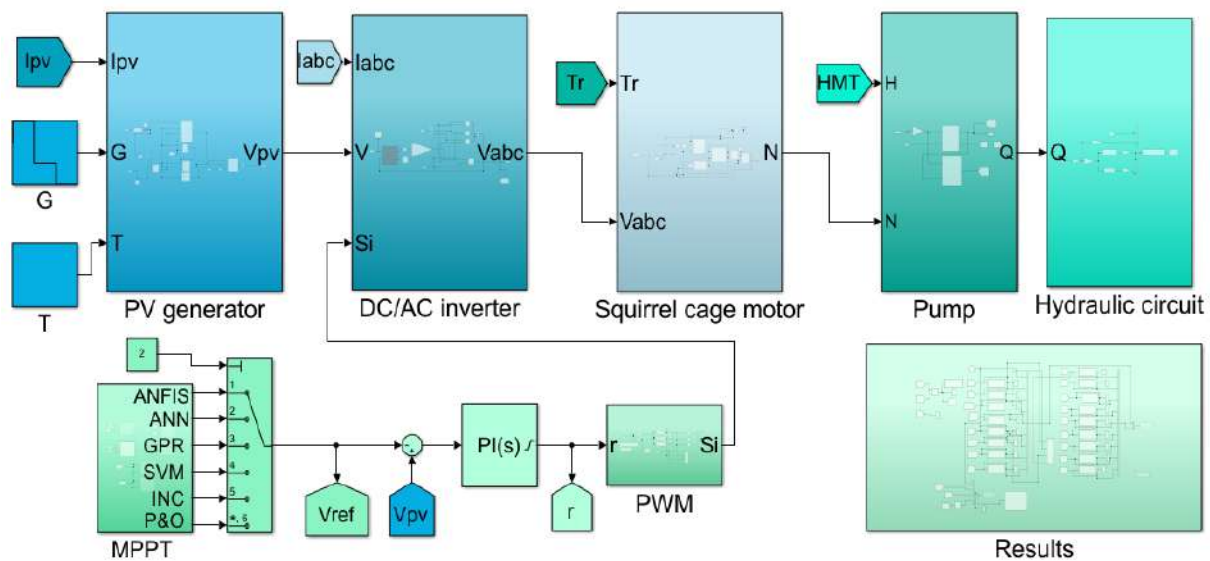


Fig. 4. Dynamic modeling of the PVWPS under Matlab/Simulink

The MPPT control strategy consists of two elements: the first one is the MPPT based on ANFIS, and the second element is the PI controller. The ANFIS uses actual irradiance and temperature to predict the optimal voltage used as a reference signal for the controller PI. The last adjusts the ratio ($r = V/f$) of the PWM. The parameters obtained for the PI controller are: proportional and integral gains ($k_p = 0.25$ and $k_i = 0.542$).

Compared to the previous study conducted in reference [18], the present work employs the KC200GT PV module. To this end, the parameters of the PV module were identified using three methods: Newton-Raphson, fsolve, and PSO. Table 4 summarizes the estimated parameters obtained using these different methods.

Table 4. Identified parameters for the KC200GT PV module

Parameter	Analytical method [20]	Lambert W function [20]	Ref. [21]	Newton-Raphson	fsolve	PSO
$R_s (\Omega)$	0.308	0.284057	0.23	0.3144	0.31074	0.2349
$R_{sh} (\Omega)$	193.049394	157.85322	113.997	186.7473	192.3	167.4838
A	1.075812	1.075816	1.06	1.06	1.07	1.1348
$I_s (\mu A)$	2.15236	2.19541	1.75	1.5440	1.9042	5.8094
$I_{ph} (A)$	8.2233	8.21	8.193	8.2238	8.22327	8.18281

Regarding the accuracy of the results (see Table 5), the PSO method provides excellent agreement with the manufacturer's data with a lower RMSE of 0.0358 and a higher R^2 of 0.9901. In contrast, the Newton-Raphson and fsolve methods exhibit relatively higher RMSE values of 0.1195 and 0.1190, respectively; this demonstrates lower accuracy in the region of the MPP (see Fig. 6).

Additionally, comparisons with results reported in references [20] and [21] reveal notable deviations in the open-circuit voltage region and to the right of the MPP. The simulated I-V curves of the KC200GT PV module using parameters estimated by the three methods at $T = 25^\circ\text{C}$ and $G = 1000\text{ W/m}^2$ are presented in Fig. 6.

Table 5. Performance indicators for the identification methods

	Analytical method [20]	Lambert W function [20]	Ref. [21]	Newton-Raphson	fsolve	PSO
RMSE	0.1189	0.1002	0.0858	0.1195	0.1190	0.0358
R-squared	0.8908	0.9224	0.9431	0.8896	0.8905	0.9901

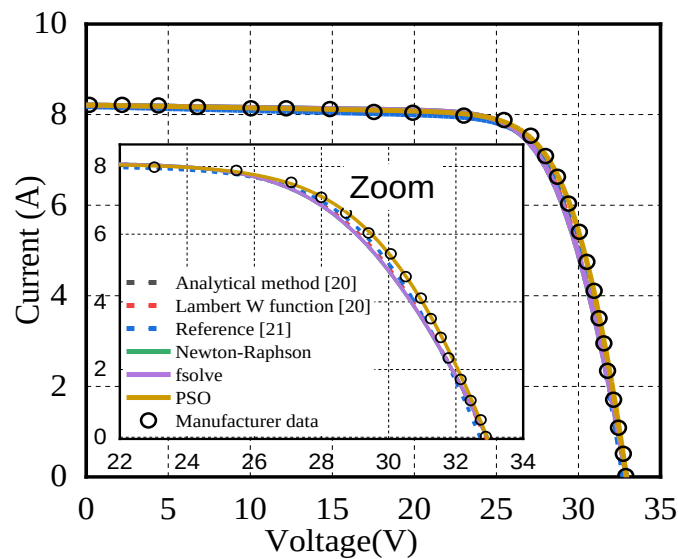


Fig. 5. I-V characteristic for the KC200GT PV module

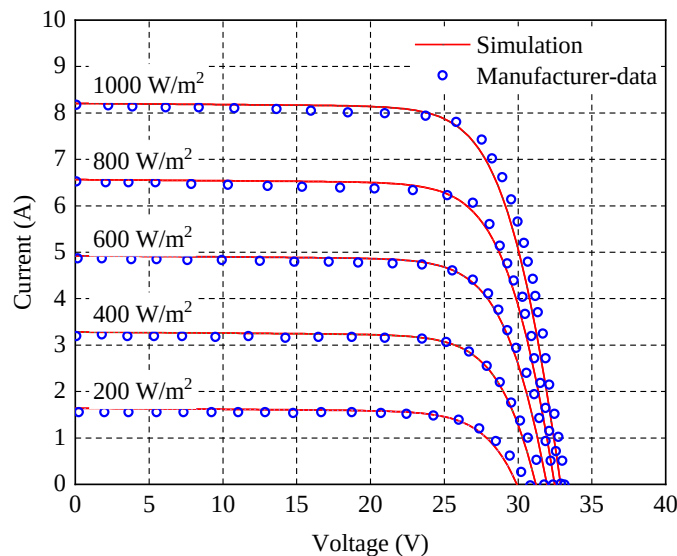


Fig. 6. I-V characteristic under different irradiances and constant temperature $T = 25^\circ\text{C}$

The I-V characteristics of the target PV module, provided by the manufacturer and simulation, are depicted in Fig. 7 with irradiance levels ranging from 200 W/m^2 to 1000 W/m^2 and constant temperature maintained at 25°C . It can be seen that the modeling by the one-diode model of the solar cell gives a good agreement between the provided manufacturer data and simulated results.

Table 6. Coefficients for computing (H) and (T_r) for different rotational speeds

Coefficient	h_0	h_1	h_2	c_0	c_1	c_2
Value	74.9	3.559	-5.594	1.957	3.476	-0.6107

The second element of the PV pumping system, which has changed according to the previous work in [17] is the centrifugal pump. Notice that the pump manufacturer gives the $H(Q)$ and $\eta(Q)$ characteristics for a single nominal speed. For this last, it is possible to calculate the load torque by utilizing eq (12). Table C of Appendix C illustrates the manufacturer's data of the pump for a nominal rotational speed $N_n = 2900$ rpm; eq (13) and eq (14) are used to obtain (H) and (T_r) for different speeds. The characteristics $H(Q, N)$ and $T_r(Q, N)$ of the selected pump (Bombas Ideal VIP 9/6) are illustrated in Figs. 7–8, respectively. The coefficients reported in Table 6 are obtained with polynomial interpolation by using data reported in Appendix C.

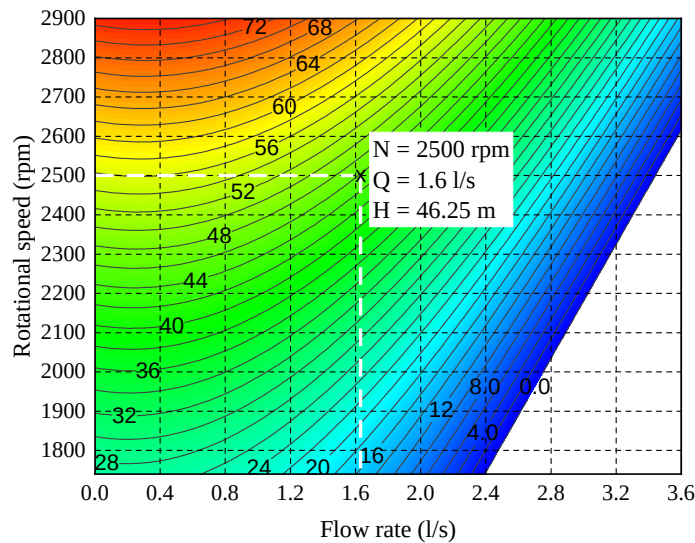


Fig. 8. Total head curves

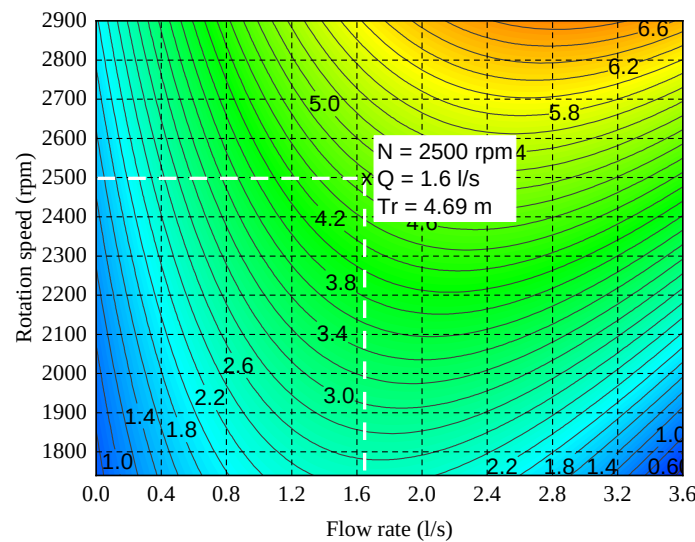


Fig. 9. Load torque curves

Fig. 8 and Fig. 9 show, for $N = 2500$ rpm and $Q = 1.6$ l/s, the obtained values of the total head $H = 46.25$ m and load torque $T_r = 4.69$ N·m, respectively. In the following, the simulation results of PVWPS for variations in irradiance and temperature will be presented.

4.1 Effect of Irradiance

The simulation was performed over a duration of 4 s. To evaluate the dynamic response of the MPPT, a step change in solar irradiance from 700 W/m^2 to 500 W/m^2 was applied at $t = 2 \text{ s}$, while the operating temperature was maintained at 25°C . The simulations depicted in Fig. 10 to Fig. 19 represent, respectively, the variations over time of voltage and current at PV generator terminals, PV power, line voltages and line currents at the motor terminals, rotational speed, electromagnetic and load torques, total head, water flow rate, and total head losses. Fig. 10 depicts the profile of the voltage at the PV generator terminals and the reference voltage predicted by ANFIS over time. It can be observed that the proposed control strategy has operated the photovoltaic generator near its optimal voltage, where it effectively adjusts to the irradiance change for a relatively short duration of 0.04 s . It is also noticeable a slight decrease in the optimal voltage during sunlight variations, attributed to the negligible impact of irradiance levels on V_{oc} voltage. Consequently, the effect of irradiance change on V_{MPP} is deemed negligible as well. On the contrary, in Fig. 11, a significant impact on the current of the PV generator can be observed, with a decrease from 5 to 3.7 A ; this reduction is accompanied by a decrease in generated power from 2048 W to 1498 W (Fig. 12); this is attributed to the notable effect of sunlight levels on the I_{sc} current. According to Fig. 13, the transient regimes of the motor are relatively insignificant, where the current demand during the transient regime is approximately 1.5 times higher than that in the nominal regime.

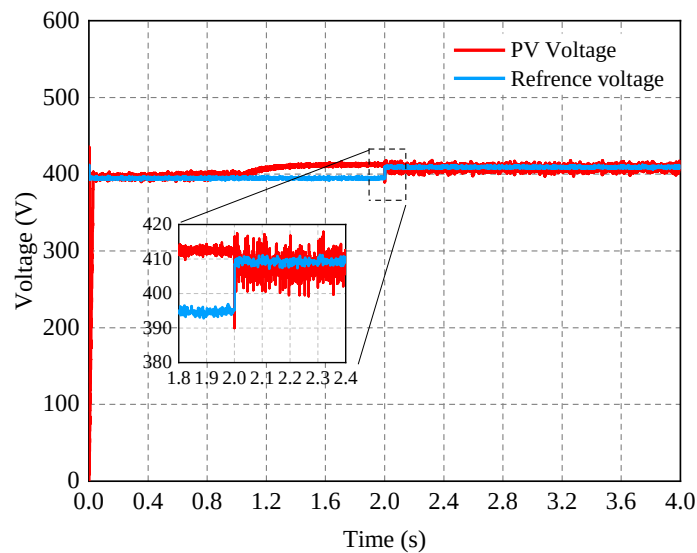


Fig. 10. PV voltage response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

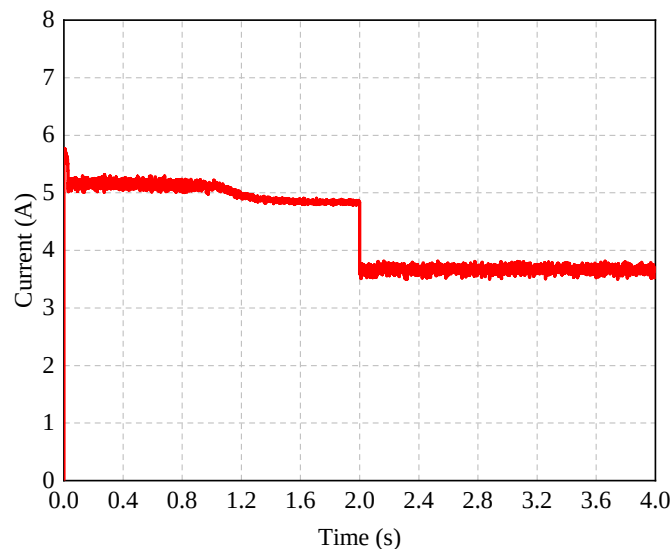


Fig. 11. PV current response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

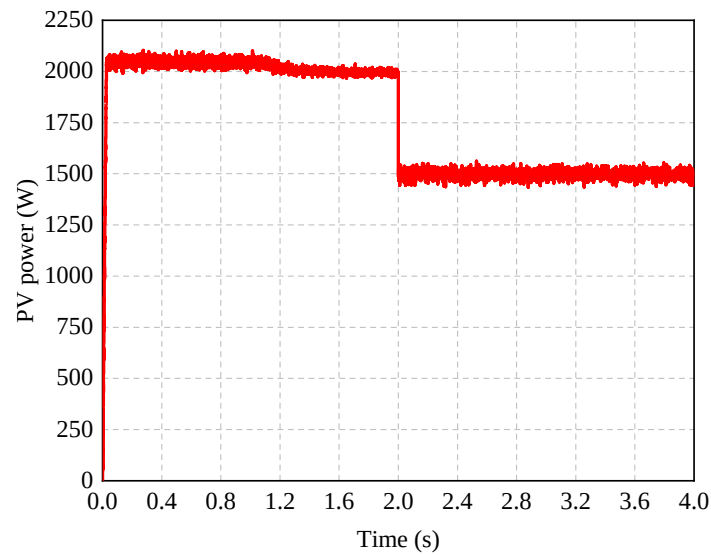


Fig. 12. PV power response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

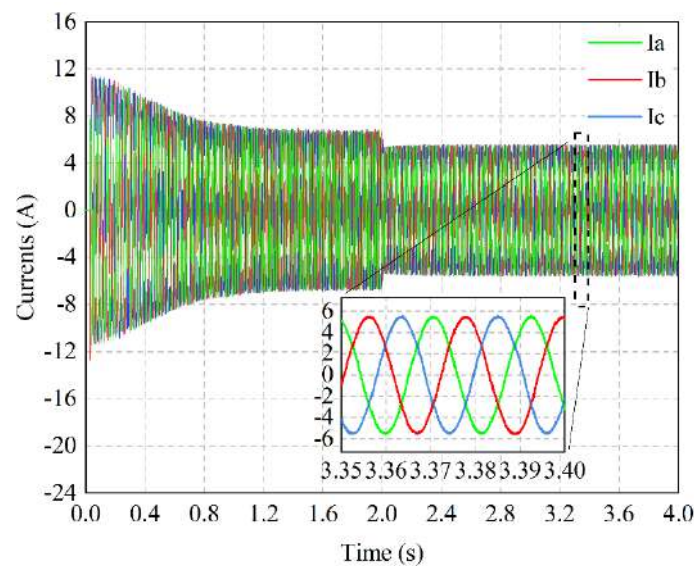


Fig. 13. Line current response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

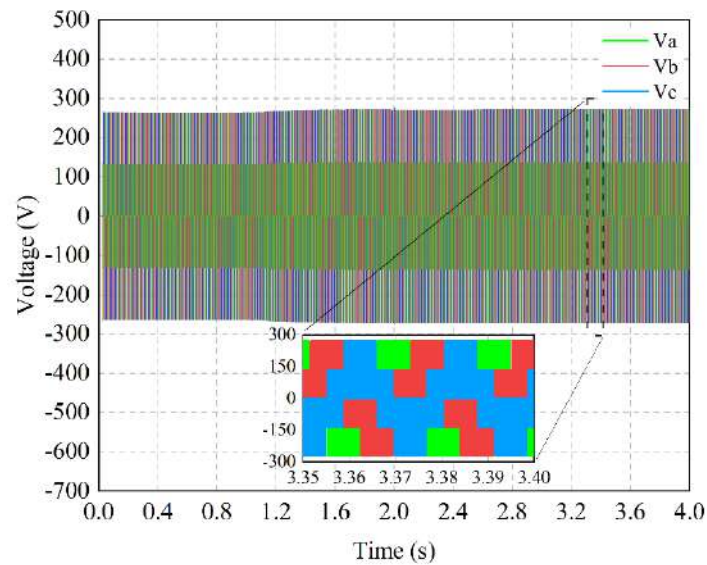


Fig. 14. Line voltages response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

The proposed control strategy enables a gradual start-up of the load to ensure a current demand lower than that of the short circuit of the PV generator. According to Fig. 14, the motor is supplied with voltages closer to the nominal values. Additionally, it can be observed that the voltage has increased by a small value of 13 V during the variation in irradiance. As shown in Fig. 16, the motor took 1.2 s for its rotational speed to reach the nominal speed. This period also corresponds to the transient oscillation phase of the electromagnetic torque, as depicted in Fig. 15. In addition, the decrease in electromagnetic torque and the rotational speed indicate that the system has shifted to another operating point to adapt the load to the available power. The variance between the electromagnetic torque and the resistive torque is due to the influence of viscous friction occurring within the motor.

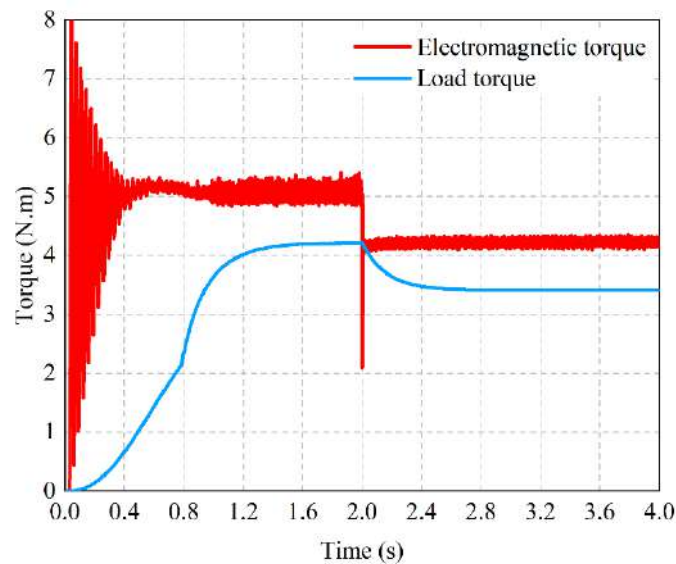


Fig. 15. Electromagnetic and load torque responses to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

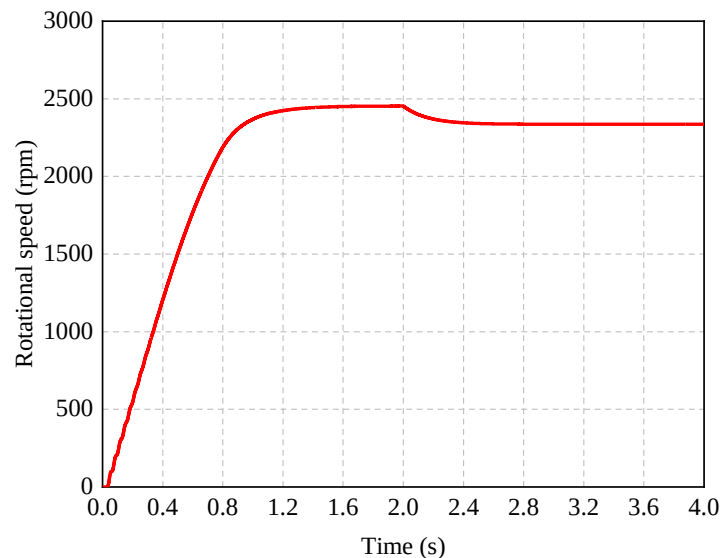


Fig. 16. Motor rotational speed response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

As shown in Fig. 18, the pump only starts pumping water at a speed of 2154 rpm. It is also observed that there is a 0.8 s delay before the commencement of water flow from the pump. This period corresponds to the duration required for the pipeline filling before the circuit begins discharging water externally. Additionally, this period also corresponds to the time required for the total head to exceed the static head and dynamic head, see Fig. 17. It is noteworthy that in reality, the pump must overcome both the head losses, the dynamic head, and the static head of the hydraulic circuit to pump water. As can be seen from Fig. 19, a significant decrease in flow rate is observed during the decrease of irradiance, where it decreased from 0.775 l/s to 0.49 l/s. This is attributed to the

decrease in generated power. The head losses are dependent on the flow rate, and their magnitude decreases as the flow rate decreases (Fig. 19).

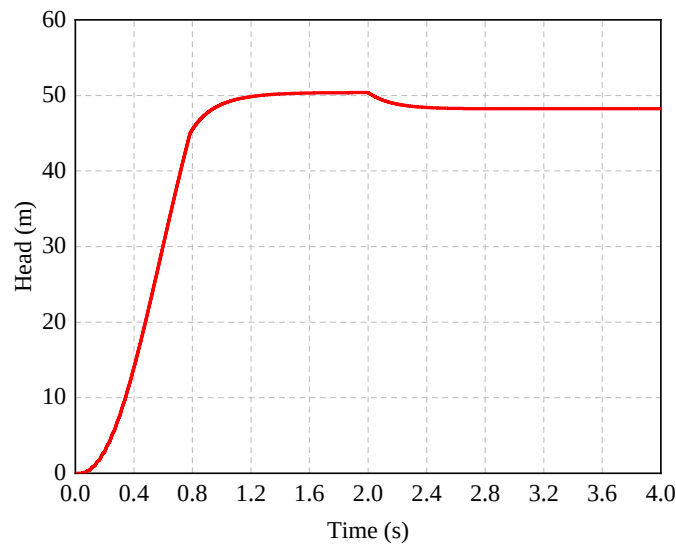


Fig. 17. Total pump head response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

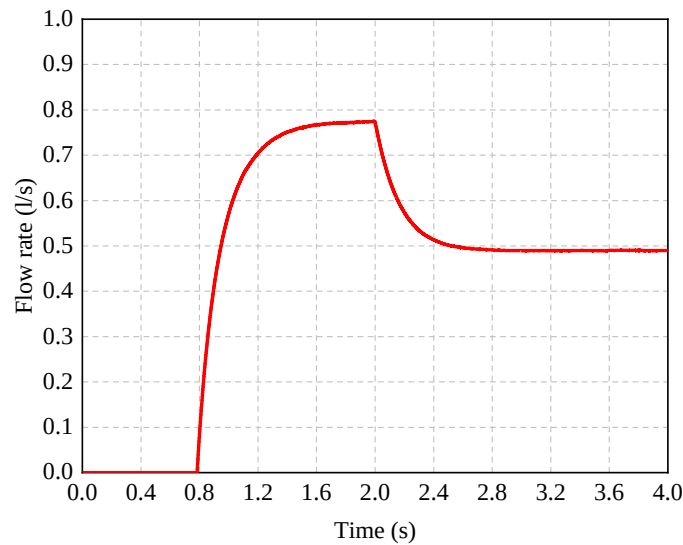


Fig. 18. Water flow rate response to an irradiance drop ($G: 700 \rightarrow 500 \text{ W/m}^2$)

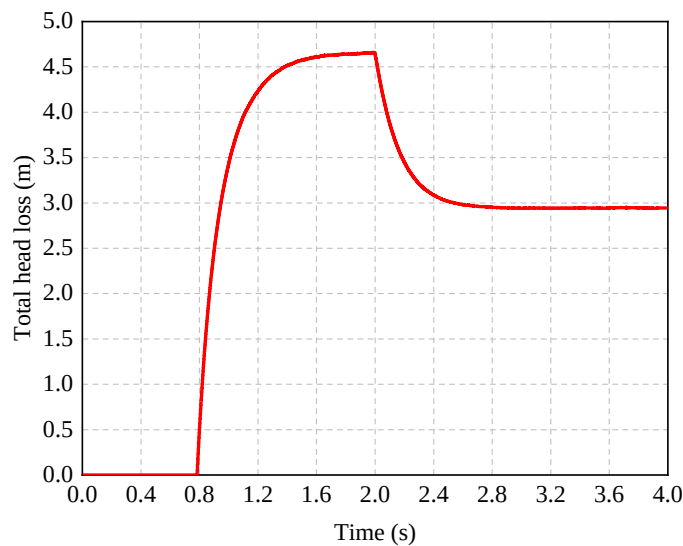


Fig. 19. Total head losses response to an irradiance step ($G: 700 \rightarrow 500 \text{ W/m}^2$)

4.1 Effect of Temperature

Subsequently, a step change in temperature from 25°C to 35°C was applied at $t=2.0$ s, while keeping irradiance constant at 600 W/m². The simulation results are depicted in Fig. 20 to Fig. 28, which represent, respectively, the variations over time of voltage and current at PV generator terminals, PV power, line voltages and line currents at the motor terminals, rotational speed, electromagnetic and load torques, total head, and water flow rate. From Fig. 20, a significant impact of temperature change on the PV voltage is observable, as it decreased from 405 V to 373 V; this is attributed to the notable influence of temperature levels on the V_{oc} voltage. As depicted in Fig. 21, the temperature variations have a negligible impact on the PV current; this is attributed to the insignificant effect of temperature levels on the I_{sc} current. According to Fig. 22, the increase in temperature leads to a notable reduction in the electrical power of the PV generator.

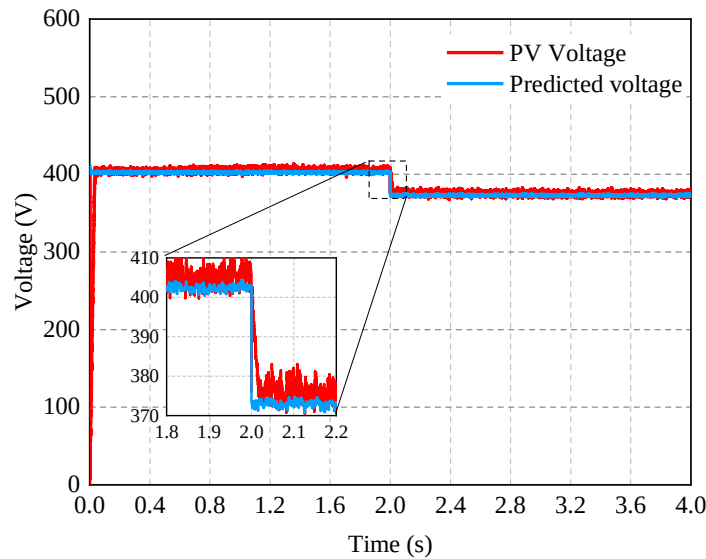


Fig. 20. PV voltage response to an increase in temperature (T: 25 → 35 °C)

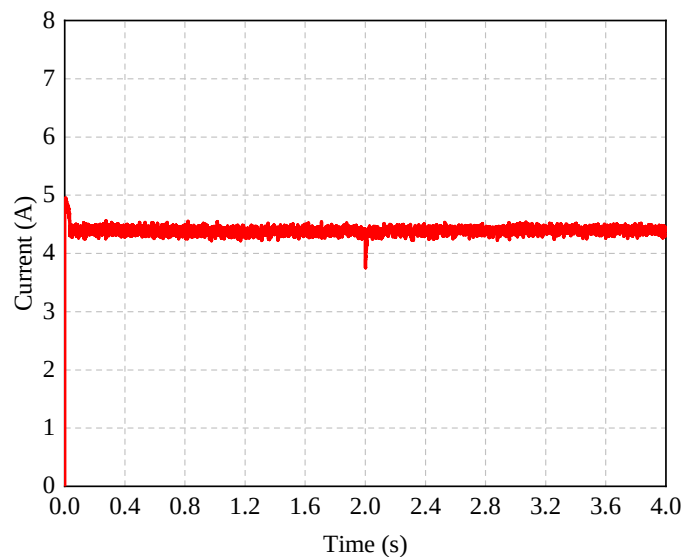


Fig. 21. PV current response to an increase in temperature (T: 25 → 35 °C)

From Fig. 23, it can be observed that the current demand did not change during the temperature variation. Conversely, the voltage at the motor terminals underwent a significant decrease, reaching 25 V during the temperature change (see Fig. 24). It is observed that upon reaching the steady state, the electromagnetic torque exhibits a fluctuation of approximately 0.25 Nm. This variation is attributed to the switching at the inverter and the requirements of the PMW regarding switching times. Based on Fig. 28, it is evident that the water flow rate increases to approximately

0.62 l/s when the system operates at 25 °C. Conversely, at 45 °C, there is a notable decrease in the flow rate.

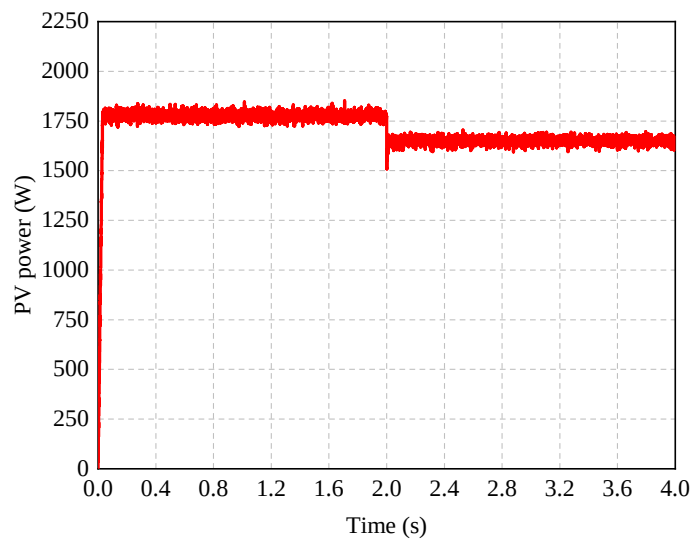


Fig. 22. PV power response to an increase in temperature (T: 25 → 35 °C)

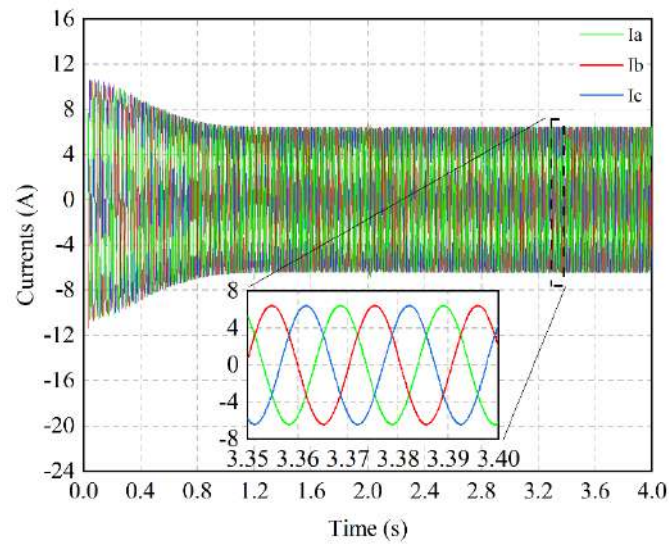


Fig. 23. Line current response to an increase in temperature (T: 25 → 35 °C)

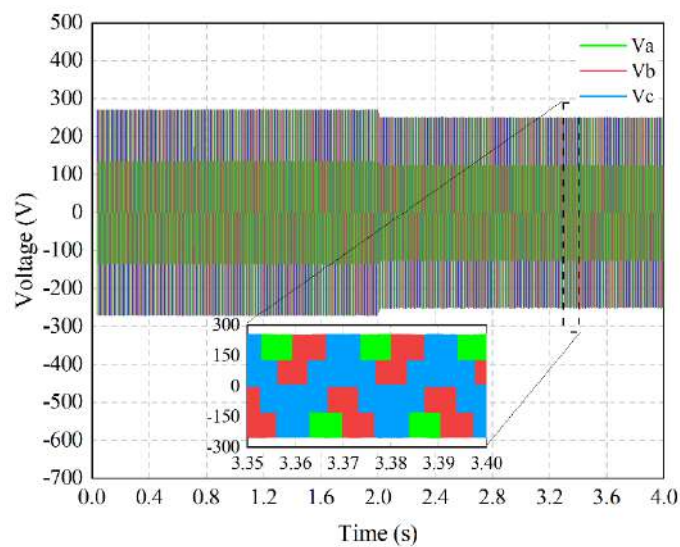


Fig. 24. Line voltages response to an increase in temperature (T: 25 → 35 °C)

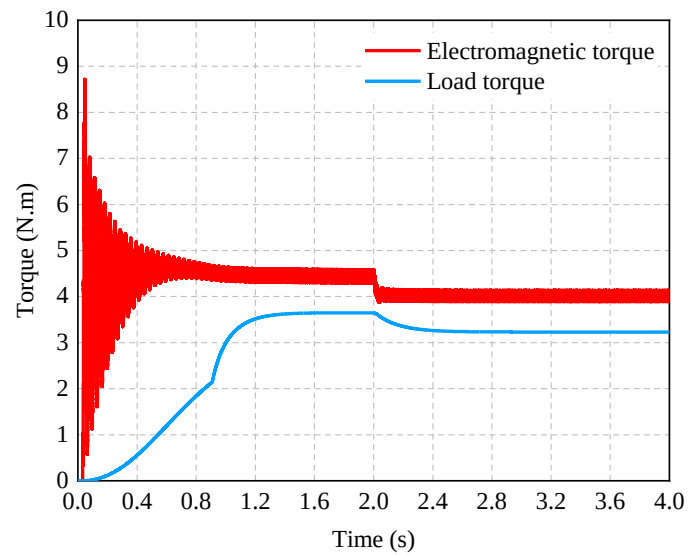


Fig. 25. Electromagnetic and load torque responses to an increase in temperature (T: 25 $\rightarrow$ 35°C)

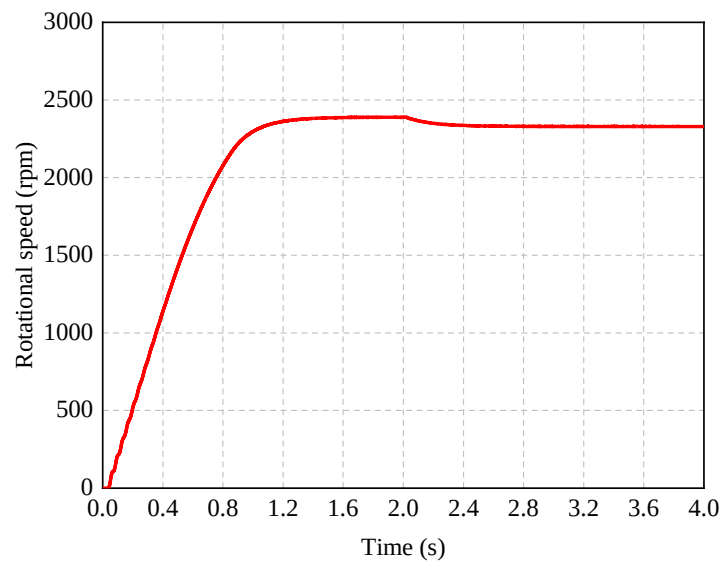


Fig. 26. Motor rotational speed response to an increase in temperature (T: 25 $\rightarrow$ 35 °C)

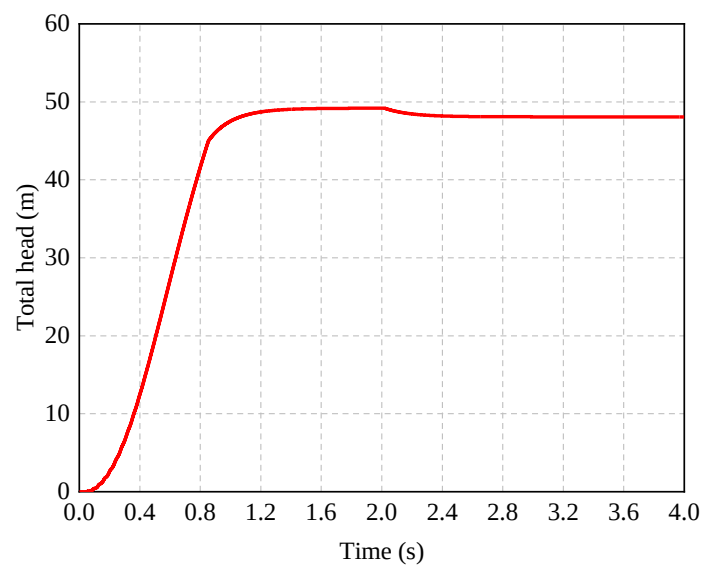


Fig. 27. Total pump head response to an increase in temperature (T: 25 $\rightarrow$ 35 °C)

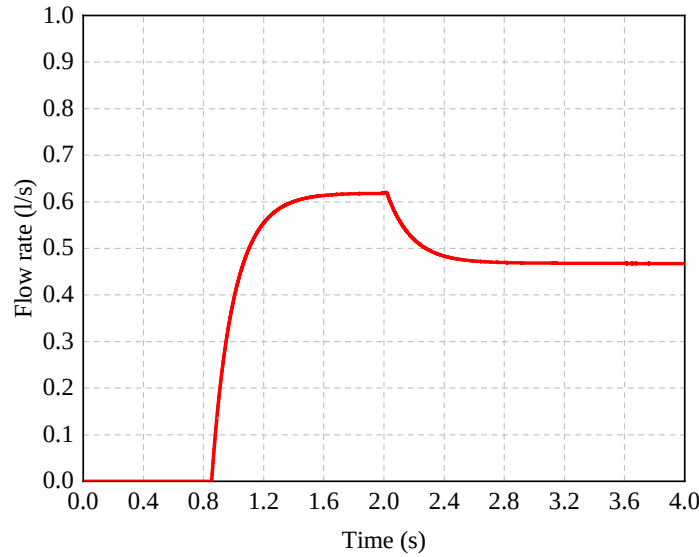


Fig. 28. Water flow rate response to an an increase in temperature (T: 25 → 35 °C)

5. Conclusions

This paper presents a dynamic simulation for a PVWPS to analyze its performance under varying irradiance and ambient temperature. The major conclusions are as follows:

- The parameters of the target PV module KC200GT were identified using three methods: Newton–Raphson, fsolve, and particle swarm optimization (PSO). In terms of identification accuracy, PSO shows excellent agreement between the simulated I–V curve and the manufacturer’s data, with a higher R^2 of 0.9901 and a lower RMSE of 0.0358. The identified parameters are then used to accurately reproduce the behavior of the target PV module within the dynamic simulation.
- The manufacturer’s data for the centrifugal pump, usually provided for a single nominal speed, were used to develop a pump model capable of predicting the flow rate, head, and load torque at different rotational speeds.
- Four machine-learning models (ANN, ANFIS, SVM, and GPR) were developed and compared to predict the MPP voltage of the PV generator. Based on this comparative study, ANFIS stands out due to its high predictive accuracy, outperforming the other models and conventional MPPT methods. Furthermore, the ANFIS model was combined with a PI controller to ensure MPP tracking, which effectively maintains the PV voltage close to the optimal value with a fast response time of 0.04 s.
- To assess the effectiveness and robustness of the PVWPS control strategy, sudden changes in irradiance and ambient temperature were applied. While variations in irradiance have a negligible effect on the optimal MPP voltage, a sudden drop in irradiance significantly reduces the PV current and power, leading to a noticeable decrease in water flow rate. An increase in ambient temperature causes a marked reduction in PV-generated power due to the drop in PV voltage. In contrast, the PV current remains almost unchanged, which also results in a lower water flow rate. These results confirm the direct coupling between PV power and the hydraulic performance of the pumping system.

Appendix A

A.1 ANN Approach

A standard feedforward artificial neural network (ANN) was employed. Its architecture consists of input, hidden, and output layers. The hidden layer contains neurons that perform internal processing. The neuron's output (s_j) is calculated as follows:

$$s_j = g \left(\sum_{i=1}^m w_{ij} \cdot e_i - b_j \right) \quad (\text{A.1})$$

The weights (w_{ij}) and bias (b_j) are adjustable parameters. Users can select the transfer function, number of hidden layers and neurons, and training algorithm to optimize performance. A linear transfer function was chosen for the output, and a hyperbolic tangent sigmoid one for the hidden layer to obtain better performances. The used training function is the Bayesian Regularization. After the training phase for different numbers of hidden neurons, the best configuration is given for a 2-3-1 structure.

A.2 ANFIS

The ANFIS architecture is characterized by five layers. For an ANFIS configured with two inputs, denoted as x_1 and x_2 , the output of each layer is determined by the following equations:

$$L_{1,i} = \mu A_i(x_1), \text{ for } i = 1, 2 \dots j \quad (\text{A.2})$$

$$L_{1,i} = \mu B_i(x_2), \text{ for } i = 1, 2 \dots j \quad (\text{A.3})$$

$$L_{2,i} = W_i = \mu A_i(x_1) \cdot \mu B_i(x_2), \text{ for } i = 1, 2 \dots j^2 \quad (\text{A.4})$$

$$L_{3,i} = \bar{W}_i = \frac{W_i}{\sum_{i=1}^{j^2} W_i} \quad (\text{A.5})$$

$$L_{4,i} = \bar{W}_i f_i = \bar{W}_i (p_i x_1 + q_i x_2 + r_i) \quad (\text{A.6})$$

$$L_{5,i} = \sum_{i=1}^{j^2} \bar{W}_i f_i = \frac{\sum_{i=1}^{j^2} W_i f_i}{\sum_{i=1}^{j^2} W_i} \quad (\text{A.7})$$

where, ($\mu A_i, \mu B_i$): Membership functions, (p_i, q_i, r_i): Linear parameters.

A.3 GPR model

Given a training dataset of input (x_i) and target (y_i) values, the relationship between them can be modeled as follows:

$$y_i = f(x_i) + \zeta_i \quad (\text{A.8})$$

The model includes additive Gaussian noise with a mean of zero and variance. In a distributed Gaussian process, $f(x_i)$ can also be represented as:

$$f(x_i) \approx GP(m(x_i), K(XX')) \quad (\text{A.9})$$

where, $K(X, X')$: Covariance matrix, $m(x_i) = E[f(x_i)]$: Expectation function for input x_i .

A.4 SVM model

The learned parameters of the SVM regression model comprising support vectors, coefficients, and bias term are extracted to formulate the predictive regression function done by:

$$f(x) = \sum_{i=1}^n \alpha_i K(X_i, x) + b \quad (\text{A.10})$$

For the present study, the SVM model was trained by an RBF kernel function given by:

$$K(x_i, x_j) = G(x_i, x_j) = \exp\left(-\frac{\|x_i - x_j\|^2}{2\sigma^2}\right) \quad (\text{A.11})$$

where, σ^2 : Variance.

Appendix B

Table B. System Parameters

Element	Parameter	Value
PV Generator (STC)	Module type	KC200GT
	Maximum power	200 W
	Voltage at MPP	26.3 V
	Current at MPP	7.61 A
	Short circuit current	8.21 A
	Open circuit voltage	32.9 V
	Temperature coefficient of Isc	3.18×10^{-3} A/°C
	Number of cells	54 per module
Squirrel cage motor	Module's configuration	14 in series
	Nominal power	2200 W
	Nominal speed	2900 rpm
	Nominal line-to-line voltage	380 V
	Nominal frequency	50 Hz
	Stator winding resistance	6.18 Ω
	Stator inductance	0.688 H
	Mutual inductance	0.678 H
	Rotor winding resistance	4.38 Ω
	Rotor inductance	0.688 H
	Pole pair	1
	Rotor inertia moment	0.012 kg·m ²
Hydraulic circuit	Viscous torque constant	0.0033 N·m·s
	Static head	45 m
	Length of pipe	47 m
	Inside pipe diameter	0.04 m
	Pipe wall roughness	0.15 mm
	Computed friction factor	0.0502
	Total fitting loss coefficient	13.5
	Specific drawdown	-1.67 m/(m ³ /h)

Appendix C

Table C. Manufacturer Data of Centrifugal Pump (Bombas Ideal VIP 9/6) for $N_n = 2900$ rpm

Q (l/s)	H (m)	η (%)	Computed T_r (N·m)
0.83522	74.14586	45.24	4.42235
1.11089	71.82246	50.61	5.09244
1.38758	68.57708	55.09	5.58015
1.67142	65.36857	58.39	6.04412
1.94709	61.23808	59.56	6.46701
2.22378	55.37426	59.56	6.67876
2.49945	48.33031	57.49	6.78744
2.77614	41.32323	53.88	6.87756
3.05181	33.98424	48.51	6.90689

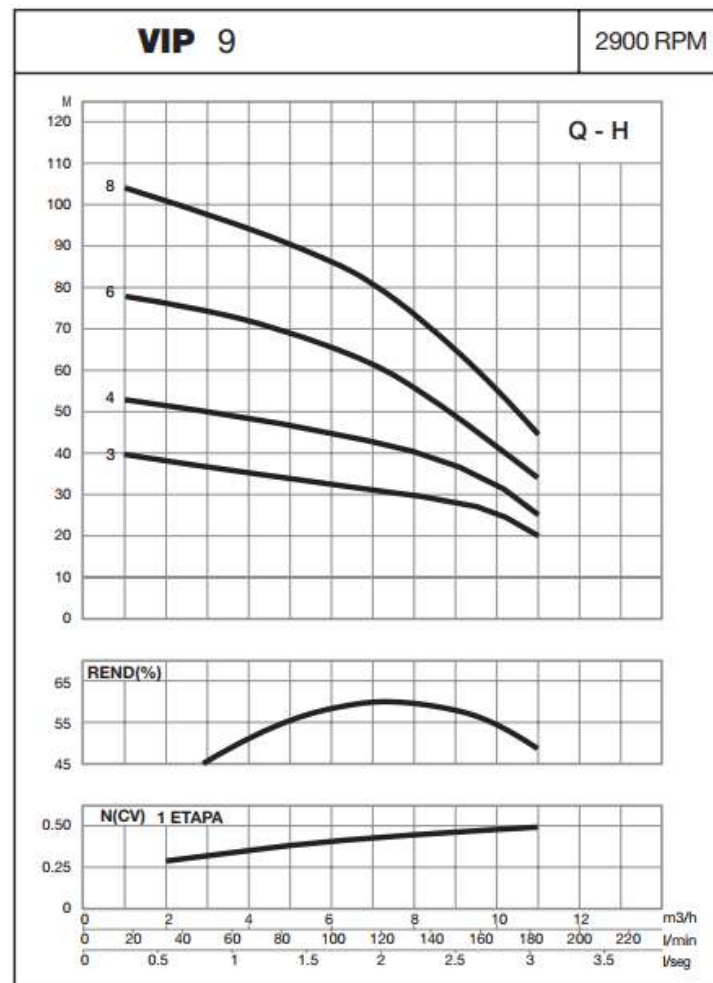


Fig. C. The efficiency curve of the pump (Bombas Ideal VIP 9/6)

References

- [1] Haddad S, Benganem M, Mellit A, Daffallah KO. ANNs-based modeling and prediction of hourly flow rate of a photovoltaic water pumping system: experimental validation. *Renewable and Sustainable Energy Reviews*, 2015; 43:635-643. <https://doi.org/10.1016/j.rser.2014.11.083>
- [2] De Leone R, Pietrini M, Giovannelli A. Photovoltaic energy production forecast using support vector regression. *Neural Computing and Applications*, 2015; 26(8):1955-1962. <https://doi.org/10.1007/s00521-015-1842-y>
- [3] Nageem R, R J. Predicting the power output of a grid-connected solar panel using multi-input support vector regression. *Procedia Computer Science*, 2017; 115:723-730. <https://doi.org/10.1016/j.procs.2017.09.143>
- [4] Mahesh PV, Meyyappan S, Alla R. Support vector regression machine learning based maximum power point tracking for solar photovoltaic systems. *International Journal of Electrical and Computer Engineering Systems*, 2023; 14(1):100-108. <https://doi.org/10.32985/ijeces.14.1.11>
- [5] Chandra S, Gaur P, Pathak D. Radial basis function neural network based maximum power point tracking for photovoltaic brushless DC motor connected water pumping system. *Computers and Electrical Engineering*, 2020; 86:106730. <https://doi.org/10.1016/j.compeleceng.2020.106730>
- [6] El Haji I, Megrini M, Kchikach M, Sahbani S, Gaga A, El Hasnaoui A. Performance optimization of symmetrical multi-level boost converter using hybrid MPPT-ANN for solar energy applications. *Results in Engineering*; 2025; 26:104729. <https://doi.org/10.1016/j.rineng.2025.104729>
- [7] Sharkawy A-N. The effect of increasing hidden layers on the performance of the deep neural network: Modelling, investigation, and evaluation. *Res. Eng. Struct. Mater.*, 2025; 11(4): 1633-1651. <http://dx.doi.org/10.17515/resm2024.442st0909tn>
- [8] Alice Hepzibah A, Premkumar K. ANFIS current-voltage controlled MPPT algorithm for solar powered brushless DC motor based water pump. *Electrical Engineering*, 2019; 102(1):1-11. <https://doi.org/10.1007/s00202-019-00885-8>

- [9] Sarkar R, Kumar JR, Sridhar R, Vidyasagar S. A new hybrid BAT-ANFIS-based power tracking technique for partial shaded photovoltaic systems. *International Journal of Fuzzy Systems*, 2021; 23(5):1313-1325. <https://doi.org/10.1007/s40815-020-01037-y>
- [10] Pachaivannan N, Subburam R, Padmanaban M, Subramanian A. Certain investigations of ANFIS assisted CPHO algorithm tuned MPPT controller for PV arrays under partial shading conditions. *Journal of Ambient Intelligence and Humanized Computing*, 2021; 12(10):9923-9938. <https://doi.org/10.1007/s12652-020-02738-w>
- [11] Ahmed MM, Hassanien WS, Enany MA. Modeling and evaluation of SC MPPT controllers for PVWPS based on DC motor. *Energy Reports*, 2021; 7:6044-6053. <https://doi.org/10.1016/j.egyr.2021.09.055>
- [12] Hilali A, El Ouanjli N, Mahfoud S, Al-Sumaiti AS, Mossa MA. Optimization of a solar water pumping system in varying weather conditions by a new hybrid method based on fuzzy logic and incremental conductance. *Energies*, 2022; 15(22):8518. <https://doi.org/10.3390/en15228518>
- [13] Boussafa A, Rabeh R, Ferfra M, Chennoufi K. Experimental test of optimizing maximum power point tracking performance in solar photovoltaic arrays based on backstepping control and optimized by genetic algorithm. *Results in Engineering*, 2024; 23:102746. <https://doi.org/10.1016/j.rineng.2024.102746>
- [14] Juarez-Perez EJ, Momblona C, Casas R, Haro M. Enhanced power-point tracking for high-hysteresis perovskite solar cells with a galvanostatic approach. *Cell Reports Physical Science*. 2024; 5(4):101213. <https://doi.org/10.1016/j.xcrp.2024.101885>
- [15] Chen B. High frequency design and efficiency optimization of LLC resonant converter based on SiC devices. *Applied and Computational Engineering*, 2025; 145:176-181. <https://doi.org/10.54254/2755-2721/2025.22243>
- [16] Chandel SS, Nagaraju Naik M, Chandel R. Review of solar photovoltaic water pumping system technology for irrigation and community drinking water supplies. *Renewable and Sustainable Energy Reviews*, 2015; 49:1084-1099. <https://doi.org/10.1016/j.rser.2015.04.083>
- [17] Santbergen R, van Zolingen RJC. The absorption factor of crystalline silicon PV cells: a numerical and experimental study. *Solar Energy Materials and Solar Cells*, 2008; 92(4):432-444. <https://doi.org/10.1016/j.solmat.2007.10.005>
- [18] Belarbi M. Modeling and simulation of a PV pumping system. Magister, University of Sciences and Technology - Oran, Algeria, 2006.
- [19] Bose BK. Power electronics and motor drives: advances and trends. Amsterdam: Elsevier Academic Press, 2006. <https://doi.org/10.1109/CIEP.2006.312138>
- [20] Nassar-Eddine I, Obbadi A, Errami Y, El Fajri A, Agunaou M. Parameter estimation of photovoltaic modules using iterative method and the Lambert W function: a comparative study. *Energy Conversion and Management*, 2016; 119:37-48. <https://doi.org/10.1016/j.enconman.2016.04.030>
- [21] Benahmida A, Maouhoub N, Tifidat K, Sahsah H. Extraction of photovoltaic generator parameters through combination of an analytical and iterative approach. *International Journal of Electrical and Computer Engineering*, 2022; 12(5):4529-4537. <https://doi.org/10.11591/ijece.v12i5.pp4529-4537>

Recent trends and open challenges in battery management systems: A comprehensive review with hardware-in-the-loop simulation for electric vehicle applications

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Abstract

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State of life

The paper highlights a novel approach that provides a system-level Battery Management System (BMS) review for electric vehicles (EVs), particularly with respect to Hardware-in-the-Loop (HIL) validation. This bridges the gap between algorithmic research and real-time BMS testing. Energy storage system (ESS) technology remains a challenge for EVs, expected to dominate future transport. A sophisticated BMS is crucial for monitoring battery health, ensuring safety, and extending lifespan. Using voltage, current, and temperature measurements, the BMS estimates parameters like SOC, SOH, impedance, capacity, power fade, and remaining life, vital for safety devices, charge balancing, thermal management, and optimal charging. Resilient BMS includes precise characterization, reliable estimation, and effective control. Development requires comprehensive testing of analog I/O and embedded code; HIL simulations offer advantages over traditional testing, providing reproducibility, efficiency, and safety, especially for ranges beyond normal limits. HIL is invaluable for early development, particularly for logic and fault testing. This review covers Li-ion battery modeling using experimental data, implemented in real time with Opal-RT HIL. A scalable model with electronic devices enables thorough BMS testing, demonstrating the validation of key functions, including protection, temperature, current, voltage monitoring, and cell balancing. HIL simulation is essential for modern BMS development.

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1. Introduction

A quarter of the world's energy use is attributed to gasoline-powered cars [1]. When EVs take their position in the not-so-distant future, rechargeable batteries may help. Rechargeable batteries (RBs) are now used in power tools, home appliances, renewable energy systems, portable electronics, and airplanes in addition to electric cars. The performance of RBs has been a focus for the US, Japan, China, and Germany in recent years, with notable progress made in this area [2]. Many energy storage technologies have gained acceptance for use in transportation in recent years, comprising

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Zinc-bromide flow battery (ZBFB), Lead-acid, Lithium-ion (Li-ion), Sodium nickel chloride (NaNiCl), Vanadium redox flow battery (VRFB), Nickel-Cadmium (NiCd), and Sodium sulphur (NaS) batteries [3]. The Li-ion battery is one of the best, offering a long lifespan, high power density, low discharge rate, high efficiency, and high dependability [4]. Furthermore, the costs of Li-ion batteries are decreasing, making them more attractive in the EV sector [5], and as a result, the market for Li-ion batteries is expanding [6]. Thermal runaway, an irreversible chemical process triggered by several factors, including overvoltage and extreme temperatures, can occur in lithium-ion batteries. Thermal runaway and other safety-related issues are more likely to occur when the battery needs to be fast-charged, which is particularly important in the use of electric vehicles [7-8]. Various factors impact the effectiveness of energy storage in batteries; in particular, the use of EVs aims to maximize efficiency. Charge efficiency, for instance, is the proportion of the total energy required for charging [9]; high charging demands result in significant energy loss in the form of heat. Appropriate SOC estimation algorithms ensure that efficiency is enhanced by assisting in the design of minimal battery-pack configurations based on specific criteria. The dependability, efficiency, and safety of a battery-powered system are ensured by a Battery Management System (BMS). BMS algorithms aim to improve battery efficiency in multiple ways; optimal charging algorithms aim to minimize heat waste and reduce degradation of the battery's state of health. One typical cause of safety and reliability problems with Li-ion batteries is short-circuited cells, which can also result from individual cells in a battery pack becoming unbalanced over time [10-12]. Energy Storage System (ESS) technology poses significant challenges for electric vehicles (EVs), primarily due to trade-offs in Li-ion battery chemistries regarding energy density, power rating, safety, cost, and lifespan [13-15]. While high-energy-density batteries like NMC and NCA offer longer ranges [16-17], they degrade more quickly and present thermal safety risks compared to LFP batteries [18-19]. Battery degradation is influenced by complex factors, such as temperature and charging or discharging rates [20], with fast charging exacerbating the issue [21]. Additionally, the environmental impact of Li-ion batteries, including the use of critical materials and low recycling rates [22], underscores the need for balanced advancements in battery technology [23], control strategies, and sustainable practices [24].

We may effectively employ HIL tests to verify these operations. The benefits of several rechargeable batteries and BMS technologies—such as heat management, SOH, and SOC estimations—are presented in this overview. Next, a thorough description of the BMS HIL test procedure in the EV application was given. Future directions for research have also been discussed. So, the novelty and the contribution of the work are mentioned as follows;

- Heat management is discussed and verified for improving the battery management system.
- Estimates like SOH and SOC for rechargeable batteries are presented.
- A thorough description of the BMS HIL test is presented for the EV application.

The paper aligns strongly with several Sustainable Development Goals, particularly SDG 7: Affordable and Clean Energy, SDG 9: Industry, Innovation and Infrastructure, and SDG 13: Climate Action, by advancing the development of efficient and reliable BMS for EVs. Through the use of HIL simulation, the study promotes innovative and safe testing methods that enhance battery performance, lifespan, and safety, which are critical for the widespread adoption of energy storage systems and clean transportation. Improved monitoring and estimation of battery parameters such as state of charge and health contribute to optimized energy use and reduced resource waste, supporting SDG 12: Responsible Consumption and Production. Additionally, by enabling more dependable electric vehicles, the research contributes to cleaner urban mobility and supports SDG 11: Sustainable Cities and Communities, ultimately helping reduce carbon emissions and combat climate change.

2. Review of Research Findings

2.1 Energy Storage Systems

A motor, a power converter, a mechanical transmission, and an ESS comprise the main parts of an electric vehicle. Globally, various energy storage systems are currently available, including lead-

acid, VRFB, ZBFB, NaS, NaNiCl, and Li-ion. The table below lists all the features of the batteries previously discussed.

Lithium-ion batteries have the highest power and energy densities, whereas VRFB batteries have the longest life cycles, as shown in the table above. Regarding cost, nominal voltage, and life cycle, Li-ion batteries are ideal for electric vehicle applications. The spider chart for the chemistries of the various cells is displayed in Figure 1 to facilitate easy understanding and comparison [26].

Table 1. Characteristics of the various types of energy storage systems [25]

Battery type	Energy Density (Wh/L)	Power Density (W/L)	Nominal Voltage (V)	Life cycle	DOD (%)	Round-trip efficiency (%)	Estimated Cost USD/kWh
Lead acid	50-80	10-400	2.0	1500	50	82	105-475
NaS	140-300	140-180	2.08	5000	100	80	263-735
NaNiCl	160-275	150-270	-	3000	100	84	315-488
NiCd	60-150	80-600	1.3	2500	85	83	-
VRFB	25-33	1-2	1.4	13000	100	70	315-1050
ZBFB	55-65	1-25	1.8	10000	100	70	525-1680
Li-ion	200-400	1500-10000	4.3	10000	95	96	200-1260

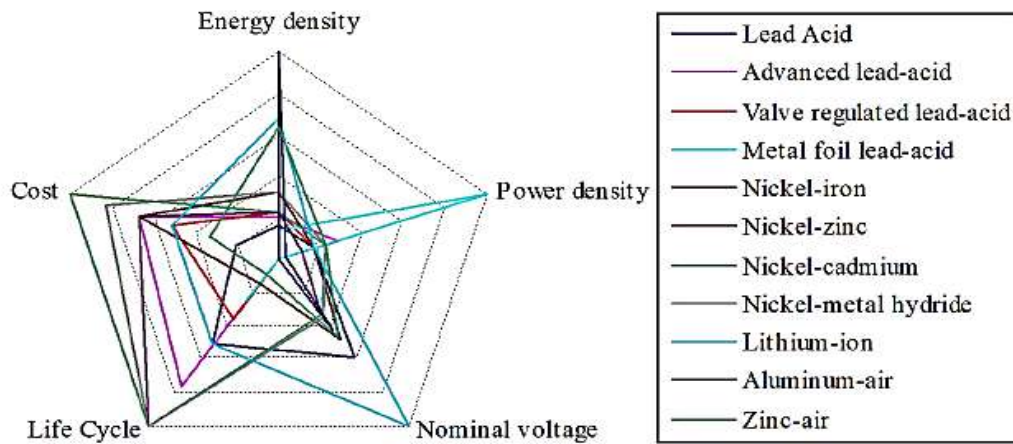


Fig. 1. Spider chart of chemistries of various cells [26-28]

2.1.1 Lithium-Ion Battery

A Lithium-ion cell is composed of a separator, two current collectors, a cathode (negative electrode), an anode (positive electrode), and both. Li-ion manganese oxide (LMO), Li-iron phosphate (LFP), Li-Sulphur (LiS), Li-Titanate Oxide (LTO), or Li-nickel-manganese-cobalt oxide (LNMC) are the materials generally found in the anode, whereas graphite typically acts as the negative electrode. Ethyl or diethyl carbonate can be used as the electrolyte. As negative and positive current collectors, respectively, copper and aluminium are employed.

Table 2. Comparison of several Li-ion battery types [2,27]

Type	Energy Density (Wh/kg)	Power Density (W/kg)	Life Cycle	Estimated Cost (USD/kWh)	Safety	Maturity
LMO	160	200	2000	360	Good	Commercial
LFP	120	200	2500	360	Good	Commercial
LNMC	200	200	2000	360	Good	Commercial
LTO	70	1000	10000	860	Good	Demo
LiS	500	-	100	-	Good	R&D Stage

The following Table [2,] compares several Li-ion battery types. Lithium-ion batteries are the best choice for EVs; however, they must still possess a long-life cycle, improved performance, better thermal control, and lower capital costs.

2.1.2 Battery Management System (BMS)

The architecture and design of the BMS are dealt with at a conceptual level in the current work. However, a comprehensive system-level embedded BMS architecture, comprising sensing layers, real-time estimation modules, control logic, safety monitoring, and communication interfaces, has not been fully developed within the current scope. The current work provides the necessary functional building blocks and requirements for such development, while hardware-software co-design and real-time development will be carried out in future work. In the current study, the parameters of the equivalent circuit model (ECM) and thermal model were primarily taken from the literature and datasheet values, and were adapted for HIL simulation compatibility, rather than fully characterized through experimental parameter identification and optimization. The experimental identification and optimization of ECM and thermal model parameters using laboratory test data (such as pulse testing, Hybrid Pulse Power Characterization (HPPC), and thermal cycling) will be considered in future work.

A robust and advanced BMS is needed when ESS has adequately matured. The BMS consists of three main components in EVs: controls, sensors, and actuators. An efficient BMS performs the following primary tasks. i) protects the battery; ii) operates within safe limits for voltage, current, and temperature; and iii) precisely measures and evaluates the battery's state [28].

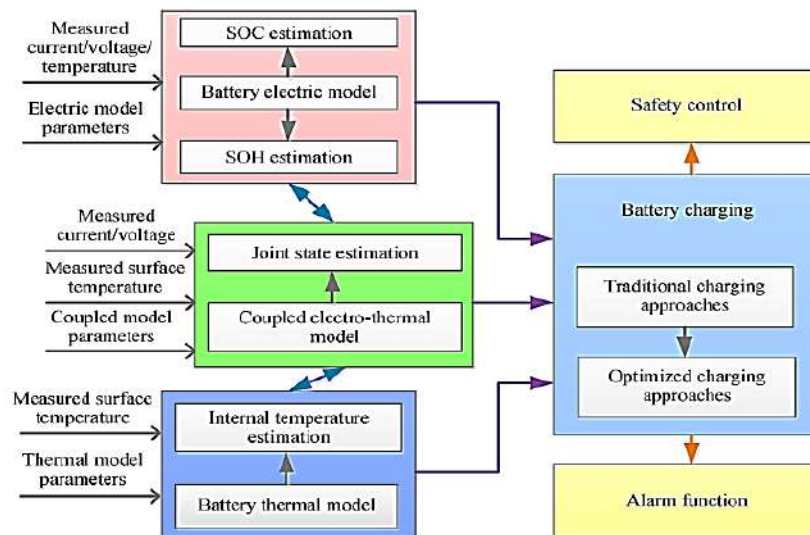


Fig. 2. Relation of key technologies in the BMS [29]

2.1.3 State of Health Estimation

In this study, thermal management is not explicitly included as the focus is on developing a framework for electrical behavior modelling and estimation. While thermal phenomena are crucial for battery safety and other functions in electric vehicles, incorporating electro-thermal modelling would complicate the system. This work lays the groundwork for future complete BMS implementations that will address thermal effects.

Degradation of batteries results in a reduction in their energy storage capacity and fast charging/discharging performance. The degree of aging of batteries can be assessed quantitatively using the SOH index. Model-based and experimental analytic methods are the two groups into which SOH estimation techniques can be divided [30], as illustrated in Figure 3 below. The battery SOH has no specific definition. A broad explanation of battery SOH is as follows:

$$SOH(t) = SOH(t_0) + \int_{\tau=t_0}^t \partial_{func}(I, T, SOC, others) d\tau \quad (1)$$

where SOH (t_0) represents the initial battery SOH, an aging rate function is highly dependent on several variables, including temperature, current, and SOC. Other variables that SOH represents include mechanical vibrations and over-potential.

The capacity/energy measurement test is suitable for bench tests or stable conditions, as it requires precise data collection and high integrity of the charging and discharging processes. Ohmic resistance measurement is a simple, accurate, and real-time method that requires pulse voltage and current. Impedance measurement requires an electrochemical workstation or AC excitation apparatus [31]. Electrochemical Impedance Spectroscopy (EIS) can yield distinctive parameters for calibrating SOH. A multi-step derivation technique indirectly estimates internal resistance or capacity by analyzing various health indicators, including film resistance, the capacity-OCV-SOC response surface, the voltage response trajectory, the incremental capacity curve, and ultrasonic wave response. Battery SOH can be assessed by combining multiple health parameters.

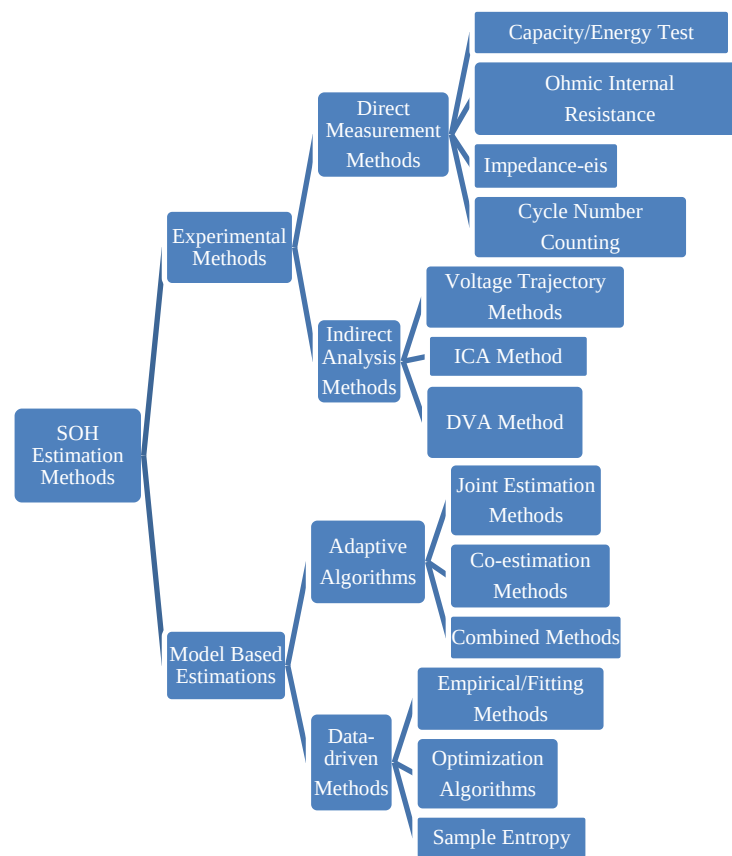


Fig. 3. Flow chart showing estimation methods of SOH [31]

The empirical/fitting approach is a data-driven method for estimating battery life based on available aging data. It uses polynomial, exponential, power law, logarithmic, and trigonometric functions, often computing quickly and with minimal computational complexity. The Arrhenius equation is used to model temperature-dependent processes, such as diffusion coefficients and creep rates, demonstrating its effectiveness in SOH calculations [32]. The Arrhenius equation can also be used to describe the temperature-dependent aging rate of batteries, as shown in the following equation;

$$\frac{dC}{dn} = \Lambda e^{-\frac{\Delta E}{R_g T}} = \Lambda e^{-\frac{\lambda}{T}} \quad (2)$$

Where $\frac{dC}{dn}$ represents the change rate of cell capacity relative to the aging cycle, Λ indicates the pre-exponential factor, R_g is defined as the general gas constant, i.e., 8.314 J/(mol K), ΔE denotes the activation energy, in units of J/mol, and T refers to the absolute temperature, in units of K. Λ and λ represent two unknown parameters needing to be calibrated. Integrating Eq. gives;

$$C_r = -\Lambda n_c e^{-\frac{\lambda}{T}} \quad (3)$$

Where C_r indicates the threshold of capacity reduction, indicating the degradation of the cell, and n_c denotes the cycling life of the cell. Two different temperature points, T_1 and T_2 ($T_1 > T_2$), were selected.

$$\Delta n_c = \frac{C_r}{\Lambda} \left(e^{-\frac{\lambda}{T_2}} - e^{-\frac{\lambda}{T_1}} \right) \quad (4)$$

The influence of temperature on the cell lifetime is described quantitatively by Δn_c , which is defined as the lifetime gap between two temperature levels. The Arrhenius equation can be used to evaluate the battery's SOH at several temperatures once its characteristics have been determined.

The current method for estimating SOH differs from those used in neural networks or machine learning topologies. Neural networks are used for voltage curve estimation, while machine learning topology utilizes changes in temperature, current, and internal circuit characteristics. Support vector machine approaches classify SOH; partial charging voltage curves are estimated. Charging time and capacity required for online estimation. The terminal voltage under the hybrid pulse power characterization current profile is identified using a sample entropy approach [33-37].

2.1.4 Optimal Charging

The safety mechanisms in the system include fault detection, isolation, protection logic, and abuse handling, ensuring reliable operation of EV components under abnormal conditions. However, a lack of description regarding the fault conditions tested in HIL simulations hinders understanding of the mechanisms' effectiveness during actual faults. Battery charging using modern technology is outdated, time-consuming, and insecure compared to gasoline [38]. Optimal charging algorithms (OCA) have gained attention, with the continual trickle-current-based strategy being a popular technique [39]. However, shortening the charging time by increasing the charging current results in the terminal voltage exceeding the safety threshold [40-41]. When the terminal voltage reaches a threshold, the initial higher current needs to be reduced [42].

The constant-current constant-voltage (CC-CV) strategy is a popular method for quick charging, offering shorter charging times and longer battery life cycles [43]. Taguchi-based battery charging techniques use orthogonal arrays for methodical solution selection [44-45]. Boost charging delivers high current to depleted batteries, while pulse-charging techniques require brief rest intervals or intentional discharging phases [46-48]. Battery charging profile optimization can be achieved using soft-computing techniques, such as an ant colony method and a universal voltage protocol [49-50]. These methods aim to improve cycle life and charging efficiency by establishing a specific charging profile based on the battery's SOH [51]. An optimization method, utilizing a cost function that combines energy loss and time-to-charge, has been employed to determine the optimal charging plan [52-53]. Alternative methods include data mining, neural networks, and genetic algorithm-based tactics, as well as grey-predicted charging systems [54-56].

In charging optimization, advantage is taken of battery models that were developed in earlier studies. The effects of temperature, health state, and charging-voltage threshold on charging outcomes are examined for lithium nickel manganese cobalt oxide (LiNMC) and lithium iron phosphate (LiFePO₄) batteries [51]. Constant-current-constant-voltage (CC-CV) charging is considered the most popular method for charging lithium-ion batteries. However, the battery's performance and lifespan are affected by excessive temperature increases during the CC stage. It follows that an extension of the CV process is required. An ideal charging algorithm based on the Genetic Algorithm (GA) is proposed below to minimize the charging temperature increment while

maintaining a reasonable charging time. The correlation equation can be expressed as follows if the battery temperature is measured following a 0.1% SOC increment.

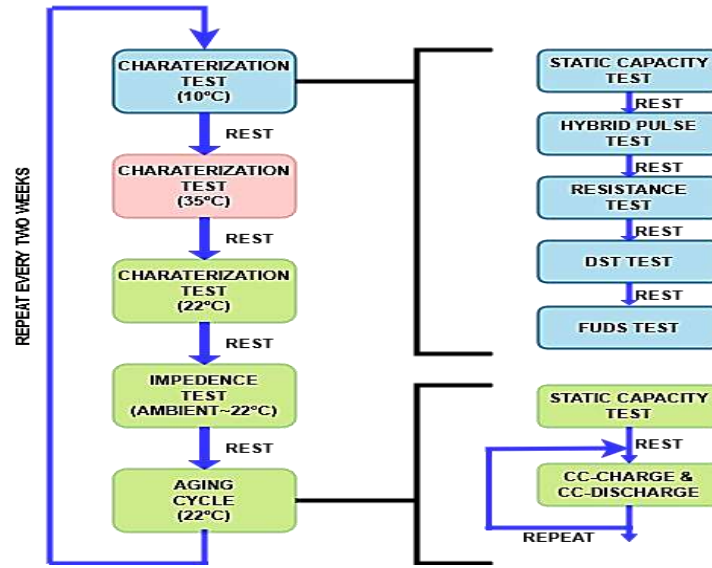


Fig. 4. The test chart for the collection of battery data. [52]

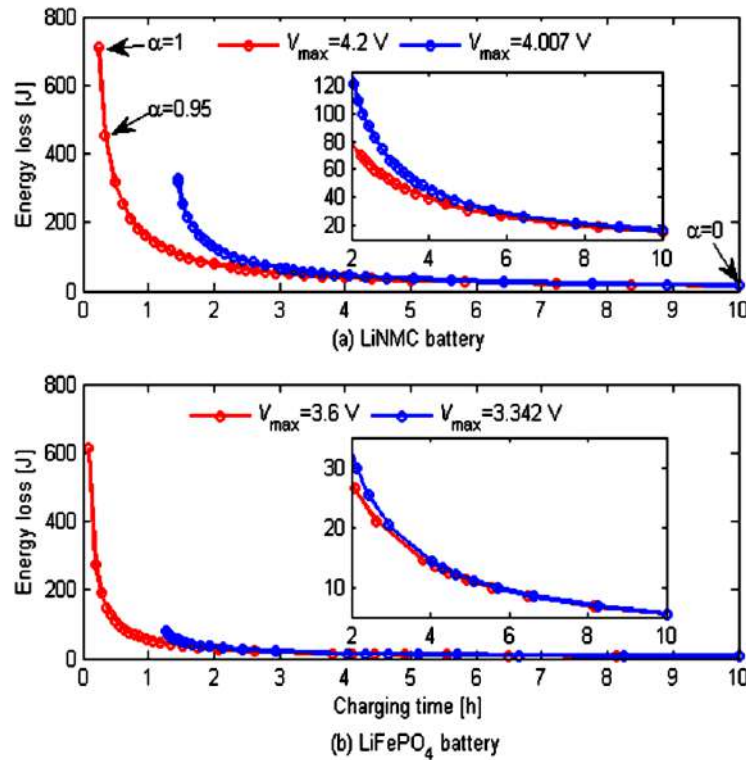


Fig. 5. Pareto charts of the multi-objective optimization for the two types of Li-ion batteries at 22°C: (a) LiNMC battery; (b) LiFePO₄ battery, where α is the weighting factor that takes values from 0 to 1, which reflects the proportionate importance of the two objectives. [52]

$$T(S) = T(S - 0.1\%) + \frac{I^2 \cdot R_0 + I \cdot V_p - A \cdot h(T(S - 0.1\%) - T_{air})}{mC} \quad (5)$$

where S is SOC, I is charging current, R_0 is the Ohmic resistance of the battery, V_p is polarization voltage, h is the heat transfer coefficient, T_{air} is ambient temperature, m is the mass of the battery, A is the surface area of the battery, and finally C is the specific heat capacity.

We define $\beta(I) = (I^2 \cdot R_0 - I \cdot V_p + A \cdot h \cdot T_{air}) / mC$ and $\alpha = 1 - Ah / mC$

Now, the above equation can be written as

$$T(S) = \alpha \cdot T(S - 0.1\%) \cdot (S - 0.1\%) + \beta(I) \quad (6)$$

The solution of the equation can be calculated. The solution of the equation can be calculated.

$$T(S) = \alpha^k \cdot T(0) + \sum_{l=0}^k \alpha^{k-1-l} \cdot \beta(I_{l+1}) \quad (7)$$

Where $k = \frac{S}{0.1\%}$

Now R_0 and V_p need to be appropriately estimated. These parameters can be calculated by the Hybrid Pulse Power Characterization (HPPC) test and the Equivalent Circuit Model (ECM).

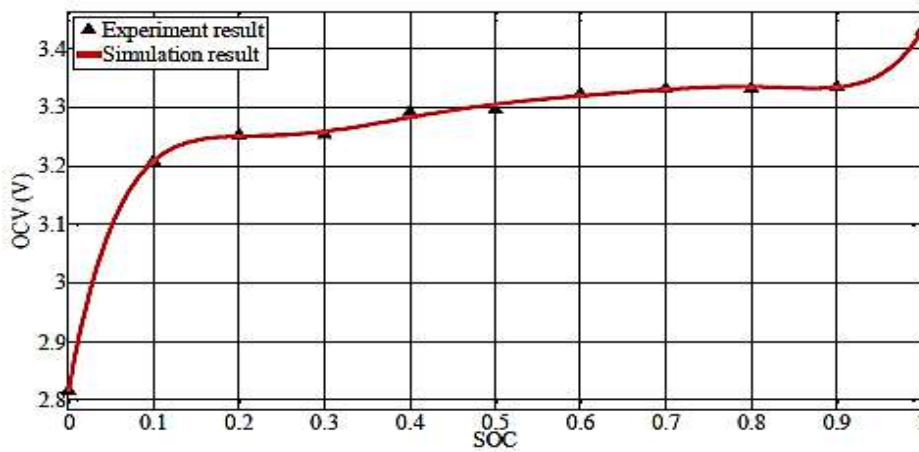


Fig. 6. Measured OCV curve with different SOC values [48]

Fig. 6 shows the measured OCV curve with different SOC values. The relation between OCV and battery SOC can be standardized as follows –

$$f_{OCV}(S) = P_1 S^7 + P_2 S^6 + P_3 S^5 + P_4 S^4 + P_5 S^3 + P_6 S^2 + P_7 S + P_8 \quad (8)$$

Similarly, this figure shows the measured ohmic resistance with changing SOC. The relationship can be formulated as follows;

$$f_{R_0}(S) = a_1 S^5 + a_2 S^4 + a_3 S^3 + a_4 S^2 + a_5 S + a_6 \quad (9)$$

Polarization voltage can be calculated as,

$$V_p = V_t - f_{OCV}(S) - I \cdot f_{R_0}(S) \quad (10)$$

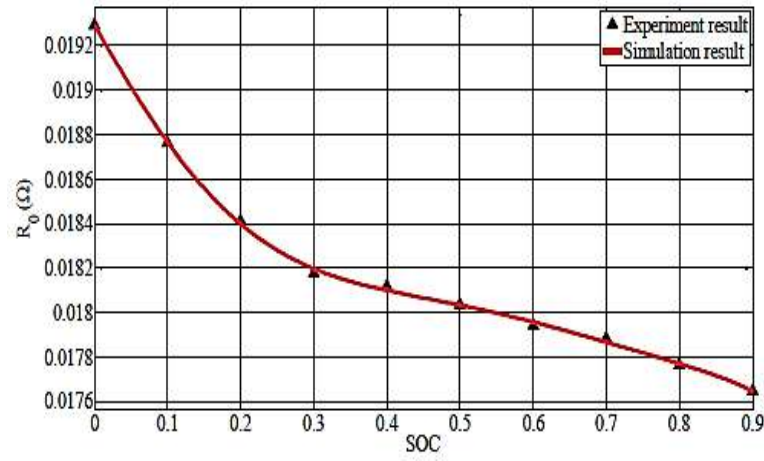


Fig. 7. Plot between battery ohmic resistance and SOC [48]

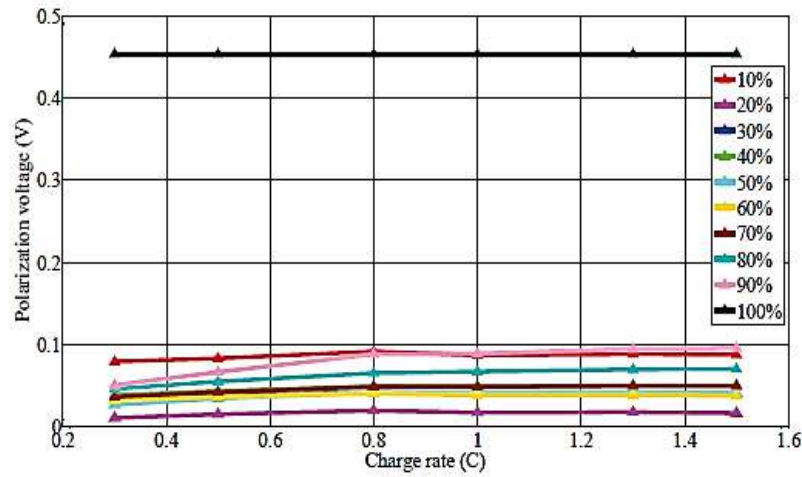


Fig. 8. Polarization voltage versus charging rate with different SOC levels [48]

The difference between the end and beginning temperatures can be characterized as the objective function, considering the influence of battery temperature and the battery charging temperature model. Assuming that the charging process takes n steps to attain the target SOC, the initial temperature $T(0)$ is T_0 , the final temperature $T(n)$ is T_n , and therefore the temperature increment is $\Delta T = T_n - T_0$

$$J = T_n - T_0 = \alpha^k \cdot T(0) + \sum_{l=0}^k \alpha^{k-1-l} \cdot \beta(I_{l+1}) - T_0 \quad (11)$$

$$= (\alpha^k - 1) \cdot T_0 + \sum_{l=0}^{1000n} \alpha^{1000n-1-l} \cdot \beta(I_{l+1}) \quad (12)$$

The optimization goal is to determine the optimal charging current, I_1 , to minimize the objective function, J , during the charging process. Battery charging time is a significant index to evaluate a charging strategy, which is different as the time taken to charge the battery from the initial SOC.

$S_{initial}$ to target SOC, S_{target} . Now, the sum of the charging time at each step, i.e. $t_{total} = \sum_{l=1}^n \frac{\Delta C_l}{I_l}$. Where

ΔC_l denotes the value at the different steps of the charging capacity.

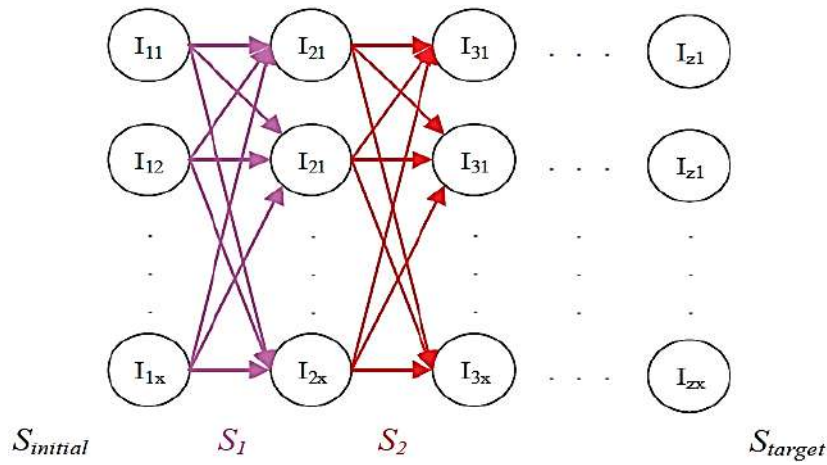


Fig. 9. Temperature variation profile [48]

The total charging time is lower than the constant current charging strategy. Therefore $t_{total} \leq t_{cc}$, Where t_{cc} refers to the charging time required when charging the battery at a constant current, as specified by the battery manufacturer. The charging current and voltage should be lower than the maximum permissible charging current and the maximum cut-off voltage, respectively, i.e. $V_t \leq V_{cut}$; $I_t \leq I_{max}$.

The implementation process is shown in the figure below. When charging from $S_{initial}$, it is assumed that x charging values and z step charging stages exist; therefore, x^z types of current combinations are obtained. Each type of charging current combination corresponds to a specific charging temperature increment mode; the minimum temperature increment current combination is selected as the optimal charging current.

2.1.5 Battery Thermal Management

Thermal behaviour, including temperature, is crucial for battery performance and lifespan in electric vehicles (EVs) [57]. Models capture this, including data-driven, reduced-order thermal, as well as heat generation and heat transfer, which explain the non-uniformity within batteries. Three widely used methods for evaluating the generation of heat in batteries are presented in Eq. (13) and are utilized extensively in real-time applications [58–61].

$$\begin{cases} Q_1 = RI^2 \\ Q_2 = I(V - OCV) \\ Q_3 = I(V - OCV) + IT \frac{dOCV}{dT} \end{cases} \quad (13)$$

where R is the internal resistance of the battery, I and V represent the current and voltage of the battery, respectively, OCV represents the open circuit voltage of the battery, Q_1 represents the heat generation of the battery which is mainly due to the huge current flowing through the internal resistance of the battery, Q_2 represents the heat generation of the battery due to the over-potentials across the RC network, Q_3 represents the heat generation of the battery due to both the entropy change and Joule's heating.

The three primary methods of heat transport that occur both inside and outside of batteries are heat convection, heat conduction, and heat radiation. In order to investigate the geometrical current and heat distribution within Li-ion batteries, Guo *et al.* [62] created a three-dimensional distributed-parameter heat transfer model, which is depicted as follows:

$$\frac{\partial \rho C_p T_{3c}}{\partial t} = -\nabla(k_{3c} \nabla T_{3c}) + Q \quad (14)$$

It can also be expressed as [51]

$$\frac{\partial \rho C_p T_{3c}}{\partial t} = -\frac{\partial}{\partial x} \left(k_x \frac{\partial T_{3c}}{\partial x} \right) - \frac{\partial}{\partial y} \left(k_y \frac{\partial T_{3c}}{\partial y} \right) - \frac{\partial}{\partial z} \left(k_z \frac{\partial T_{3c}}{\partial z} \right) + Q \quad (15)$$

Where the battery density is represented by ρ , the battery heat capacity is denoted by C_p , k_{3c} is the coefficient of battery thermal conductivity (along three dimensions: k_x k_y k_z), Q means battery heat generation. A simplified one-dimensional heat conduction model can be created [63-64] in the following manner, assuming that the battery temperature distribution inside each layer plane is uniform and that only one dimension of battery heat conduction (e.g., the x dimension) is taken into consideration.

$$\frac{\partial \rho C_p T_{1c}}{\partial t} = -\frac{\partial}{\partial x} \left(k_x \frac{\partial T_{1c}}{\partial x} \right) + Q \quad (16)$$

The temperature distribution inside the battery can be described by the three-dimensional heat transfer models, which can be used to identify potential hot spots, particularly in applications involving substantial heat generation. A one-dimensional heat transfer model can represent the temperature gradient in a single direction. These heat transfer models are primarily used in offline simulations because their computational overhead is frequently too high for real-time applications.

Considering that the primary mechanism of heat transport is conduction, the heat generation is dispersed equally inside the battery, and the interior and exterior temperatures of the battery are both constant, a well-liked two-stage thermal model for battery cells [65-67] can be developed as follows-

$$\begin{cases} C_{q1} dT_{in}/dT = k_1(T_{sh} - T_{in}) + Q \\ C_{q2} dT_{sh}/dt = k_1(T_{in} - T_{sh}) + k_2(T_{amb} - T_{sh}) \end{cases} \quad (17)$$

where T_{in} and T_{sh} stand for internal temperature and surface temperature of the battery, respectively, T_{sh} means the ambient temperature around the battery, C_{q1} , and C_{q2} stands for the thermal capacities of the battery interior and surface, k_1 stands for the heat conduction between the battery surface and the interior, k_2 stands for the heat conduction between the ambience and the battery surface. Reduced-order thermal models have been developed for controlling battery thermal management, identifying heat generation and transport components. These models are based on a computational fluid dynamics (CFD) model, which has lower computation costs and similar findings to the CFD model. This approach helps in controlling battery thermal management [68-70].

2.1.6 State of Charge Estimation

SOC represents a battery's remaining capacity relative to its overall capacity, with 100% indicating a fully charged battery and 0% indicating a discharged one [13]. SOC assessment is crucial for safe operation. There are direct estimation techniques and model-based methods, with OCV and Ah methods being the primary methods. The Ah method is a general and simple method to calculate SOC [71-72], which is illustrated as follows,

$$SOC(k) = SOC(k_0) + \int_{k_0}^k \eta I(t) dt / C_n \quad (18)$$

Where $SOC(k_0)$ is the known initial SOC, η stands for the efficiency of battery charging or discharging, C_n stands for the battery nominal capacity, $I(t)$ is the current value, which is positive during charging and negative during discharging. The Ah technique is a popular choice for monitoring charging and discharging current; however, its accuracy is influenced by mistake accumulation and real-time challenges, especially when the battery's charge level is between 10% and 90% [71-72].

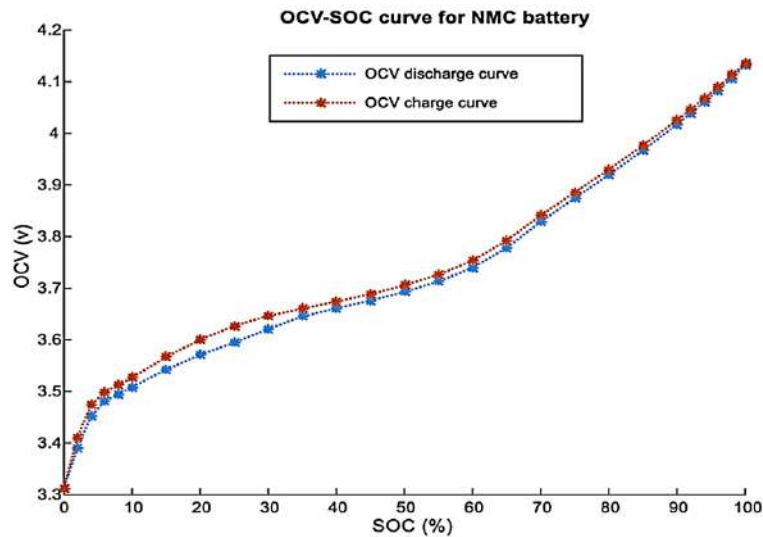


Fig. 10. Relationship between the battery OCV and SOC

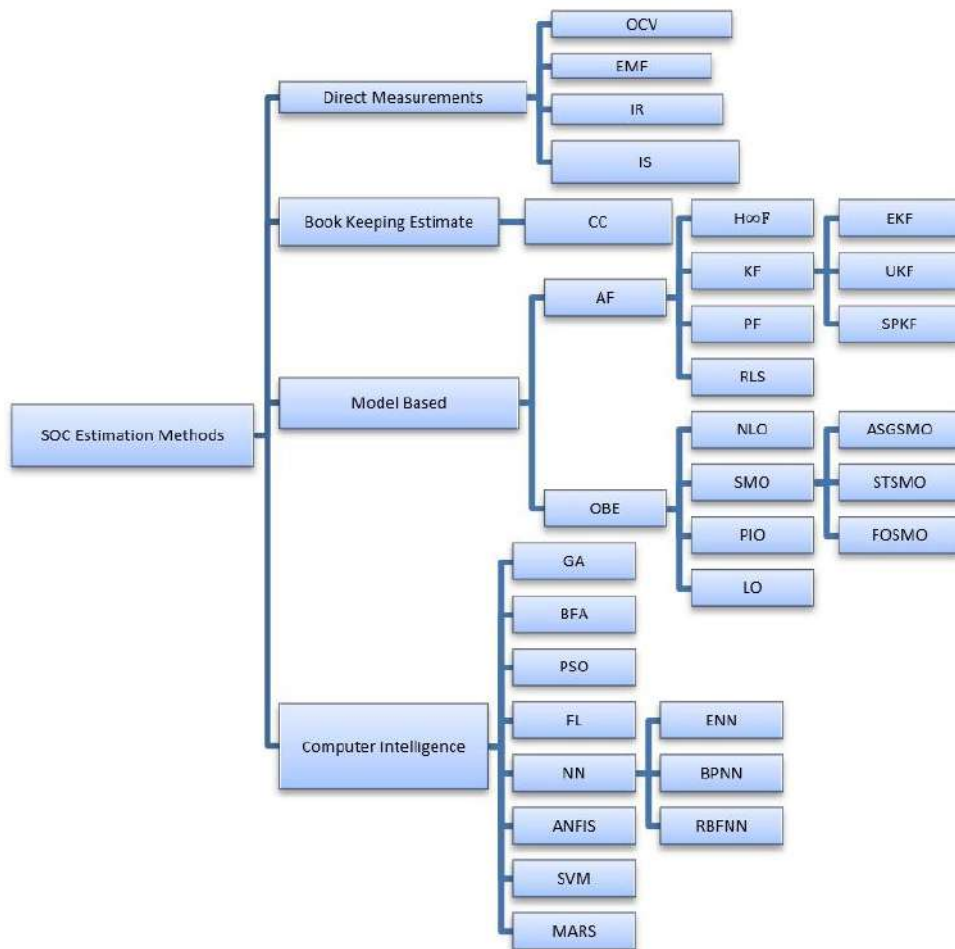


Fig. 11. Classification of SOC Estimation Methods. [28]

AF: Adaptive Filters. ANFIS: Adaptive neuro-fuzzy inference system. ASGSMO: Adaptive switching gain sliding observer. BFA: Bacterial foraging algorithm. BPNN: Back propagation neural network. CC: Coulomb Counting. EMF: Electromotive force. ENN: Elman Neural Network. EKF: Extended Kalman Filter. FL: Fuzzy Logic. FOSMO: Fractional order sliding mode observer. GA: Genetic Algorithm. $H_\infty F$: H infinity filter. IR: Internal Resistance IS: Impedance spectroscopy. KF: Kalman filter. MARS: Multivariate adaptive regression splines LO: Luenberger-

based observer. NLO: Nonlinear observer. NN: Neural network. OBE: Observer-based estimation. OCV: Open circuit voltage. PF: Particle filter. PIO: Proportional integral observer. PSO: Particle swarm optimization. RLS: Recursive least square filter. RBFNN: Radial basis function neural network. SPKF: Sigma point Kalman filter. SVM: Support vector machine. SMO: Sliding mode observer. STSMO: Super twisting sliding mode observer. UKF: Unscented Kalman filter.

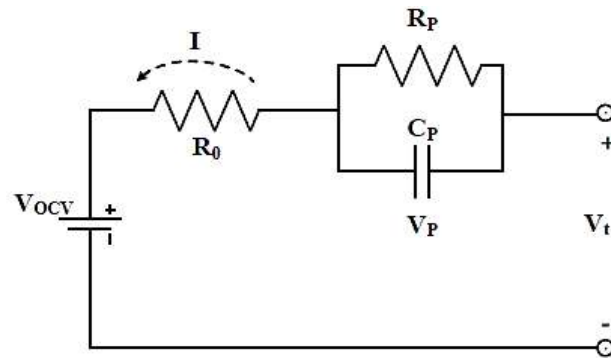


Fig. 12. Equivalent circuit of the battery

OCV is a popular strategy for measuring battery rest time, but it's not suitable for EV applications due to long resting periods and the varying relationship between SOC and OCV. Real-time OCV acquisition can resolve this issue, allowing online SOC estimates using model-based techniques. The battery's SOC is estimated using the battery equivalent circuit model and electrochemical model in standard state space [73-74]. State observers such as H_∞ , sliding mode, extended Kalman, adaptive Kalman, and Kalman filters are used for online estimation, affecting the accuracy of model-based techniques.

3. Methods and Data

3.1 Hardware-in-the-loop (HIL) Testing of ECU

The hardware part of the OPAL-RT system is the HIL platform, which may be used for real-time validation of the EV model and control logic. The OPAL-RT system is a simple simulation tool that enables deep correlation among electrochemical, thermal, and system-level phenomena. HIL testing is performed by omitting all of each ECU's interfaces and components that the HIL system simulates [75-77]. Equivalent electrical outputs, such as digital, analog, or variable resistance outputs, are always used to represent the sensors that are often attached [76]. The ECU's control outputs to actual or substitute loads are typically connected at the outputs, and the appropriate HIL system inputs are used to measure the ECU's control outputs [77]. Plausibly simulating sensor values at the sensor simulation HIL system's output is enabled by the controlled system's real-time competent simulation model. Rest bus simulation refers to the absence of communication on the HIL bus, as the HIL bus controller simulates it [66]. By extending these interfaces, the HIL system can be used to precisely or repeatedly generate different types of failures as needed [77-78].

3.2 HIL Test for Battery Management System (BMS)

Battery models for testing a BMS must depict the battery as a network of interconnected single cells, rather than models for simulating the car's electrical system in the traditional low-voltage range [79]. The SOC and cell voltage of a single battery cell are then modelled through a cell representation [80]. Leakage currents, dynamic behavior during abrupt load changes, and technical variations in behavior during charging and discharging are all considered [81-82]. The integration of the BMS HIL system into a vehicle simulation as a whole is the most intricate HIL simulation scenario [80]. The electrical ESS in a standard HEV or EV interacts with the gearbox, electric drives, and combustion engine, among other drivetrain parts. This study shows how thermal models and simple protection/control logic may be combined within a HIL simulation environment to assess the thermal monitoring and safety capabilities of BMS. This is a useful testbed for evaluating temperature measurement, threshold protection, and system behaviour before implementation in actual EV hardware. The "scalable battery model" lacks sufficient detail on critical aspects,

including pack-level dynamics, multi-cell imbalances, aging propagation, and adaptability across different chemistries, which are essential for effective system-level modelling and simulation.

3.3 Structure of HIL System with RT-LAB

In this work, the HIL battery model's accuracy, including voltage and temperature error, is not quantitatively compared to experimental reference data, as it is derived from literature to demonstrate HIL feasibility. Future work will aim to validate the model's accuracy experimentally. The electrical and thermal models in the Hardware-in-the-Loop (HIL) environment are connected via bidirectional electro-thermal coupling.

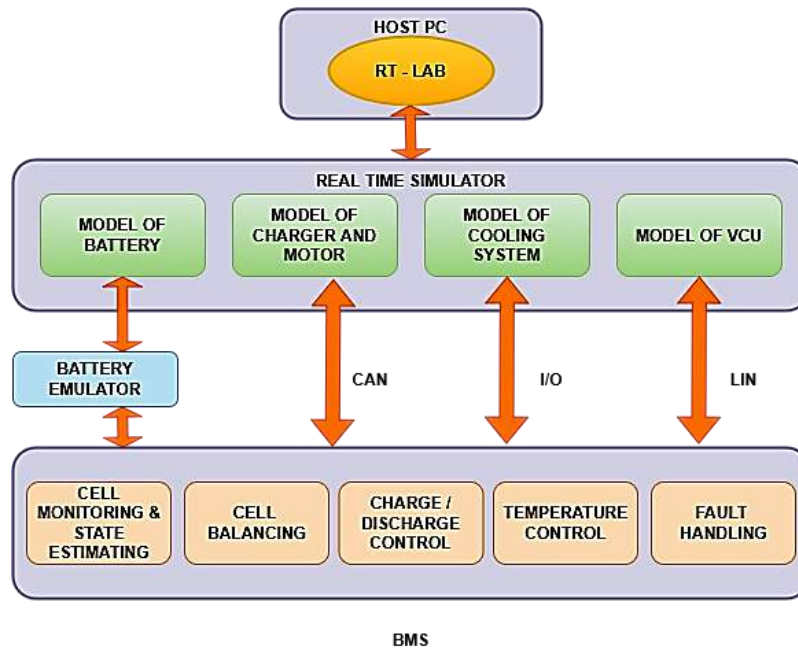


Fig. 13. Structure of HIL System

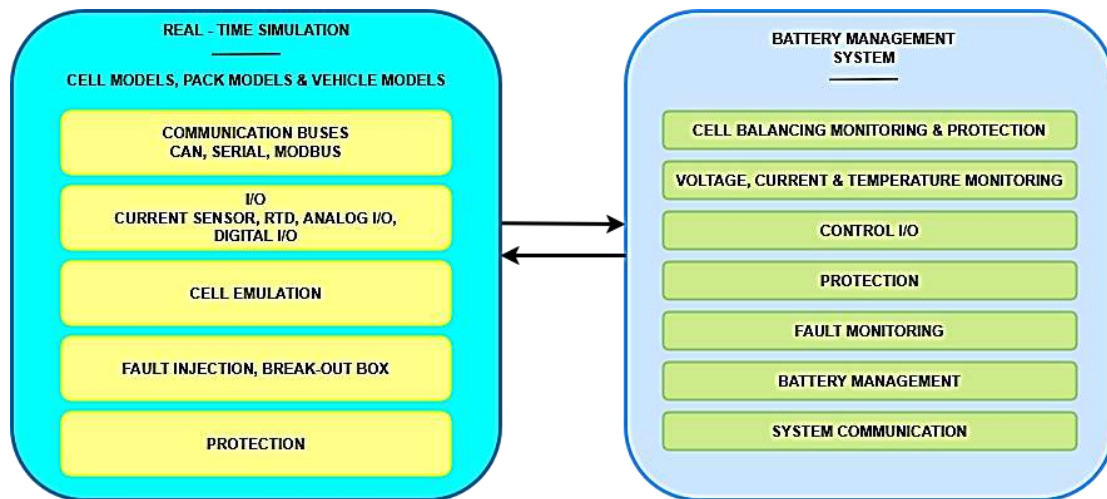


Fig. 14. Real-time HIL functional schematic

The electrical battery model calculates internal losses, like Joule heating, which the thermal model uses to determine temperature based on these heat sources. This temperature feedback is crucial for revising parameters in the electrical model and ensuring electro-thermal consistency. For SOC and SOH estimations, temperature influences the OCV-SOC relationship and internal resistance, thereby affecting estimation accuracy, though its impact varies by strategy. While sophisticated BMS can directly integrate temperature, most HIL simulations use fixed parameters, limiting robustness during temperature variations. Overall, the HIL architecture allows for parameter

revision based on temperature but does not enable comprehensive temperature-aware SOC/SOH estimation, highlighting a limitation of the approach [83-88].

The HIL system, as seen in Fig. 13 below, consists of the host PC, a battery emulation, a real-time simulator, and the BMS. The host PC serves as an interface between the user and the system, running testing and simulation control applications. It provides a platform for creating simulation models and managing the simulation process using MATLAB and RT-LAB tools, respectively. We can download the simulation model to the real-time simulator and run it using RT-LAB software. A multicore CPU and an FPGA card are found in the RT-LAB HIL box, the real-time simulator that is capable of sophisticated real-time computation and I/O processing. It uses digital and analog I/Os and the communication bus to interact with the BMS while computing all the models [89].

The real-time simulator manages the battery emulator, which simulates cell voltage, current balance, battery temperature, and battery defects. Other components of the BMS are connected to the real-time simulator via CAN buses and I/Os, while the monitoring and balancing unit is directly connected to the battery emulator. We can test the following BMS function items using this HIL system:

- Cell voltage monitoring
- Cell balancing
- State estimating
- Temperature control
- Fault handling
- Communication with VCU

3.4 Battery Simulation with High Electrical Requirements

A single battery cell's voltage is primarily determined by its SOC, temperature, and charge/discharge current. When the cell is not loaded, the SOC falls, and the cell voltage steadily declines.

3.4.1 Cell Model

A voltage source simulates the open-circuit voltage ($V_{Bat,0}$), and different impedances model the static and dynamic voltage behavior in a cell voltage model. The simplest possible substitution circuit for a car battery simulation is shown in the above picture. It comprises one RC element for double-layer capacity and charge transfer, and another RC element for the diffusion effect. The impedance spectroscopy approach is typically used to parameterize the cell voltage model. The values for the components of the replacement circuit are determined from measured spectrograms. An optimization algorithm can be used to define an automated procedure for the parameterization process. [90]. The multi-cell and pack-level SOH estimation was not covered in this study, as it focuses only on cell/module-level review and HIL validation/module level and platform feasibility. Detailed pack-level SOH modelling and scalability analysis require extensive multi-cell data, integrated thermal and aging models, and complex embedded system implementation, which are beyond the scope of this work.

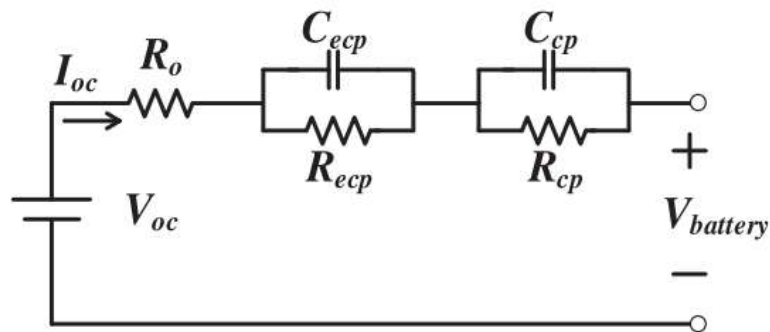


Fig. 15. Battery cell model

3.4.2 Cell Network

It is theoretically possible to create a series network of cells by joining several examples of the previously mentioned cell model. Thus, for the HIL simulation of a series cell topology, a novel strategy was adopted. Based on the assumption that a battery is composed of identical cells that consistently exhibit similar static and dynamic characteristics, the battery model is split into two sections: a reference section and a delta section.

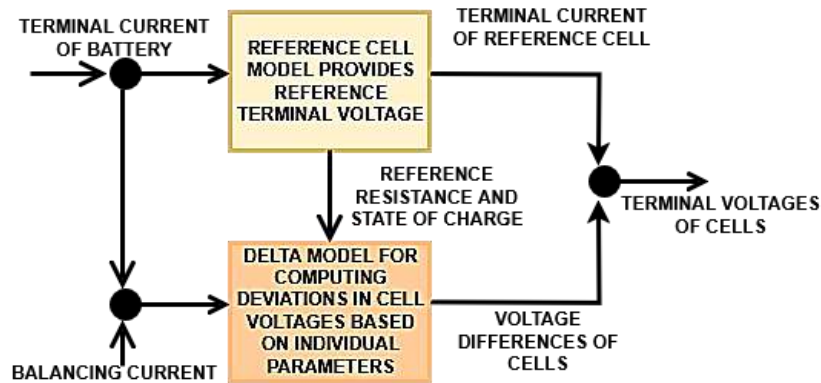


Fig. 16. Topology of the ASM multi-cell model

The delta model uses a linearized method to compute the difference between each single-cell voltage and the reference voltage. For every cell in the delta model, the capacity, starting SOC, and deviation of the internal impedances from the reference value can be provided. The new modelling strategy saves computing time on a real-time system by a factor of 12, while saving a significant amount of time in offline simulation on a PC. Fig. 17 illustrates the simulation of cell discharge using the multi-cell method compared to a model with individual cells. The cell voltage response, which is essential to the BMS's capability to balance cells, demonstrates minimal differences, particularly in the 40% to 80% SOC range.

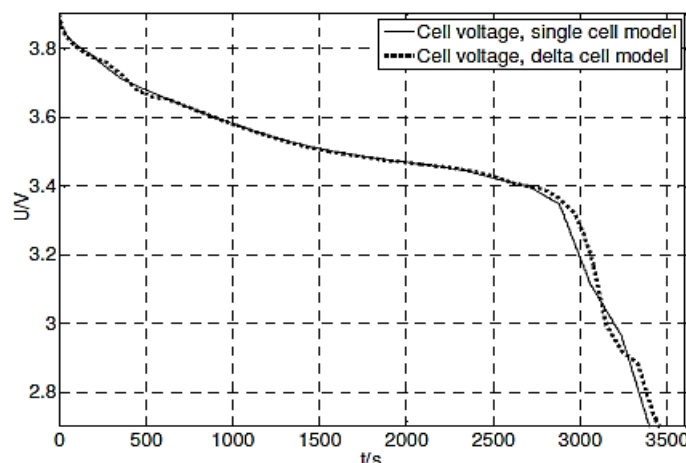


Fig. 17. Comparison of single and delta cell models

The currents in each circuit must be calculated for a parallel connection, and these currents are based on the battery's terminal voltage. For this computation, the Thevenin technique is used in the model. [91] An energy balance is used in the temperature model to determine the battery temperature.

3.4.3 Battery Model Integration

For instance, the BMS HIL system's DS1006 processor (dSpace) board can calculate the battery model in real-time when combined with models of electric drives and combustion engines, such as the ASM Engine. This enables the BMS to be tested in common hybrid drivetrain conditions. The drivetrain model can be tied to vehicle dynamics and environment simulation, such as ASM Traffic,

to verify GPS and map-based control methods for the drivetrain. In the test system, these model configurations can be calculated in real-time. While accurate emulators and sensors are employed, the current work considers the electrical, thermal, aging, and control subsystems primarily independently and does not yet correlate and validate the closed-loop interactions among them; however, multi-physics correlation and pack-level closed-loop validation are specifically planned as part of the future work scope.

3.4.5 System Setup

Several interconnected cell-simulation models are required to replicate the behavior of an advanced electric vehicle battery. Battery emulators are the heart and soul of BMS HIL. It consists of the modules listed below, as shown in Figure 18. They are mentioned below-

- Communication module;
- FIU module; Cell voltage module;
- Battery temperature module;
- Balancing the current module;

A battery pack with up to 200 cells can be simulated by connecting multiple emulators in series. The battery emulator is specifically made to mimic battery cells and has 24 series-connected voltage output channels. The emulator also includes fault injection units and channels that simulate temperature. The real-time simulator, as depicted in Fig. 19, uses the CAN bus to control the battery emulator. It receives readback voltage and current data from the battery emulator, which also receives set points for cell temperatures, voltages, and FIU states.

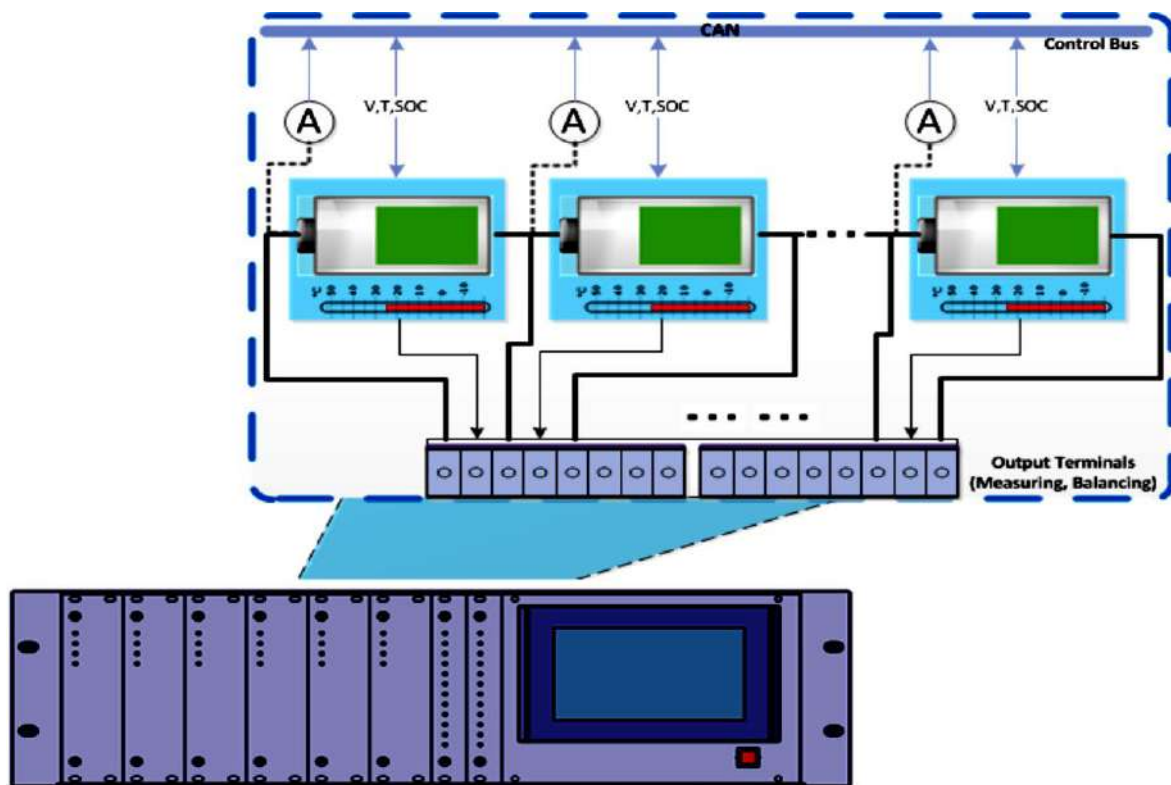


Fig. 18. Battery Emulator Structure [92]

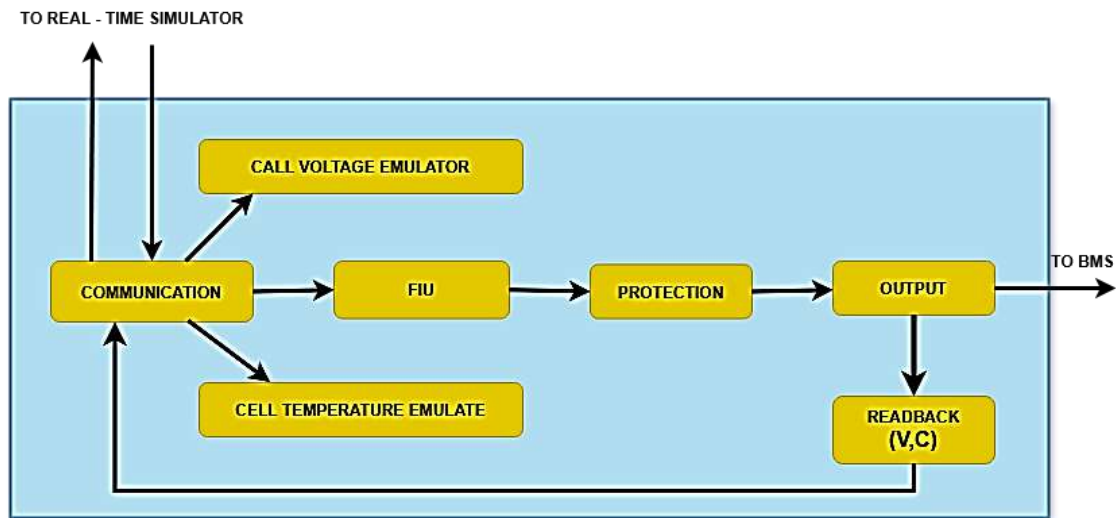


Fig. 19 Structure of the battery emulator [93-94]

3.6 Implementation

System engineering defines NFRs as specific criteria used to analyze a system's functioning, whereas FRs are defined as specific behaviors of a system's functions. Broadly speaking, NFRs specify how a specific system should function and behave, while FRs specify what a certain system should accomplish. ISO 26262 provides further information regarding functional safety requirements. Before doing HIL testing on any ECUs, it is necessary to have classified test cases. Based on these, we will execute the test and conclude.

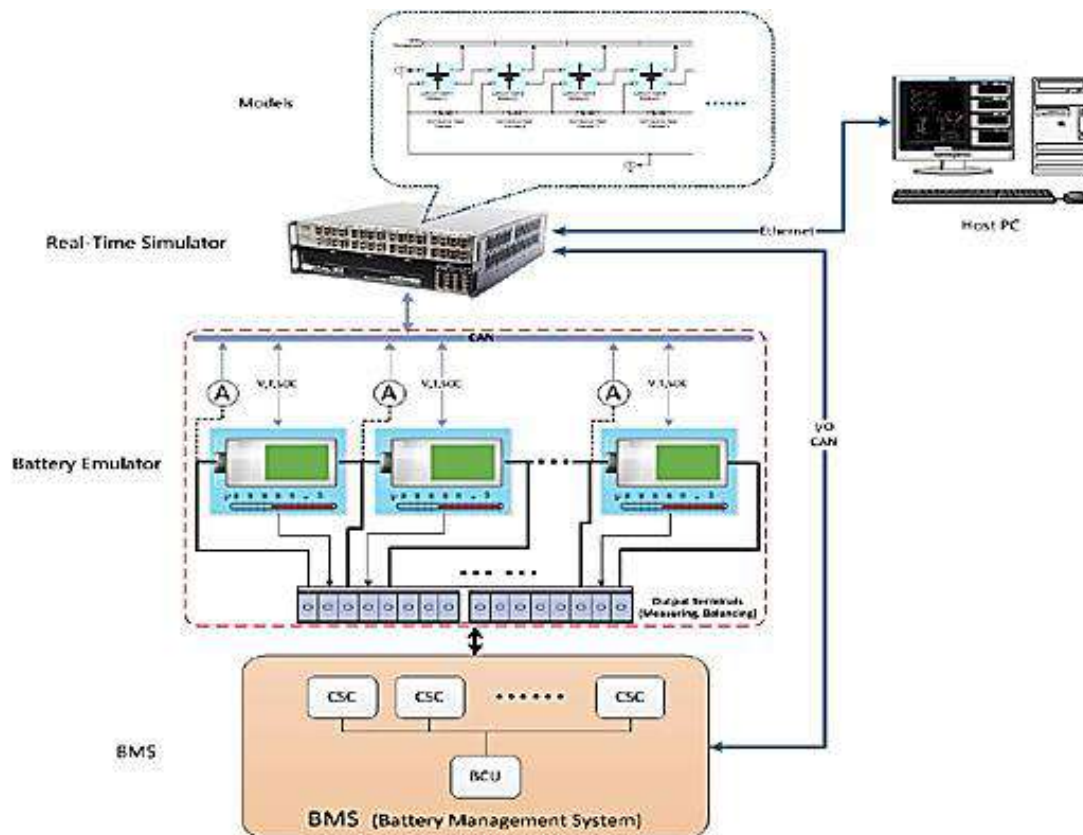


Fig. 20. Real-time Simulator OP5600 [95]

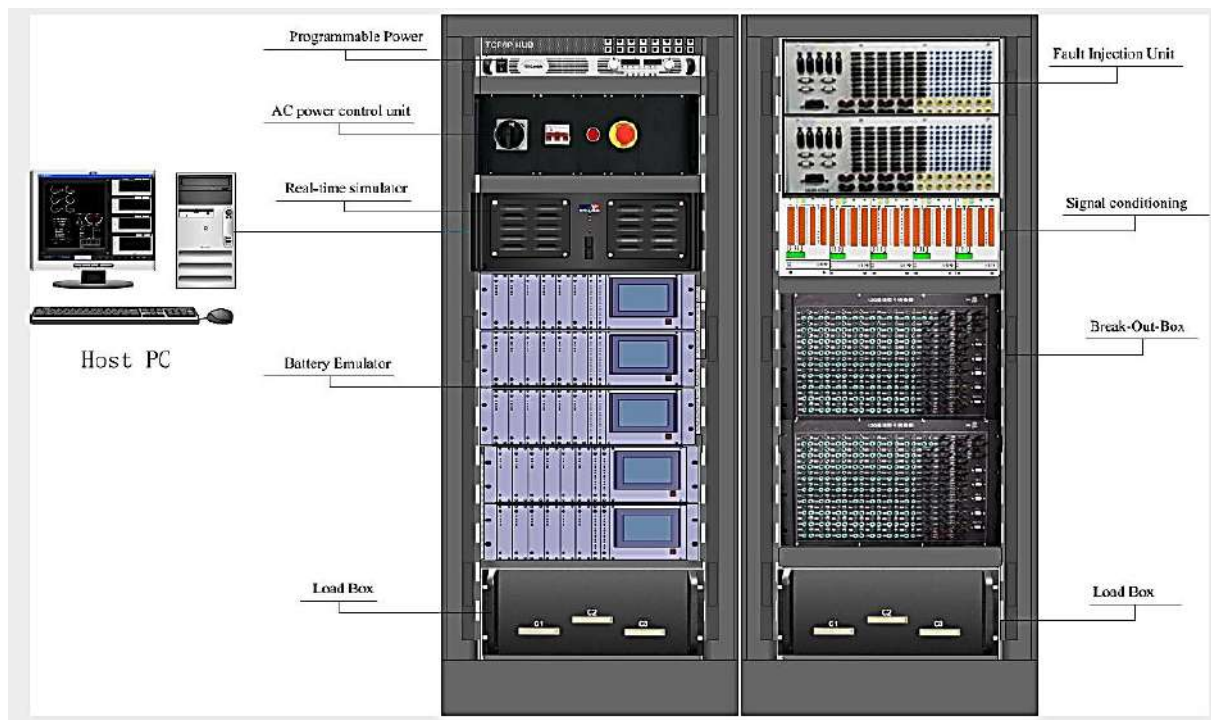


Fig. 21. HIL system hardware connection [95]

The diagram in Figure 21 displays the hardware connection for the HIL setup. Through Ethernet, bidirectional communication is established between the host PC and the real-time simulator. The real-time simulator utilizes the CAN bus to interface with the battery emulator and the I/Os, as well as the CAN bus, to communicate with the BMS central ECU. The battery emulator is directly connected to the local ECU or BMM. The BMS under test (DUT) comprises multiple BMMs and a central ECU. BMMs are modularized components that track temperature, current, and battery cell voltage. The BMMs are managed by the central ECU via the real-time CAN bus. Furthermore, the CECU manages temperature, fault handling, charge/discharge control, and communication with the VCU in addition to completing SOC estimates.

The operating conditions are specified before testing begins, such as the charging or city driving processes that generate various charging sources or battery-draining loads. The IO, CAN, and battery emulator outputs are updated in real-time, based on the operational state. The BMS examines every state and adapts to any modifications. The real-time simulator receives the BMS's data and readback. After that, they are regularly updated and contrasted with the actual values. To wrap up the test cases, the test results are achieved after completing the necessary iterations. Multi-cell and pack-level SOH estimation was not covered, as the current study is only a review and HIL validation, with its scope primarily on cell/module-level ideas and platform feasibility. Comprehensive pack-level SOH modeling, estimation, and scalability studies are extensive and require large multi-cell data, integrated thermal and aging models, and large-scale embedded system implementation, which are outside the current study's scope. These are proposed for future work, including pack-level SOH fusion, weak-cell identification, imbalance-aware aging modeling, and scalable, real-time implementation for EV-grade BMS.

Table 2. Test cases suggested for testing a BMS for its Functional Requirements (FRs) and Non-Functional Requirements (NFRs) [92-103]

No	Test Case	Description	End-of-test Criteria
1	Idle or stand-by	System settings are controlled. The fault criteria are defined in the BMS. The next functions are examined in steady-state: data management, monitoring, hazard detection, and hazard response.	All functions tested in steady state

2	Temperature, Current, and Voltage sensing	All sensors are calibrated, if possible. Then, full-range accuracy tests are performed while keeping the BMS under various conditions, e.g., in a temperature chamber.	All conditions tested
3	Dynamic discharge	By employing a real dynamic discharge profile of an EV, the pack is discharged to room temperature.	BMS stops the discharge to avoid over-discharge
4	Over-temperature during discharge	Applying a real dynamic discharge profile of an EV, the pack is discharged at high temperature.	The pack reaches maximum temperature, and the BMS stops the discharge.
5	Overvoltage during regenerative braking	Employing a real dynamic discharge profile of an EV, the pack is completely discharged when a very high regenerative current is encountered.	The BMS interrupts the regenerative charging current.
6	Short Circuit	Shorts at different places in the pack. Case I: internal or external short close to cell tabs. Case II: external short through fuses or shunt resistor, Case III: External short through fuse and switch box (relays or fuses)	The short circuit current is zero.
7	CCCV Charge	Conventional CCCV charge with active/passive balancing.	End of charge
8	Charge test at low temperatures	The charge is allowed, but the pack temperature is below the charging limit. Temperature rises gradually due to the heating system.	Pack temperatures over the threshold, and charging starts
9	Diagnosis	Case I: Simulated SOC vs. BMS-approximated SOC throughout a real dynamic discharge profile from an EV. Case n: New cases based on the BMS diagnosis features.	End of charge and discharge
10	Isolation Monitor	Single isolation failure on the positive/negative terminal of a floating voltage pack	Isolation fault detected
11	Global power consumption	Case I: Full discharge, taking into account a dynamic discharge profile from an EV, and subsequent CCCV charge, Case II: Stand-by test for a period of time.	BMS power consumption is measured under all conditions

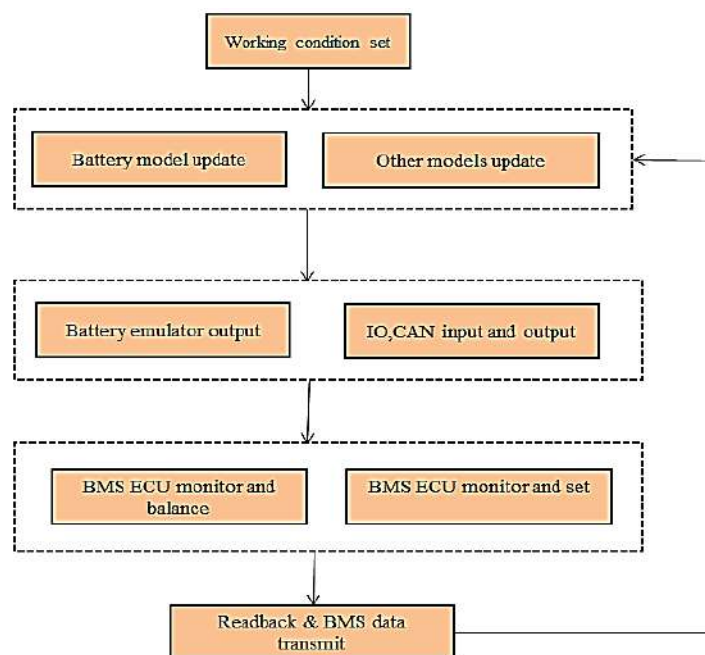


Fig. 22. Working process of BMS testing

4. Results and Discussion

This section displays the experimental findings of the BMS's communication with the system-level HIL-simulated battery. Regarding test case 2 (Table I), Figure 23 provides details about the calibration of the pack current sensor. Calibrating the sensor is one of the first steps in evaluating the accuracy of a BMS product. However, for two arbitrary sensing lines in the BMS, the figure shows exemplary data on the accuracy of cell voltage sensors at various temperatures and voltage levels.

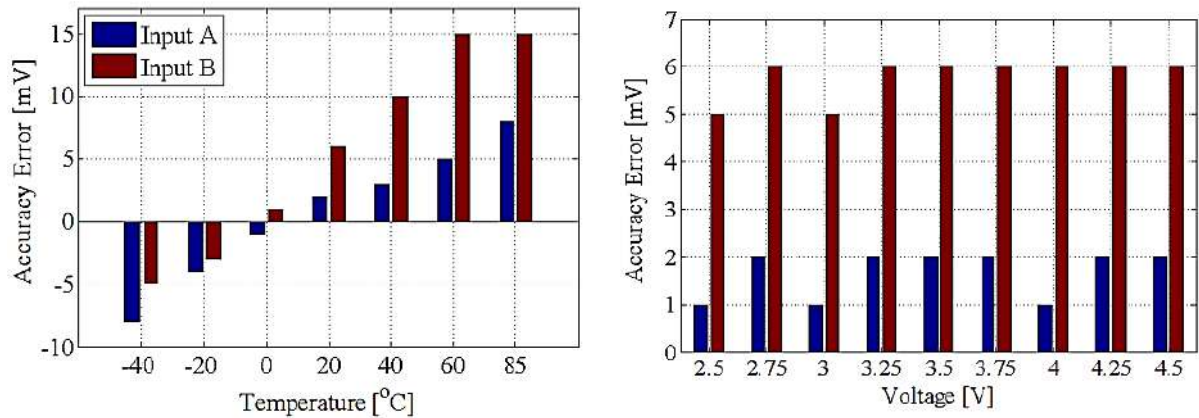
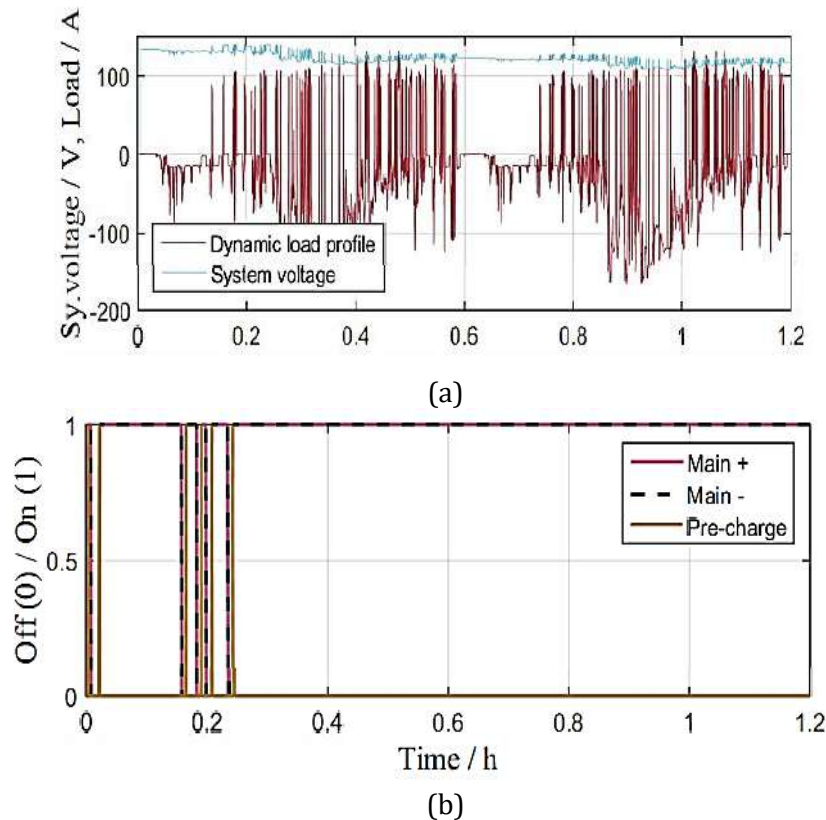
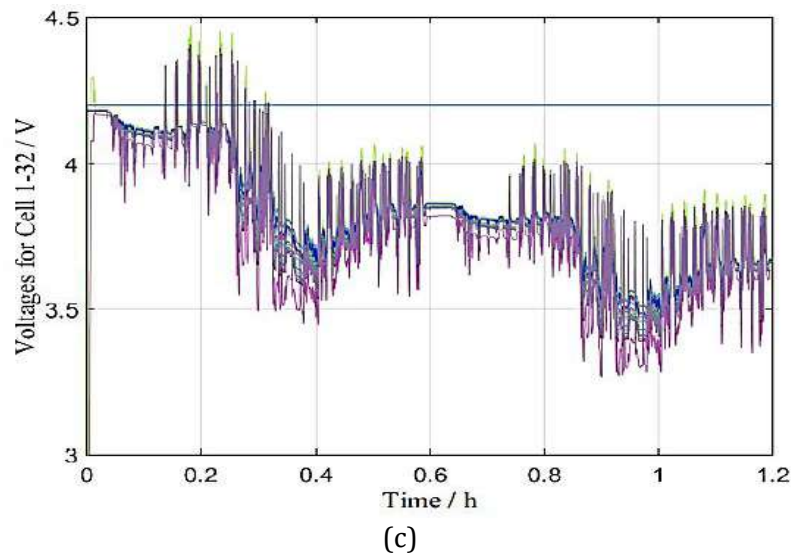


Fig. 23. Test case 2 revealing cell voltage sensing accuracy, (a) Cell voltage measurement accuracy at various temperatures and (b) a) Cell voltage measurement accuracy at various voltages [95]

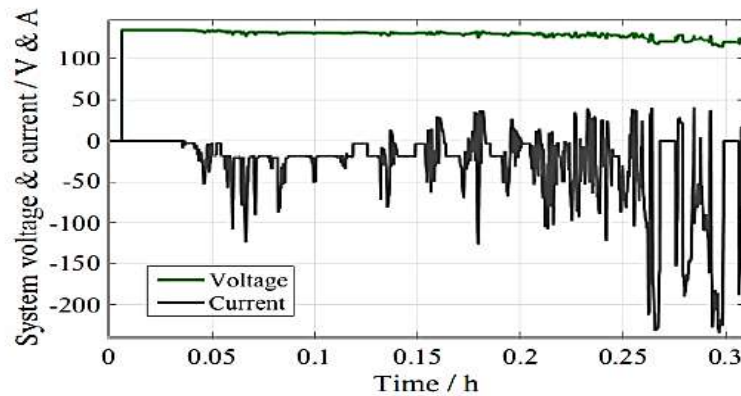
Figure 24 illustrates the plotted results of an overvoltage that occurred during regenerative braking, corresponding to test case 5 (Table I). A robust regeneration current at high SOC levels may cause an overvoltage in specific battery cells. To stop the excessive inrush current, the BMS recognizes the hazard and opens the switch box's relays. It is possible to design the BMS to close the relays and resume discharge mode operation after a predetermined amount of time.



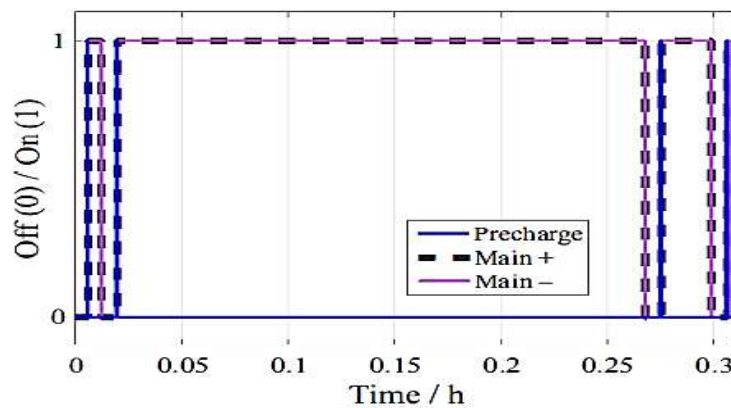


(c)
Fig. 24. Test case 5: Overvoltage during regenerative braking (a) Pack voltage and pack current in time (b) Switch box contactor logic and (c) Cell voltages in time and max. voltage threshold [95]

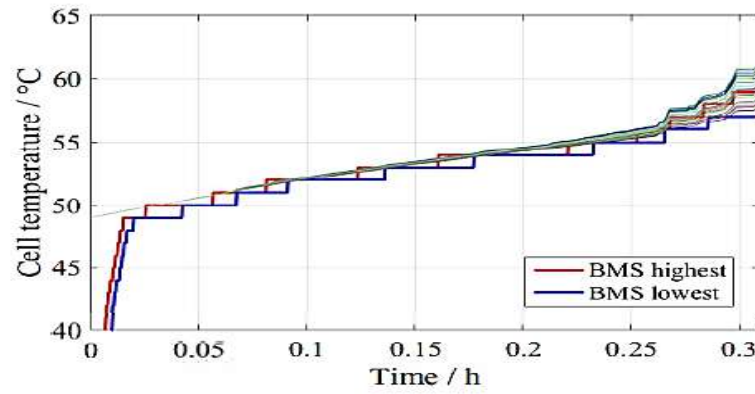
Test case 4 (Table I), which involves over-temperature during discharge, is illustrated in Fig. 25. For this test, predetermined high starting temperatures and constrained cooling capacities are utilized. As a result, during discharge, the cells in the pack overheat. Once more, the BMS recognizes the danger and stops the discharge by opening the relays in the switch box. The BMS can be set to shut the relays and resume operating in discharge mode after a predetermined amount of time once the issue has been resolved.



(a)



(b)



(c)

Fig. 25. Test case 4: over temperature during discharge (a) Pack voltage and pack current in time (b) Switch box contactor logic and (c) Experimental vs BMS measured cell temperature [95]

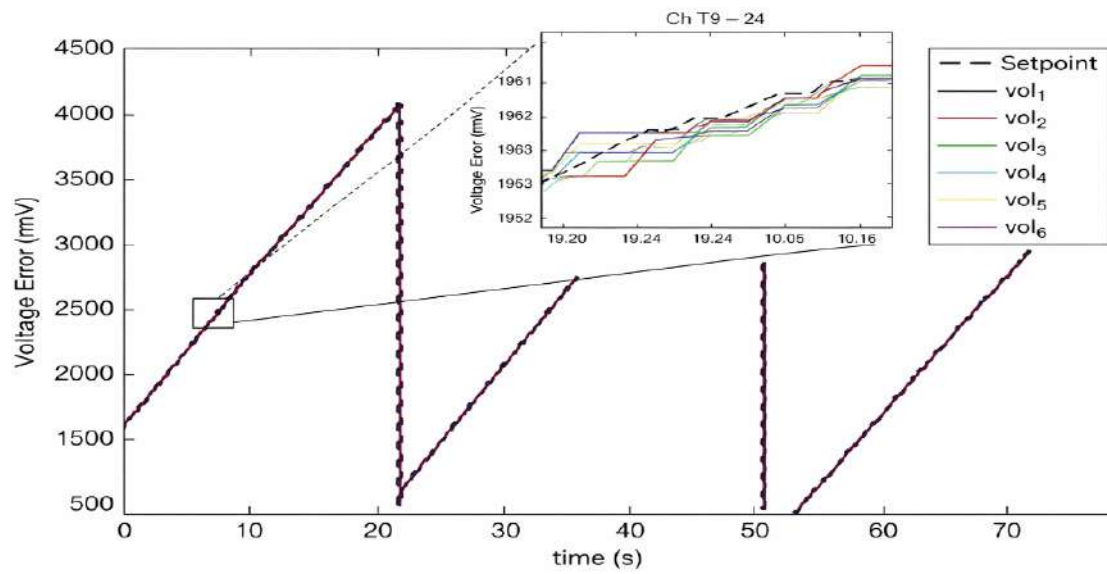


Fig. 26. Output accuracy of battery emulator [80]

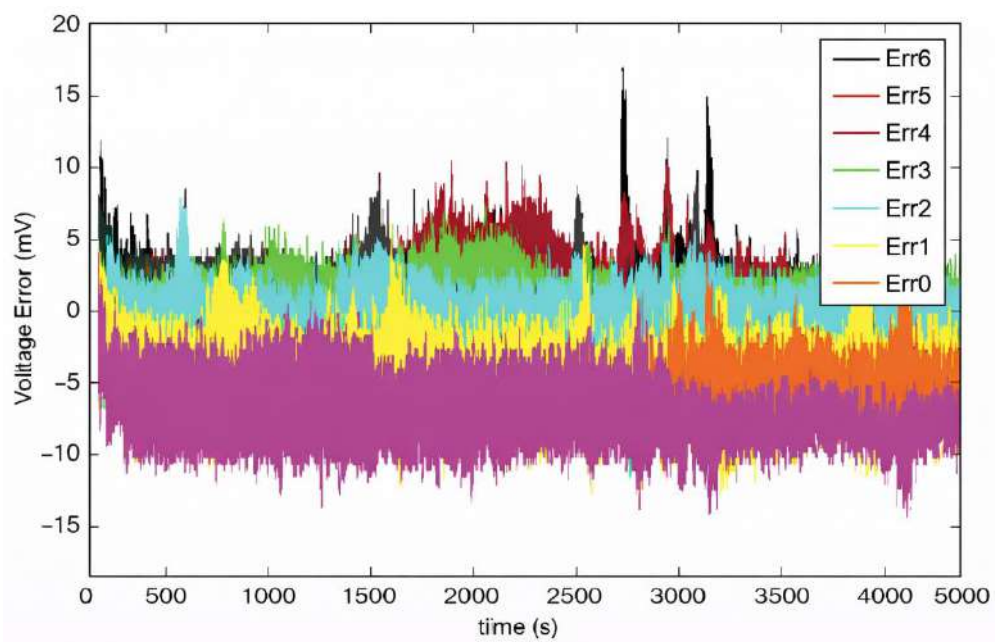


Fig. 27. Voltage monitoring error of BMS [80]

The figure presents the system behavior during operation in three subplots. Figure (a) shows the system voltage and current over time, where the voltage remains relatively stable while the current exhibits fluctuations and negative peaks due to load variations. Figure (b) illustrates the switching states of the pre-charge, main positive, and main negative contactors, indicating their on/off operation during the process. Figure (c) shows the cell temperature evolution, where both the highest and lowest cell temperatures gradually increase from about 40 °C to around 60 °C, demonstrating the thermal response of the battery pack during operation.

Figure 26 shows the voltage error response of six voltage channels (vol_1 – vol_6) compared with a reference setpoint over 0–70 s. The setpoint follows a repeating ramp pattern, and all channels closely track it with minimal deviation. The inset (19.20–19.24 s) highlights millivolt-level tracking accuracy, indicating stable and high-performance voltage control or estimation. The output of the battery emulator is compared with the set point in Fig. 26. As shown, the outputs of all six channels provide very accurate results, with errors of less than ± 2 mV. This indicates that the battery emulator's readback and output are reliable and represent the actual value.

Figure 27 shows the voltage error variation of six signals (Err_1 – Err_6) over 0–5000 s. Most errors remain within -10 mV to $+10$ mV, indicating stable and accurate voltage estimation. Occasional peaks around 2500–3200 s appear due to transient disturbances. The voltage-monitoring error of the BMMs (LECU) under test is shown in Fig. 27, along with a comparison to the battery emulator's readback value, which remains approximately 10 mV.

5. Conclusions

The work focuses on foundational modelling and framework development for estimating SOC, SOH, and SOP, rather than introducing new estimation algorithms. Key contributions include the establishment of accurate battery behavior models essential for any estimation approach, and a validated simulation framework for performance evaluation under realistic conditions. It also identifies practical limitations and sensitivities of SOC/SOH/SOP indicators to measurement noise and operating conditions, informing future estimator designs. Furthermore, it specifies the necessary inputs and metrics for applying advanced methods, such as EKF/UKF and neural networks, in future development, thereby creating a reliable baseline for subsequent SOC, SOH, and SOP estimation algorithms. It summarizes and discusses current approaches and their validation in a real-time HIL environment using OPAL-RT/RT-LAB, rather than creating new algorithms for SOC/SOH estimation, thermal management, or AI-based diagnosis. The manuscript offers an implementation-focused perspective on BMS, linking recent algorithms and architectures to HIL validation. It highlights new insights into testability, real-time capability, and prototyping feasibility, addressing the gap between simulation-based research and practical EV BMS development not previously emphasized in reviews from 2019 to 2024. This study contrasts several important BMS technology features with the benefits of Li-ion batteries and other ESS. Its long life, high energy and power density, and nominal voltage make the Li-ion battery a highly recommended choice for electric vehicle applications. However, lowering capital costs remains a significant obstacle. This review also thoroughly categorized the best charging topologies, thermal management, and SOH and SOC estimations for Li-ion batteries. As they become available in the literature, several algorithms and technologies have been presented. Upcoming difficulties have also been highlighted. Consequently, to address these challenging problems, it is necessary to establish confidence intervals for the methods and algorithms under consideration. Afterward, RT-LAB is used to verify the BMS by interfacing with the HIL system. The HIL setup can model up to N cells/modules in real time on the target hardware, adhering to a simulation time step of $\Delta t = 1$ ms. Beyond this limit, real-time constraints are not satisfied. The minimum stable real-time step for the model is $\Delta t_{\min} = 0.5$ ms.

A general discussion of testing definitions and aims was presented, emphasizing the fact that there is a wide range of opinions. Various test cases have been suggested to verify the dependability of the system. Since it is not always possible to verify every scenario that theoretical techniques provide, some essential elements of the system are often preferred as theoretical frameworks. On a commercial HIL simulator, an effective approach to generating SW and testing techniques for BMS

at the system level was demonstrated. Extensive experimental data were collected after comprehensive test cases were suggested. Different scenarios were shown to illustrate the interactions between the real-time simulator and BMS. The conclusion lacks explicit identification of technical limitations, applicability scope, or future research directions for a comprehensive BMS framework, which are anticipated to be addressed in future work. The conclusion lacks explicit identification of technical limitations, applicability scope, or future research directions for a comprehensive BMS framework, which are anticipated to be addressed in future work. The research does not include a validation or benchmarking framework, focusing instead on reviewing existing methodologies and showcasing the feasibility of HIL simulation. It lacks systematic performance comparisons under EV drive cycles, benchmarking against reference models, validation with experimental baselines, and evaluation using recognized performance metrics.

5.1 Limitations of the Method

The manuscript offers an overview of BMS research and HIL platform significance for validation but inadequately discusses practical implementation methods for real-time embedded systems. It fails to address essential aspects like microcontroller computational complexity, real-time capabilities of SOC/SOH estimation algorithms, stability under varying EV loads, and sensor noise considerations. Key elements such as sampling rate, memory and power usage, task scheduling, communication delays, and robustness to hardware issues are also overlooked, raising concerns about the transition from simulation to real-time applications in vehicles and the technology's readiness.

- **Model Accuracy:** The real-time battery model may not capture all complex electrochemical phenomena and non-linear behaviors, particularly under extreme operating conditions or advanced aging scenarios.
- **Hardware Constraints:** Opal-RT platform computational limitations may restrict model complexity and resolution, while processing delays could introduce timing inaccuracies not present in actual battery systems.
- **Limited Validation Scope:** Testing appears focused on standard operating conditions, potentially excluding extreme scenarios like thermal runaway, and may not encompass the full range of Li-ion chemistries or environmental conditions.
- **Scalability Uncertainty:** While described as scalable, real-time performance with large EV battery packs (hundreds of cells) remains unclear, with computational requirements potentially increasing exponentially.
- **BMS Function Coverage:** The study may not address all advanced BMS functions, such as predictive maintenance, adaptive algorithms, or cybersecurity features, limiting comprehensive system validation.
- **Experimental Data Limitations:** Model characterization may be restricted to specific battery types and conditions, with measurement uncertainties potentially affecting validation accuracy.
- **Industry Application Gap:** Results may not translate directly to all commercial BMS platforms due to hardware/software differences, and cost-benefit analysis versus traditional testing is not addressed.
- **Technology Evolution:** Rapid battery technology advancement may quickly outdate models, while emerging technologies like solid-state batteries may require entirely different approaches.
- **Benchmarking Deficiency:** Limited comparison with other HIL platforms or real-world testing results makes it difficult to quantify simulation accuracy and effectiveness.
- **Long-term Validation:** Insufficient long-term degradation data may affect the accuracy of state-of-health and remaining useful life estimations critical for BMS performance.
- **Advanced Technology:** AI can be implemented to increase the efficiency of the BMS
- **Maintenance:** Improper maintenance of the BMS may degrade the efficiency of the usage
- **Space Requirement:** The requirement for space can be optimized to increase its efficiency

5.2 Future Prospectives

Some prospects of Li-ion battery technologies are mentioned below.

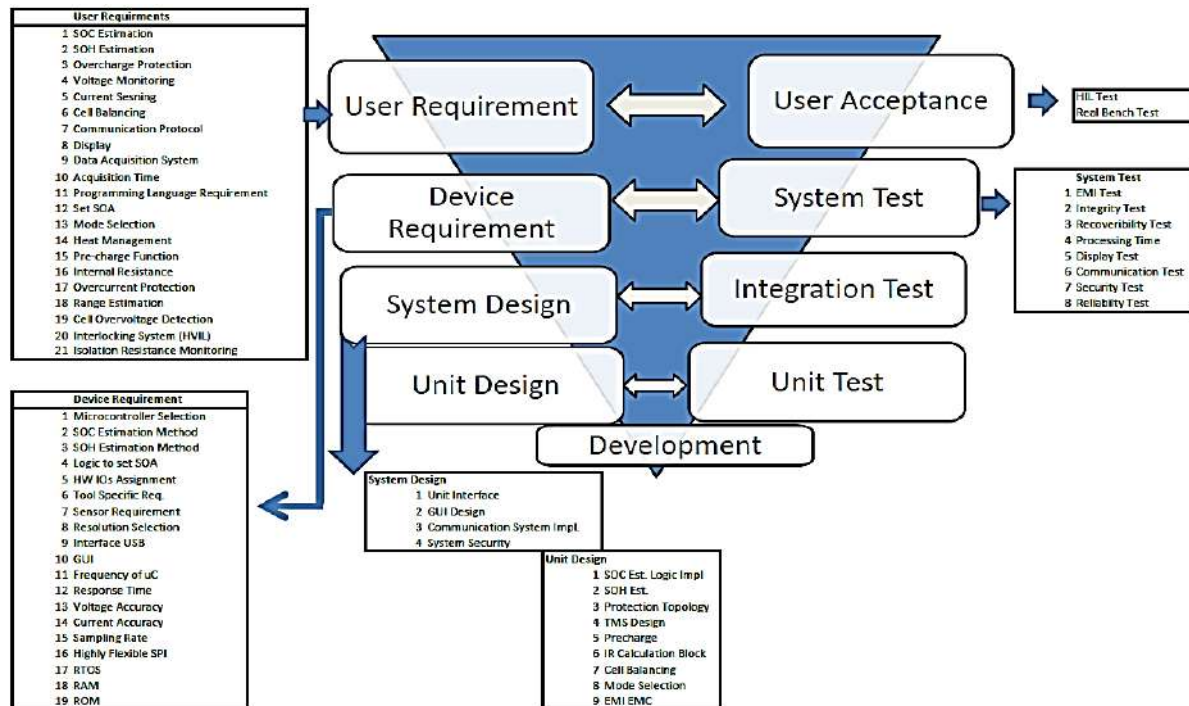
- There should be some efficiency improvements, Depth of Discharge (DOD), and energy density of Li-ion batteries. The main concern is the increase in capital cost.
- The RC-structured ECM estimation could be more precise using some complex mathematical models.
- Most of the research of the recent era has been conducted in a lab environment. But the actual scenario could differ if we run the tests under different environmental conditions, such as snowy, humid, hot, or cold.
- Errors in SOC or SOH estimations could be lessened with further research.
- The different algorithms used in battery models could be more optimized.
- Most of the time, current sensors are corrupted by measurement noise.
- Real-time SOH estimation is a big problem.

The current study lacks integration of advanced methods and does not evaluate their feasibility for embedded systems. It fails to address critical questions regarding data requirements, training/validation processes, generalization for various cells and temperatures, computational and memory demands on ECUs, real-time inference delays, interpretability, and safety certification. Consequently, the manuscript primarily serves as a survey or HIL demonstration, not demonstrating how AI or hybrid BMS approaches can be embedded and validated for electric vehicle-grade implementation.

While the proposed HIL development methodology can be applied to commercial BMS hardware, its suitability for production-grade automotive systems remains unproven. It lacks explicit consideration of automotive communication protocols and various safety and validation standards, including ISO 26262, ASPICE, ISO 12405, and SAE J2464. Consequently, the framework is perceived as more research-oriented than validated for production use.

6. Appendix

V-Model Development of BMS: A Case Study



BMS Generic HIL Test Cases

Sl. No.	Test Case	Description of Test	Criteria for Test Completion
1	Wake Up	Wake-up signal is sent to BMS, and receives confirmation from BMS to HIL	If firmly get a Rx signal from BMS
2	Check Sum	Every Signal should come from the BMS to the host PC via HIL communication buses.	Every signal code should be floated.
3	Idle or Standby	Making the system ready to use, but not connecting any load. The fault criteria are set in the BMS.	Readiness time was within the time limit, and no faults were detected.
4	Isolation Monitoring	Isolation resistance between the chassis and the battery pack should be within the limit. An external sliding resistor could be connected to create a manual situation.	If any leakage happens, BMS should send signals to VCU and take proper action according to ISO26262 (ASIL-C)
5	Battery CAN_H & CAN_L signals	CAN High and CAN Low should come from all BMMs to the BMS	Rx and Tx are coming properly.
6	V CAN_H & V CAN_L	Vehicle CAN communication between VCU and BMS	Rx and Tx are coming properly.
7	Cell Balancing	Active and passive balancing should be correct, and the time taken for balancing should match the predefined value.	Balancing is happening correctly.
8	Pre-charge	To charge the battery, the OCV of the battery should be equal to the charger's output voltage to avoid sparks and hazards. To achieve this, a capacitor, a resistor, and a relay-based circuit are employed. Testing the recharge operation.	Pre-charge circuitry is functioning properly, ensuring the safe operation of the relay.
9	HVIL Check	Due to an interruption in the current flow through the High Voltage Interlocking Loop, the vehicle should stop. Moreover, BMS should disconnect the battery from the vehicle system. (ASIL C)	Manually created a faulty situation with the help of an NC switch, and the BMS stops operation.
10	Temperature Sensing	A 12V temperature signal in all the BMMs from the temperature sensors, thus in the BMS via CAN	If Rx signals are coming into the BMS
Sl. No.	Test Case	Description of Test	Criteria for Test Completion
11	Temperature Cut Off	Based on temperature rises and dips, the BMS should take action by sending a proper signal to the Coolant tank. If the maximum temperature is reached, the BMS should disconnect the battery.	Proper coolant rate as predefined calculated rate and stop battery in overtemperature
12	Current Sensing	Current-sensing signals from all current sensors to the BMS.	Getting an accurate sensing signal.
13	Current Cut Off	Required charging and discharging currents should be measured. Beyond the specified limits for minimum and maximum currents, the BMS should take appropriate action.	If the maximum current is reached, it should stop draining the battery.
14	Voltage Sensing	Voltage-sensing signals from each cell should come to the BMS via the BMMs from the voltage sensors.	Getting all signal codes in the HIL host PC.
15	Voltage Cut Off	Overvoltage and undervoltage situations should be addressed by the BMS through appropriate action.	Successfully enabling the cut-off circuitry operation.
16	SOC Estimation	Are the estimated SOC and emulated SOC the same or not	If they are equal
17	CCCV Charge	Conventional CCCV charging happens with successful active/passive balancing.	End of the charge
18	Under and Overcharge	SOC levels typically range from 20% to 80%. The SOC limits for both cases should be within the same range.	Successfully enabled the protection circuitry and disengaged the battery for both cases.
19	Coolant Flow	Calibrated temperature rise Vs. Coolant flow rate should be maintained. Checking predefined flow rate Vs. temperature curve.	The curve is true for every operating point.
20	Handshaking with a charger	Before charging starts, proper handshaking should occur between the BMS and the CCU (Charger Control Unit) over a dedicated CAN channel.	Handshake successful and reached the ready-to-charge state.

21	Regenerative Braking	In the case of regenerative braking, the BMS should act accordingly.	Action taken is successful.
22	Data Acquisition	All data is stored in the BMS.	Data can be extracted and matched with predefined set points.
23	Diagnosis	Checking whether any fault happens or not through the OBD connector and the handle given in the BMS. Diagnosis happens according to ISO14229.	Successfully diagnosed.

References

- [1] Bang H, Dave A, Tzortoglou FN, Wang S, Malikopoulos AA. On Mobility Equity and the Promise of Emerging Transportation Systems. *IEEE Trans Intell Transp Syst.* 2025;26(7):10623-10637. <https://doi.org/10.1109/TITS.2025.3548885>
- [2] Saeed A, Alawi A, Sharqawy MH, Al Janaideh M. Advancements in Temperature-Sensing Technologies for Lithium-Ion Batteries in Electric Vehicle Thermal Management Systems: A Comprehensive Review. *IEEE Sens J.* 2025;25(23):42442-42461. <https://doi.org/10.1109/ISEN.2025.3616085>
- [3] Hu B, Ioannou PA. Survey of Electric Truck Technologies and Infrastructure Support. *IEEE Intell Transp Syst Mag.* 2025;17(5):72-90. <https://doi.org/10.1109/ITS.2025.3573644>
- [4] Yadav IC, Bajpai RS, Gupta S, et al. Hybrid fuzzy-MPC based multi-objective control strategy for fast charging of electric vehicles with advanced battery thermal management and renewable grid support. *Sci Rep.* 2025;15:42342. <https://doi.org/10.1038/s41598-025-26279-4>
- [5] Chen J, Zhang X, Zhou Y, Chen G, Zhai Q, Cao Y. Optimal Joint Scheduling of Distribution Network and Electric Vehicle Tracks Considering Actual Carbon Emission Responsibility. *IEEE Trans Transp Electrif.* 2025;11(5):11075-11087. <https://doi.org/10.1109/TTE.2025.3570747>
- [6] Roy S, Majumder S, De K, Ahmad F, Boumaiza A. Electric vehicles and environmental sustainability: Integrating material, energy, and emission flows in a system-level LCA. *Sustain Dev.* 2026;34(1):851-872. <https://doi.org/10.1002/sd.70269>
- [7] Taylor W, Krithivasan G, Nelson JJ. System safety and ISO 26262 compliance for automotive lithium-ion batteries. In: *Proceedings of the 2012 IEEE Symposium on Product Compliance Engineering*; 2012 Nov 5-7; Portland, OR, USA. p. 1-6. <https://doi.org/10.1109/ISPC.2012.6398297>
- [8] Hussein HH, Batarseh I. A review of charging algorithms for nickel and lithium battery chargers. *IEEE Trans Veh Technol.* 2011;60:830-838. <https://doi.org/10.1109/TVT.2011.2106527>
- [9] Dey S, Mohon S, Pisu P, Ayalew B. Sensor fault detection, isolation, and estimation in lithium-ion batteries. *IEEE Trans Control Syst Technol.* 2016;24:2141-2149. <https://doi.org/10.1109/TCST.2016.2538200>
- [10] Kang Y, Duan B, Zhou Z, Shang Y, Zhang C. Online multi-fault detection and diagnosis for battery packs in electric vehicles. *Appl Energy.* 2020;259:114170. <https://doi.org/10.1016/j.apenergy.2019.114170>
- [11] Wang Z, Hong J, Liu P, Zhang L. Voltage fault diagnosis and prognosis of battery systems based on entropy and Z-score for electric vehicles. *Appl Energy.* 2017;196:289-302. <https://doi.org/10.1016/j.apenergy.2016.12.143>
- [12] Dutta S, Majumder S, De K, et al. Electric vehicle battery recycling for sustainability: environmental and water benefits. *Clean Techn Environ Policy.* 2026;28:28. <https://doi.org/10.1007/s10098-025-03402-7>
- [13] Padmaraj O, Satyanarayana N, Suthanthiraraj SA, Venkateswaran C. Issues and challenges of rechargeable lithium batteries. *Rechargeable Lithium-Ion Batteries.* 2020:38-98. <https://doi.org/10.1201/9781351052702-2>
- [14] Sethi P, Pathak M, Sharma PK, Saxena V, Dixit SK. A comprehensive research paper on lithium-ion batteries: Advances and challenges of battery management systems (BMS). Vol 1, Issue 4. 2025.
- [15] Nekahi A, Madikere Raghunatha Reddy AK, Li X, et al. Rechargeable Batteries for the Electrification of Society: Past, Present, and Future. *Electrochem Energy Rev.* 2025;8:1. <https://doi.org/10.1007/s41918-024-00235-8>
- [16] Yap JW, Lee M, El Idrissi F, Ramesh P, Yoon DY, Kang CY, et al. Unveiling real-world aging mechanisms of lithium-ion batteries in electric vehicles. *J Energy Storage.* 2025;130:117420. <https://doi.org/10.1016/j.est.2025.117420>
- [17] Madani SS, Shabeer Y, Allard F, Fowler M, Ziebert C, Wang Z, et al. A Comprehensive Review on Lithium-Ion Battery Lifetime Prediction and Aging Mechanism Analysis. *Batteries.* 2025;11:127. <https://doi.org/10.3390/batteries11040127>
- [18] Toprak MM, Kok C, Mert SO. Analysis of various flow field designs for PEM fuel cells used in vehicular systems through 3D modelling. *Res Eng Struct Mater.* 2022;8(1):179-203. <http://dx.doi.org/10.17515/resm2021.328en0805>
- [19] Lozano Ruiz P, Damianakis N, Ram Chandra Mouli G. Physics-Based and Data-Driven Modeling of Degradation Mechanisms for Lithium-Ion Batteries-A Review. *IEEE Access.* 2025;13:21164-21189. <https://doi.org/10.1109/ACCESS.2025.3535918>

- [20] Yardimci AI, Acikbas Y, Capan R. SEM images of carbon nanotubes (CNTs): Effect of voltage and spot size on image resolutions. *Res Des*. 2024;1(1):43-51. <https://doi.org/10.17515/rede2024.004en1116rs>
- [21] Ma X, Meng Z, Bellonia MV, et al. The evolution of lithium-ion battery recycling. *Nat Rev Clean Technol*. 2025;1:75-94. <https://doi.org/10.1038/s44359-024-00010-4>
- [22] Meng W. Recycling technologies of spent lithium-ion batteries and future directions: A review. *Trans Nonferrous Met Soc China*. 2025;35(1):271-295. [https://doi.org/10.1016/S1003-6326\(24\)66679-3](https://doi.org/10.1016/S1003-6326(24)66679-3)
- [23] Coskun T, Çetkin E. Vascularized mini cooling channels to achieve temperature uniformity: Battery thermal management and electronic cooling. *Res Eng Struct Mater*. 2023;9(3):675-685. <http://dx.doi.org/10.17515/resm2022.585ma1121>
- [24] Ralon P, Taylor M, Ilas A, Diaz-Bone H, Kairies K. Electricity storage and renewables: Costs and markets to 2030. Abu Dhabi: International Renewable Energy Agency; 2017.
- [25] Westbrook MH. The electric car: Development and future of battery, hybrid and fuel-cell cars. Stevenage: IET; 2001. <https://doi.org/10.1049/PBPO038E>
- [26] Hu X, Zou C, Zhang C, Li Y. Technological developments in batteries: A survey of principal roles, types, and management needs. *IEEE Power Energy Mag*. 2017;15:20-31. <https://doi.org/10.1109/MPE.2017.2708812>
- [27] Ali MU, Zafar A, Nengroo SH, Hussain S, Alvi MJ, Kim HJ. Review towards a Smarter Battery Management System for Electric Vehicle Applications: A Critical Review of Lithium-ion Battery State of Charge Estimation. *Energies*. 2019;12:446. <https://doi.org/10.3390/en12030446>
- [28] Liu K, Li K, Peng Q, Zhang C. A brief review on key technologies in the battery management system of electric vehicles. Springer; 2018. p. 1-18. <https://doi.org/10.1007/s11465-018-0516-8>
- [29] Ali MU, Zafar A, Nengroo SH, Hussain S, Alvi MJ, Kim HJ. Review towards a smarter Battery Management System for Electric Vehicle Applications: A Critical Review of Lithium-ion Battery State of Charge Estimation. *Energies*. 2019;12:446. <https://doi.org/10.3390/en12030446>
- [30] Xiong R, Tian J, Mu H, Wang C. A systematic model-based degradation behaviour recognition and health monitoring method for lithium-ion batteries. *Appl Energy*. 2017;207:372-383. <https://doi.org/10.1016/j.apenergy.2017.05.124>
- [31] Lam L, Bauer P. Practical capacity fading model for Li-ion battery cells in electric vehicles. *IEEE Trans Power Electron*. 2013;28:5910-5918. <https://doi.org/10.1109/TPEL.2012.2235083>
- [32] Eddahech A, Briat O, Bertrand N, Deléage JY, Vinassa JM. Behavior and state-of-health monitoring of Li-ion batteries using impedance spectroscopy and recurrent neural networks. *Int J Electr Power Energy Syst*. 2012;42:487-494. <https://doi.org/10.1016/j.ijepes.2012.04.050>
- [33] Borahel RDS, Telli GD, Bolzoni AA, Isoldi LA, dos Santos ED, Luiz Rocha AO. Numerical analysis of the impact of PCM reservoir width on lithium-ion battery cooling: A constructal design study. *Res Eng Struct Mater*. 2025;11(5):2421-2435. <https://doi.org/10.17515/resm2025-557en1202rs>
- [34] Feng X, Weng C, He X, Han X, Lu L, Ren D, et al. Online State-of-Health Estimation for Li-Ion Battery Using Partial Charging Segment Based on Support Vector Machine. *IEEE Trans Veh Technol*. 2019;68:8583-8592. <https://doi.org/10.1109/TVT.2019.2927120>
- [35] Deng Y, Ying H, E J, Zhu H, Wei K, Chen J, et al. Feature parameter extraction and intelligent estimation of the State-of-Health of lithium-ion batteries. *Energy*. 2019;176:91-102. <https://doi.org/10.1016/j.energy.2019.03.177>
- [36] Quin T, Zeng S, Guo J, Skaf Z. A Rest Time-Based Prognostic Framework for State of Health Estimation of Lithium-Ion Batteries with Regeneration Phenomena. *Energies*. 2016;9:896. <https://doi.org/10.3390/en9110896>
- [37] Cope RC, Podrazhansky Y. The art of battery charging. In: *Proceedings of the 14th Annual Battery Conference on Applications and Advances*; 1999 Jan 12-15; Long Beach, CA, USA. p. 233-235. <https://doi.org/10.1109/BCAA.1999.795996>
- [38] Ikeya T, Sawada N, Takagi S, Murakami J, Kobayashi K, Sakabe T, et al. Multi-step constant-current charging method for electric vehicle, valve-regulated, lead/acid batteries during night time for load-levelling. *J Power Sources*. 1998;75:101-107. [https://doi.org/10.1016/S0378-7753\(98\)00102-5](https://doi.org/10.1016/S0378-7753(98)00102-5)
- [39] Ikeya T, Sawada N, Murakami J, Kobayashi K, Hattori M, Murotani N, et al. Multi step constant-current charging method for an electric vehicle nickel/metal hydride battery with high-energy efficiency and long cycle life. *J Power Sources*. 2002;105:6-12. [https://doi.org/10.1016/S0378-7753\(01\)00907-7](https://doi.org/10.1016/S0378-7753(01)00907-7)
- [40] Liu Y, Luo Y. Search for an Optimal Rapid-Charging Pattern for Li-Ion Batteries Using the Taguchi Approach. *IEEE Trans Ind Electron*. 2010;57:3963-3971. <https://doi.org/10.1109/TIE.2009.2036020>
- [41] Vo TT, Chen X, Shen W, Kapoor A. New charging strategy for lithium-ion batteries based on the integration of Taguchi method and state of charge estimation. *J Power Sources*. 2015;273:413-422. <https://doi.org/10.1016/j.jpowsour.2014.09.108>
- [42] Notten P, het Veld JO, van Beek J. Boostcharging Li-ion batteries: A challenging new charging concept. *J Power Sources*. 2005;145:89-94. <https://doi.org/10.1016/j.jpowsour.2004.12.038>

- [43] Purushothaman B, Landau U. Rapid Charging of Lithium-Ion Batteries Using Pulsed Currents. *J Electrochem Soc.* 2006;153:A533-A542. <https://doi.org/10.1149/1.2161580>
- [44] Zhang J, Yu J, Cha C, Yang H. The effects of pulse charging on inner pressure and cycling characteristics of sealed Ni/MH batteries. *J Power Sources.* 2004;136:180-185. <https://doi.org/10.1016/j.jpowsour.2004.05.008>
- [45] Chen L. A Design of an Optimal Battery Pulse Charge System by Frequency-Variied Technique. *IEEE Trans Ind Electron.* 2007;54:398-405. <https://doi.org/10.1109/TIE.2006.888796>
- [46] Behara NSR, Rao PS. Vibration energy harvesting using telescopic suspension system for conventional two-wheeler and EV. *Res Eng Struct Mater.* 2023;9(3):687-707. <http://dx.doi.org/10.17515/resm2022.589me1212>
- [47] Chen Z, Shu X, Li X, Xiao R, Shen J. An optimal charging algorithm for lithium-ion batteries considering temperature rise minimization. In: *Chinese Automation Congress (CAC)*; 2017; Jinan, China. <https://doi.org/10.1109/CAC.2017.8243195>
- [48] Liu YH, Teng JH, Lin YC. Search for an optimal rapid charging pattern for lithium-ion batteries using ant colony system algorithm. *IEEE Trans Ind Electron.* 2005;52:1328-1336. <https://doi.org/10.1109/TIE.2005.855670>
- [49] Guo Z, Liaw BY, Qiu X, Gao L, Zhang C. Optimal charging method for lithium-ion batteries using a universal voltage protocol accommodating aging. *J Power Sources.* 2015;274:957-964. <https://doi.org/10.1016/j.jpowsour.2014.10.185>
- [50] Guo Z, Qiu X, Hou G, Liaw BY, Zhang C. State of health estimation for lithium-ion batteries based on charging curves. *J Power Sources.* 2014;249:457-462. <https://doi.org/10.1016/j.jpowsour.2013.10.114>
- [51] Hu X, Li S, Peng H, Sun F. Charging time and loss optimization for LiNMC and LiFePO4 batteries based on equivalent circuit models. *J Power Sources.* 2013;239:449-457. <https://doi.org/10.1016/j.jpowsour.2013.03.157>
- [52] Aliev R, Aliev R, Guirimov B, Uyar K. Dynamic data mining technique for rules extraction in a process of battery charging. *Appl Soft Comput.* 2008;8:1252-1258. <https://doi.org/10.1016/j.asoc.2007.02.015>
- [53] Guo G, Xu P, Bai Z, Zhou S, Xu G, Cao B. Optimization of Ni-MH Battery Fast Charging in Electric Vehicles Using Dynamic Data Mining and ANFIS. In: *Advanced Intelligent Computing Theories and Applications.* Berlin/Heidelberg: Springer; 2008. p. 468-475. https://doi.org/10.1007/978-3-540-85984-0_56
- [54] Petchjaturporn P, Khaehintung N, Sunat K, Sirisuk P, Kiranon W. Implementation of GA-trained GRNN for Intelligent Fast Charger for Ni-Cd Batteries. In: *Proceedings of the 2006 CES/IEEE 5th International Power Electronics and Motion Control Conference*; 2006 Aug 14-16; Shanghai, China. Vol 1, p. 1-5. <https://doi.org/10.1109/IPEMC.2006.4777963>
- [55] Chen L, Hsu RC, Liu C. A Design of a Grey-Predicted Li-Ion Battery Charge System. *IEEE Trans Ind Electron.* 2008;55:3692-3701. <https://doi.org/10.1109/TIE.2008.928106>
- [56] Richter F, Kjelstrup S, Vie PJ, et al. Thermal conductivity and internal temperature profiles of Li-ion secondary batteries. *J Power Sources.* 2017;359:592-600. <https://doi.org/10.1016/j.jpowsour.2017.05.045>
- [57] Dai H, Zhu L, Zhu J. Adaptive Kalman filtering based internal temperature estimation with an equivalent electrical network thermal model for hard-cased batteries. *J Power Sources.* 2015;293:351-365. <https://doi.org/10.1016/j.jpowsour.2015.05.087>
- [58] Raijmakers LH, Danilov DL, van Lammeren JP, et al. Non-zero intercept frequency: An accurate method to determine the integral temperature of Li-ion batteries. *IEEE Trans Ind Electron.* 2016;63(5):3168-3178. <https://doi.org/10.1109/TIE.2016.2516961>
- [59] Lee KT, Dai MJ, Chuang CC. Temperature-compensated model for lithium-ion polymer batteries with extended Kalman filter state of- charge estimation for an implantable charger. *IEEE Trans Ind Electron.* 2018;65(1):589-596. <https://doi.org/10.1109/TIE.2017.2721880>
- [60] Mehne J, Nowak W. Improving temperature predictions for Li-ion batteries: Data assimilation with a stochastic extension of a physically-based, thermo-electrochemical model. *J Energy Storage.* 2017;12:288-296. <https://doi.org/10.1016/j.est.2017.05.013>
- [61] Guo M, Kim GH, White RE. A three-dimensional multi-physics model for a Li-ion battery. *J Power Sources.* 2013;240:80-94. <https://doi.org/10.1016/j.jpowsour.2013.03.170>
- [62] Jeon DH, Baek SM. Thermal modeling of cylindrical lithium-ion battery during discharge cycle. *Energy Convers Manag.* 2011;52(8-9):2973-2981. <https://doi.org/10.1016/j.enconman.2011.04.013>
- [63] Jaguemont J, Omar N, Martel F, et al. Streamline three-dimensional thermal model of a lithium titanate pouch cell battery in extreme temperature conditions with module simulation. *J Power Sources.* 2017;367:24-33. <https://doi.org/10.1016/j.jpowsour.2017.09.028>
- [64] Lin X, Perez HE, Siegel JB. Online parameterization of lumped thermal dynamics in cylindrical lithium-ion batteries for core temperature estimation and health monitoring. *IEEE Trans Control Syst Technol.* 2013;21(5):1745-1755. <https://doi.org/10.1109/TCST.2012.2217143>

- [65] Shah K, Vishwakarma V, Jain A. Measurement of multiscale thermal transport phenomena in Li-ion cells: A review. *J Electrochem Energy Convers Storage*. 2016;13(3):030801. <https://doi.org/10.1115/1.4034413>
- [66] Chen D, Jiang J, Li X. Modelling of a pouch lithium-ion battery using a distributed parameter equivalent circuit for internal non-uniformity analysis. *Energies*. 2016;9(11):865. <https://doi.org/10.3390/en9110865>
- [67] Muratori M, Canova M, Guezennec Y. A reduced-order model for the thermal dynamics of Li-ion battery cells. *IFAC Proc Vol*. 2010;43(7):192-197. <https://doi.org/10.3182/20100712-3-DE-2013.00190>
- [68] Kim Y, Mohan S, Siegel SJ. The estimation of temperature distribution in cylindrical battery cells under unknown cooling conditions. *IEEE Trans Control Syst Technol*. 2014;22(6):2277-2286. <https://doi.org/10.1109/TCST.2014.2309492>
- [69] Hu X, Asgari S, Yavuz I. A transient reduced order model for battery thermal management based on singular value decomposition. In: *Proceedings of 2014 IEEE Energy Conversion Congress and Exposition (ECCE)*; 2014; Pittsburgh. p. 3971-3976. <https://doi.org/10.1109/ECCE.2014.6953941>
- [70] Lin C, Yu Q, Xiong R. A study on the impact of open circuit voltage tests on state of charge estimation for lithium-ion batteries. *Appl Energy*. 2017;205:892-902. <https://doi.org/10.1016/j.apenergy.2017.08.124>
- [71] Grandjean TR, McGordon A, Jennings PA. Structural identifiability of equivalent circuit models for Li-ion batteries. *Energies*. 2017;10(1):90. <https://doi.org/10.3390/en10010090>
- [72] Tang SX, Camacho-Solorio L, Wang Y. State-of-charge estimation from a thermal-electrochemical model of lithium-ion batteries. *Automatica*. 2017;83:206-219. <https://doi.org/10.1016/j.automatica.2017.06.030>
- [73] Li J, Wang L, Lyu C. State of charge estimation based on a simplified electrochemical model for a single LiCoO₂ battery and battery pack. *Energy*. 2017;133:572-583. <https://doi.org/10.1016/j.energy.2017.05.158>
- [74] Wang Y, Zhang C, Chen Z. On-line battery state-of-charge estimation based on an integrated estimator. *Appl Energy*. 2017;185:2026-2032. <https://doi.org/10.1016/j.apenergy.2015.09.015>
- [75] Acuña DE, Orchard ME. Particle-filtering-based failure prognosis via sigma-points: Application to lithium-ion battery state-of-charge monitoring. *Mech Syst Signal Process*. 2017;85:827-848. <https://doi.org/10.1016/j.ymssp.2016.08.029>
- [76] Zou C, Manzie C, Nešić D. Multi-time-scale observer design for state-of-charge and state-of-health of a lithium-ion battery. *J Power Sources*. 2016;335:121-130. <https://doi.org/10.1016/j.jpowsour.2016.10.040>
- [77] Xiong B, Zhao J, Su Y. State of charge estimation of vanadium redox flow battery based on sliding mode observer and dynamic model including capacity fading factor. *IEEE Trans Sustain Energy*. 2017;8(4):1658-1667. <https://doi.org/10.1109/TSTE.2017.2699288>
- [78] Ye M, Guo H, Cao B. A model-based adaptive state of charge estimator for a lithium-ion battery using an improved adaptive particle filter. *Appl Energy*. 2017;190:740-748. <https://doi.org/10.1016/j.apenergy.2016.12.133>
- [79] Gehring J, Schütte H. A Hardware-in-the-Loop Test Bench for the Validation of Complex ECU Networks. *SAE Technical Paper 2002-01-0801*; 2002. <https://doi.org/10.4271/2002-01-0801>
- [80] Schütte H, Wältermann P. Hardware-in-the-Loop Testing of Vehicle Dynamics Controllers - A Technical Survey. *SAE Technical Paper 2005-01-1660*; 2005. <https://doi.org/10.4271/2005-01-1660>
- [81] Schütte H, Plöger M. Hardware-in-the-Loop Testing of Engine Control Units - A Technical Survey. *SAE Technical Paper 2007-01-0500*; 2007. <https://doi.org/10.4271/2007-01-0500>
- [82] Kandilli C, Uzel M. Evaluation of annual performance of a photovoltaic thermal system integrated with natural zeolites. *Res Eng Struct Mater*. 2020;6(4):363-374. <https://doi.org/10.17515/resm2020.200en0618>
- [83] Isella A, Manca D. A framework for the design of battery energy storage systems in Power-to-X processes. *J Energy Storage*. 2025;123:116744. <https://doi.org/10.1016/j.est.2025.116744>
- [84] Aher P, Deshmukh R, Chavan C, et al. A comprehensive equivalent circuit model of Li-ion batteries for SOC estimation in electric vehicles based on parametric sensitivity analysis. *Ionics*. 2025;31:287-303. <https://doi.org/10.1007/s11581-024-05950-2>
- [85] Li H, Zhao H, Liu D, Li Z, Hu Q. Electro-thermal coupling modeling and heat generation decoupling analysis of semi-solid-state lithium-ion battery. *Electrochim Acta*. 2025;512:145455. <https://doi.org/10.1016/j.electacta.2024.145455>
- [86] Mukherjee S, Chowdhury K. State of charge estimation techniques for battery management system used in electric vehicles: a review. *Energy Syst*. 2025;16:1521-1564. <https://doi.org/10.1007/s12667-023-00631-x>
- [87] Soyoye BD, Bhattacharya I, Anthony Dhason MV, Banik T. State of Charge and State of Health Estimation in Electric Vehicles: Challenges, Approaches and Future Directions. *Batteries*. 2025;11:32. <https://doi.org/10.3390/batteries11010032>

- [88] Opal-RT. Products [Internet]. Opal-RT website; 2026 [cited 2026 Feb 25]. Available from: <http://www.opal-rt.com/product>
- [89] Casado P, Blanes JM, Garrigós A, Marroquí D, Torres C. Evaluation of commercial Li-Ion 18650 battery cells for deep space applications. *J Power Sources*. 2025;638:236552. <https://doi.org/10.1016/j.jpowsour.2025.236552>
- [90] Pande AS, Soni BP, Sharma AK. Modeling of one and two RC model and state estimation of Lithium-Ion Battery using Thevenin's equivalent Circuit Model. *Procedia Comput Sci*. 2025;259:494-503. <https://doi.org/10.1016/j.procs.2025.03.351>
- [91] Chung L, Nixon BA, Yu E, Mylopoulos J. *Non-Functional Requirements in Software Engineering*. Boston: Kluwer Academic Publishers; 2000. <https://doi.org/10.1007/978-1-4615-5269-7>
- [92] Khaleduzzaman S, Wood A, Srivastava A. Real-Time Hardware-in-the-Loop Modeling and Emulation for Utility-Scale Energy Storage Systems: A Review, Challenges and Path Forward. *IEEE J Emerg Sel Top Ind Electron*. 2025;6(4):1190-1201. <https://doi.org/10.1109/JESTIE.2025.3571436>
- [93] Marhold C, Rohleder C, Salinesi C, Doerr J. Clarifying Non-Functional Requirements to Improve User Acceptance - Experience at Siemens. In: *Proceedings of the 15th International Working Conference, REFSQ 2009*; 2009 Jun 8-9; Amsterdam, The Netherlands. https://doi.org/10.1007/978-3-642-02050-6_12
- [94] Fleischer C, Sauer DU, Barreras JV, Schaltz E, Christensen AE. Development of software and strategies for Battery Management Testing on HIL Simulator. In: *11th International Conference on Ecological Vehicles and Renewable Energies (EVER)*; 2016. <https://doi.org/10.1109/EVER.2016.7476438>
- [95] Dutta S, Debnath T, Amitabh K, Chauhan M, Majumder S, De K. Electric Mobility in North East India: Transforming the Region's Transportation Landscape. In: *2025 First International Conference on Advances in Computer Science, Electrical, Electronics, and Communication Technologies (CE2CT)*; 2025; Bhimtal, Nainital, India. p. 377-382. <https://doi.org/10.1109/CE2CT64011.2025.10941116>
- [96] Kandilli C. Exploring the potential of natural zeolites to enhance solar energy systems: A concise review. *Res Des*. 2024;1(1):13-21. <https://doi.org/10.17515/rede2024-001en1101rv>
- [97] Ozmen HB, Tanriverdi B. Concrete paradox: Economic importance, environmental impacts, and the sustainability of concrete material. *Res Des*. 2025;2(2):66-84. <https://doi.org/10.17515/rede2025-008en0801rs>
- [98] Monaco R, Liu X, Murino T, Cheng X, Nielsen PS. A non-functional requirements-based ontology for supporting the development of industrial energy management systems. *J Clean Prod*. 2023;414:137614. <https://doi.org/10.1016/j.jclepro.2023.137614>
- [99] Dutta S, Majumder S. Efficient Bi-Directional Charging Solutions for Compact Electric Vehicles for Sustainable Mobility. In: *2024 International Conference on Computer, Electronics, Electrical Engineering & their Applications (IC2E3)*; 2024; Srinagar Garhwal, Uttarakhand, India. p. 1-6. <https://doi.org/10.1109/IC2E362166.2024.10826619>
- [100] Dutta S, Majumder S, De K. Parameter Sizing in Bi-Directional Charging for Compact EVs. In: *2024 IEEE Silchar Subsection Conference (SILCON 2024)*; 2024; Agartala, India. p. 1-6. <https://doi.org/10.1109/SILCON63976.2024.10910648>
- [101] Sun B, Zhao X, He X, Ruan H, Zhu Z, Zhou X. Virtual Battery Pack-Based Battery Management System Testing Framework. *Energies*. 2023;16:680. <https://doi.org/10.3390/en16020680>
- [102] Saleem MU, Shakir M, Usman MR, Bajwa MHT, Shabbir N, Ghahfarokhi PS, et al. Integrating Smart Energy Management System with Internet of Things and Cloud Computing for Efficient Demand Side Management in Smart Grids. *Energies*. 2023;16:4835. <https://doi.org/10.3390/en16124835>

Experimental evaluation of the synergistic strengthening of concrete beams with hybrid GFRP and JFRP systems exploring bond behavior and structural performance

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Abstract

This study explores the structural behavior and load transfer characteristics of reinforced concrete (RC) beams externally retrofitted with hybrid fiber reinforced polymer (FRP) systems combining synthetic glass fibers (GFRP) and natural jute fibers (JFRP). Fourteen full-scale RC beams were tested under four-point bending to examine how different fiber types and strengthening schemes influence both strength and deformation capacity. To monitor precise structural response, the experimental setup utilized a spreader beam for symmetrical loading and LVDTs positioned at mid-span and supports to account for settlement. While existing research often focuses on mono-material reinforcement, this work provides a new contribution by conducting a comprehensive experimental evaluation of strategic fiber positioning on full-scale specimens. The experimental results demonstrate that different configurations yield distinct performance benefits: the JFRP U-wrap configuration (JBJS) achieved a 38% increase in ultimate strength with proper anchorage, while the GFRP U-wrap reached a 44% gain. A primary novelty of this investigation is the identification of the JBGS hybrid configuration (jute along the soffit and glass on the sides) as the most balanced and resilient system tested. Despite the higher material ductility of GFRP, this specific hybrid arrangement achieved a 28% strength gain and a 71% increase in ductility index, suggesting effective stress redistribution across the reinforcement layers. Failure observations extracted from detailed crack analysis point to more gradual crack development and improved load retention in hybrid systems. These findings offer insight into performance-driven design for sustainable structural rehabilitation by demonstrating how fiber synergy can be strategically optimized to balance mechanical efficiency and eco-efficiency.

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1. Introduction

Reinforced concrete (RC) remains the most widely used construction material in civil infrastructure [1-4], including bridge decks [5, 6], roadway support systems [7], and structural slabs [4, 8-11]. However, RC members are susceptible to degradation under flexural fatigue, cracking, corrosion of internal reinforcement, and environmental exposure over time [12-14].

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These deteriorations compromise serviceability and safety, prompting the need for reliable strengthening and rehabilitation techniques [15-21]. The urgency of such interventions is further emphasized by the global need for seismic retrofitting, which is essential to enhance the resistivity and overall lifespan of critical elements like beam-column joints [22].

Fiber-reinforced polymer (FRP) composites have emerged as effective materials for retrofitting RC elements due to their high strength-to-weight ratio [23], corrosion resistance, and ease of application [24-32]. The increasing use of these composites is justified by their superior stiffness and favorable response under static, free vibration, and buckling conditions compared to traditional metallic materials [33]. Externally bonded FRP systems, particularly those based on glass or carbon fibers, have shown considerable success in restoring structural capacity [34-38]. However, conventional FRP systems often exhibit brittle failure modes and are associated with high production costs and environmental burdens [39-41]. These limitations have led researchers to explore the integration of natural fibers as sustainable alternatives or supplements in structural strengthening applications [42-45]. One notable example of such innovation is the investigation into using bamboo as a substitute for structural steel, which has shown promise in low-cost housing due to its appreciable tensile resistance [46]. Among natural fibers, jute has attracted interest due to its low cost, biodegradability, and acceptable tensile performance [47-52]. Nevertheless, the mechanical properties of jute are inferior to those of synthetic fibers, limiting its application in high-performance strengthening unless appropriately combined [53-57]. Recent studies have proposed hybrid systems, combining synthetic and natural fibers, to exploit their complementary properties [58-62]. Such systems aim to harness the stiffness and strength of synthetic FRPs alongside the ductility and energy absorption capacity of natural fibers, offering improved failure behavior and sustainability [63-67].

Despite the growing interest in composite materials and sustainable alternatives [68, 69], a significant research gap remains regarding the practical application of hybrid systems for the flexural strengthening of full scale reinforced concrete beams. Most existing studies are limited to theoretical reviews or experiments conducted at the material scale using small coupons which do not accurately capture the complex bond mechanics and load distribution found in real structural elements. Furthermore, while the general benefits of individual synthetic or natural fibers are well documented, there is a critical lack of evidence concerning how their strategic positioning influences structural synergy. The specific question of how to arrange high stiffness glass fibers alongside high ductility jute fibers to prevent premature debonding while maximizing both strength and deformation has not been sufficiently explored. This research addresses these deficiencies by providing a comprehensive experimental evaluation of various hybrid wrapping configurations. By investigating the interaction between fiber placement and structural performance in fourteen full scale specimens, this study establishes a clearer understanding of how to optimize hybrid reinforcements for safer and more efficient structural rehabilitation.

The beams were divided into three main categories. The first included single-material systems, where GFRP or JFRP sheets were bonded using the wet lay-up technique to either the bottom face (GB and JB) or both the bottom and side faces in a U-wrap configuration (GBGS and JBJS). The second category consisted of hybrid systems: JBGS (jute at the bottom, glass on the sides) and GBJS (glass at the bottom, jute on the sides). The final category included an unstrengthened control group (CB). All FRP fabrics were applied using an epoxy resin and arranged in a single layer per face, with particular attention given to fabric alignment and bond quality to ensure uniform load transfer. This experimental matrix was carefully designed to isolate and compare the effects of fiber type (natural vs. synthetic), reinforcement layout (bottom-only vs. U-wrap), and fiber positioning (glass vs. jute in critical tension or shear zones) on key structural parameters including strength, stiffness, ductility, and failure mode. By doing so, the study provides direct insight into how strategic fiber selection and placement can influence the mechanical behavior of strengthened RC elements under flexural loading.

The study aims to evaluate each configuration in terms of its contribution to load capacity, ductility, stiffness, and failure mechanism, thereby addressing key uncertainties in hybrid FRP retrofitting performance. In particular, it explores whether the combination of GFRP's high tensile strength and

JFRP's ecological and damping properties can produce a pseudo-ductile response, mitigating sudden brittle failure through a more progressive damage mechanism while maintaining mechanical efficiency. The findings provide experimental evidence to support design-oriented applications of hybrid FRP systems and demonstrate how fiber placement can be strategically optimized to balance performance and sustainability. This work contributes to advancing the development of eco-efficient, structurally reliable retrofitting methods for RC members in civil infrastructure, including pavements, bridge decks, and structural slabs.

2. Materials

The materials used in this study were selected to replicate realistic construction and retrofitting conditions while enabling a comparative evaluation of synthetic, natural, and hybrid fiber-reinforced polymer systems. This section describes the properties of the concrete matrix, epoxy adhesive, internal steel reinforcement, and externally bonded FRP composites employed in the fabrication and strengthening of the RC beam specimens.

Table 1. Concrete mix composition and design parameters per cubic meter, optimized for structural beam casting and flexural testing

Category	Material	Quantity	Remarks
Binder	Portland Cement (CEM I 42.5 R)	350 kg	High early strength, good durability
Fine Aggregates	Natural Sand (0/3 mm)	648 kg	Clean, well-graded
Coarse Aggregates	Boussaada Sand	97 kg	Local source, filler optimization
	Crushed Gravel (8/15 mm)	710 kg	Structural-grade angular aggregate
	Crushed Gravel (15/25 mm)	400 kg	Enhanced packing and strength
Water	Potable Water	190 L	W/C ratio of 0.54
Mix Proportions	Water-to-Cement Ratio	0.54	Targeting balanced strength/workability
	Aggregate-to-Cement Ratio	5.3	Moderately lean mix
	Total Aggregate Content	1855 kg	Consistent with normal-weight concrete
Performance Targets	Target Slump	~90 mm	Suitable for beam casting
	Design Compressive Strength	25 MPa @ 28 days	Confirmed via cube testing
	Admixtures	None	Mix achieved target without additives

2.1. Concrete

The concrete used in all beam specimens was designed to achieve a target compressive strength of approximately 25 MPa after 28 days of water curing. The mix consisted of CEM I 42.5 R Portland cement, fine aggregates (natural 0/3 mm sand and Boussaada sand), two grades of crushed gravel (8/15 mm and 15/25 mm), and potable water. A water-to-cement ratio of 0.5 was maintained to ensure sufficient workability and strength development. Mixing was carried out in a rotary drum mixer until a uniform consistency was achieved. After casting, the beams were demolded after 24 hours and cured in water at ambient temperature for 28 days. The detailed proportions of the mix components per cubic meter are provided in Table 1.

In addition to mix proportions, the water-cement ratio and fresh concrete density were determined and are presented in Table 2. The measured water-cement ratio was 0.54, slightly above the theoretical design value due to adjustments made for workability during casting. The average density of the concrete was 2390 kg/m³, which is consistent with standard values for normal-weight structural concrete. This parameter is particularly important in the present study, as it directly influences the dead weight of the beams and the internal stress distribution under flexural loading. Accurate density values also ensure consistency when comparing load-deflection behavior

across beams, especially in cases where strengthening systems alter stiffness and ductility characteristics.

Table 2. Physical and strength-related properties of fresh and hardened concrete used for casting full-scale RC beam specimens

Property	Value	Remarks
Water–Cement Ratio (w/c)	0.54	Balanced for strength and workability
Fresh Density	2390 kg/m ³	Consistent with normal-weight concrete
Hardened Density (28 days)	~2400 kg/m ³	Slight increase due to moisture loss
28-Day Compressive Strength	25 MPa	Based on cube/cylinder testing
Concrete Type	Normal-weight	No lightweight or heavyweight aggregates used
Air Content	Not measured	Estimated under 2% due to compacting process
Slump (Workability)	~90 mm	Verified during specimen casting

The dimensions and reinforcement of the beam specimens were determined based on the design principles of Eurocode 2 [70]. The specimens were specifically engineered as under-reinforced sections to ensure a ductile flexural failure mode, characterized by the yielding of the longitudinal steel reinforcement 14 mm prior to concrete crushing. To prevent premature diagonal tension failure, the stirrup spacing was calculated to provide a shear capacity significantly higher than the expected flexural capacity, ensuring that the effectiveness of the hybrid FRP strengthening could be fully evaluated in bending.

2.2. Epoxy Resin

A two-component epoxy adhesive system, NOVA MASTICOLLE, was used to bond the fiber-reinforced polymer (FRP) sheets to the concrete surfaces. This adhesive, widely employed in structural applications, offers high bond strength, compatibility with various fiber types, and ease of application using the wet lay-up method. The system, illustrated in Fig. 1, consists of a base resin (component A) and a hardener (component B), which were mixed in a weight ratio of 2:1 as recommended by the manufacturer.



Fig. 1. Two-component NOVA MASTICOLLE epoxy adhesive system used for bonding FRP sheets: Component A (resin) and Component B (hardener)

The two components were manually blended using mechanical stirring until a uniform, bubble-free mixture was achieved. The epoxy was applied to both the concrete substrate and the fiber fabric using brushes and rollers to ensure full impregnation and contact. The product exhibits a tensile strength of approximately 30 MPa and a modulus of elasticity of 3.2 GPa, making it suitable for load transfer in externally bonded reinforcement systems. Its low shrinkage, strong adhesion, and sufficient working time under ambient conditions contributed to consistent and reliable composite bonding across all specimens. The role of NOVA MASTICOLLE was essential in enabling effective

stress transfer between the concrete beams and the FRP reinforcements, regardless of fiber type or configuration.

2.3. Steel Reinforcement

All beams were reinforced internally using steel bars arranged in a conventional flexural configuration. The reinforcement layout consisted of two 14 mm diameter high-adherence steel bars placed in the tension zone (bottom), two 8 mm diameter mild steel bars in the compression zone (top), and 6 mm diameter closed stirrups spaced at 100 mm along the length of the beam for shear reinforcement.

Table 3. Mechanical properties and functional roles of high-adherence and mild steel reinforcement used in RC beam specimens.

Steel Type	High-Adherence Steel	Mild Steel
Application in Beam	Bottom longitudinal bars	Top longitudinal + stirrups
Bar Diameter (mm)	14	8 (top) / 6 (stirrups)
Yield Strength (MPa)	500	235
Ultimate Strength (MPa)	550	410
Elongation (%)	12	25
Young's Modulus (MPa)	210	210
Poisson's Ratio	0.30	0.30
Remarks	Primary tensile reinforcement	Used for compression and shear

Two types of steel were used: high-adherence steel for the main tension reinforcement, and mild steel for compression and stirrup reinforcement. Mechanical properties of both steel types were obtained from uniaxial tensile tests conducted on representative samples. The high-adherence steel exhibited a yield strength of 500 MPa, an ultimate strength of 550 MPa, and an elongation capacity of 12%. The mild steel showed a lower yield strength of 235 MPa, an ultimate strength of 410 MPa, and greater ductility, with a 25% elongation capacity. Both steel types had a Young's modulus of 210,000 MPa and a Poisson's ratio of 0.3. These properties are summarized in Table 3.

The mechanical behavior of the internal steel reinforcement and the externally bonded FRP sheets was verified through uniaxial tensile testing. Fig. 2 illustrates the stress-strain relationships for these materials, highlighting the distinct yield plateau of the steel bars and the linear-elastic failure modes of the GFRP and JFRP composites.

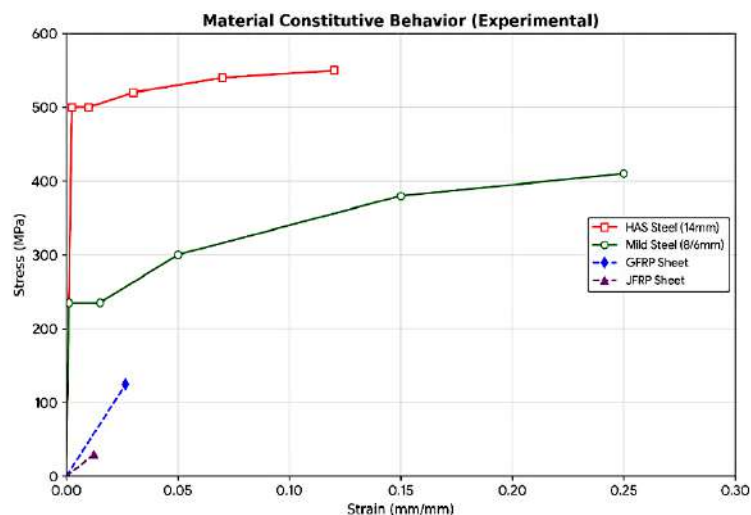


Fig. 2. Tensile stress-strain behavior of internal steel reinforcement (High-Adherence and Mild steel) and external FRP composites (GFRP and JFRP)

2.4. FRP Systems

Three externally bonded fiber-reinforced polymer (FRP) systems were employed in this study: glass fiber-reinforced polymer (GFRP), jute fiber-reinforced polymer (JFRP), and hybrid GFRP-JFRP systems. All fabrics were applied using the wet lay-up technique with NOVA MASTICOLLE epoxy resin, following surface preparation and primer application. The fibers were installed in single layers with their principal direction aligned along the longitudinal axis of the beams to ensure full contribution in flexural loading.

The GFRP sheets were unidirectional, with a nominal areal density of 850 g/m². They offered high tensile strength and stiffness, with a tensile strength of approximately 3200 MPa, an elastic modulus of 73 GPa, and an ultimate strain of 4.4%. The JFRP sheets, made from natural jute fibers, had lower strength but higher deformability, with a tensile strength of 250 MPa, an elastic modulus of 10 GPa, and an ultimate strain of 2.5%. The choice of jute was motivated by its environmental benefits, cost-effectiveness, and compatibility with sustainable construction strategies.

Hybrid configurations were developed by combining GFRP and JFRP sheets in specific geometric arrangements. In the JBGS configuration, jute fibers were bonded to the bottom face (tension zone) of the beam, while glass fibers were applied along the side faces to provide shear confinement. In the GBJS configuration, the positions were reversed, with GFRP at the bottom and JFRP on the sides. This hybridization aimed to exploit the stiffness and strength of synthetic fibers while benefiting from the energy absorption and ductility of natural fibers.

Table 4. Mechanical characteristics of jute and glass FRP composites prepared by wet lay-up technique, used for externally bonded reinforcement.

FRP Type	Jute FRP (JFRP)	Glass FRP (GFRP)
Elastic Modulus (GPa)	2.79	5.85
Tensile Strength (MPa)	28.54	108.96
Ultimate Stress (MPa)	29.60	124.50
Thickness (mm)	2.25	1.64
Ultimate Strain (%)	1.24	2.66

All fabrics were cut to the required dimensions and applied after saturating the surface with resin. Care was taken to ensure uniform pressure during bonding, avoid air entrapment, and maintain continuity between the fibers and the concrete substrate. The mechanical properties of both fiber types are presented in Table 4.

3. Apparatus and Operational Procedures

This section presents the experimental procedures used to investigate the flexural behaviour of RC beams strengthened with GFRP, JFRP, and hybrid FRP systems. The process included specimen preparation, application of external strengthening systems, and structural testing under four-point bending to evaluate load-bearing capacity, deformation behaviour, and failure characteristics.

The experimental program was conducted in several sequential phases to evaluate the effectiveness of the proposed hybrid strengthening systems. The overall methodology, encompassing material characterization, specimen fabrication, strengthening strategies, and structural testing, is summarized in the flowchart presented in Fig. 3.

3.1. Key Experimental Assumptions

To ensure the reliability of the comparative analysis between the fourteen specimens, the following experimental assumptions and controls were maintained:

- **Batch Homogeneity:** The concrete properties and internal steel reinforcement characteristics are assumed to be uniform across all specimens, as they were sourced from the same respective batches and subjected to identical curing conditions.

- **Bond Consistency:** The mechanical surface preparation (grinding) is assumed to have provided a standardized roughness, ensuring that variations in bond behavior are attributable to the fiber configuration rather than surface inequality.
- **Application Uniformity:** It is assumed that the manual wet lay-up process achieved full resin impregnation and a consistent adhesive thickness across all strengthened beams.
- **Loading Symmetry:** The four-point bending setup assumes perfectly symmetrical load application and leveled supports, maintaining a constant moment zone in the middle third of the span.
- **Environmental Stability:** Laboratory conditions were kept stable during the curing and testing phases to ensure that the mechanical response of the natural jute fibers was not influenced by significant humidity or temperature fluctuations.

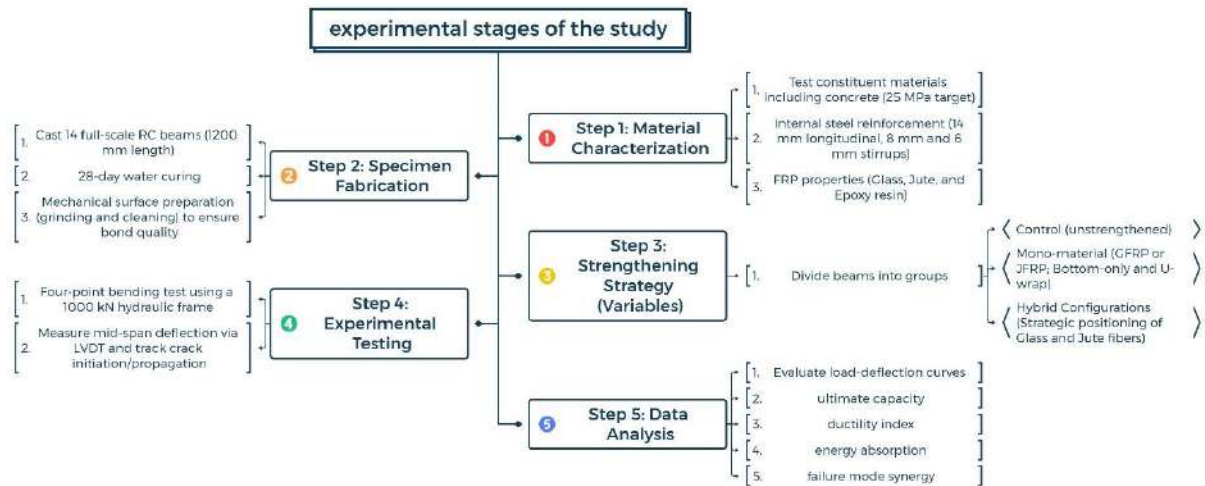


Fig. 3. Flowchart illustrating the experimental stages of the study, from material characterization to the evaluation of hybrid strengthening efficiency

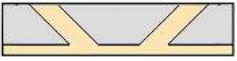
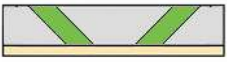
3.2. Specimen Preparation

A total of fourteen full-scale RC beams were fabricated using identical formwork and reinforcement detailing. Each beam measured 600 mm in length, with a cross-section of 150 mm × 100 mm. The longitudinal reinforcement consisted of two 14 mm diameter high-adherence steel bars placed at the bottom (tension zone) and two 8 mm diameter mild steel bars placed at the top (compression zone). Transverse shear reinforcement was provided by 6 mm diameter closed stirrups spaced at 100 mm along the span. After casting, the beams were demolded after 24 hours and cured in water for 28 days.

Following curing, the concrete surfaces of all beams designated for strengthening were mechanically ground to remove surface laitance and expose the aggregate structure. Compressed air was used to clean dust and debris. This surface preparation was essential to ensure optimal bond conditions for the externally bonded FRP sheets. During specimen preparation, specific measures were implemented to ensure the reliability of the FRP-concrete bond:

- **Bond Integrity:** To eliminate air voids and ensure full resin impregnation, a ribbed roller was used during the wet lay-up, applying constant pressure to prevent premature debonding.
- **Moisture Control:** Given the hydrophilic nature of jute fibers, the JFRP sheets were pre-dried and stored in a low-humidity environment to prevent moisture from compromising the epoxy bond.
- **Surface Consistency:** Hand-held diamond-cup grinding was utilized to achieve a uniform ICRI CSP-3 roughness profile across all 14 beams, minimizing variability in bond performance.
- **Overlap Management:** In hybrid layouts (GBJS/JBGS), localized thickness at fiber intersections was mitigated by a graduated resin application to maintain a consistent cross-sectional profile.

Table 5. Experimental groups and external FRP strengthening configurations for RC beam specimens

Group	Bottom Reinforcement	Side Reinforcement	FRP Strengthening Configurations	Specimen Code	Description
0	None	None		CB	Control beam (no external reinforcement)
1	GFRP	None		GB	GFRP applied to bottom face only
2	JFRP	None		JB	JFRP applied to bottom face only
3	GFRP	GFRP		GBGS	GFRP applied to bottom and both side faces (U-wrap)
4	JFRP	JFRP		JBJS	JFRP applied to bottom and both side faces (U-wrap)
5	GFRP	JFRP		GBJS	GFRP bottom + JFRP sides (hybrid configuration)
6	JFRP	GFRP		JBGS	JFRP bottom + GFRP sides (hybrid configuration)

While the experimental program was limited to two specimens per configuration ($n=2$) due to laboratory resource constraints, the high degree of consistency and low variability observed between the paired results justify the reliability of the reported trends. These findings provide a primary evaluation of the proposed hybrid systems, though larger sample sizes in future studies will further validate the statistical dispersion of these performance enhancements.

3.3. Strengthening Schemes and Specimen Grouping

The fourteen beam specimens were divided into seven groups based on the type and location of externally bonded reinforcement. Each group consisted of two identical beams. The experimental matrix was designed to assess the influence of fiber type (GFRP or JFRP), fiber position (bottom-only or U-wrap), and hybrid configurations (mixed fiber types) on flexural behavior. A summary of the reinforcement configurations and corresponding specimen codes is provided in Table 5.

The control group (CB) had no external reinforcement. Groups GB and JB were strengthened only at the bottom face with GFRP and JFRP, respectively. Groups GBGS and JBJS were fully wrapped in a U-shaped configuration (bottom and both side faces) using the same fiber type. The hybrid systems included JBGS, with jute at the bottom and glass on the sides, and GBJS, with the reverse arrangement. All strengthening was performed using the wet lay-up method.

The geometric dimensions, internal steel reinforcement, and experimental configuration of the beam specimens are detailed in Fig. 4. All beams shared a total length of 600 mm and a rectangular cross-section of 150 mm × 100 mm, with consistent internal steel detailing across all groups. To ensure precise data acquisition, the four-point bending setup featured a loading head connected to a spreader beam for uniform load distribution. As shown in the schematic, LVDTs were strategically positioned at the mid-span to capture maximum deflection and at the supports to monitor and account for any potential settlement during the testing process.

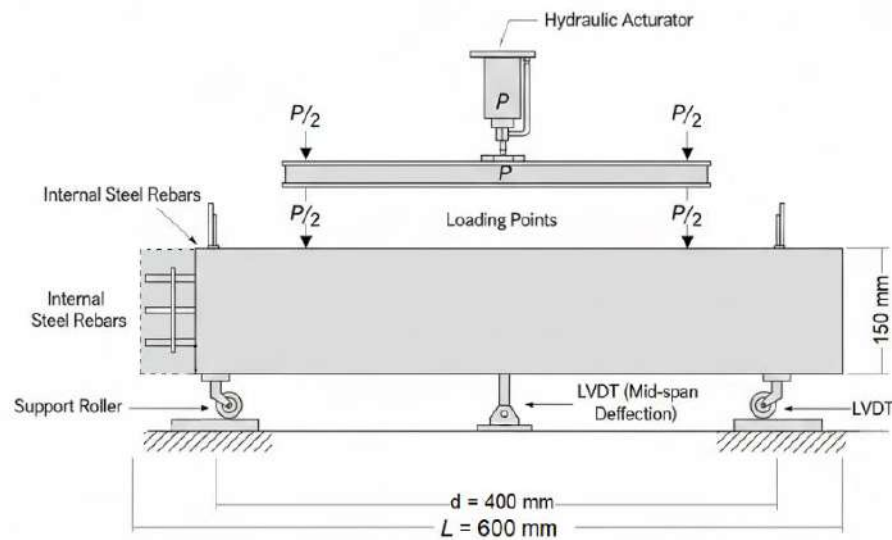


Fig. 4. Experimental test setup for a 600 mm unstrengthened RC beam under four-point loading, showing the internal reinforcement (T14 and T8 bars), stirrups (R6), and the LVDT positioning at critical measurement points

3.4. Test Setup and Instrumentation

All beams were tested under four-point bending using a hydraulic loading frame with a maximum capacity of 1000 kN. Each specimen was simply supported over a clear span of 1000 mm, with two concentrated point loads applied symmetrically at one-third points along the span, creating a constant moment region in the middle third (300 mm). This configuration is widely adopted in flexural testing to isolate bending effects and allow comparison of stiffness, strength, and ductility parameters.

Roller supports were used at both ends to allow horizontal movement and rotation, reducing constraint effects. The applied load was transmitted through a spreader beam connected to the actuator, ensuring uniform force distribution at both loading points. Loading was applied in displacement control mode at a constant rate of 2 mm/min until failure.



Fig. 5. Application of FRP strengthening systems and layout of tested configurations: bottom-only, U-wrap, and hybrid reinforcement schemes

Instrumentation was implemented to monitor both the global and localized behavior of the beams throughout testing. The applied load was measured in real time using a calibrated load cell integrated into the hydraulic actuator, with continuous data acquisition ensuring high accuracy. Mid-span deflection was recorded using a Linear Variable Differential Transformer (LVDT)

positioned vertically at the center of the span. This sensor was connected to a digital acquisition system to track displacement throughout the loading process.

In addition to quantitative data, qualitative observations were also conducted. A crack monitoring system was implemented by marking the beam surfaces at regular 50 mm intervals, allowing for accurate tracking of crack formation and propagation. Crack initiation, development, and final failure modes were recorded using visual inspection and high-resolution photographic documentation. These procedures provided a comprehensive assessment of structural behavior under flexural loading and allowed for precise correlation between strengthening configuration and failure mechanism. The external appearance of the strengthened beams, showing the applied FRP systems across different configurations (bottom-only, U-wrap, and hybrid), is illustrated in Fig. 5.

4. Results and Discussion

This section presents and interprets the experimental results obtained from the four-point bending tests conducted on fourteen RC beam specimens. The beams were strengthened using various externally bonded FRP configurations GFRP, JFRP, and hybrid GFRP-JFRP systems. The analysis is organized into six subsections covering failure mechanisms, load-deflection behavior, strength performance, deformability, ductility, and the overall contribution of each configuration to structural performance.

4.1. Failure Modes

For the mechanical tests of the concrete specimens, Table 6 shows the measured properties after 28 days of curing. The compressive strength reached 36.6 MPa, determined through standard testing as illustrated in Fig. 6(a), which confirms the adequate load-bearing capacity of the mix. The tensile strength was found to be 2.79 MPa, obtained using the indirect splitting method as shown in Fig. 6(b), reflecting the brittle behavior typically associated with cementitious composites.

While the concrete mix was initially proportioned for a target compressive strength of 25 MPa, experimental testing at 28 days revealed a mean compressive strength of 36.6 MPa. This increase is attributed to the high quality of local aggregates and optimal laboratory curing conditions, which promoted a more extensive hydration process than initially anticipated. Consequently, to maintain analytical rigor, the measured value of 36.6 MPa was adopted for all subsequent theoretical calculations, flexural capacity predictions, and comparative performance analyses throughout this study. The average specimen weight was 15.47 kg, which was recorded to monitor density and ensure consistency across batches.



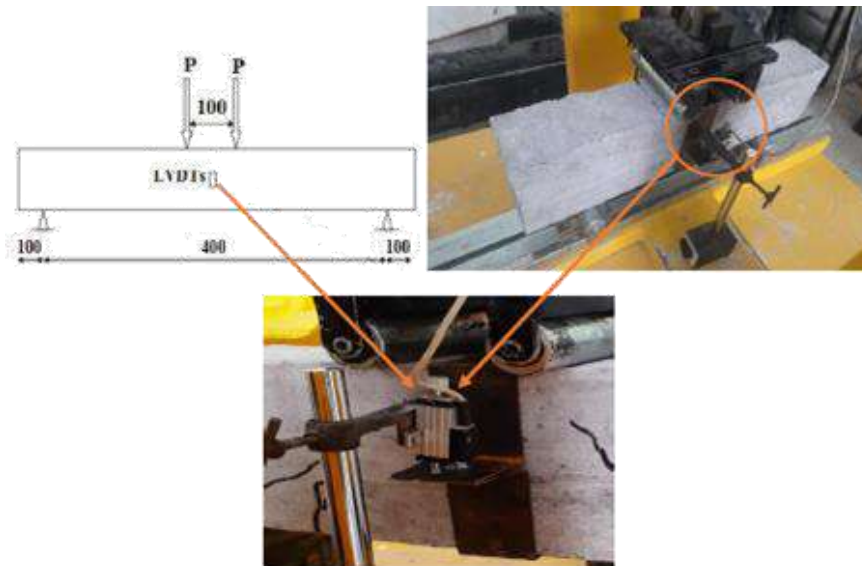


Fig. 6. Experimental test setups for concrete specimens: (a) compression testing showing the applied loading mechanism, and (b) indirect splitting tensile test used to determine the tensile strength of concrete

Failure patterns observed during testing varied significantly across the different strengthening configurations and are summarized in Table 7. The control beam (CB) failed in a typical flexural mode, with pronounced mid-span cracking followed by concrete crushing in the compression zone.

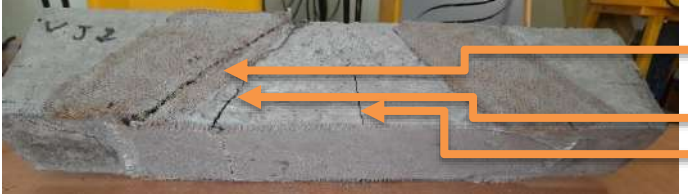
Table 6. Measured mechanical properties of 28-day cured concrete specimens.

Property	Value	Remarks
Compressive Strength ($F_{c(28)}$)	36.6 MPa	Determined via standard cube testing
Tensile Strength ($F_{t(28)}$)	2.79 MPa	Estimated using indirect (splitting) method
Average Specimen Weight	15.47 kg	Prism/cube weight used for density tracking

Beams strengthened with GFRP only (GB and GBGS) primarily failed due to delamination or debonding of the fiber sheet, occasionally accompanied by concrete crushing near the loading points. The brittle nature of the glass fibers contributed to sudden load drops after peak strength.

Table 7. Observed failure modes of reinforced concrete beams under four-point bending for different FRP strengthening configurations

Beam code	RC Beam concrete	Failure mode
CB		1- Crashing of concrete 2- Propagation of the shear cracks 3- Shear cracks
JB		1- Propagation of the shear cracks 2- Shear cracks 3- Fracture of JFRP bands and crashing of concrete
GB		1- Propagation of the shear cracks 2- Shear cracks 3- Debonding of GFRP sheets and crashing of concrete

Beam code	RC Beam concrete	Failure mode
JBJS		1- Shear cracks 2- Fracture of JFRP sheets 3- Flexural cracks
GBGS		1- Propagation of the shear cracks 2- Shear cracks 3- Debonding of GFRP sheets and crashing of concrete
JBGS (Hybrid reinforcement)		1- Crashing of concrete 2- Debonding of GFRP sheets 3- Flexural cracks
GBJS (Hybrid reinforcement)		1- Fracture of JFRP and GFRP sheets 2- Shear cracks 3- Flexural cracks

JFRP-reinforced beams (JB and JBJS) exhibited rupture of the jute sheets without prior delamination, indicating effective bonding but early material failure due to the lower tensile capacity of natural fibers. These beams showed more gradual crack propagation and distributed damage.

Hybrid configurations presented mixed failure modes. The JBGS beam (jute at the bottom, glass on the sides) failed by rupture of the jute fibers at mid-span and localized side debonding of the GFRP. In contrast, the GBJS beam (glass at the bottom, jute on the sides) experienced rupture of both fiber types. These hybrid beams exhibited more stable failure progression, with delayed crack growth and better energy absorption than mono-material systems.

4.2. Analysis of Cracking Characteristics

To evaluate the influence of hybrid FRP strengthening on the serviceability of the beams, the cracking characteristics were extracted and analyzed. As summarized in Table 8, the reinforcement configuration significantly influenced the crack distribution and widths.

Table 8. Summary of measured crack characteristics at ultimate load.

Beam code	No. of Cracks	Avg. Crack Spacing (mm)	Avg. Crack Width (mm)	Failure Mode	Typical Literature Ranges
CB	6	185	5.2	Flexural (Concrete crushing)	RC beams: crack spacing ~150–200 mm, crack width ~4–6 mm [71]
GB	4	95	4.2	FRP Debonding	GFRP RC beams: spacing 70–120 mm, width 2–5 mm [71]
JB	5	130	4.9	FRP Rupture	Natural FRP beams: spacing 100–140 mm, width 3–5 mm [72]

Beam code	No. of Cracks	Avg. Crack Spacing (mm)	Avg. Crack Width (mm)	Failure Mode	Typical Literature Ranges
JBJS	8	110	3.8	Hybrid Delamination	Hybrid FRP: spacing 60–120 mm, width 1–4 mm [72]
GBGS	4	100	4.1	Flexural (Concrete crushing)	GFRP-steel hybrid: spacing 70–130 mm, width 2–4.5 mm [73]
JBGS (Hybrid reinforcement)	2	55	2.2	Flexural (Concrete crushing)	Hybrid (natural + glass): spacing 50–80 mm, width 1–2.5 mm [72]
GBJS (Hybrid reinforcement)	3	72	2.5	Flexural (Concrete crushing)	Hybrid (glass + jute): spacing 60–90 mm, width 2–3 mm [72]

The results in Table 8 show that the strengthened beams exhibited reduced crack spacing and widths compared to the control beam (CB), indicating a more efficient distribution of tensile stresses along the span. The hybrid beams demonstrated the most significant improvements, with minimal crack spacing of approximately 55 mm for JBGS and 72 mm for GBJS, and average crack widths of 2.2 mm and 2.5 mm, respectively. These reductions are substantially lower than those of the control and FRP-only beams, suggesting that the combination of Jute and Glass fibers enhances bond-slip behavior and limits the widening of individual flexural cracks.

When compared with literature data, beams reinforced exclusively with FRP, such as GFRP or natural FRP, typically show moderate reductions in crack spacing but relatively larger crack widths due to the lower elastic modulus and bond characteristics of FRP compared to steel. Glass-FRP beams in previous studies exhibited crack spacings of 70 to 120 mm and crack widths of 2 to 5 mm, consistent with the GB beam in this study. Hybrid reinforcement systems that combine FRP with steel or different fiber types have been reported to improve crack control by enhancing load distribution and tension stiffening, often achieving smaller crack widths and spacings than FRP-only beams. The present hybrid configurations (JBJS, JBGS, GBJS, GBGS) follow the same trends, with JBGS and GBJS achieving up to 70 percent reduction in crack spacing and 58 percent reduction in crack width. These results confirm that the proposed hybrid reinforcement schemes effectively improve crack distribution, control crack opening, and enhance the serviceability performance of the beams.

4.3. Load-Deflection Behavior

Figs 5-8 present the load-deflection curves for all beam configurations. The control beam demonstrated the expected elastic-plastic response with a peak load of approximately 49.5 kN. Strengthened beams exhibited increased stiffness and load capacity, with varying levels of post-yield deformation.

Fig. 7 shows that GB (GFRP-bottom-only) reached a peak load of 63.7 kN, a 29% increase over the control, with moderate deflection. JB (JFRP-bottom-only) showed a more ductile profile but a lower peak load of 56.1 kN, representing only a 13% strength gain. As seen in Fig. 8, U-wrapped beams exhibited superior performance. GBGS (GFRP on bottom and sides) reached 71.3 kN (44% improvement), while JBJS (JFRP on bottom and sides) achieved 68.3 kN (38% gain). Both showed improved deflection over bottom-only counterparts, with JBJS being more ductile. Hybrid systems, illustrated in Fig. 9, demonstrated balanced structural behavior. JBGS achieved a load of 63.3 kN and the highest deflection (20.4 mm), showing strong energy dissipation. GBJS reached 59.9 kN with intermediate deformability.

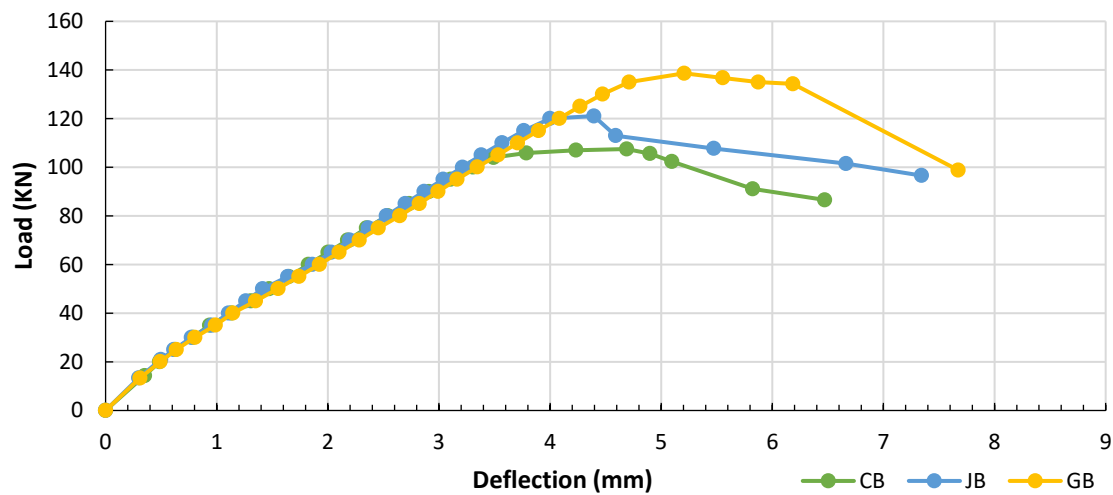


Fig. 7. Load-deflection curves for beams strengthened only at the bottom face, compared to the unstrengthened control beam

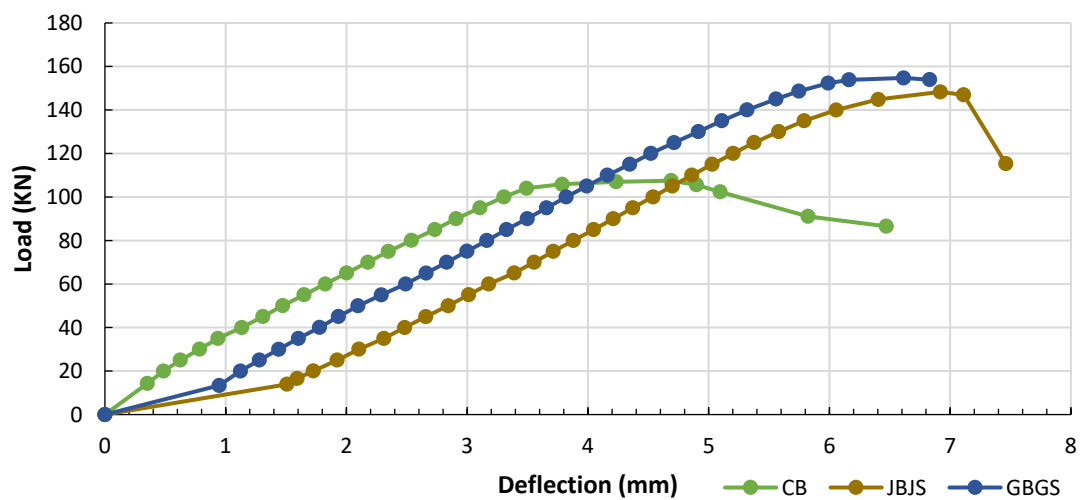


Fig. 8. Load-deflection curves for beams strengthened on the bottom and side faces using the same FRP type, compared to the control beam

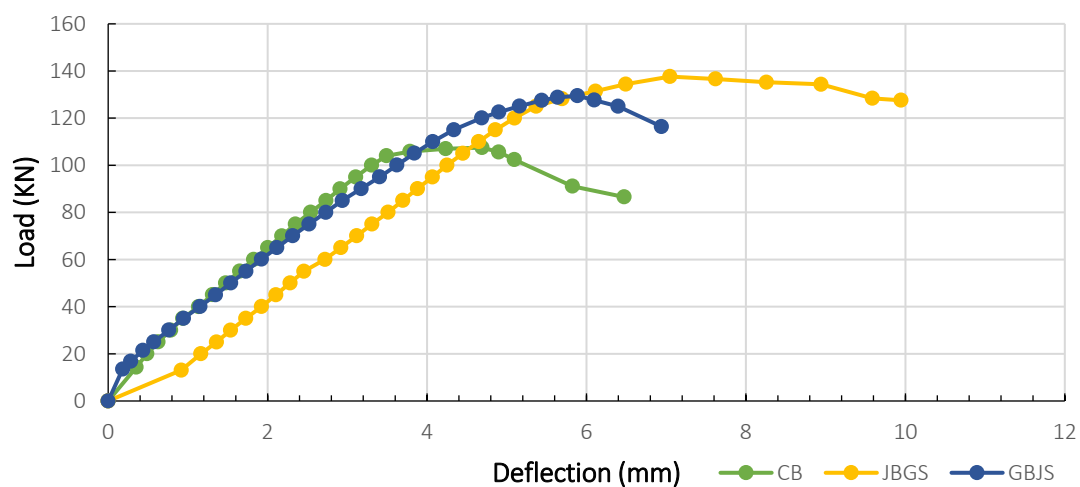


Fig. 9. Load-deflection curves for hybrid-strengthened beams (GFRP and JFRP in combined configurations), compared to the control beam

4.4. Ultimate Load Performance

The percentage gains in peak load relative to the control beam are presented in Fig. 10 and quantified in Table 9. The GBGS beam exhibited the highest increase (44%), followed by JBJS (38%), JBGS (28%), GB (29%), and GBJS (21%). The JB beam showed the lowest gain at 13%.

Table 9. Summary of mechanical performance parameters including ultimate load, and mid-span deflection, for all strengthened and control RC beams, with percentage gains to evaluate strengthening efficiency.

Beams	f_u (kN)	f_{us} (kN)	δ_u (mm)	δ_{us} (mm)	$\frac{f_{us}-f_u}{f_u}$ (%)	$\frac{\delta_{us}-\delta_u}{\delta_u}$ (%)
CB	107.5	/	4.68	/	/	/
JB	/	121.7	/	4.4	+13	-6
GB	/	138.6	/	5.2	+29	+11
JBJS	/	148.3	/	6.92	+38	+47
GBGS	/	154.7	/	6.61	+44	+41
JBGS	/	137.6	/	7.04	+28	+50
GBJS	/	129.6	/	5.88	+21	+25

As shown in Table 9, although GFRP has a higher ultimate deformation capacity, the inclusion of JFRP layers in the hybrid configuration (JBGS, GBJS) resulted in a more uniform crack distribution and a gradual loss of stiffness prior to failure. This suggests that JFRP acts as a 'sacrificial' or 'shock-absorbing' component that prevents the catastrophic and explosive delamination often associated with pure GFRP reinforcement. These results confirm that U-wrap configurations significantly enhance flexural strength by improving anchorage and stress redistribution. Hybrid systems demonstrated comparable gains to synthetic FRPs, validating the contribution of natural fibers when placed in strategic locations.

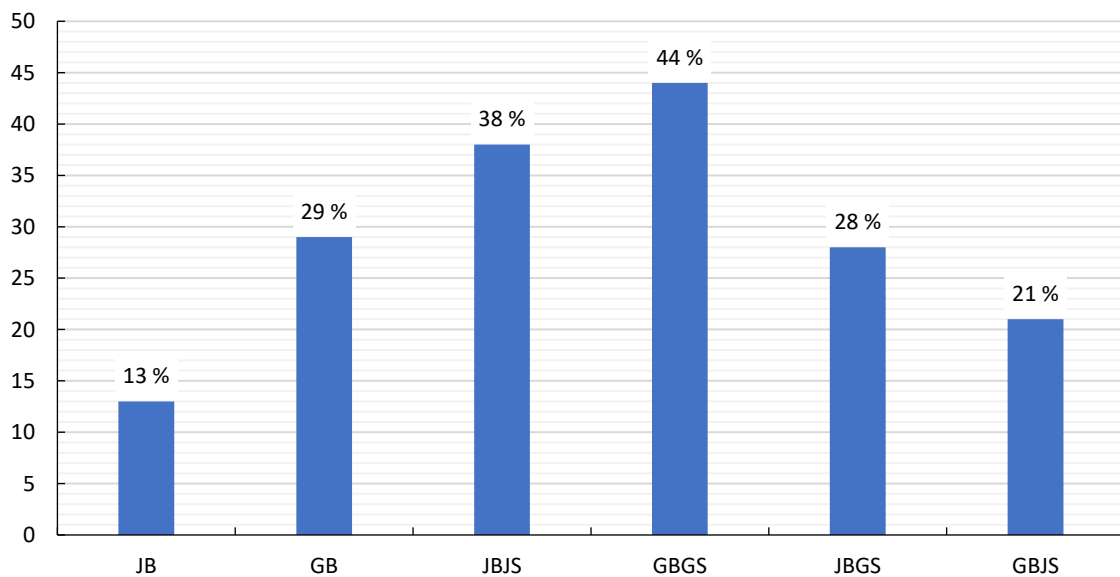


Fig. 10. Percentage improvement in ultimate load for all FRP-strengthened beams compared to the control

The mechanical superiority of the JBGS system over the GBJS and mono-material configurations can be attributed to optimized strain compatibility and stress redistribution. In the JBGS layout, the high-modulus GFRP side layers act as a stiff 'anchorage frame' that stabilizes the JFRP soffit, effectively delaying the onset of interfacial debonding. This configuration facilitates a more uniform stress transfer across the concrete-FRP interface, allowing the jute fibers to reach higher strain levels before failure. In contrast, the GBJS system lacks this external confinement, leading to localized stress concentrations and premature delamination. This anchorage effect in the JBGS

specimens transforms the failure mode from a sudden, brittle loss of bond to a controlled, pseudo-ductile response, explaining the substantial 71% increase in the ductility index.

4.5. Analytical Evaluation of Hybrid Synergy

To quantitatively define the "synergy" observed in the hybrid configurations, a Synergy Index (S_{yn}) was implemented. This analytical approach evaluates whether the hybrid system performs better than the theoretical sum of its individual components. The index is derived from the Rule of Mixtures (RoM) as follows:

$$S_{yn} = \frac{P_{exp(hybrid)}}{\sum P_i V_i} \quad (1)$$

Where; $P_{exp(hybrid)}$ is the experimental ultimate load of the hybrid specimen (e.g., JBGS or GBJS), P_i represents the ultimate load of the mono-material reference beams strengthened at the bottom (GB for glass and JB for jute), V_i is the relative contribution of each fiber type (taken as 0.5 for the hybrid configurations).

As shown in Table 9, the theoretical benchmark for the hybrid systems, calculated as the average of GB (138.6 kN) and JB (121.7 kN), is 130.15 kN. The JBGS configuration (Jute bottom + Glass sides) achieved an experimental ultimate load of 137.6 kN, resulting in a Synergy Index of 1.057. This value, being greater than 1.0, provides analytical evidence of a positive synergistic effect. It demonstrates that the strategic spatial positioning of glass fibers as side reinforcement prevents the premature debonding of the jute fibers on the bottom face, allowing the hybrid system to exceed its theoretical additive capacity. This quantitative result analytically confirms the "synergy" mentioned throughout this study. The side reinforcement (Glass in JBGS) acts not just as additional strength, but as a protective mechanism that restrains the dilation of the concrete and prevents the premature debonding of the jute fibers on the bottom face, resulting in a system that is more resilient than the sum of its parts.

4.6. Mid-Span Deflection at Ultimate Load

Fig. 11 shows the improvement in mid-span deflection across all beams. JBGS recorded the highest deflection gain (50%), followed by JBJS (47%) and GBGS (41%). GB and GBJS reached 11% and 25% respectively, while JB surprisingly exhibited a 6% reduction, attributed to premature fiber rupture without sufficient anchorage. These findings highlight the ability of hybrid and U-wrapped systems to improve deformability. JBGS, in particular, combined a ductile failure mechanism with increased capacity, offering improved serviceability and resilience.

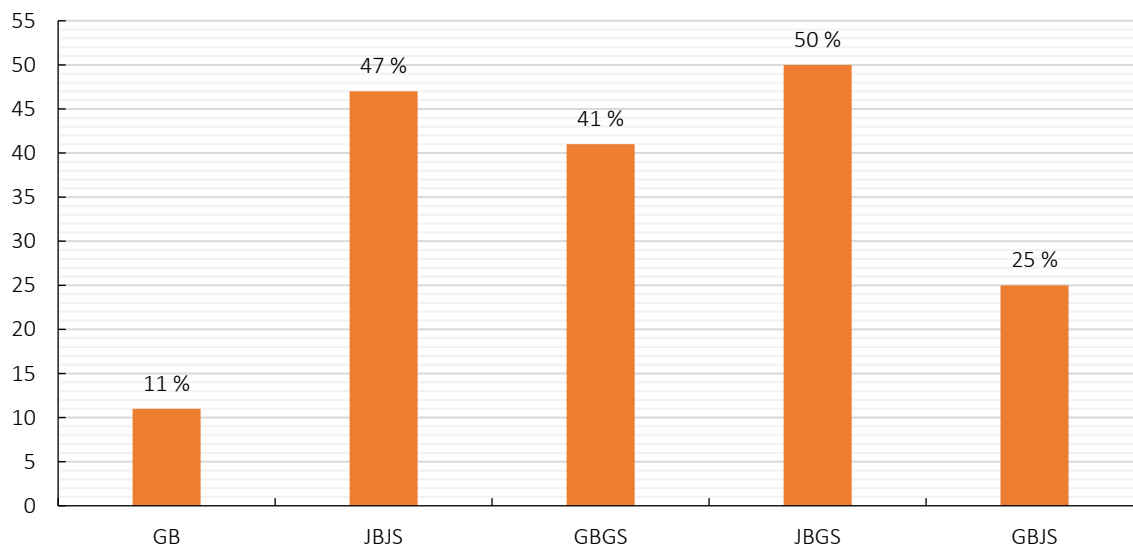


Fig. 11. Percentage increase in mid-span deflection at peak load for all strengthened beams relative to the control beam

4.7. Ductility Index

The ductility index (μ), defined as the area under the load–deflection curve normalized to the control beam, is summarized in Fig. 12 and Table 9. The GBGS beam achieved the highest gain in ductility index (86%), followed by JBJS (80%), GB (78%), JBGS (71%), and JB (62%). GBJS reached 60%, slightly lower due to its stiffer GFRP base and early jute rupture. To quantify the ability of the hybrid JFRP/GFRP system to sustain inelastic deformation, the ductility index (μ) was calculated for all specimens. This index is defined as the ratio of the displacement at the ultimate load to the displacement at the first yielding of the internal longitudinal steel reinforcement, as shown in the following equation:

$$\mu = \frac{\delta_u}{\delta_y} \quad (2)$$

Where; δ_u is the ultimate displacement, δ_y is the yield displacement.

These results confirm that U-shaped and hybrid systems not only improve ultimate strength but also significantly enhance energy absorption and post-yield deformation. The JBGS configuration, in particular, provides an optimal balance between stiffness and ductility. The superior performance of the JBGS configuration compared to GBJS is attributed to the optimized layering sequence that enhances strain compatibility and interfacial stress transfer. In the JBGS system, the inner Jute layer acts as a compliant buffer that uniformly distributes concrete expansion and micro-cracking, allowing the high-stiffness outer Glass layer to reach its ultimate capacity more effectively. This creates a synergetic, multi-stage failure mode that significantly increases energy dissipation and results in a 71% ductility gain. Conversely, in the GBJS setup, the high-stiffness Glass fibers are placed in direct contact with the concrete, subjecting them to localized stress concentrations and premature brittle failure. Because the outer Jute layer in GBJS remains at a lower tension state when the internal Glass layer fractures, the system fails to achieve the same "Hybrid Effect," leading to reduced structural resilience and lower overall ductility.



Fig. 12. Percentage gain in ductility index for all strengthened beams compared to the unstrengthened referenc.

The ductility of the strengthened beams was evaluated using an energy-based index, defined as the ratio of total energy absorption to elastic energy.

$$E_D = \frac{\int_0^{\delta_u} P(\delta) d\delta}{\frac{1}{2} P_y \delta_y} \quad (3)$$

This method was chosen over traditional displacement-based indices because the hybrid GFRP/JFRP system exhibits a pseudo-ductile, multi-stage failure rather than a single discrete yield

point. Physically, this index represents the capacity of the hybrid configuration to dissipate energy through progressive fiber micro-cracking and controlled interfacial slip. While this approach effectively captures the toughening effect of the jute fibers, its limitation lies in the sensitivity to the post-peak slope; the index may over-estimate ductility if a significant residual load is maintained despite extensive structural damage.

The enhanced performance of the hybrid system is fundamentally rooted in fiber modulus heterogeneity. By combining high-modulus Glass fibers with lower-modulus Jute fibers, a staggered mechanical activation is achieved. Initially, the stiffer Glass fibers bear the majority of the load, providing the necessary stiffness to arrest micro-cracks. As the system approaches peak strain, the modulus mismatch allows the more compliant Jute fibers to continue dissipating energy and bridging macro-cracks even after the Glass fibers begin to fracture. This heterogeneity prevents a sudden, brittle catastrophic failure, instead facilitating a progressive 'pseudo-ductile' response that explains the 71% increase in the ductility index for the JBGS configuration.

4.8. Evaluation of Elastic Stiffness

The elastic stiffness (K) was evaluated to determine the influence of the strengthening configuration on the structural rigidity of the beams during the serviceability stage. The stiffness is defined as the slope of the load-deflection curve within the linear elastic range:

$$K = \frac{\Delta P}{\Delta \delta} \quad (4)$$

Where ΔP and $\Delta \delta$ represent the increment in load and mid-span deflection, respectively, prior to the yielding of the internal steel reinforcement. The results are summarized in Table 10.

Table 10. Elastic stiffness of plain steel and FRP-strengthened beams

Beam Designation	Strengthening Configuration	Elastic Stiffness (kN/mm)	Increase/decrease over Control (%)
CB	None (Control)	22.97	—
GB	GFRP	26.65	16.1%
JB	JFRP	27.66	20.42%
JBJS	JFRP	21.43	-7.08%
GBGS	Hybrid	23.4	1.87%
JBGS	Hybrid	19.54	-17.55%
GBJS	Hybrid	22.05	-4.17%

As shown in Table 10, the impact of FRP strengthening on elastic stiffness varied significantly depending on the fiber type and layout. The JB specimen exhibited the highest stiffness enhancement of 20.42%, followed closely by the GB specimen at 16.1%. Among the hybrid configurations, GBGS showed a slight improvement of 1.87%, effectively maintaining the rigidity of the control beam while providing the benefits of hybridization.

Conversely, the JBGS and JBJS configurations showed a reduction in initial stiffness of 17.55% and 7.08%, respectively. This decrease in the linear-elastic slope suggests that the specific arrangement of jute and glass layers in these specimens may have introduced a slight increase in initial compliance or localized micro-cracking during the early loading stages before the FRP contribution was fully mobilized. Despite these variations in the serviceability range, these configurations ultimately contributed to significant gains in ultimate capacity and ductility as discussed in previous sections.

5. Conclusions

This study experimentally evaluated the flexural behavior of reinforced concrete beams externally strengthened with GFRP, JFRP, and hybrid GFRP–JFRP systems. Fourteen beams were tested under four-point bending to assess the effects of fiber type, layout configuration, and positioning on

strength, deformability, and failure mode. Based on the analysis of load–deflection behavior, failure patterns, and ductility, the following conclusions can be drawn:

- GFRP-based U-wrap reinforcement (GBGS) provided the highest strength gain, with a 44% increase in ultimate load relative to the control beam.
- The JBJS (JFRP U-wrap) configuration demonstrated a 38% increase in strength and the second-highest ductility, highlighting the effectiveness of natural fibers when provided with proper anchorage.
- The JBGS (jute bottom, glass sides) hybrid system achieved a 28% increase in load capacity, a 50% gain in deflection, and a 71% rise in ductility index, identifying it as the most balanced and resilient system tested.
- Failure modes were significantly affected by fiber type and positioning. GFRP systems were prone to debonding, JFRP systems failed by fiber rupture, while hybrid systems exhibited mixed and more stable failure behavior.
- U-shaped configurations improved both strength and deformability by enhancing confinement and delaying premature debonding, especially in hybrid reinforcements.

The results confirm that hybrid GFRP–JFRP systems, when strategically arranged, can achieve competitive performance compared to fully synthetic systems while introducing ductility and sustainability advantages. This approach offers a viable, eco-efficient alternative for structural strengthening applications in civil infrastructure, including beams, slabs, and pavement substructures.

Despite the significant performance gains observed in the JBJS and JBGS configurations, several limitations of the current study should be acknowledged to guide future investigations:

- Long-term Durability: Investigation is needed on JFRP degradation under environmental stressors like moisture, UV, and concrete alkalinity.
- Cyclic Loading: Testing under fatigue and seismic-type loading is required to verify the hybrid system's performance in infrastructure applications.
- Bond-Slip Behavior: Micro-scale analysis of the hybrid FRP-to-concrete interface is necessary to better define debonding mechanisms.
- Size Effect: Validation on larger-scale structural elements is recommended to confirm the scalability of the JBGS configuration.

Future investigations should incorporate standardized pull-off and bond-slip testing to characterize the interfacial constitutive laws and develop analytical load transfer models that account for the complex stress distribution between synthetic and natural fiber layers. Furthermore, to transition current qualitative sustainability claims into demonstrated outcomes, future research must include Life Cycle Assessments (LCA) to provide quantitative environmental indicators and durability testing under accelerated aging to evaluate the long-term performance of jute fibers. Additionally, exploring the use of bio-based resins in place of traditional epoxies could further optimize the eco-efficiency of the system, aligning with the growing demand for fully sustainable structural rehabilitation.

Future research should focus on long-term durability under environmental exposure, fatigue resistance under cyclic loads, and the development of predictive models for hybrid FRP systems. Optimization of fiber orientation, layer thickness, and anchorage details will be essential for translating these findings into standardized design guidelines.

Nomenclature

Symbol	Description	Unit
a/b	Shear span-to-effective depth ratio	—
E_c	Modulus of elasticity of concrete	GPa
K	Elastic stiffness	kN /mm
M_n	Nominal flexural strength	kN ·m
P_u	Ultimate load capacity	kN
V_n	Nominal shear strength	kN

Symbol	Description	Unit
δ_u	Mid-span deflection at ultimate load	mm
δ_y	Mid-span deflection at steel yielding	mm
μ	Ductility index	—
E_D	Energy-based ductility index	—
ρ	Reinforcement ratio	%
ΔP	Change in applied load	kN
$\Delta \delta$	Change in deflection	kN
S_{yn}	Synergy Index	—
$P_{exp (hybrid)}$	Experimental ultimate load of the hybrid specimen	kN
P_i	Ultimate load	kN
V_i	Relative contribution of each fiber type	kN

References

- [1] Mosley WH, Bungey JH, Hulse R. Reinforced concrete design. Springer; 1999. <https://doi.org/10.1007/978-1-349-14911-7>
- [2] Wight JK, MacGregor JG. Reinforced concrete. Pearson Education UK Sydney, NSW; 2016.
- [3] Chatbi M, et al. Nano-Clay Platelet Integration for Enhanced Bending Performance of Concrete Beams Resting on Elastic Foundation: An Analytical Investigation. Materials. 2023;16(14):5040. <https://doi.org/10.3390/ma16145040>
- [4] Kecir A, Chatbi M, Harrat ZR, Bouiadjra MB, Bouremana M, Krouer B. Enhancing the Mechanical Performance of Concrete Slabs through the Incorporation of Nano-sized Iron Oxide Particles (Fe₂O₃): Non-local Bending Analysis. Periodica Polytechnica Civil Engineering. 2024;68(3):842-858. <https://doi.org/10.3311/PPci.23016>
- [5] Reynolds CE, Steedman JC, Threlfall AJ. Reinforced concrete designer's handbook. CRC Press; 2007.
- [6] Benfrid A, Benbakhti A, Harrat ZR, Chatbi M, Krouer B, Bouiadjra MB. Thermomechanical Analysis of Glass Powder Based Eco-concrete Panels: Limitations and Performance Evaluation. Periodica Polytechnica Civil Engineering. 2023;67(4):1284-1297. <https://doi.org/10.3311/PPci.22781>
- [7] Delatte N. Concrete pavement design, construction, and performance. CRC Press; 2018. <https://doi.org/10.1201/9781482288483>
- [8] Hassoun MN, Al-Manaseer A. Structural concrete: theory and design. John Wiley & Sons; 2020.
- [9] Chatbi M, Krouer B, Benatta MA, Harrat ZR, Amziane S, Bouiadjra MB. Bending analysis of nano-SiO₂ reinforced concrete slabs resting on elastic foundation. Structural Engineering and Mechanics, An Int'l Journal. 2022;84(5):685-697.
- [10] Harrat ZR, et al. Modeling the Thermoelastic Bending of Ferric Oxide (Fe₂O₃) Nanoparticles-Enhanced RC Slabs. Materials. 2023;16(8):3043. <https://doi.org/10.3390/ma16083043>
- [11] Benfrid A, Chatbi M, Harrat ZR, Bouiadjra MB. Selective Integration of Waste-Derived Glass Nanopowders in Structural Wall Concrete: Improving Thermal Efficiency and Elasto-Mechanical Properties for Sustainable Construction. Periodica Polytechnica Civil Engineering. 2025;69(3):954-965. <https://doi.org/10.3311/PPci.39913>
- [12] Mehta PK. Concrete: Structure, properties and materials. 1986.
- [13] De Schutter G. Damage to concrete structures. Taylor & Francis; 2017. <https://doi.org/10.1201/b12914>
- [14] Harrat ZR, et al. Bending analysis of nano-Fe₂O₃ reinforced concrete slabs exposed to temperature fields and supported by viscoelastic foundation. Advances in Concrete Construction. 2024;17(2):111-126.
- [15] Woodson RD. Concrete structures: protection, repair and rehabilitation. Butterworth-Heinemann; 2009.
- [16] Modi PI, Patel CN. Repair and Rehabilitation of Concrete Structures. PHI Learning Pvt. Ltd.; 2015.
- [17] Asmara YP. Concrete Reinforcement Degradation and Rehabilitation: Damages, Corrosion and Prevention. Springer Nature; 2023. <https://doi.org/10.1007/978-981-99-5933-4>
- [18] Moulay-Ali A, Abdeldjalil M, Khelafi H. An experimental study on the optimal compositions of ordinary concrete based on corrected dune sand-Case of granular range of 25 mm. Case Studies in Construction Materials. 2021;14:e00521. <https://doi.org/10.1016/j.cscm.2021.e00521>
- [19] Benbakhti A, Benfrid A, Harrat ZR, Chatbi M, Bouiadjra MB, Krouer B. An Analytical Analysis of the Hydrostatic Bending to Design a Wastewater Treatment Plant by a New Advanced Composite Material. Journal of Composite & Advanced Materials/Revue des Composites et des Matériaux Avancés. 2024;34(2):177-188. <https://doi.org/10.18280/rcma.340207>
- [20] Dine Elhennani S, et al. Buckling and Free Vibration Analyses of Various Nanoparticle Reinforced Concrete Beams Resting on Multi-Parameter Elastic Foundations. Materials. 2023;16(17). <https://doi.org/10.3390/ma16175865>

- [21] Thirupathi S, Palanisamy S, Asokan V. Enhancing sustainability in construction: Exploring the self-healing mechanisms of bio-concrete. *Res Eng Struct Mater.* 2025;11(6):2731-2744. <https://doi.org/10.17515/resm2025-517ma1105rs>
- [22] Sreadha AR, Pany C, Varkey MV. A Review on Seismic Retrofit of Beam-Column Joints. *International Journal for Modern Trends in Science and Technology.* 2020;6(9):80-93. <https://doi.org/10.46501/IJMTST060914>
- [23] Sangeetha P, Prithvi R, Ashwin KV, Murali B, Yaashika M. Structural behavior of deficient hollow steel columns strengthened using GFRP. *Res Eng Struct Mater.* 2025;11(6):2745-2760. <https://doi.org/10.17515/resm2025-541st1121rs>
- [24] Van Den Einde L, Zhao L, Seible F. Use of FRP composites in civil structural applications. *Construction and Building Materials.* 2003;17(6-7):389-403. [https://doi.org/10.1016/S0950-0618\(03\)00040-0](https://doi.org/10.1016/S0950-0618(03)00040-0)
- [25] Belarbi A, Dawood M, Acun B. Sustainability of fiber-reinforced polymers (FRPs) as a construction material. In: *Sustainability of construction materials.* Elsevier; 2016. p. 521-538. <https://doi.org/10.1016/B978-0-08-100370-1.00020-2>
- [26] Hota G, Liang R. Advanced fiber reinforced polymer composites for sustainable civil infrastructures. In: *Proceedings of the International Symposium on Innovation & Sustainability of Structures in Civil Engineering*; 2011; Xiamen University.
- [27] Alzate A, Arteaga Á, de Diego A, Perera R. Shear strengthening of reinforced concrete beams using fibre reinforced polymers (FRP). *European Journal of Environmental and Civil Engineering.* 2009;13(9):1051-1060. <https://doi.org/10.1080/19648189.2009.9693172>
- [28] Maraveas C, Miamis K, Vrakas AA. Fiber-reinforced polymer-strengthened/reinforced concrete structures exposed to fire: A review. *Structural Engineering International.* 2012;22(4):500-513. <https://doi.org/10.2749/101686612X13363929517613>
- [29] Aouadi A, Ghernouti Y. Comportament flexibil-forfecare a grinzilor de beton armat Întărit de diverse rețele de compozite încorporate. *Revista Romana de Inginerie Civila.* 2020;11(2):196-207. <https://doi.org/10.37789/rjce.2020.11.2.4>
- [30] Frih A, et al. A novel hyperbolic integral-Quasi-3D theory for flexural response of laminated composite plates. *Geomechanics and Engineering.* 2023;34(3):233-250.
- [31] Chatbi M, Harrat ZR, Ghazoul T, Bouiadjra MB. Free vibrational analysis of composite beams reinforced with randomly aligned and oriented carbon nanotubes, resting on an elastic foundation. *Journal of Building Materials and Structures.* 2022;9(1):22-32. <https://doi.org/10.34118/jbms.v9i1.1895>
- [32] Lakhder M, Benfrid A, Bouiadjra MB, Chatbi M, Harrat ZR, Benbakhti A. The Mechanical Bending Behavior of a New Metal Alloy Nano-Reinforced Plate Incorporating Tungsten Nanoparticles. *NIPES - Journal of Science and Technology Research.* 2025;7(3):423-440. <https://doi.org/10.37933/nipes/7.3.2025.1682>
- [33] Sreadha AR, Pany C. Static, Free Vibration and Buckling Analysis of Composite Panels; A Review. *Advanced Journal of Graduate Research.* 2020;9(1):21-45. <https://doi.org/10.21467/ajgr.9.1.21-45>
- [34] Ramesh B, Eswari S, Sundararajan T. Flexural behaviour of glass fibre reinforced polymer (GFRP) laminated hybrid-fibre reinforced concrete beams. *SN Applied Sciences.* 2020;2(2):204. <https://doi.org/10.1007/s42452-020-1966-2>
- [35] Iskender M, Karasu B. Glass fibre reinforced concrete (GFRC). *El-Cezerî Fen ve Mühendislik Dergisi.* 2018;5(1):136-162. <https://doi.org/10.31202/ecjse.371950>
- [36] Aslani F, Gunawardena Y, Dehghani A. Behaviour of concrete filled glass fibre-reinforced polymer tubes under static and flexural fatigue loading. *Construction and Building Materials.* 2019;212:57-76. <https://doi.org/10.1016/j.conbuildmat.2019.03.321>
- [37] Correia JR. Fibre-reinforced polymer (FRP) composites. In: *Materials for construction and civil engineering: science, processing, and design.* Springer; 2014. p. 501-556. https://doi.org/10.1007/978-3-319-08236-3_11
- [38] Lee SW, Hong KJ, Park S. Current and future applications of glass-fibre-reinforced polymer decks in Korea. *Structural Engineering International.* 2010;20(4):405-408. <https://doi.org/10.2749/101686610793557672>
- [39] Frigione M, Lettieri M. Durability issues and challenges for material advancements in FRP employed in the construction industry. *Polymers.* 2018;10(3):247. <https://doi.org/10.3390/polym10030247>
- [40] Zaman A, Gutub SA, Wafa MA. A review on FRP composites applications and durability concerns in the construction sector. *Journal of Reinforced Plastics and Composites.* 2013;32(24):1966-1988. <https://doi.org/10.1177/0731684413492868>
- [41] Wu Z, Wang X, Zhao X, Noori M. State-of-the-art review of FRP composites for major construction with high performance and longevity. *International Journal of Sustainable Materials and Structural Systems.* 2014;1(3):201-231. <https://doi.org/10.1504/IJSMSS.2014.062757>

- [42] Chen C, et al. Comparative analysis of natural fiber reinforced polymer and carbon fiber reinforced polymer in strengthening of reinforced concrete beams. *Journal of Cleaner Production*. 2020;263:121572. <https://doi.org/10.1016/j.jclepro.2020.121572>
- [43] Ahamed MS, Ravichandran P, Krishnaraja A. Natural fibers in concrete-A review. In: IOP Conference Series: Materials Science and Engineering. IOP Publishing; 2021. vol. 1055, no. 1, p. 012038. <https://doi.org/10.1088/1757-899X/1055/1/012038>
- [44] Yan L, Chouw N. Natural FRP tube confined fibre reinforced concrete under pure axial compression: A comparison with glass/carbon FRP. *Thin-Walled Structures*. 2014;82:159-169. <https://doi.org/10.1016/j.tws.2014.04.013>
- [45] Abdelhak A, Abdelkader F, Fakhreddine D, Imene B, Adel D, Chebabhi L. Reduce energy consumption by using glass fiber reinforced concrete. *Brazilian Applied Science Review*. 2024;8(2):e74713. <https://doi.org/10.34115/basrv8n2-019>
- [46] Sreadha AR, Pany C. Review on Fabrication of Bamboo Composite Materials Reinforced Concrete. *Journal of Science & Technology*. 2020;5(3):258-279. <https://doi.org/10.46243/jst.2020.v5.i3.pp258-279>
- [47] Islam MS, Ahmed SJ. Influence of jute fiber on concrete properties. *Construction and Building Materials*. 2018;189:768-776. <https://doi.org/10.1016/j.conbuildmat.2018.09.048>
- [48] Zakaria M, Ahmed M, Hoque MM, Islam S. Scope of using jute fiber for the reinforcement of concrete material. *Textiles and Clothing Sustainability*. 2016;2(1):11. <https://doi.org/10.1186/s40689-016-0022-5>
- [49] Tan H, Yan L, Huang L, Wang Y, Li H, Chen JY. Behavior of sisal fiber concrete cylinders externally wrapped with jute FRP. *Polymer Composites*. 2017;38(9):1910-1917. <https://doi.org/10.1002/pc.23761>
- [50] Zakaria M, Ahmed M, Hoque M, Shaid A. A comparative study of the mechanical properties of jute fiber and yarn reinforced concrete composites. *Journal of Natural Fibers*. 2020.
- [51] Gulzar MA, et al. Influence of jute fiber on tensile, electrical, and permeability characteristics of slag concrete: a better, cheaper, and eco-friendly substitute for conventional concrete. *Journal of Natural Fibers*. 2023;20(1):2170947. <https://doi.org/10.1080/15440478.2023.2170947>
- [52] Idder A, Hamouine A, Labbaci B, Abdeldjebar R. The porosity of stabilized earth blocks with the addition plant fibers of the date palm. *Civil Engineering Journal*. 2020;6(3):478-494. <https://doi.org/10.28991/cej-2020-03091485>
- [53] Khan MB, et al. Effects of jute fiber on fresh and hardened characteristics of concrete with environmental assessment. *Buildings*. 2023;13(7):1691. <https://doi.org/10.3390/buildings13071691>
- [54] Pradeep P, Dhas JER, Ramachandran M, Retnam BSJ. Mechanical Characterization of jute fiber over glass and carbon fiber reinforced polymer composites. *International Journal of Applied Engineering Research*. 2015;10(11):10392-10396.
- [55] Sridhar J, Gobinath R, Kirgiz MS. Comparative study for efficacy of chemically treated jute fiber and bamboo fiber on the properties of reinforced concrete beams. *Journal of Natural Fibers*. 2022;19(15):12224-12234. <https://doi.org/10.1080/15440478.2022.2054894>
- [56] Ahmad J, Arbili MM, Majdi A, Althoey F, Deifalla AF, Rahmawati C. Performance of concrete reinforced with jute fibers (natural fibers): A review. *Journal of Engineered Fibers and Fabrics*. 2022;17:15589250221121871. <https://doi.org/10.1177/15589250221121871>
- [57] Akid ASM, Wasiew QA, Sobuz MHR, Rahnam T, Tam VW. Flexural behavior of corroded reinforced concrete beam strengthened with jute fiber reinforced polymer. *Advances in Structural Engineering*. 2021;24(7):1269-1282. <https://doi.org/10.1177/1369433220974783>
- [58] Varma DA, Sarker PK, Madhavan MK, Jayanarayanan K. Experimental and analytical assessment of strengthening of concrete cylinders using jute/basalt hybrid textile fiber reinforced mortar. In: *Structures*. Elsevier; 2024. vol. 70, p. 107807. <https://doi.org/10.1016/j.istruc.2024.107807>
- [59] Maciel LP, et al. Experimental Analysis of Shear-Strengthened RC Beams with Jute and Jute-Glass Hybrid FRPs Using the EBR Technique. *Buildings*. 2024;14(9):2893. <https://doi.org/10.3390/buildings14092893>
- [60] Benaddache L, et al. Comparative study on externally bonded heat-treated jute and glass fiber reinforcement for repair of pre-cracked high performance concrete beams. *Archives of Civil and Mechanical Engineering*. 2024;24(2):82. <https://doi.org/10.1007/s43452-024-00899-5>
- [61] Sen T, Reddy HJ. Strengthening of RC beams in flexure using natural jute fibre textile reinforced composite system and its comparative study with CFRP and GFRP strengthening systems. *International Journal of Sustainable Built Environment*. 2013;2(1):41-55. <https://doi.org/10.1016/j.ijisbe.2013.11.001>
- [62] Djeddi F, Ghernouti Y, Abdelaziz Y, Alex L. Strengthening in flexure-shear of RC beams with hybrid FRP systems: Experiments and numerical modeling. *Journal of Reinforced Plastics and Composites*. 2016;35(22):1642-1660. <https://doi.org/10.1177/0731684416662532>
- [63] Nurazzi N, et al. A review on mechanical performance of hybrid natural fiber polymer composites for structural applications. *Polymers*. 2021;13(13):2170. <https://doi.org/10.3390/polym13132170>

- [64] Ribeiro F, Correia L, Sena-Cruz J. Hybridization in FRP composites for construction: State-of-the-art review and trends. *Journal of Composites for Construction*. 2024;28(4):04024024. <https://doi.org/10.1061/JCCOF2.CCENG-4486>
- [65] Alabi OO, Olaleye AO, Akande TO, Gbadeyan OJ, Fasina AO. Advanced engineering applications of natural fiber composites for wind turbine blades. *Sustainable Smart Composites*. 2025;67-86. <https://doi.org/10.1201/9781003527275-4>
- [66] Hason MM, Hanoon AN, Saleem SJ, Hejazi F, Al Zand AW. Characteristics of experimental ductility energy index of hybrid-CFRP reinforced concrete deep beams. *SN Applied Sciences*. 2021;3(2):200. <https://doi.org/10.1007/s42452-021-04202-6>
- [67] Harris HG, Somboonsong W, Ko FK. New ductile hybrid FRP reinforcing bar for concrete structures. *Journal of Composites for Construction*. 1998;2(1):28-37. [https://doi.org/10.1061/\(ASCE\)1090-0268\(1998\)2:1\(28\)](https://doi.org/10.1061/(ASCE)1090-0268(1998)2:1(28))
- [68] V BK, M CCH, Lingeshwaran N, S KI. Structural application of fiber reinforced concrete. *Res Eng Struct Mater*. 2025;11(5):1913-1946. <http://dx.doi.org/10.17515/resm2024.305st0531rs>
- [69] Chatbi M, Lozančić S, Harrat ZR, Hadzima-Nyarko M. Computational Models for the Vibration and Modal Analysis of Silica Nanoparticle-Reinforced Concrete Slabs with Elastic and Viscoelastic Foundation Effects. *Modelling*. 2026;7(1):8. <https://doi.org/10.3390/modelling7010008>
- [70] Comité Européen de Normalisation. Eurocode 2: Design of concrete structures-Part 1-1: General rules and rules for buildings. London: British Standard Institution; 2004. p. 37.
- [71] Barris C, Torres L, Vilanova I, Miàs C, Llorens M. Experimental study on crack width and crack spacing for Glass-FRP reinforced concrete beams. *Engineering Structures*. 2017;131:231-242. <https://doi.org/10.1016/j.engstruct.2016.11.007>
- [72] Alsuhaibani E, Aldukail A. Experimental Study on Hybrid Steel-GFRP Reinforced Concrete Beams with Different Stirrup Types and Spacings. *Buildings*. 2025;15(22):4047. <https://doi.org/10.3390/buildings15224047>
- [73] Bui LVH, Stitmannathum B, Ueda T. Ductility of concrete beams reinforced with both fiber-reinforced polymer and steel tension bars. *Journal of Advanced Concrete Technology*. 2018;16(11):531-548. <https://doi.org/10.3151/jact.16.531>

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Ascertaining the physical and mechanical performance of concrete modified with locally sourced coconut fiber ash

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Abstract

This current study evaluated the physical and mechanical performance of concrete modified with locally sourced coconut fiber ash. Concrete specimens were prepared with Portland limestone cement (PLC) partially replaced with locally sourced coconut fiber ash (CFA) in proportions of 0%, 5%, 10%, 15%, and 20%. Tests, including slump, XRF, flexural strength, and compressive strength tests, aided in evaluating the physical and mechanical performance of CFA-modified concrete. XRF analyses revealed the presence of essential elements in cements in CFA: Ca (2.3150 %), Si (3.0900%), and Fe (0.4790%). This affirmed the potential of CFA as a supplementary cementitious material (SCM). Workability of CFA-modified concretes was low: ranged from 20mm to 40 mm for 5% to 20% partial replacement levels. Also, density and water absorption (WA) decreased as CFA concentration in concrete increased. Concrete was classified as normal-weight concrete, with values ranging between 2000 kg/m³ - 2600 kg/m³, just like the control. None of the WA values was less than the control for 28 days and 70 days of curing. Compressive and flexural strengths rose to 10% optimum partial replacement levels, beyond which loss of strength set in for days 7, 28 and 70. Empirically, the elemental composition of CFA locally sourced in Ghana was established, affirming its use as an SCM. It also generates an equation model to guide construction practitioners in the use of CFA as an SMC in PLC concrete.

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1. Introduction

Cement is the main binding material in concrete for civil infrastructure purposes. Globally, it is estimated that cement production accounts for about 7% contribution of carbon dioxide emissions in the atmosphere [1, 2], significantly contributing to the global climate change being experienced [3, 4]. Whereas sustainable development goal (SDG) 13 calls for actions to combat climate change, and SDG 12 promotes ensuring responsible consumption and production, which, among others, targets reduction in the use of fossil fuel, creating green and decent jobs, promoting quality of life for all, and reducing waste generation through reuse, prevention, recycling, and reduction [5]. The overreliance on some conventional materials, such as Portland cement in the 21st century for concrete and infrastructure construction is a major setback towards the realization of SDGs 12 and

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13, and sustainability in concrete production. It is estimated that the demand for cement increases at a rate of 9% per year across the globe [7, 8]. In view of this, an increasing number of studies within the domain of construction materials have explored the potential of some agricultural by-products, including coconut fiber ash (CFA), whose carbon net worth is lower than cement after recycling, as a supplementary cementitious material (SCM) for concrete production within some national contexts: Nigeria, India, and Malaysia (see [7, 8]). However, none of the studies had evaluated the potential of locally sourced coconut fiber ash from Ghana as a supplementary cementitious material in concrete production. This lack of empirical studies within the context of Ghana is a major setback to Ghana's use of CFA for concretes, thereby reducing the carbon footprint of cement in concrete production. Thus, leaving the effort of the construction industry in Ghana to combat climate change in limbo. Sourcing CFA locally reduces the use of fossil fuel in transporting CFA and Portland cements into Ghana and limits the impact of cement production activities on the environment [1,5,9]. Regarding the use of CFA as an SCM, there is a lack of consensus among researchers regarding the optimal percentage of CFA to replace cement, the elemental composition of CFA, and the performance of CFA-modified concrete. This had been attributed to the varying processes in obtaining CFA, including processing temperature and the origin of the coconut fiber for CFA, among others [1,7, 8]. Furthermore, whereas CFA had been used to replace Ordinary Portland cement (OPC) in some existing studies (see [7, 8]), little is known, if any, of studies that partially replaced Portland limestone cement (PLC) with CFA. PLC is the cement commonly found in Ghana and used in construction. Thus, the relevance of the current study as it seeks to address the weaknesses highlighted in existing studies. It also provides empirical evidence on the potential of CFA locally sourced from Ghana as an SCM in concrete production. This addresses the lack of literature within the context of Ghana on CFA as an SCM, which had derailed the effort of recycling or repurposing CFA for construction purposes in Ghana towards attaining sustainability in concrete production. Finding an alternative use for CFA in concrete promotes agricultural waste valorization. Properties germane to the functional performance of cement-based concrete include the physical and mechanical properties [1,7, 8]. It is against this backdrop that this current study seeks to evaluate the physical and mechanical performance of concrete modified with locally sourced coconut fiber ash. The specific objectives that guided the study were:

- To evaluate the physical performance of CFA-modified concrete, and
- To evaluate the mechanical performance of CFA-modified concrete.

Physical and mechanical performance of concrete had been evaluated in previous studies using a combination of parameters such as slump value, temperature, compressive strength, tensile strength, density, and water absorption (see [1, 8]). Regarding the physical performance of concrete, studies including Arimanwa et al. [10] report that the density of CFA concrete specimens increases with an increase in curing age, and for each curing age, density decreases with an increase in CFA concentrations, while none of the concrete specimens with CFA outperformed that of the control specimen [10]. Water absorption was also found to increase as CFA concentration in concrete increases, while the control specimen records the least absorption value [10]. Regarding mechanical performance of CFA concrete, studies including Sanjay and Rajeev [6] report a decreasing compressive strength as CFA concentration increases [6]; whereas according Waqar et al. [8], compressive strength, tensile strength, and flexural strength increase up to the optimum partial replacement level and decrease afterwards as the percentage of CFA replacement for cement increases [8]. In this current study, a combination of slump value, temperature, compressive strength, flexural strength, density, and water absorption aided in evaluating the physical and mechanical performance of CFA-modified concrete. Although this study does not seek to generalize its findings to be the same across the globe, due to the variety in processing the CFA amidst the origin of the coconut species for the CFA, the outcome offers lessons for countries such as, Burkina Faso, Togo, and Côte d'Ivoire, whose coconut species share close resemblances with Ghana. The study helps in practice as it establishes the optimum partial replacement level essential for construction practitioners in the application of CFA as an SCM in concrete production. It also generates an equation model to guide construction practitioners in the use of CFA as an SMC in PLC concrete.

2. Overview of CFA Use in Concrete Within Some National Contexts

In a CFA study in Nigeria by Anifowoshe and Nwaiwu [7], coconut fiber ash was obtained by drying the fresh coconut fibers and then burning the fibers in an oven at a temperature between 600°C - 700°C. The ash was made to cool and sieved using a 150-micron sieve. CFA was used to partially replace ordinary Portland cement in proportions of 0%, 10%, 30%, 50%, 70% and 100% and cured for 7, 14, 28, 42, 63, and 90 days. Flexural strength and compressive strength were determined, as well as the oxide composition of CFA and ordinary Portland cement. The ratio mix of concrete was 1:2:4. The water-cement ratio was 0.55. The results indicated that compressive strength increases with curing time. Also, as the percentage of partial replacement increases, compressive strength decreases. Regarding flexural strength, it was found that flexural strength increases with curing time. Also, flexural strength decreases with increasing level of CFA replacing cement partially. In a related study, CFA was reaffirmed to be an SCM, and that it increases compressive strength and flexural strength of concrete when it partially replaces cement in a concrete mix [11].

In another related study, cement was partially replaced with CFA in proportions of 0%, 5%, 10%, and 15 %. Concrete specimens were tested at ages 7, 14, and 28 days. The study, among others, concluded that compressive strength decreased as the concentration of CFA in concrete increased [12]. In another study by Waqar et al. [8] in Malaysia, CFA partially replaced ordinary Portland cement in proportions of 3%, 6%, 9%, 12%, 15% and 18%. The standard size for coarse aggregate used was 10 mm, whereas the fine aggregate ranged between 0.4- and 2.52-mm. Water-cement ratio was 0.3. A 1% water-reducing superplasticizer, Sika ViscoCrete – 2192, was also added. Workability, tensile strength, flexural strength, and compressive strength properties of concrete were evaluated. Compressive strength increased up to 9% partial replacement and decreased afterwards as the percentage of CFA replacement for cement increased. For each level of CFA replacement, compressive strength increased with age of curing. A similar account was recorded for the flexural and tensile strengths. For flexural strength, 12% CFA recorded the optimum strength beyond which loss of binding integrity sets in. For splitting tensile strength, the optimum strength was at 9% CFA partially replacing cement. The strength performance beyond 28 days was not investigated.

In a study by Sanjay and Rajeev [6] in India, CFA was obtained by drying and burning coconut fiber in the open air with a temperature range of 600 °C to 700 °C. CFA was sieved using a 150-micron sieve. Concrete specimens were prepared with CFA partially replacing ordinary Portland cement in proportions of 0%, 5%, 10%, 15%, 20%, and 25 %. Super plasticizer was added (2.0% by weight of cement). Concrete specimens were tested at ages 7, 28, 60, and 90 days of curing. The slump test results showed that workability of the concrete decreased as the CFA content increased. The compressive strength of CFA concrete increased with curing age but decreased as CFA percentage increased. None of the compressive strength values exceeded that of the control; however, 5% was the optimum among the CFA-modified concrete specimens [6]. The study did not evaluate the flexural strength of the concrete.

Thus, from the literature reviewed, the optimum level of CFA replacement varied from country to country, as well as the temperature for processing the material. None of the studies had Ghana as its focus, or partially replaced Portland limestone cement (PLC) with CFA. The elemental composition of CFA locally sourced from Ghana is not known. Thus, the relevance of the current study as it addresses these weaknesses in addition to demonstrating the physical and mechanical performance of CFA-modified concrete.

2.1 Theoretical Basis of the Study

2.1.1 Loss of Clinker Theory

The loss of clinker theory argues that partially replacing PLC with SCM beyond a certain threshold leads to loss of cementitious binder due to the dilution effect [13]. Hence, establishing the optimum level of partial replacement of CFA in CFA-modified concrete is essential and provides guidance for construction practitioners when preparing CFA-modified concretes.

2.1.2 Synthesis of Literature

Based on the theoretical basis and empirical literature review, this current study conceptualizes that partially replacing PLC with CFA leads to an improvement in the mechanical performance of concrete (compressive and flexural strength) until optimum performance is attained, beyond which the loss of clinker effect sets in. More, CFA concentration in a concrete mix decreases the slump, density, and water absorption of concrete.

3. Materials and Methods

Portland Limestone Cement (PLC), CEM II/B-L, rated strength class 42.5 R, was used in this study. X-ray fluorescence (XRF) analysis was performed to determine the elemental composition of cement. Standard elemental composition requirements in PLC include: Ca (Calcium), Si (Silicon), Al (Aluminum), Fe (Iron), Mg (Magnesium), S (Sulphur), Na (Sodium), K (Potassium) [14, 15]. CFA was used as an SCM to replace PLC partially in a CFA-modified concrete. Also, XRF analysis was performed to determine the elemental composition of the locally sourced CFA and to compare it with PLC to ascertain its suitability as an SCM. Potable water, per BS EN 1008:2002, was used for the concrete specimens [16]. It was clean and odorless when smelled. The sieve analysis test was performed in accordance with BS EN 933-1:2012 and BS 1377-3:1990 to determine the particle size distribution of the aggregates for the concrete specimens [17,18].

3.1 Experimental Procedure

Coconut fiber, an agricultural by-product, was cleaned and air-dried under the sun for 7 hours to remove moisture. The sample was burnt in a regulated oven into ashes at a temperature of 500 degrees for 3 hours. It was further sieved to obtain the CFA for the experiment [1, 8]. In this case, the 75 μ m sieve was used to get a finer CFA particle to replace cement partially. According to Ranatunga et al. [1], the fineness of an SCM is essential as it influences the strength development. Concrete mix design was developed based on the trial mix method to guide the process (see Table 1). Firstly, the control concrete, which had 0% CFA, was prepared to achieve the targeted compressive strength of 20 N/mm² at 28 days of curing using cube moulds:150mm x 150 mm x 150mm. In the trial mix design, the water-to-cement ratio was selected as 0.60. Then, PLC was partially replaced with CFA in proportions of 5%, 10%, 15% and 20%. Concrete mixes were named as CFA 0, CFA 5, CFA 10, CFA 15, and CFA 20, when CFA partially replaced PLC in proportions of 0%, 5%,10%,15%, and 20%, respectively. Three concrete specimens each were tested at age 7 days, 28 days, 70 days for compressive strength following BS EN 12390-3:2019 [19], density of concrete was determined in accordance with BS EN 12390 -7:2009 [20], water absorption test of concrete in accordance with BS EN 1881-122: 2011[21], and flexural strength following BS EN 12390-5: 2019 [22]. Concrete was mixed and cured in accordance with BS 1881-125: 1986 and BS EN 12390-2, 2000 [23, 24]. Since CFA is a pozzolanic material that reacts slowly during hydration, compared to cements, compressive and flexural strengths of concrete at age 70 days were essential to understand the strength gained over a longer period. The grading test for coarse aggregates for the concrete mix followed the procedures outlined in BS EN 933-1:2012, which stipulate the sieving method for determining aggregate size [17]. The sieves used were: 22mm, 20mm, 19mm, 14mm, 12.5mm,10mm, 4.75mm, and 2.36 mm. Grading test for fine aggregate (sand) for concrete was done in accordance with BS EN 933-1:2012 [17] and classified following BS 882:1992 and BS EN 12620:2013 [25, 26]. The specific sieves used for the test were: 5mm, 4.75mm, 2.36mm, 1.18mm, 0.6mm, 0.3mm, 0.15mm, and 0.075mm.

Table 1. Mix design for C20 concrete with cement partially replaced with a CFA

Mix ID	CFA (Kg)	Cement (Kg)	Sand (Kg)	Crushed rock (Kg)	Water-cement ratio (0.60) (Kg)
(CFA 0)	0	1.27	1.91	3.81	0.76
(CFA 5)	0.06	1.21	1.91	3.81	0.76
(CFA 10)	0.13	1.14	1.91	3.81	0.76
(CFA 15)	0.19	1.08	1.91	3.81	0.76
(CFA 20)	0.25	1.02	1.91	3.81	0.76

4. Results and Discussions

From Figure 1, 57.38% of the sample passed through the 0.6mm sieve. Thus, the fine aggregate falls within zone II: medium sand, reference to BS 882:1992 and BS EN 12620: 2013 [25, 26]. This was an indication that the fine aggregate was suitable for the production of general concrete.

From Figure 2, the coarse aggregate for the concrete specimens was well graded with an upper limit diameter (D) not exceeding 20mm and a lower limit diameter (d) not exceeding 4.75mm (see Figure 2). The aggregate met the grading limits for 20mm coarse aggregate for concrete, in reference to BS 882:1992 and BS EN 12620:2013 [25, 26]. This indicates well-graded coarse aggregate for a concrete mix.

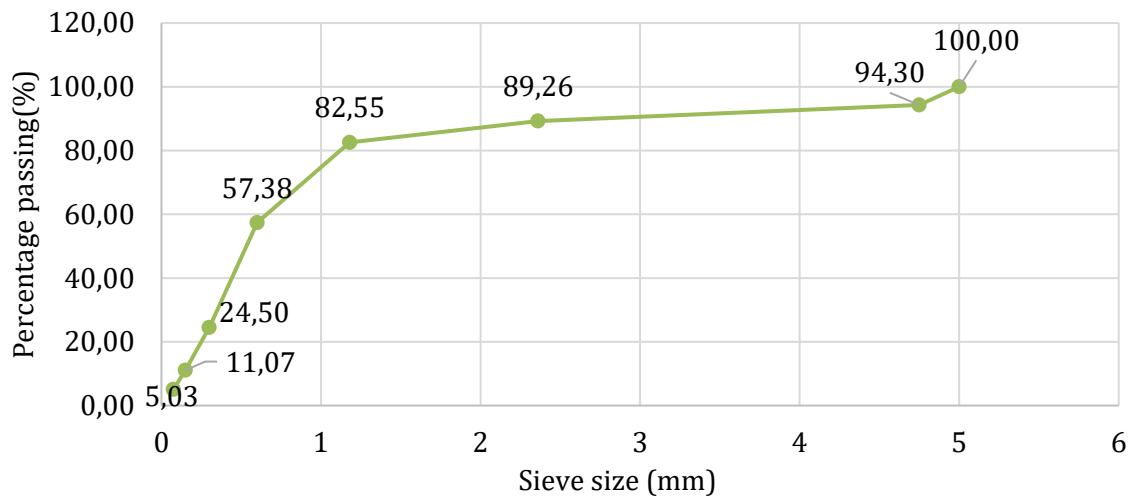


Fig.1. Gradation curve for fine aggregate

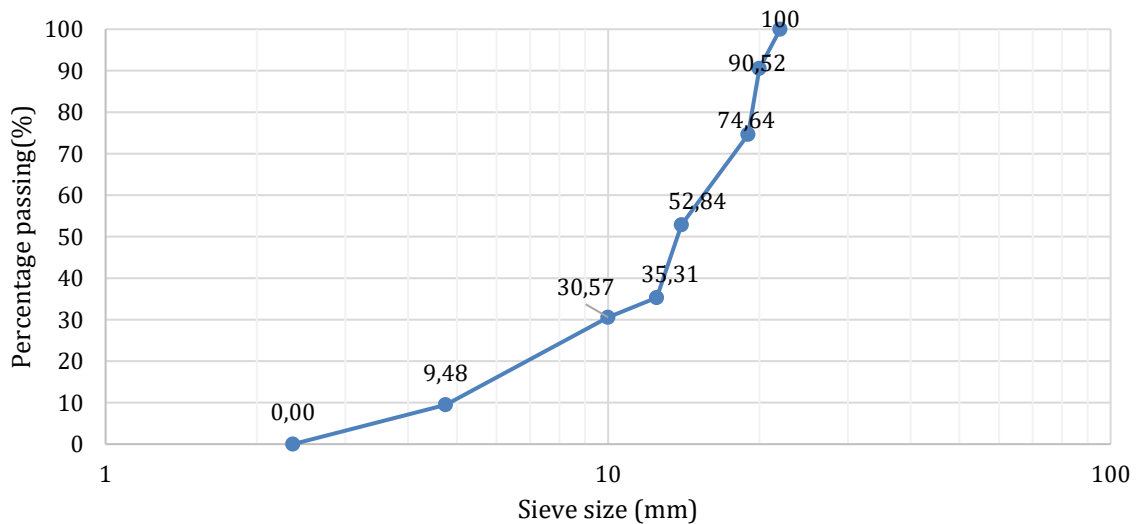


Fig. 2. Gradation curve for coarse aggregates

Table 2. Properties of CFA and PLC

Mater.	Ca	Al	Fe	Si	S	Mg	P	K	Cl	Ti
Cement (%)	44.150	4.1800	3.4650	14.1500	1.8650	2.7750	<0.003075	1.1000	0.0344	0.2680
CFA (%)	2.3150	0.8575	0.4790	3.0900	1.3150	3.6050	2.8850	25.6000	6.0600	0.0465

From Table 2, the elemental composition of CFA was comparable to that of the PLC cement. This affirms the potential of CFA as an SCM. The CFA elements revealed the richness of CFA in Mg (3.6050%), Si (3.0900%), and Ca (2.3150%). Si and Ca are essential elements for the formation of calcium silicate hydroxide and portlandites during hydration, critical for continuous hydraulic and pozzolanic reactivity for continuous strength development and durability of cement-based products [1]. Objective one is to evaluate the physical performance of CFA-modified concrete.

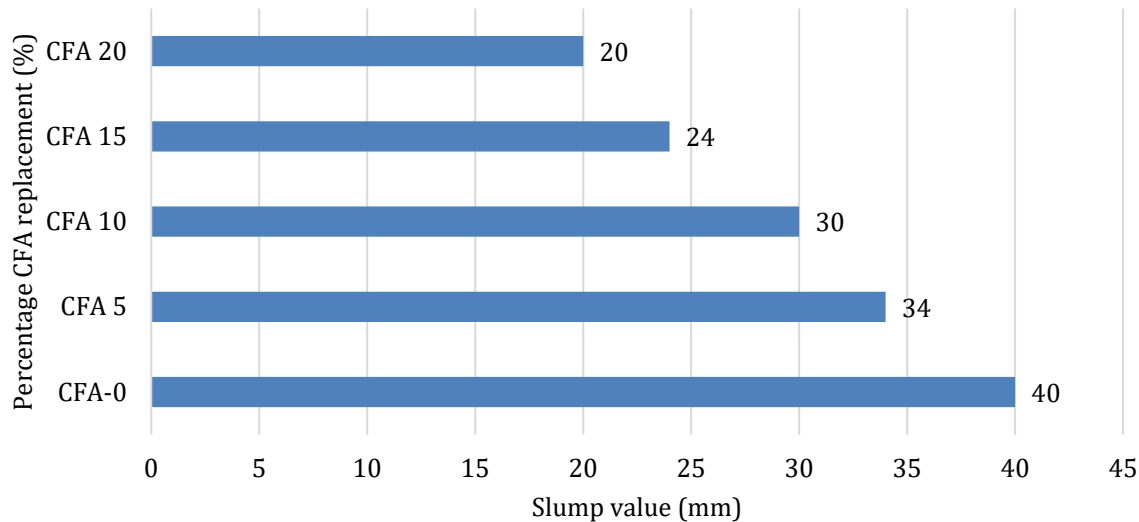


Fig. 3. Workability (slump) of CFA-Modified concretes

From Figure 3, none of the slump values exceeded that of the control specimen. With reference to BS EN 206-1:2000, true slump was ideal for concrete mix, and it ranged from (10mm to 220mm), specifically S1: 10-40 mm slump was suitable for low-workability applications [27]. Thus, with a slump of 20mm to 40mm, recorded by CFA 0, CFA 5, CFA 10, CFA 15, and CFA 20, in this study, the CFA-modified concrete specimens could be classified as S1 slump and ideal for low workability applications. Likewise, from Figure 3, the slump value decreased with a percentage rise in CFA. This indicates a decrease in workability as the CFA concentration increases in the concrete specimens. This affirmed the position of Sanjay and Rajeev [6] that workability decreases when the concentration of CFA as a partial replacement for cement in concrete increases.

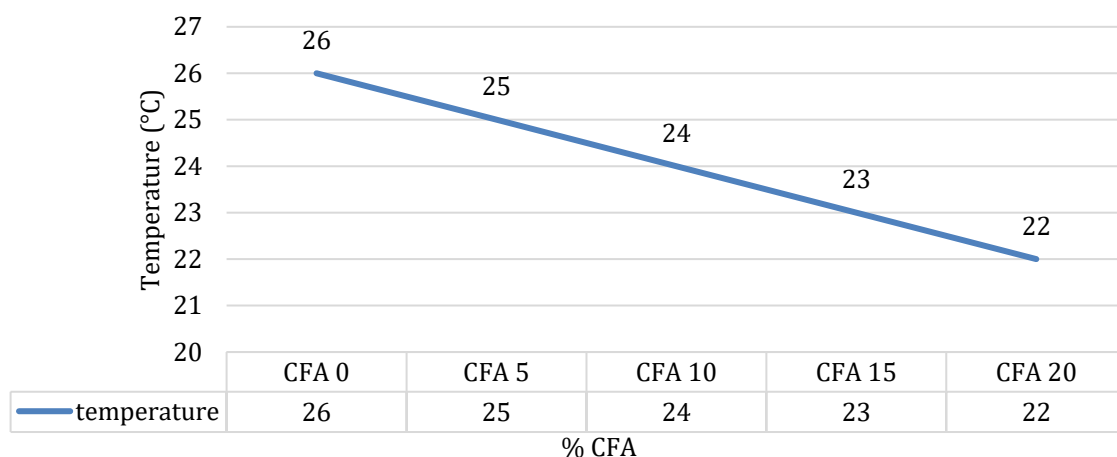


Fig. 4. Influence of CFA on temperature

The temperature for freshly mixed normal concrete should typically range between 10 °C and 32 °C, with 20 °C–27 °C being considered the optimal range for normal concrete [28]. Thus, from Figure 4, the temperature of freshly mixed concrete ranged from 26°C to 22 °C. The trend suggested a decreasing temperature of fresh concrete as the percentage of CFA concentration increased from

0% to 20%. According to Mehta and Monteiro [29], as the level of partial replacement increased, it reduced the heat generated during hydration, thereby lowering the temperature of the fresh mixed concrete [29].

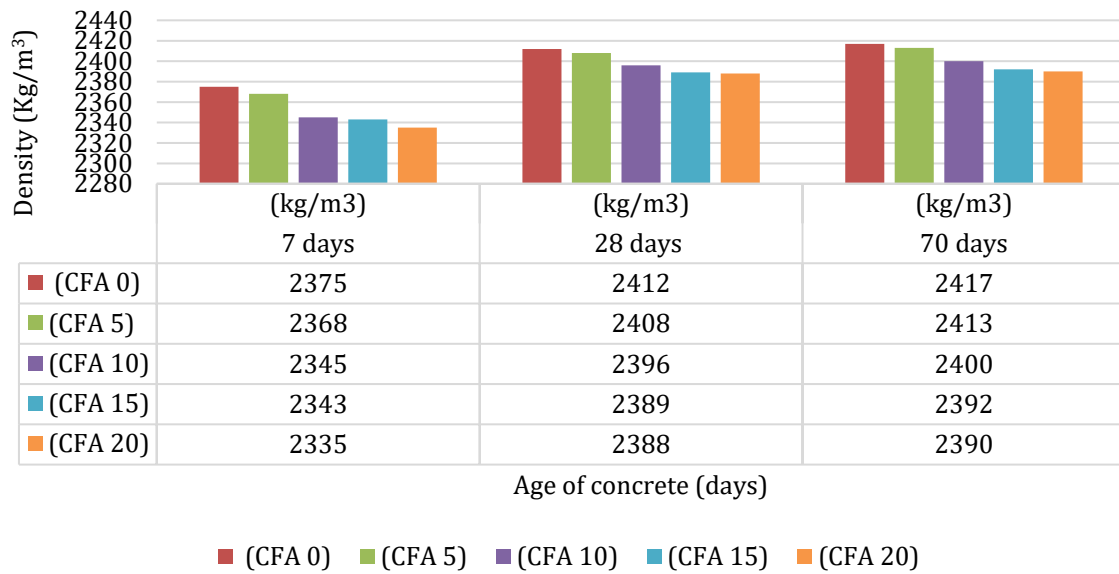


Fig. 5. Density of CFA-modified concretes

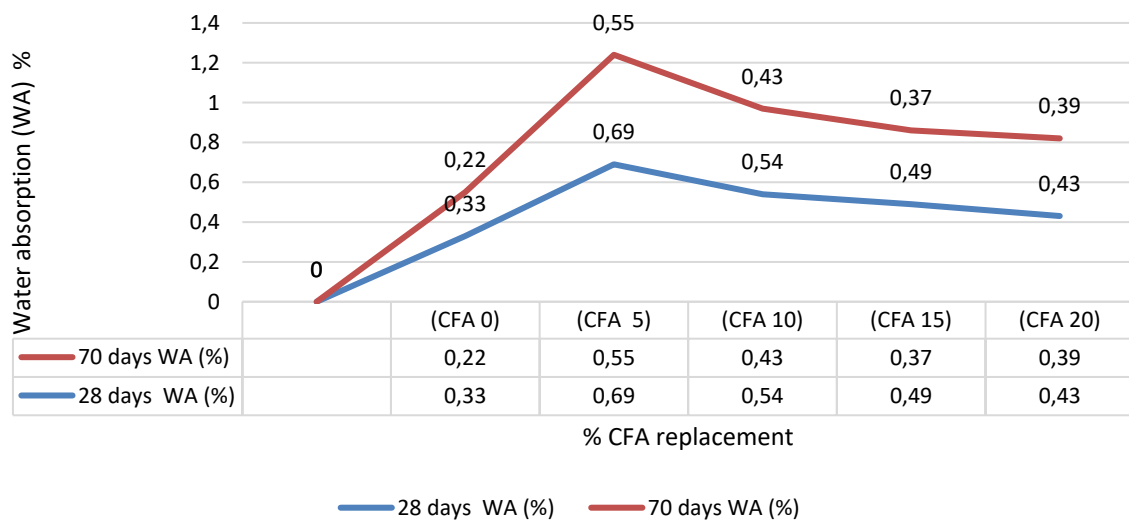


Fig. 6. Water absorption of CFA-modified concretes

According to Arimanwa et al. [10], the density of CFA concrete specimens increases with an increase in curing age, and for each curing age, density decreases with an increase in CFA concentrations, while none of the concrete specimens with CFA concentrations outperformed that of the control specimen [10]. This position was affirmed by the findings of this current study. From Figure 5, the density of concrete increased with age of curing (7 days, 28 days, and 70 days), and within a specific curing age, such as 7 days, 28 days, and 70 days, density was found to decrease as the percentage of CFA replacement increased. Also, none of the densities outperformed that of the control concrete for ages 7 days, 28 days, and 70 days. A phenomenon Ranatunga et al [1] and Arimanwa et al. [10] attributed to the use of comparatively less dense SCM, in this case CFA, to replace a denser PLC in terms of material densities. From Figure 5, in accordance with the BS EN 206-1:2000 [27], the CFA-modified concretes (from 0% to 20% CFA replacement levels) could be classified as normal-weight concretes since the average oven-dry density values were more than 2000 kg/m³ but did not exceed 2600 kg/m³.

Water absorption was found to increase as CFA concentration in concrete increases, while the control specimen recorded the least water absorption value [10]. Contrary to this assertion, from Figure 6 of this study, for (CFA 0), (CFA 5), (CFA 10), (CFA 15), and (CFA 20) concrete mixes, water absorption (WA) of concrete decreased with increasing age of curing for both 28 days and 70 days of curing. It also rose up to (CFA 5) and decreased thereafter with an increasing percentage of CFA replacement for each of the days of curing. Again, it was observed that none of the specimens with CFA recorded a WA value that was lower than the control specimen. A phenomenon, Ranatunga et al. [1] attributed to a decrease in total voids in concrete as a result of an increase in C-S-H gel formation during hydration [1].

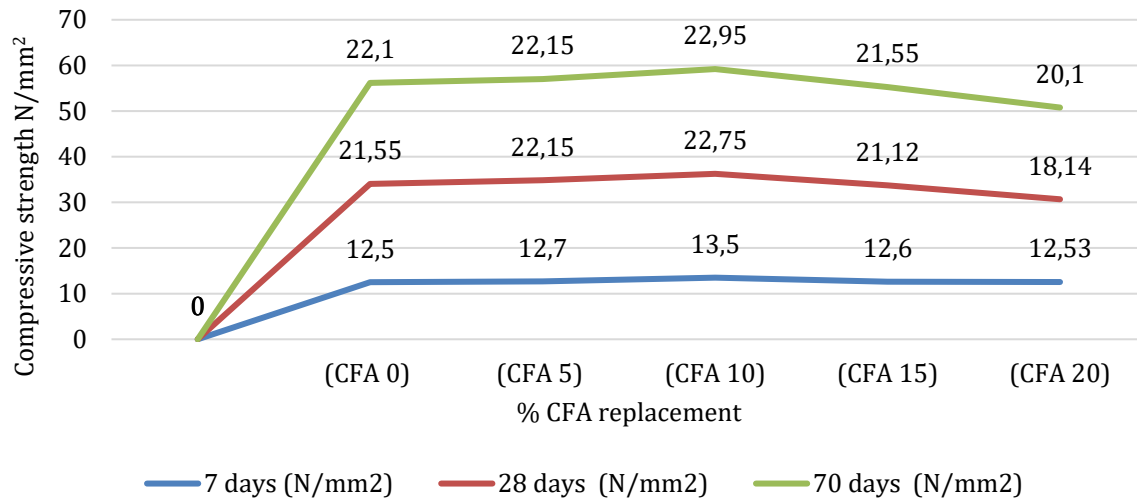


Fig. 7. Compressive strength of CFA-modified concretes and ages of curing

The targeted compressive strength at 28 days of curing for this study was 20 N/mm². According to Waqar et al. [8] compressive strength of CFA concrete increased up to 9 % partial replacement and decreased afterwards as the percentage of CFA replacement for cement increased. For each level of CFA replacement, compressive strength increases with age of curing [8]. From Figure 7, the compressive strength of the concrete specimens increased from ages 7 days, through 28 days, to 70 days. An indication of continuous hydration and pozzolanic reaction within the concrete specimens [1]. Also, for all the days (7 days, 28 days, and 70 days), compressive strength rose progressively to 10% partial replacement level, beyond which any additional increase in CFA resulted in a loss of clinker effect. This 10% optimum replacement level was higher than 9% recorded by Waqar [8] in Malaysia, and 5 % by Sanjay and Rajeev [6] in India. Thus, the case in Ghana suggested an improvement in the optimum level of partially replacing cement with CFA. This affirmed that the behaviour of CFA concrete is country-specific owing to the origin of the material and the temperature at which the material was processed, among others [1]. At age 7 days, (CFA 5), (CFA 10), and (CFA 15) improved early compressive strength of concrete better than the control, while at (CFA 5) and (CFA 10), CFA improved compressive strength at 28 days and 70 days better than the control. Furthermore, in this study, a regression model was developed to practically guide construction practitioners for the ease of applying the CFA concrete mix for optimum results, as presented in Table 3.

From Table 3, R^2 values ranged between 0 and 1. According to Kutner et al. [30] and Montgomery et al. [31], an R^2 value below 0.2 means a very weak relationship, 0.2-0.4 shows a weak relationship, 0.4-0.6 means a moderate relationship, 0.6-0.8 shows a strong relationship, and values above 0.8 implies a very strong relationship between the variables [30, 31]. The R^2 values for the 7-day compressive strength model, 28-day compressive strength model, and 70-day compressive strength model were 0.514, 0.975, and 0.903, respectively (see Table 3). Indicating a moderate to very strong relationship. From Table 3, (Y) is the compressive strength, and the compressive strength models confirmed that the optimum level of partial replacement was 10%.

Table 3. Equation to guide optimization of compressive strength of CFA concretes

Model	Equation	R ² (goodness of fit)	Remarks
For 7-Day Strength (Y ₇)	$Y_7 = [-0.0064x^2 + 0.1272x + 12.454]$	0.514	An indication that 51.4 % of the change in compressive strength of concrete at age 7 days can be predicted from the proportions of CFA, a moderate model fit.
For 28-Day Strength (Y ₂₈)	$Y_{28} = [-0.026829x^2 + 0.379571x + 21.371]$	0.975	An indication that 97.5 % of the change in compressive strength of concrete at age 28 days can be predicted from the proportions of CFA, a very strong model fit.
For 70-Day Strength (Y ₇₀)	$Y_{70} = [-0.014857x^2 + 0.2051x + 21.947]$	0.903	An indication that 90.3 % of the change in compressive strength of concrete at age 70 days can be predicted from the proportions of CFA, a very strong model fit.

According to [8] flexural strength of CFA concrete increased progressively up to 12 % optimum partial replacement and decreased afterwards as the percentage of CFA replacement for cement increased. This position was supported by the findings of this current study. From Figure 8, flexural strength increased from day 7 days to 70 days for each of the specimens, an indication of continuous hydration and pozzolanic reactivity within the concrete specimens. However, the optimum level of partial replacement was 10%, and when compared to the study in Malaysia by [8], it was lower. Thus, affirming that the behavior of CFA is context-specific.

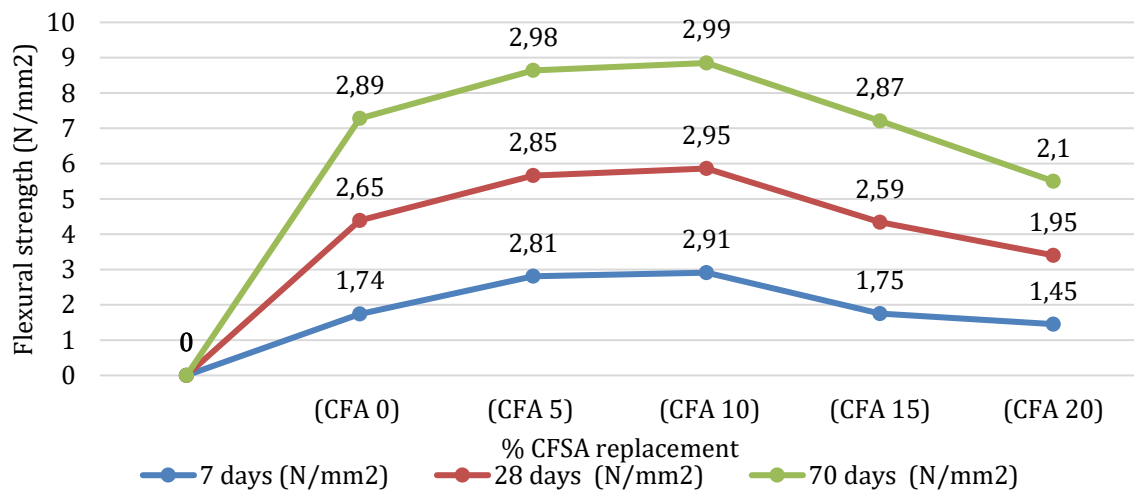


Fig. 8. Flexural strength of CFA-modified concretes and ages of curing

5. Conclusions

This current study sought to evaluate the physical and mechanical performance of concrete-modified with locally sourced coconut fiber ash. Base on the findings of the study, it could be concluded that:

- CFA in Ghana is a potential supplementary cementitious material for PLC. Elements of cement were also found to be present in CFA. The elements include Ca (2.3150 %), Si (3.0900%), and Fe (0.4790%). This affirmed the potential of CFA as a supplementary cementitious material (SCM).

- In relation to the physical performance of the CFA-modified concrete specimens, it was observed that the workability of CFA concrete decreased as the concentration of CFA in concrete increased. Workability of CFA-modified concretes was low: ranging from 20mm to 34 mm for 5% to 20% partial replacement levels. Temperature continually decreased as CFA concentration increased from 0% to 20%. The density of CFA concrete decreased with an increase in the concentration of CFA in a concrete mix. Water absorption of CFA concrete decreased with increasing concentration of CFA in a concrete mix. None of the WA values was less than the control for 28 days and 70 days of curing.
- Regarding the mechanical performance of CFA-modified concrete, compressive strength and flexural strength rose progressively to 10% as the concentration of CFA in concrete increased, then declined beyond the 10% optimum partial replacement level, as a result of the dilution effect of concrete. At age 28 days of curing, CFA 10 concretes recorded the maximum compressive strength of 22.75 N/mm² and flexural strength of 2.95 N/mm², exceeding compressive strength values of 21.55N/mm² and flexural strength of 2.65N/mm² recorded by the control specimens.
- In accordance with the BS EN 206-1:2000 [27], CFA concrete specimens and the control specimen were classified as normal-weight concrete.
- The study found an alternative use for coconut fiber, otherwise a waste material polluting the environment, in CFA-modified concrete. This promotes agricultural waste valorization and material circularity.
- Empirically, the elemental composition of CFA, sourced locally from Sekondi-Takoradi in Ghana, has been established, which hitherto was largely absent in existing literature. This has broadened the frontiers of existing literature on CFA as an SCM.
- Practically, it has developed a regression equation that will aid construction practitioners in Ghana in determining the optimal level of partially replacing PLC with CFA in concrete mix in order to maximize compressive strength.
- Moreover, this study provides the basis for future studies on sourcing SCMs locally for the intended purpose.
- By finding alternative uses for CFA in concrete production will lessen the overreliance on cement for concrete, which will translate into reducing the carbon footprint of concrete, thereby contributing to addressing SDG 13: climate action, and SDG 12: responsible consumption and production. It also promotes waste valorization.
- Policy-wise, knowledge of the optimal replacement level and physical and mechanical performance of CFA-modified concretes will inform the development of standards by the government and policy makers in developing relevant standards and codes to regulate the use of CFA as SCM.

References

- [1] Ranatunga KS, del Rey Castillo E, Toma C. Evaluation of the optimal concrete mix design with coconut shell ash as a partial cement replacement. *Constr Build Mater.* 2023;401:132929. <https://doi.org/10.1016/j.conbuildmat.2023.132978>
- [2] Bali M, Abdelali H, Cherradi T. Towards sustainable concrete: reviewing the potential and challenges of plant fiber reinforcement with a life cycle case study using date palm fibers. *Res Eng Struct Mater.* 2025. <http://dx.doi.org/10.17515/resm2025-1111ma0823rv>
- [3] Bheel N, Mahro SK, Adesina A. Influence of coconut shell ash on workability, mechanical properties, and embodied carbon of concrete. *Environ Sci Pollut Res.* 2021;28(5):5682-5692. <https://doi.org/10.1007/s11356-020-10882-1>
- [4] Pahuja V, Ghosh P, Kumar A. Experimental investigation of bond behavior between bamboo and concrete: a comparative analysis of bamboo surface treatments. *Res Eng Struct Mater.* 2026;12(1):1-15. <http://dx.doi.org/10.17515/resm2025-658ma0204rs>
- [5] United Nations. Resolution adopted by the General Assembly on 6 July 2017, Work of the Statistical Commission pertaining to the 2030 Agenda for Sustainable Development (A/RES/71/313). New York: United Nations; 2017.
- [6] Sanjay S, Rajeev C. Effect of coconut fibre ash on strength properties of concrete. *Int J Eng Res App.* 2015;5(4):33-35.

- [7] Anifowoshe F, Nwaiwu NE. The use of coconut fibre ash as a partial replacement for cement. *Br J Appl Sci Technol*. 2016;17:1-11. <https://doi.org/10.9734/BJAST/2016/25926>
- [8] Waqar A, Bheel N, Almujiab HR, Benjeddou O, Alwetaishi M, Ahmad M, et al. Effect of Coir Fibre Ash (CFA) on the strengths, modulus of elasticity and embodied carbon of concrete using response surface methodology (RSM). *Results Eng*. 2023. <https://doi.org/10.1016/j.rineng.2023.100883>
- [9] Awolusi T, Oginni O, Aluko O, Azab M. Enhancing the mechanical and durability quality of fibre-modified concrete using eco-sustainable wastes and lime powder. *Res Eng Struct Mater*. 2026. <http://dx.doi.org/10.17515/resm2026-1223me1009rs>
- [10] Arimanwa M, Anyadiegwu P, Ogbonna N. The potential use of Coconut Fibre Ash (CFA) in Concrete. *Int J Eng Sci (IJES)*. 2020;9(1):68-74.
- [11] Oglekwu F, Auta S, James O. An Overview: Investigating the effect of Coconut Fibre Ash on the flexural strength of revibrated concrete beam. In: *International Civil Engineering Conference*. 2025.
- [12] Vagestan P, Periyasamy M, Pathikonda P, Kiran U, Yadav S, Zahid M, et al. Enhancing the strength properties of concrete using coconut fiber and coconut fiber ash. *AIP Conf Proc*. 2024;3037. <https://doi.org/10.1063/5.0196073>
- [13] Ambreshwar, Brunda G, Reddy P. Investigation on partial replacement of eggshell powder with cement. In: *International Conference on Modern Trends in Civil Engineering Infrastructure Development and Management*. 2023. p. 275-283. https://doi.org/10.1007/978-981-96-1554-4_26
- [14] British Standards Institution. BS EN 196-2: Methods of testing cement - Part 2: Chemical analysis of cement. London: BSI; 1995.
- [15] British Standards Institution. BS EN 197-1: Cement - Part 1: Composition, specifications and conformity criteria for common cements. London: BSI; 2000.
- [16] British Standards Institution. BS EN 1008:2002: Mixing water for concrete - specification for sampling, testing and assessing the suitability of water. London: BSI; 2002.
- [17] British Standards Institution. BS EN 933-1:2012: Tests for geometrical properties of aggregates - Part 1: Determination of particle size distribution - Sieving method. London: BSI; 2012.
- [18] British Standards Institution. BS 1377-2:1990: Methods of test for soils for civil engineering purposes - Part 2: Classification tests. London: BSI; 1990.
- [19] British Standards Institution. BS EN 12390-3:2019: Testing hardened concrete - Part 3: Compressive strength of test specimens. London: BSI; 2019.
- [20] British Standards Institution. BS EN 12390-7:2009: Testing hardened concrete - Part 7: Density of hardened concrete. London: BSI; 2009.
- [21] British Standards Institution. BS EN 1881-122:2011: Testing concrete - Part 122: Method for determination of water absorption. London: BSI; 2011.
- [22] British Standards Institution. BS EN 12390-5:2019: Testing hardened concrete - Part 5: Flexural strength of test specimens. London: BSI; 2019.
- [23] British Standards Institution. BS 1881-125:1986: Testing concrete - Part 125: Methods for mixing and sampling fresh concrete in the laboratory. London: BSI; 1986.
- [24] British Standards Institution. BS EN 12390-2:2000: Testing hardened concrete - Part 2: Making and curing specimens for strength tests. London: BSI; 2000.
- [25] British Standards Institution. BS 882:1992: Specification for aggregates from natural sources for concrete. London: BSI; 1992.
- [26] British Standards Institution. BS EN 12620:2013+A1:2019: Aggregates for concrete. London: BSI; 2013.
- [27] British Standards Institution. BS EN 206-1:2000: Concrete - specification, performance, production and conformity. London: BSI; 2000.
- [28] American Concrete Institute. Guide to Hot Weather Concreting (ACI 305R-19). Farmington Hills, MI: ACI; 2019.
- [29] Mehta P, Monteiro P. *Concrete: Microstructure, Properties, and Materials*. 4th ed. New York: McGraw-Hill Education; 2014.
- [30] Kutner MH, Nachtsheim CJ, Neter J. *Applied Linear Regression Models*. 4th ed. Chicago: McGraw-Hill/Irwin; 2004.
- [31] Montgomery DC, Peck EA, Vining GG. *Introduction to Linear Regression Analysis*. 6th ed. Hoboken, NJ: Wiley; 2021.

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Nonlinear numerical analysis of concrete members subjected to temperature-induced volumetric changes

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Abstract

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The present paper investigates the capability of Nonlinear Finite Element Analysis (NLFEA) in predicting the behavior of Reinforced Concrete (RC) members subjected to combined gravity and temperature loads. The experimental and NLFEA results show that temperature changes have notable influence on the initiation of cracking, the stiffness degradation and ultimate load-carrying capacity. The NLFEA outcomes were compared to the experimental data of RC beams subjected to uniform and through-thickness temperature gradients. The NLFEA modelling technique, presented in this paper, reasonably well simulated the load-displacement history, cracking patterns and failure modes. Cracking loads and ultimate capacities for all tested specimens were estimated within 10% of the experimental data. In some cases, there was a slight overprediction of cracking loads and slight under-prediction of ultimate strengths; however, the agreement was generally consistent. Mesh sensitivity studies proved that the NLFEA solution converged and showed more accurate results as the discretization is refined. The conclusions emphasize the significance of considering nonlinear material behavior, tension stiffening and differential thermal expansion of steel and concrete for the thermo-mechanical analysis. In summary, the results indicate that the proposed NLFEA model offers a robust and accurate prediction of RC structural behavior under combined thermal and gravity loading.

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1. Introduction

For RC members experiencing volumetric changes, structural behavior of such members is controlled by the extent and pattern of cracking [1-6]. Stiffness and stress-carrying capacity of the section drops off when cracking takes place; therefore, cross-sectional area contributes to some extents to carrying internal forces. Therefore, the correct calculation of volumetric change-induced stress must take this stiffness reduction into account. Herein, the area stiffness is modified during FEA in the linear analysis. If a completely uncracked section is assumed the stress level overestimate is obtained, and this influences incorrectly the reliability of internal force estimate.

Hence, it is important to estimate the effective section properties with care to obtain accurate results. Modulus of elasticity is an important property of concrete in determining the stress-strain behavior. Once the cracking sets in, this relation ceases to be linear [3-7]. The combination of volumetric change and stiffness reduction leads to effective values of structural deformation and internal stresses non-identical with those at elastic stage. This interaction is very important for correct structural evaluation and better performance prediction and design options.

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Effect of temperature fluctuations becomes more significant in flat slabs and the other long span systems [12,14]. Induced axial forces affect both flexural and shear action. At present, design codes or standards [17-22] give provisions to adjust one-way shear strength due to axial force influence. This is accounted for by recognizing that compression increases and tension decreases shear strength. However, limited recommendations are available in codes and standards with respect to this effect on the two-way shear resistance. This is a limitation in the literature, for overlong concrete structures under combined load conditions including gravity loads and volumetric variations.

Although international code provisions are available for thermal analysis, the guidelines provided of these codes for RC and prestressed concrete structures are still limited. Current design guidelines show significant degree of variation in the stiffness modification factors and do not include a coherent structure for simulating thermally induced volumetric variations. Moreover, the existing provisions do not present a comprehensive or systematic approach to combine thermal actions and mechanical loads in restrained members where internal force redistribution controls the behavior. Furthermore, the assumption of simplified or nominal temperature distribution through member thickness introduces an error in predicted thermal stresses. Further, present codes do not adequately differentiate between the essentially different thermal reactions of RC and prestressed concrete, with reference to prestress loss effects, cracking control and serviceability issues. These gaps reveal that current code-based requirements are inadequate for accurate prediction of the thermal response and resilient performance, suggesting the need for a more comprehensive and systematically validated framework.

The crack initiation and shear failure in the interface were caused by early uniaxial tensile stress, as observed from the literature, whereas axial compressive stress retarded cracking. However, the majority of such understandings are about one-way shear and two-way shear behavior under an axial load is still not well understood. Paucity of comprehensive provisions on such retrofit measures in codes and standards like ACI, Eurocode, and ECP indicates the pressing need for more experimental as well as numerical studies. And besides the changing volumes, this extended structural continuity has structural implications. Elimination of expansion joints can improve the symmetry and efficiency in lateral load-resisting systems. When the expansion joints divide the structure, they may result in non-symmetric stiffness distribution and consequently decrease seismic response of whole buildings as well as a wind loading capacity. This can be explained schematically through Fig. (1). On the other hand, no knot in a joint-free setting implies more regular behavior and better robustness. But this strengthened mechanical stability must be weighed against the elevated volumetric changes at the system.

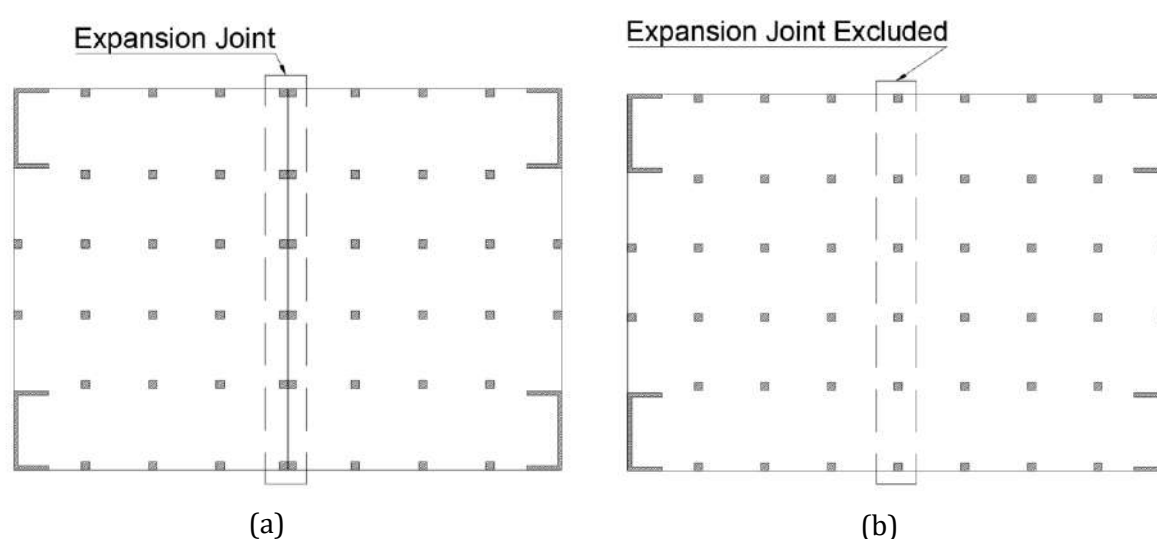


Fig. 1. (a) A structure with unsymmetric lateral force resisting system due to the presence of expansion joint, (b) A structure with symmetric lateral force resisting system after exclusion of expansion joint

1.1 Research Objectives

This study highlights the necessity of considering material nonlinearity. Once cracked the internal distribution of forces and structural behavior both under gravity loads as well as volumetric changes are fundamentally different. The main aim of this study is to develop a validated NLFEA method considering the coupled action of gravity and temperature loading in the RC members. This method attempts to improve the accuracies of stresses and deformations considering thermal effects. Thus, this may lead to better estimates of serviceability and durability for RC structures. The proposed tool is aimed to fill the gap between numerical modelling and practical design. Furthermore, this can help structural engineers to take more effective decisions about the overall thermal performance of RC elements under restrained situations. To offer a better insight into the thermal response of RC members, by explicitly considering the role of concrete cracking and hence reduced effective stiffness, it is worth pursuing a further investigation.

2. Experimental Data

To determine the accuracy and robustness of the developed NLFEA technique performed in this study, a thorough review was carried out for previous experimental investigations concerning the combined action between gravity loads and volumetric changes. These studies give valuable insights to the behavior of concrete members under combined thermal and gravity loading. They present the complex interaction between material cracking and structural capacity. The experimental programs presented in previous research serve as an important foundation for analyzing the predictive capability of the numerical models developed throughout this study. Correspondingly, the main observations, behavior, and failure characteristics derived from these experimental tests are combined and presented in the following section for comparison with the NLFEA results of this paper. The selected experimental data capture the dominating thermo-mechanical mechanisms which control the structural response. These studies investigated both uniform temperature variations across concrete sections as well as the effect of temperature gradient. Accordingly, experimental tests represent an ideal benchmark for the performance of numerical models in simulating temperature-induced changes and load carrying capacity.

2.1 DeRosa et al.

DeRosa et al. [8] performed an experimental investigation to investigate the influence of temperature variations on RC members. The study was based on stiffness, strength and crack width against temperature changes. It also studied the mechanical performance of RC beams under freeze-thaw cycles and low temperatures. The testing facility for all experiments was a temperature-controlled room with load frame. The tests were done to study the behavior of RC elements under axial tension in addition to thermal cycles. The specimens tested were full-size RC beams subjected to different temperatures. Four beams with the dimensions of 200 mm × 400 mm × 4200 mm and span length of 3400 mm were tested under four-point bending. Two of the beams were tested at room temperature (+21 °C) and two to (−20°C). Beams 1 and 3 were without shear Reinforcement. Stirrups placed at 175 mm spacing were provided in beams 2 and 4. Loading sequence of the four specimens is illustrated in Table (1).

Table 1. Loading sequence of DeRosa et al. specimens [8]

Beam Number	Stirrups Layout	Load (kN)				Exposure
		Initial Loading	Sustained 48h	Cyclic (5 Cycles)	Final Static	
1	No stirrups	0 - 75	90	75 - 50 - 75	75 up to failure	-20 ⁰ C
2	With Stirrups	0 - 75	90	75 - 50 - 75	75 up to failure	-20 ⁰ C
3	No stirrups	0 - 75	90	75 - 50 - 75	75 up to failure	+21 ⁰ C
4	With Stirrups	0 - 75	90	75 - 50 - 75	75 up to failure	+21 ⁰ C

2.2 Mirzazadeh et al.

Mirzazadeh et al. [9] experimental studied the static performance of RC beams under thermal gradient field through the depth. These differentials were to represent actual conditions of thermal exposure, most pertinent being structures of bridges. The test program included experimentations with four full scale RC beams. The four-step loading sequence was applied to each of the beams. In the first stage, a static load was gradually increased to 90 kN. This load was maintained for 48 h. Then ten loading cycles were performed from 50 kN to 90 kN. Each beam was then tested to failure. The specimens had the same geometry as used by DeRosa et al. [8]. But in contrast to the previous experiment, these beams were not subjected to only horizontal temperature gradients. Loading sequence for the four specimens is illustrated in Table (2). The shear reinforcement and the test temperature state already reflected in being included on the beam's labels. "NS" denotes specimens with no shear reinforcement and "WS" denotes specimens with shear reinforcement. Likewise, "R" is for testing at room temperature (+15°C) and "L" is for low temperature (-25°C). To resist shear, 10M stirrups at 175 mm centers were used in beams WSR and WSL. Contrastingly, NSR and NSL did not contain any shear reinforcement except the minimum stirrups at center of span and supports needed for construction. The four-point bending configuration was used to test all specimens. Heating pads were used on the upper side of each beam. This technique achieved the desired temperature gradient across the depth of the beam.

Table 2. Loading sequence of Mirzazadeh et al. specimens [9]

Beam Description	Beam ID	Load (kN)				Exposure
		Static	Sustained 48h	Cyclic (10 Cycles)	Final Static	
No Shear RFT Room Temp.	NSR	0 - 90	90	90 - 50 - 90	90 up to failure	+15 ⁰ C
No Shear RFT Low Temp.	NSL	0 - 90	90	90 - 50 - 90	90 up to failure	-25 ⁰ C
With Shear RFT Room Temp.	WSR	0 - 90	90	90 - 50 - 90	90 up to failure	+15 ⁰ C
With Shear RFT Low Temp.	WSL	0 - 90	90	90 - 50 - 90	90 up to failure	-25 ⁰ C

3. Nonlinear Finite Element Analysis

This research provides a numerical study on the behavior of concrete beams and long-span RC members under combined gravity-temperature loading. The investigation specifically addresses the volumetric changes due to daily and seasonal environmental processes. These are volumetric variations that can have considerable effect on the performance of overlong members and as such must be analyzed. The primary goal is to study the interaction between volumetric changes and gravity loads, and how the interaction would influence deformation of, cracking in, as well as global stability of a structure. The structural response of RC beams subjected to combined thermal and gravity loading was modelled using NLFEA. Modelling consisted of material definition, imposition of boundary conditions and a sequentially coupled thermal-stress analysis to account for temperature effects. Fig. (2) shows depiction of the created NLFEA models presenting the geometry and arrangement of reinforcement. This figure gives a graphical illustration of the numerical modelling technique. Fig. (2) shows four-point bending configuration. The ends of the specimen were modelled to simulate the support conditions in the experiments. Translation restraints were assigned according to the same conditions applied during testing. Also, two loading points were imposed to apply at them the concentrated loads as applied in experimental testing. All interface conditions and impositions in the NLFEA were made to be consistent with those in experimental setup. A simplified illustration of the supports, load positions, and direction is shown in Fig. (2) for clarity.

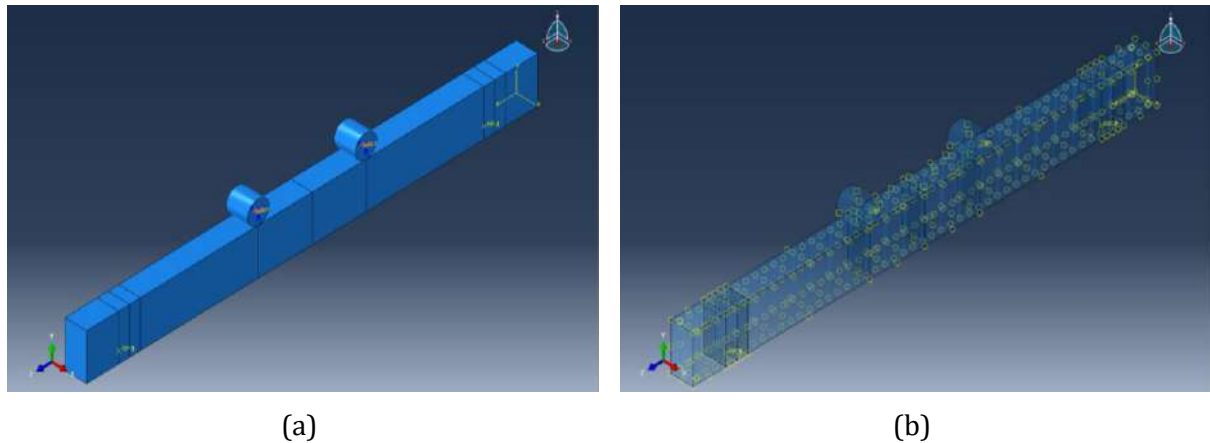


Fig. 2. (a) 3D Model for RC Beam using ABAQUS/CAE, (b) Embedded Element Technique for Reinforcement inside RC Beam

3.1 Numerical Analysis Parameters

NLFEA was conducted using ABAQUS/CAE. RC members were simulated under combined effect of the gravity and the temperature loads. Concrete was modelled by the C3D8RT 3D continuum solid element, which also enables to perform the accurate thermal and structural behavior simulation [16]. The reinforcement bars were simulated as T3D2 trusses incorporated in the volume of concrete [16]. The reinforcement was embedded in the concrete. The approach allows the steel bars to deform compatibly and in a compatible manner with the adjacent concrete, without requiring an additional interface model. It limits relative steel and concrete sliding for accurate stress transfer and composite action. This approach also simplifies the meshing by eliminating the requirement for shared or coincident nodes at interface travel. Hence, it makes the model simplified and retains the numerical accuracy. The thermal expansions of concrete and steel were defined with different values, which allows the model to calculate internal stresses due to thermally induced expansion differences between both materials.

3.1.1 Sequentially Coupled Thermal-Stress Analysis

A sequential thermal-stress analysis was employed to model the behavior of RC members subjected to biaxial loading. This approach consists of two separate computation steps. First, the temperature distribution in the member is calculated. Step also includes the thermal properties of concrete and steel, as well as the boundary conditions and environmental effects including ambient temperature and solar radiation. The emerging temperature field is subsequently provided as an input for the structural analysis. In the second step, the defined temperature field is applied to the structural model. This step calculates the stress and strain responses of RC member under the interaction between mechanical and volumetric quantities. The temperature change causes thermal expansion or contraction, resulting in thermoelastic strains. Such strains are superimposed on those due to external action from gravity loads, causing redistribution of stresses in the element.

3.2 Concrete Material Model

The CDP model is chosen for this study because it offers the best flexibility and accuracy. CDP model has several beneficial properties, as they include broader mechanical behavior of concrete. It can handle virtually unlimited loading and boundary conditions (monotonic, cyclic, dynamic), which make it become a versatile tool in both nonlinear structural analyses. It allows detailed description of concrete behavior under mechanical and volumetric loading necessary the assessing structural condition in open environment. To accurately simulate the material nonlinearity of concrete, especially in compression, the Modified Hognestad model [23] was used. In this model, the classical Hognestad curve is further developed by giving a more detailed representation of both ascending and descending branches on the stress-strain curve. In the pre-peak zone, the model expresses compressive stress (f_c) as a parabolic function of strain ϵ_c according

to Eq. (1), which is satisfied up to the peak condition for compressive strain (ϵ_{c0}), that indicates the greatest capacity to resist stress status (f_c'). The parameters that are considered for Modified Hognestad model are shown in Table (3). The formula models that stress must soften up to its maximum, in accordance with the true behavior of concrete under compression. In the post-peak part of the curve for which the strain is larger than (ϵ_{c0}) and arrives at a crushing strain $\epsilon_{cu} = 0.0038$, as shown in Eq. [2], similar, descending stress represents material softening [23-24].

This simplified linear softening curve is convenient and realistic for the simulation of decrease in bearing capacity by crushing. By employing the CDP model in conjunction with Modified Hognestad stress-strain relation, this paper accommodates gradual variation of damage and nonlinear material response. In RC members subjected to changeable environmental conditions, the combined effect of mechanical and thermal loads affects structural response which requires an integrated modelling considered as a crucial aspect for accurate prediction of lifecycle behavior. Fig. (3) presents the stress-strain curve for unconfined concrete with modified Hognestad model in compression. Concrete in compression was modeled with a modulus of elasticity (E_c) equal to $4700\sqrt{f_c'}$

$$f_c = f_c' \left[\frac{2 * \epsilon_c}{\epsilon_{c0}} - \left(\frac{\epsilon_c}{\epsilon_{c0}} \right)^2 \right] \quad (1)$$

$$f_c = 0.85 * f_c' \quad (2)$$

Table 3. CDP material model parameters

Dilation Angle (ψ)	Eccentricity (ϵ)	Stress Ratio (f_{b0}/f_{c0})	Shape Factor (K_c)	Viscosity Parameter (μ)
40	0.1	1.16	0.67	0.005

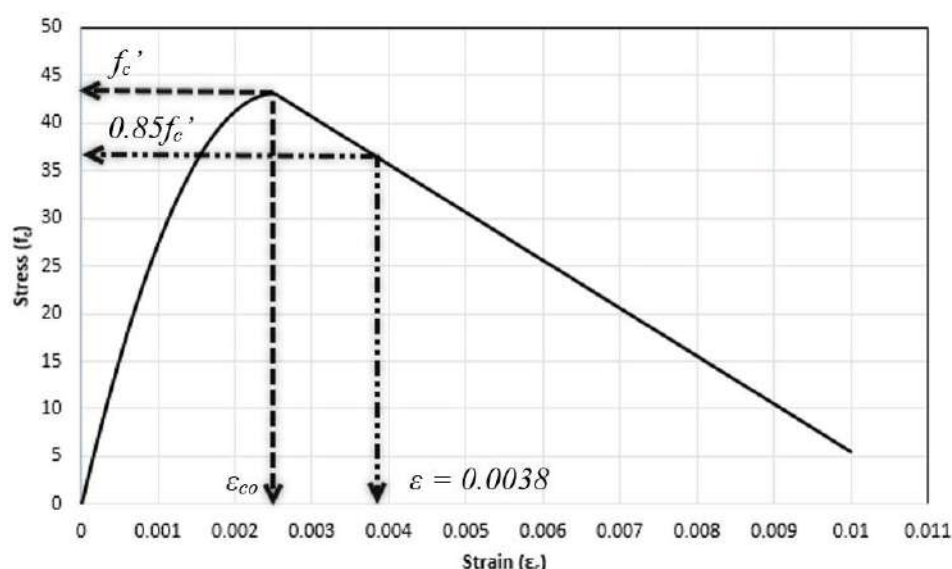


Fig. 3. Stress-strain relationship of concrete in compression using Modified Hognestad model

3.3 Concrete Modulus of Rupture

One of the most important parameters in NLFEA of RC structures is the determination of the direct tensile strength (f_t) of concrete. Researchers and design codes have offered various equations to predict f_t as a function of f_c' [17-22]. For example, ACI 435R-95 [22] proposes Eq. (3) and ACI 318-25 [18] proposes Eq. (4). Several experimental research showed that the tensile strength of concrete is more proximate to the value estimated by ACI 318-25 [18]. This corresponds to a better indication of the resistance of the material against cracking. By contrast, Eq. (3) in ACI 435R-95 [22] is conservative and often results in lower tensile strength. While this more conservative

value ensures safety in very uncertain situations or during times aggregates are needed. Hence, considering Eq. (4) in NLFEA simulations of concrete offers a more representative estimation, particularly in the study of crack growing. This is more consistent with what can be observed in the experimental results analyzed within this work. As the use of Eq. (4) in NLFEA would result in more accurate prediction of concrete performance, it will also benefit reliable structural evaluations.

$$f_t = 0.33\sqrt{f'_c} \quad (3)$$

$$f_t = \frac{2}{3} * 0.62\sqrt{f'_c} \quad (4)$$

3.4 Tension Stiffening in Concrete

Exact modeling of tension stiffening is very important for RC members model, where reinforcement is still carrying load after concrete cracks [10]. There are three possible alternatives for modelling concrete tension stiffening. The three alternatives are the stepped response, the stepwise unloading response and the abrupt unloading response. The steplike response divides the stress-strain curve into steps and therefore is computationally easier to analyses. The method described above may, however, fail to properly reflect the viscoelastic nature of tensile behavior in concrete and hence offer less accurate results. The slowly unloading response gives more realistic image, as it presents a smooth continuous decrease in stress with increasing strain past the cracking. This kind of constitutive relationship is closer to the true behavior of concrete in tension but tends to make the modelling and computation more complicated. Finally, the discontinuous unloading response presents itself as an intermediate between both by suffering an immediate decrease of stress but with a behavior more linear. This approach is a compromise between simplicity and accuracy, providing a sharp initial drop while considering extra stress reductions after some higher strain. Although it is more accurate than the stepped response, the partially unloading response may oversimplify some of the features of a fully gradual unloading response. Each approach has advantages and limitations, so the method chosen should depend on a balance between computational simplicity and model accuracy.

Collins-Mitchell approach presents a modified tension stiffening model. This is an extension of the Vecchio-Collins model that includes improvements allowing for a more realistic representation of the interaction between concrete and reinforcement after cracking. The linearity of the stress-strain response is first supposed, by Eq. (5). Where, (f_{cr}) is the tensile stress, (E_c) is the elastic modulus of concrete and (ε_1) is the tensile strain. This is valid until the onset of cracking (ε_{cr}). Beyond this point the nonlinear stress reduction is then applied to simulate a post-cracking response, which is defined by Eq. (6).

$$f_{cr} = E_c \varepsilon_1, \text{ for } \varepsilon_1 \leq \varepsilon_{cr} \quad (5)$$

$$f_{ct} = \frac{\alpha_1 \alpha_2 f_{cr}}{1 + \sqrt{500\varepsilon_1}}, \text{ for } \varepsilon_1 \geq \varepsilon_{cr} \quad (6)$$

The α_1 and α_2 parameters are material dependent constants and commonly found in the literature from testing. The $\sqrt{500\varepsilon_1}$ term thus describes the progressive softening of tensile strength as strain is increased. It represents the asymptotic behavior (when cracks widen and spacing increases, the stress transfer through the concrete matrix is reduced). The Collins-Mitchell model includes contributions from concrete and reinforcement in tension. It discretizes the tension stiffening effect of concrete, that is an interaction between cracked concrete and contained steel. This method enhances the accuracy of numerical estimates of RC structures at service and ultimate loads. With the use of such models, structural analysis enables a better representation of the true post-crack behavior and thus yielding more accurate deflection, crack width and stress distribution predictions. This results in better estimates of structure performance, especially for the members under the flexural tension and biaxial loading condition. The stress strain curve of Collins-Mitchell Model for concrete in tension is illustrated in Fig. (4).

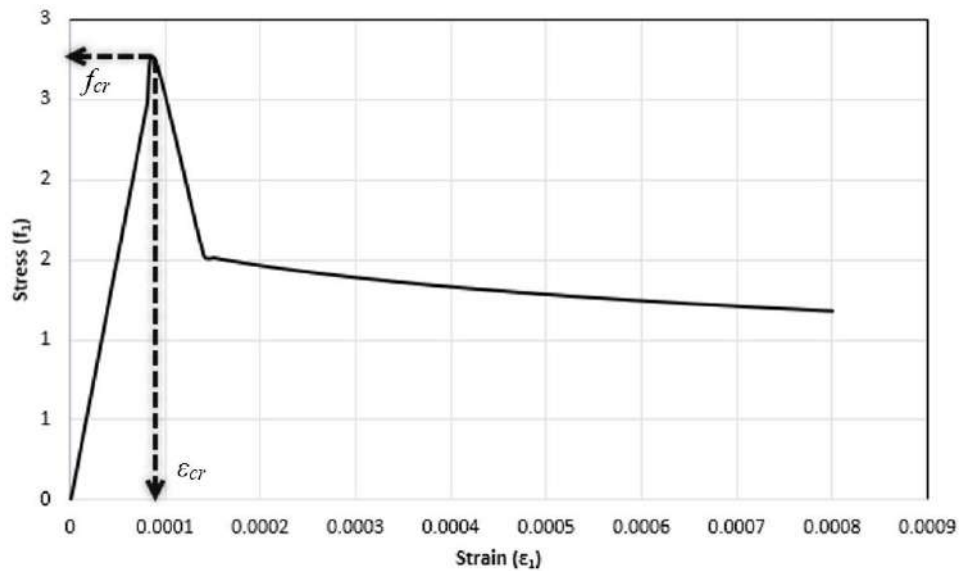


Fig. 4. Stress-strain relationship of concrete in tension using Collins-Mitchell Model

3.5 Reinforcing Steel Material Model

The Giuffre-Menegotto-Pinto model is one of the well-known constitutive models to be used for representing the nonlinear stress-strain relationship of reinforcement steel in RC structures. It represents the material behavior by a smooth and continuous curve which accounts for chronological elastic and plastic stages.

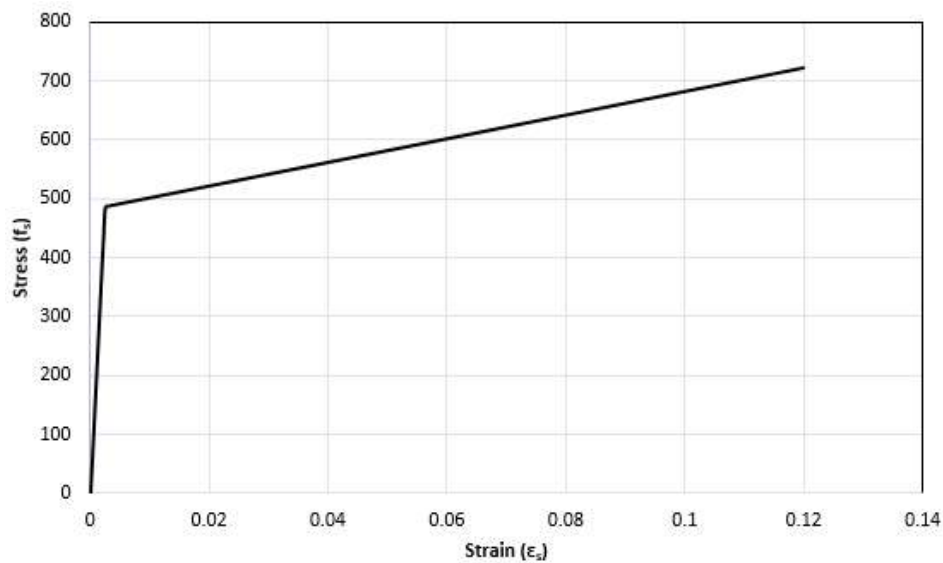


Fig. 5. Stress-strain relationship of steel reinforcement using Giuffre-Menegotto-Pinto model

First, the model is based on a relationship of linear elasticity, i.e., stress is proportional to strain (Eq. (7)), with proportionality constant being steel's modulus of elasticity. Once the material has achieved its yield point, it is said to have entered the plastic region and stress will still increase but by a decreasing amount. In this stage, the material response is given by Eq. (8).

$$f_s = E_0 * \varepsilon_s, \text{ for } f_s \leq f_y \quad (7)$$

$$f_s = f_y + b * E_0 * \varepsilon_s, \text{ for } f_s \geq f_y \quad (8)$$

Where; E_0 Initial elastic modulus of steel reinforcement, b Ratio between post-yield tangent and elastic tangent, ϵ_s Strain occurring in steel reinforcement, f_s Stress acting on steel reinforcement, f_y Stress at yielding of steel reinforcement.

4. Validation of Experimental Results

To validate the results of NLFEA obtained during this study, previous experimental results were used. Several experimental programs of concrete members under simultaneous flexure and temperature loadings were examined and according to their relevance, level of accuracy as well as accordance with the modelling approach used in this study. By comparing the predicted response of the structure from the NLFEA in terms of load–displacement curves, and crack patterns with previous experimental results, it is possible to assess thoroughly the reliability and prediction capabilities of such numerical models. Details of the key experimental results were previously summarized in this paper. It is also detailed how these have been used to validate the NLFEA results in the following sections.

4.1 Validation of NLFEA Against Experimental Results by DeRosa et al.

The loading sequence applied was divided into four stages, each stage consisting of specific steps. In the initial stage (Step (1)), a monotonic load was applied. This load increased successively from 0 kN to 75 kN. In the second stage (Step (2)), a sustained constant load of 75 kN was applied. The second step for beam (1) and beam (2) is that the ambient temperature drops to (-20°C). For the subsequent 10 steps (steps 3 to 12), the model experienced a 10-cycle load history. Each cycle comprised alternating loads of 75 kN – 50 kN was then exerted as the compliance for this kind of continuous wave test on air bearings. To evaluate cyclic behavior. Last, in step (13), the final step involved a monotonic loading phase. Starting from 75 kN, the load was increased to failure. This made it possible to evaluate ultimate load capacity. Materials properties for both concrete and reinforcing steel in the numerical modelling were defined based on information from DeRosa et al. [8]. They are given in Table (4). These properties were incorporated to guarantee the same behavior of specimens both under NLFEA simulation for comparison with actual structural test results where necessary. Relevant concrete parameters included compressive strength, tensile strength and E/P ratio. Certain reinforcing steel properties such as yield strength, ultimate strength and E were also defined. These input values guaranteed a capable foundation for studying structural reaction under the thermal and mechanical loads. Fig. (6) displays the temperature loading stage in NLFEA.

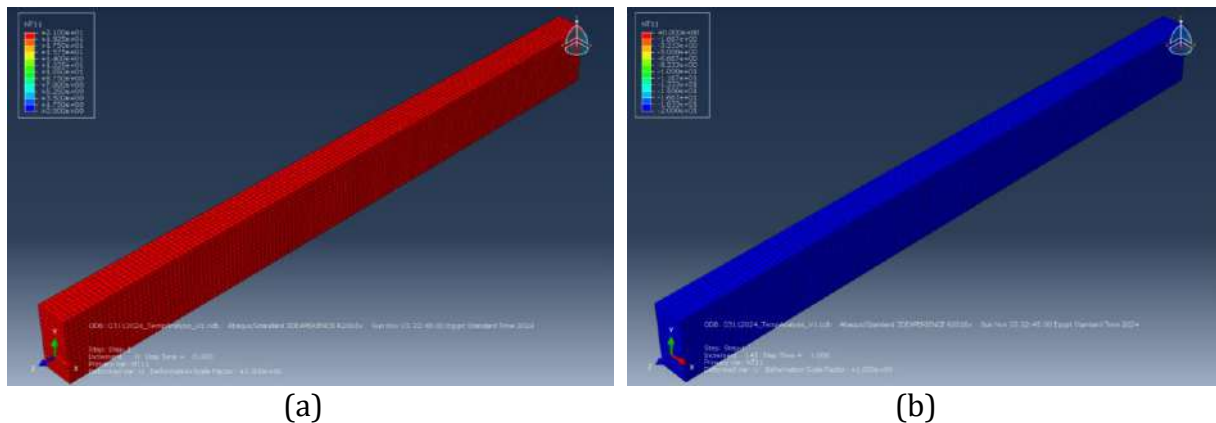
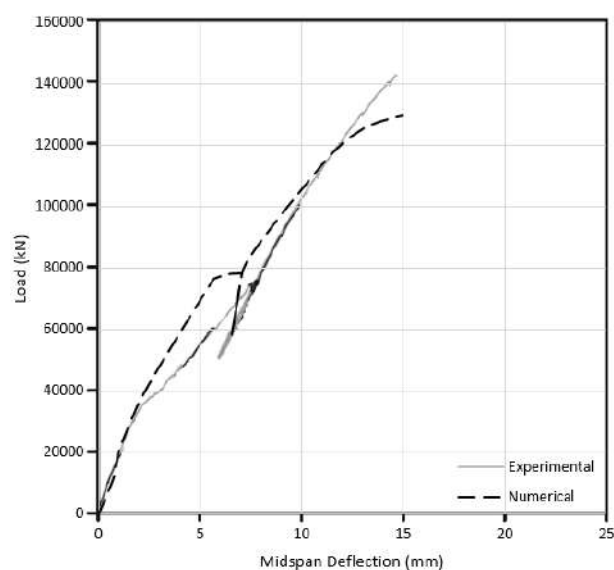


Fig. 6. (a) Initial Ambient Temperature surrounding RC Beam (Step 1), (b) Final Ambient Temperature surrounding RC Beam (Step 2)

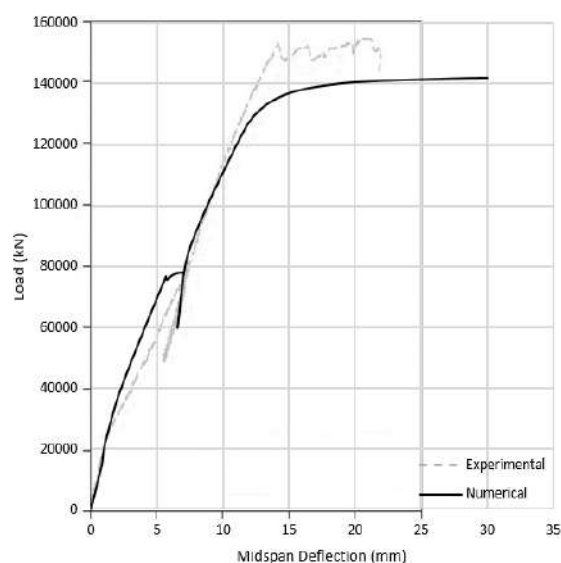
Fig. (7) displays load-displacement behavior alongside concrete cracking. Through calculations, the NLFEA results show good agreement with data from experimentations. It really confirms that the incorporated modeling approach is accurate. Fig. (7) validates the effectiveness of NLFEA in simulating complicated behavior by such concrete members when simultaneously subjected to both temperature and gravity loadings.

Table 4. Material properties of reinforcing steel and concrete of DeRosa et al. [8] specimens

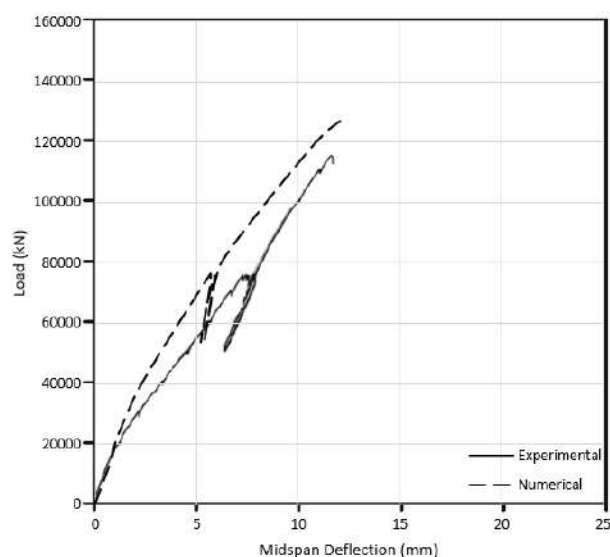
Material Property	Reinforcing Steel		Concrete
	10 M	20 M	
Modulus of elasticity (MPa)	200000	200000	28200
Poisson's ratio	0.30	0.30	0.18
Yield strength (MPa)	478	453	NA
Ultimate Strength (MPa)	576	563	NA
Compressive strength (MPa)	NA	NA	36
Density (kg/m ³)	7850	7850	2400
Coefficient of thermal expansion (/°C)	12*10 ⁻⁶	12*10 ⁻⁶	10*10 ⁻⁶



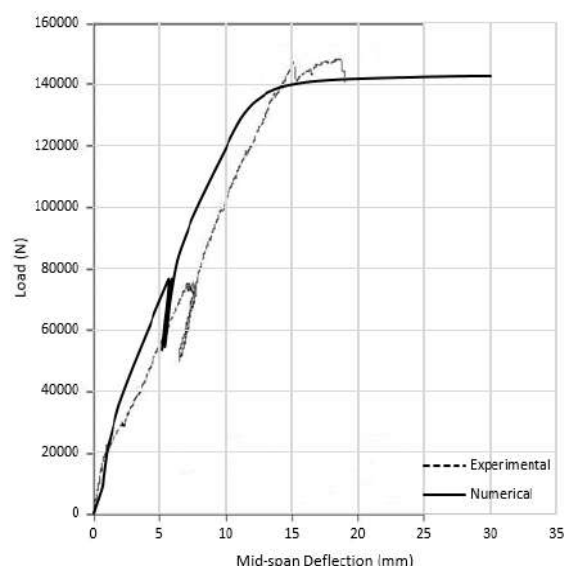
(a) Beam (1)



(b) Beam (2)



(c) Beam (3)



(d) Beam (4)

Fig. 7. Load-displacement relationship DeRosa et al. [8] specimens

4.2 Validation of NLFEA Against Experimental Results by Mirzazadeh et al.

The loading procedure was split into four distinct steps. In Step (1), load was incrementally applied and rose from 0 to 90 kN. In step (2), a sustained constant load of 90 kN was applied. During second step, temperature decreased in ambient temperature for beam (WSL) and beam (NSL), as

illustrated in the Fig. (8). For the next 10 steps (i.e., from step 3 to step 12), the beam was loaded to 90 kN, then back to 50 kN after pulling in each loading cycle and finally to 75 kN, in order to investigate the cyclic behaviour of the material. Finally, step (13) was a monotonic loading from 90 kN to failure which could be used as a measure of the ultimate carrying capacity of the beam.

ABAQUS/CAE was used to define the material properties of the two materials, concrete and reinforcing steel. The experimental data comes from Mirzazadeh et al. [9] and is summarized in Table (5). Thus, these parameters should ensure that both NLFEA and experimental results yield matching results. Compressive strength, splitting tensile strength, elastic, and Poisson's ratio are the principal inputs. Both elastic modulus and Poisson's Ratio affect the deformation of materials with respect to time. The performance of these reinforcing bars is described using its ultimate strength, yield point and Young's modulus of elasticity. Fig. (8) indicates the results of temperature-gradient applied on RC beam during experimental testing. Results of the NLFEA are shown in Fig. (9). These figures show how the RC beams responded to different loading conditions, with critical parameters such as load-displacement relationship, cracking pattern and stress distribution. The numerical results showed strong agreement with the experimental results. This confirms the accuracy of the modelling approach.

Table 5. Material properties of reinforcing steel and concrete of Mirzazadeh et al. [9] specimens

Material Property	Reinforcing Steel		Concrete
	10 M	20 M	
Modulus of elasticity (MPa)	200000	200000	30820
Poisson's ratio	0.30	0.30	0.18
Yield strength (MPa)	481	421	NA
Ultimate Strength (MPa)	669	531	NA
Compressive strength (MPa)	NA	NA	43
Density (kg/m ³)	7850	7850	2400
Coefficient of thermal expansion (/°C)	12*10 ⁻⁶	12*10 ⁻⁶	10*10 ⁻⁶

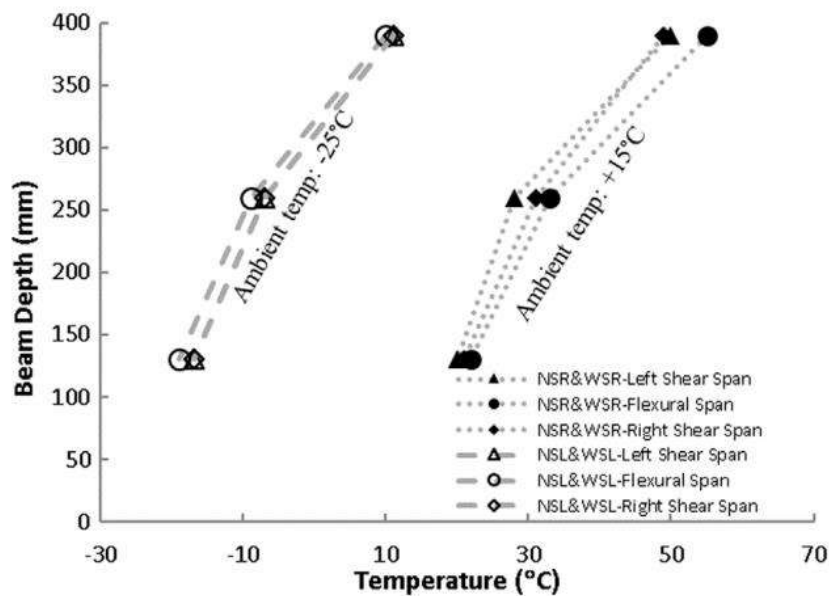


Fig. 8. Thermal gradient over the depth of the beams during tests

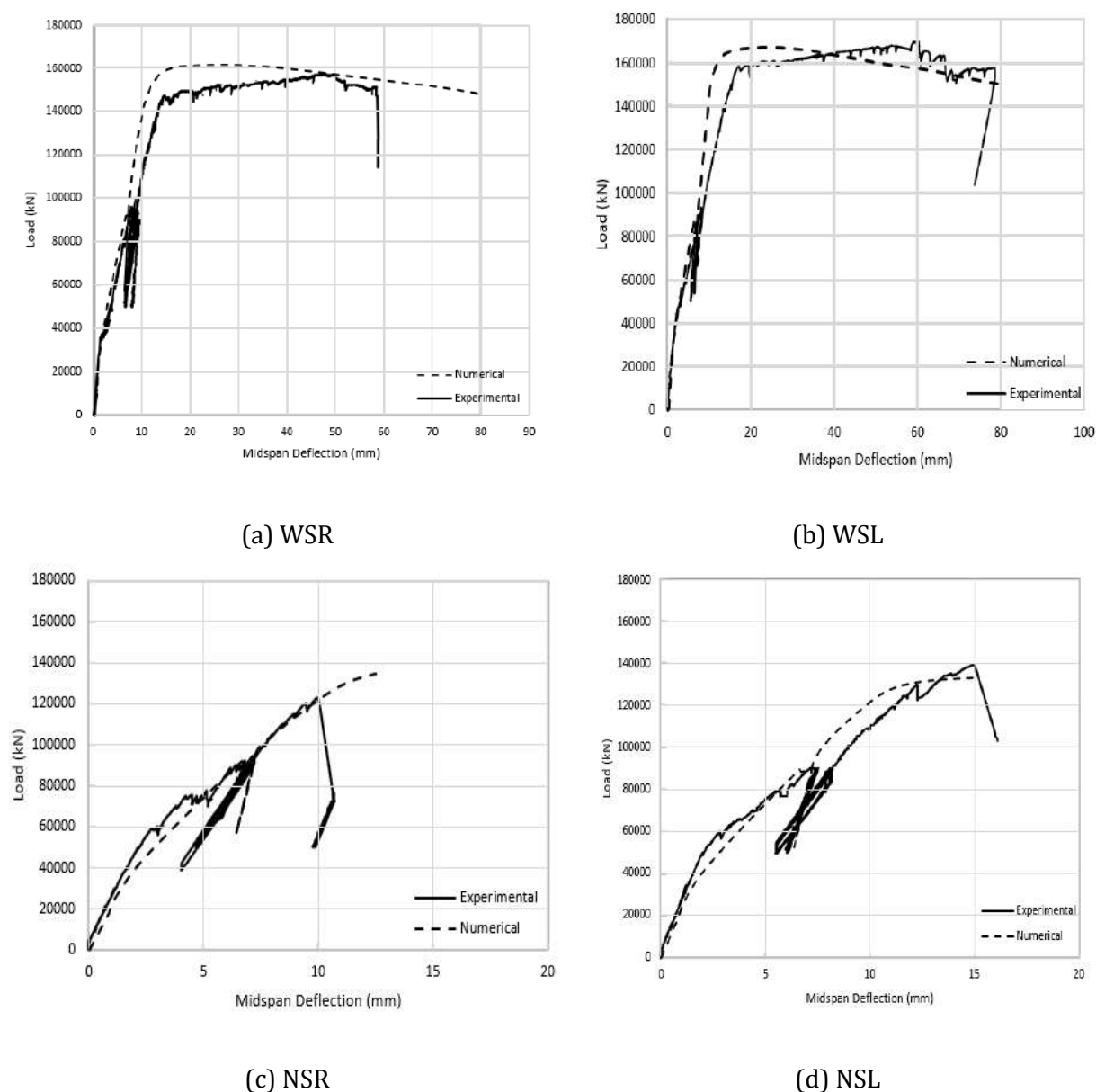


Fig. 9. Load-displacement relationship for Mirzazadeh et al. [9] specimens

4.3 Mesh Sensitivity Analysis

Mesh sensitivity refers to the impact of mesh size and quality on the accuracy of numerical predictions. Lower resolution meshes typically yield more accurate results but also increase computational cost. Sparse number of mesh elements lowers computer resource requirements but can sacrifice precision. To obtain reliable simulations at a reasonable cost, the balance between mesh size is important. Mesh supports the optimization of this balance, by allowing elements sizes to be modified according to error estimates. Mesh sensitivity is an important aspect for accurate and efficient simulations in engineering and scientific programs. The results provided in Fig. (10) with mesh sensitivity analysis reveals load-deflection curves of RC Beam (4) for M30, M40, M50 and M60 compared to experimental results. As the mesh becomes finer (from M60 to M30), NLFEA predictions are in better agreement and nearer to the experimental curve, particularly so on the ascending limb and close to the peak load. The quantification of initial stiffness and peak load for coarser meshes. M60 underestimates them; however, finer meshes (M30, M40) result in more reliable and stable results. Past the peak, all models fit the decrease in trend, but with varying amounts of smoothness. Regarding the mesh convergence of the presented results, M30, M40, and M50 are in agreement indicating increased independence at finer resolutions.

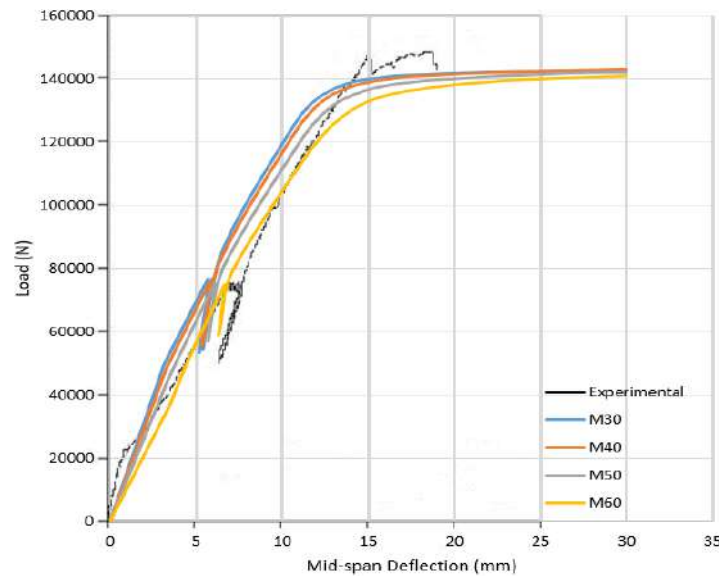


Fig. 10. Mesh sensitivity analysis carried out for Beam (4)

5. Results and Discussion

A comparison of test results and NLFEA predictions for the four RC beams tested by DeRosa et al. is summarized in Table (6). Generally, NLFEA tended to slightly over-predict the cracking loads, such as Beam 1 39kN vs. 36kN, with difference +8.3%. It also tended to underestimate the ultimate capacities, such as Beam 2 142kN vs. 153kN, with difference -6.5%. For all other beams similar minor deviations, but still within acceptable limits, were observed. In general, NLFEA-based method gave fairly accurate predictions for both cracking behavior and ultimate strength, with margin of error within 10%, hence confirming the reliability of such approach for this type of analysis.

Table 6. Comparison between experimental and NLFEA results for DeRosa et al. [8] specimens

Beam No.	Experimental Cracking Load (kN)	NLFEA Cracking Load (kN)	Difference (%)	Experimental Ultimate Load (kN)	NLFEA Ultimate Load (kN)	Difference (%)
1	36	39	8.33	142	130	-8.45
2	30	33	10.00	153	142	-6.54
3	30	28	-6.67	128	126	-1.56
4	24	25	4.17	149	142	-4.70

Table7. Comparison between experimental and NLFEA results for Mirzazadeh et al. [9] specimens

Beam ID	Experimental Cracking Load (kN)	NLFEA Cracking Load (kN)	Difference (%)	Experimental Ultimate Load (kN)	NLFEA Ultimate Load (kN)	Difference (%)
WSR	36	39	8.33	157	161	2.54
WSL	48	51	6.25	169	168	-0.60
NSR	60	52	-13.33	123	135	9.76
NSL	56	52	-7.14	139	133	-4.32

A comparison with the four RC beams tested by Mirzazadeh et al. is presented in Table (7). In general, the NLFEA results were in excellent agreement with the experimental data. For instance, the Beam WSR was also in good agreement with cracking load (+8.3%) and ultimate strength (+2.5%), and that for the Beam WSL was also in a reasonable point (cracking load +6.3%, ultimate, with difference -0.6%). For Beams NSR and NSL, larger discrepancies were found, with

underpredictions of the cracking load (-13.3% and -7.1% , respectively) but overpredictions or very small reductions for the ultimate strength. In consideration of these discrepancies, however, the NLFEA method was in reasonable agreement with cracking mode and ultimate capacity.

The comparison between the experimental load-displacement responses and those provided by NLFEA reveals an overall satisfactory agreement, with margin of error within 10%. Thereby validating the numerical model as far as it concerns the response of the structure at global level. The experimental results show evident residual deformation due to the cyclic loading conditions applied. This indicates irreversibility of the resistances accompanying crack formation in concrete and yielding of steel and accumulated damage. These residual deformations are less significant in the corresponding NLFEA results, likely a result of the idealized material constitutive behavior and damage evolution inherent to the numerical framework. In addition, according to the numerical simulations, stiffer behavior was noticed as compared to experimental results. Such a small difference in the stiffness may be attributed to mainly using the cyclic loading in the experimental tests, which leads to progressive stiffness degradation and micro-cracking phenomena of which are only partially represented within the numerical model. In general, such minor differences, good agreement between the NLFEA and experimental responses, show that the proposed developed NLFEA technique is reliable in terms of simulating complex behavior under combined loading. In general, the NLFEA predictions were well correlated with experimental data and particularly in ultimate load prediction. More variation in predicting cracking loads occurred, possibly due to sensitivity towards modeling assumptions with respect to tension stiffening and initial cracking response. However, the NLFEA model was successful in predicting the structural behavior of the RC beams subjected to the loading condition used in this study.

6. Conclusions

The thermo-mechanical response of RC members under combined gravity and temperature loading was analyzed in this study, based on a developed NLFEA approach. The numerical results demonstrate that temperature affect strongly the cracking initiation, stiffness softening and ultimate load-carrying capacity. Thus, volumetric changes need to be taken explicitly into account in structural analysis. Validation with experimental results of RC beams subjected to uniform temperature changes and through-depth temperature differences showed a efficient correlation between NLFEA results and test results. Cracking loads were in general overestimated by about 6–9% but ultimate capacities had some underestimation; however, with differences of an order of only 7% on average. The maximum error between simulation and experiment was less than 10% for all specimens tested. The load-displacement curves, initial stiffness, peak forces, post peak softening and observed cracking patterns were correctly captured by the NLFEA demonstrating a consistent description of structural response to coupled thermal and mechanical loading.

The outcomes of NLFEA demonstrate the necessity of including the concrete material nonlinearity including tension stiffening and stiffness degradation by using model in conjunction with modified Hognestad stress-strain relationship. The explicit inclusion of the differential thermal expansion between steel reinforcement and concrete was found to be essential for producing realistic internal stress states and crack propagation behavior. Mesh sensitivity analyses additionally substantiated numerical convergence, as smaller thicknesses (M30–M40) yielded stable and accurate predictions. The incorporation of the latest modulus of rupture expression from ACI 318-25 resulted in better estimates for cracking load, as compared to earlier code expressions. The proposed NLFEA framework affords a strong foundation for subsequent parametric studies to quantify the interaction of volumetric changes and gravity loads. Such findings may fill some existing gaps for influencing temperature load effects in the international design codes. The results also indicate calibrated linear finite element methods could be established to achieve NLFEA accuracies for practical designs. Such a complementary approach would make structural analysis more reliable and promote wider application and practicability of temperature-induced design provisions in engineering practice.

References

- [1] Vecchio FJ. Nonlinear Analysis of Reinforced Concrete Frames Subjected. ACI Struct J. 1987;84:492-501.
- [2] Vecchio FJ, Agostino N, Angelakos B. Reinforced concrete slabs subjected to volumetric changes. Can J Civ Eng. 1993;20(5):741-753. <https://doi.org/10.1139/193-099>
- [3] Ariyawardena N, Ghali A, Elbadry M. Experimental study on thermal cracking in reinforced concrete members. ACI Struct J. 1997;94(4):432-441. <https://doi.org/10.14359/494>
- [4] Elbadry M, Ghali A. Thermal stresses and cracking of concrete bridges. In: J Proc. 1986;83(6):1001-1009. <https://doi.org/10.14359/2670>
- [5] Elbadry M, Ghali A. Control of thermal cracking of concrete structures. ACI Struct J. 1995;92(4):435-450. <https://doi.org/10.14359/993>
- [6] Ghali A, Favre R, Elbadry M. Concrete structures: Stresses and deformations: Analysis and design for serviceability. CRC Press; 2018. <https://doi.org/10.1201/9781482289138>
- [7] Do TMD, Lam TQK, Van Thuc N. Analysis of axial stiffness reduction factors in thermal analysis of multistorey buildings. In: IOP Conf Ser Mater Sci Eng. 2020;988(1):012059. <https://doi.org/10.1088/1757-899X/988/1/012059>
- [8] DeRosa D, Hoult NA, Green MF. Effects of varying temperature on the performance of reinforced concrete. Mater Struct. 2015;48:1109-1123. <https://doi.org/10.1617/s11527-013-0218-y>
- [9] Mirzazadeh MM, Noël M, Green MF. Effects of low temperature on the static behaviour of reinforced concrete beams with temperature differentials. Constr Build Mater. 2016;112:191-201. <https://doi.org/10.1016/j.conbuildmat.2016.02.216>
- [10] Collins MP, Mitchell D. Prestressed concrete structures. Response Publications; 1997.
- [11] El-Tayeb EH, El-Metwally SE, Askar HS, Yousef AM. Thermal analysis of reinforced concrete beams and frames. HBRC J. 2017;13(1):8-24. <https://doi.org/10.1016/j.hbrj.2015.02.001>
- [12] Pooja M, Karthiyaini S. Investigation of flat slab structures with and without expansion joints for thermal stresses. Int J Civ Eng Technol. 2017;8:1287-1295.
- [13] Hisham A, Mahmoud GH, Nasr NE, Mohamed A, Rahim A. Finite Element Modelling of Structures Subjected to Thermal Loading. IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE.2018;15(6): 31-43
- [14] Hisham A, Turk BS, Nasr NE, Goda HA. Assessment of Flat Slab Axial Stiffness in Thermal Analysis. Al-Azhar University Civil Engineering Research Magazine. 2022: 44(1):133-158
- [15] Ahmed K. Temperature effects in multi-story buildings. JES J Eng Sci. 2011;39(2):249-267. <https://doi.org/10.21608/jesaun.2011.120405>
- [16] Systèmes D. Abaqus analysis user's manual. Simulia Corp. Providence, RI, USA. 2007;671(40):672.
- [17] American Concrete Institute, ACI Committee 224. Control of Cracking in Concrete Structures (ACI 224R-01). American Concrete Institute-ACI; 2001.
- [18] American Concrete Institute. Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary. American Concrete Institute; 2025.
- [19] European Committee for Standardization (CEN). Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings (EN 1992-1-1). CEN; 2004.
- [20] British Standard. Eurocode 8: Design of structures for earthquake resistance (BS EN 1998-1). 2005.
- [21] ECP Committee. Egyptian Code for Design and Construction of Concrete Structures (ECP 203-2025). Cairo, Egypt: Housing and Building National Research Center; 2025.
- [22] ACI Committee 435. Control of deflection in concrete structures (ACI 435R-95) (Reapproved 2000). American Concrete Institute; 2000.
- [23] Hognestad E. A study on combined bending and axial load in reinforced concrete members (Report No. 399). University of Illinois Engineering Experiment Station; 1951.
- [24] Lubliner J, Oliver J, Oller S, Onate E. A plastic-damage model for concrete. Int J Solids Struct. 1989;25(3):299-326. [https://doi.org/10.1016/0020-7683\(89\)90050-4](https://doi.org/10.1016/0020-7683(89)90050-4)

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Enhanced properties of recycled ceramic fine aggregates via cement and sodium bicarbonate treatment: Experimental study, microstructure analysis and modeling

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Abstract

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The aim of this contribution is to improve the physical characteristics of waste ceramic fine aggregates (WCFA) which have a particle size of 0 to 5 mm through surface chemical treatment, while diagnosing: bulk density, water absorption, and apparent porosity. The impregnation protocol involves applying a solution of cement (C) and sodium bicarbonate (SB) to the aggregate surface at the following concentrations (C: 2% and 4%; SB: 2%, 4%, 6% and 8%) and immersion durations of 12, 24, 48, and 72 hours. The microstructural examination consisting of X-ray diffraction (XRD), Fourier transform infrared spectroscopy (FTIR), and scanning electron microscopy (SEM) with Energy Dispersive Spectroscopy (EDS) was used to determine the physical effectiveness of this treatment to the WCFA accurately. A design of experiments dependent on response surface methodology was created to find the best treatment conditions. Results showed that the sample (4C4SB24H) increased its bulk density by about 11.5% and reduced water absorption by immersion and apparent porosity by 41.3% and 47.9% respectively as compared with the untreated control sample. A normal new calcite distribution was observed, which very well demonstrated the maximum reduction of surface pore spaces, thus confirming the quality of the chosen sample one more time. The statistical interpretation via Box-Behnken (BBD) suggested that the proposed model was very effective since the maximum error was only 7%.

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1. Introduction

The fast industrial development in developed countries has made waste management problems even worse. The issue is caused by environmental and economic pressures together with the depletion of natural resources [1]. Waste recovery has been made a necessity in the course of the sustainable development strategy. This method is highly pertinent to the construction industry in which the use of concrete must be limited to impacts that are not harmful to nature [2]. Various nations such as England, China, and Spain have stringent laws that govern the use of recycled

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aggregates for concrete production. These laws promote aggregates primarily made of crushed concrete and natural stone that are stuck together with mortar [3–5]. On the contrary, Germany and the Netherlands have gone a step further and have set lax rules permitting the blending of recycled aggregates from concrete, stone, ceramics, masonry, asphalt, or any other mineral materials [6,7]. The application of recycled ceramic aggregates from the sanitary industry in concrete is an area of research that is growing very fast due to the absence of specific standards for their reuse [8]. A number of studies have pointed out its potential as an active additive to cement by taking advantage of its pozzolanic activity to enhance the durability and mechanical performance of materials [9–14]. Other studies have shown that they can be effective natural aggregate partial substitutes, and the findings are quite promising depending on the substitution rates [15–23]. Ceramic waste, primarily from defective items or manufacturing scraps, is produced every day and increasingly challenging for the producers [24,25]. Poor waste management can cause significant environmental impacts, so it is essential for the construction sector to come up with eco-friendly recovery options [26]. Treatment facilities for construction and demolition waste (CDW) have shown to be cost-effective [27], still There are technical issues that need to be solved before these materials can be used as a substitute for natural aggregates. The mechanical behavior of the resulting concrete was a significant impact [28–30], the workability at the same water/cement (w/c) ratio has already been mentioned as an important factor [31], and the durability of concrete has been also the subject of discussion [32]. Recycled aggregates have been carefully chosen in some research works to give the same or better results [33], while their pre-treatment, washing, for instance, to remove fine fractions with high soluble sulfate content and water absorption was also proved to be very effective [34–36], besides, the cement with the incorporation of mineral admixtures was used [37,38]. Ceramic aggregates ensure an excellent level of resistance while still being very effective in heat and having a very low thermal expansion coefficient [39]. Replacing natural aggregates with them in concrete is the very process that helps to benefit the environment by managing the industrial waste [40,41]. Recent experiments by D. J. Anderson et al. [42], A. V. Alves et al. [43], C. Medina et al. [44], L. A. Pereira-de-Oliveira et al. [45], M. O'Farrell et al. [46], J. Silva et al. [47], have studied the introduction of ground sanitaryware and earthenware ceramics into building materials with the focus being on their usability as alternative raw materials for mortar and concrete composition. This study has demonstrated the bright potential of this waste as a source of materials with good mechanical and physical properties, thus contributing to the cause of sustainable development. Different methods of treatment have been investigated to enhance the recycling aggregate's resistance to water absorption, among which A. Barbudo et al. [48], established that recycled concrete aggregates (RCA) could substitute for natural aggregates in a maximum proportion of 20% without causing any reduction in compressive strength, and a substitution of 100% is possible when a water reducer is mixed up to offset these high absorption levels, this approach focuses primarily on adjusting the mix composition by adding water-reducing admixtures to maintain mechanical performance. Similarly, D. Matias et al. [49], pointed out that the addition of a superplasticizer partially compensates for the loss of workability and strength due to high water absorption, while improving the compactness and mechanical performance of concrete made from recycled concrete aggregates (RCA). these works brought up the topic of using water-reducing admixtures, while K. K. Sagoe-Crentsil et al. [50], have shown that industrial processing of recycled concrete aggregates (RCA) produces more spherical and smoother particles, and that pre-saturation of these aggregates in water improves the workability of concrete without significantly reducing its compressive strength at 28 days. Likewise, J. García-González et al. [51] demonstrated the feasibility of pre-saturating recycled aggregates in order to limit their water absorption. Immersion for 3 to 5 minutes significantly improves the workability of the concrete (plastic to fluid consistency), with limited loss of compressive strength ($\approx 11-13\%$). These studies focused on the method of pre-saturation of aggregates. Some studies have examined the possibility of applying various chemical treatments to recycled aggregates. For example, S. Ismail and M. Ramli [52] have shown that surface treatment of recycled aggregates, combining mild acid attack to remove mortar residues and impregnation with a cementitious solution to reduce porosity, significantly improves their physical and mechanical properties, enabling the concrete to achieve better strength and reduced shrinkage. V. Spaeth and A. Djerbi Tegguer [53] showed that polymer treatments applied to recycled aggregates form a protective film in the porous network,

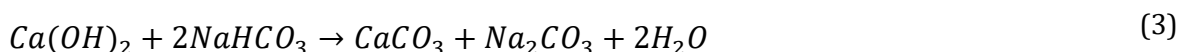
significantly reducing water absorption and strengthening the cement matrix. A. M. Grabiec et al. [54] showed that calcium carbonate biodeposition effectively reduces the water absorption of recycled aggregates, with a notable improvement for fine fractions ($\approx 30\text{--}35\%$). Confirmed by SEM observations. However, the proposed methods, such as carbonation and chemical waterproofing, have significant limitations, including prolonged reaction times and incompatibility with sustainable development requirements. The very slow and difficult alternative of bio-deposition of calcium carbonate, which has been studied by V. Achal et al. [55] and Q. Chunxiang et al. [56], still holds promise but is difficult to reconcile with the working conditions of construction sites. Some previous studies have produced contradictory results in respect to the physical properties of ceramic waste, drawing attention to large difference which varies depending on its origin, kind and manufacturing methods. For instance, Kwok Wei Shah and Ghasan Fahim Huseien [57] analyzed white ceramic tile waste, measuring a real density of 2.38 g/cm^3 , a visual density of 1.450 g/cm^3 , and a water absorption of 1.3% . On the other hand, Abadou Y et al. [58] reported values of 2.288 g/cm^3 for the real density, 1.212 g/cm^3 for the visual density, and a water absorption of as high as 30% for ceramic sanitary waste. Derrick J. Anderson et al. [42] have mentioned a real density of 2.263 g/cm^3 and a water absorption of 5.5% , while D. Tavakoli et al. [28] found 2.6 g/cm^3 , 1.750 g/cm^3 and 2% for another variety of ceramics. This evidence, supported by other research works [59–68], reveals the fundamental differences in the features of ceramic waste according to its source and chemical composition. In order to improve the physical characteristics of recycled aggregates, particularly their water absorption, for use in concrete or cementitious matrices, several treatment methods have been developed. The results obtained vary depending on the nature of the aggregates and the technique used. Mistri A et al. [65] conducted a comparative study of different treatments aimed at improving the properties of recycled aggregates (RCA). They observed a reduction in water absorption of 48.62% for a treatment combining mechanical grinding and soaking, 18.82% for autogenous cleaning, 19.36% for treatment with an acid solution (0.1 M HCl for 3 days), 4.23% for simple acid treatment (HCl), 9.47% for thermal treatment at 350°C , and 2.03% for chemical treatment with acetic acid ($\text{C}_2\text{H}_4\text{O}_2$). García-González J et al. [69] studied the improvement in the quality of mixed and ceramic recycled aggregates through calcium carbonate bio deposition and reported a decrease in water absorption of 46% (TEC-REC samples), 41% (ANTWERP) and 16% (BIERZO). Mareeswari Andal N et al. [70] analyzed the effect of chemical treatment on recycled fine aggregates and found a reduction in water absorption of 12.48% , 38.31% , and 54.35% for HCl concentrations of 0.1 M , 0.4 M , and 0.7 M , respectively. Other studies [54,71,72] have also explored different treatment techniques, but have mainly focused on the mechanical and physical properties of concrete after incorporation of the treated aggregates, rather than on the in-depth characterization of the aggregates themselves. With regard to treatment using a suspension of cement and sodium bicarbonate, to the best of our knowledge, only one study, conducted by Phan et al. [73], has been identified. The authors studied effect of recycled coarse aggregate treatment using cement and sodium bicarbonate slurry on compressive strength of hardened concrete and reported water absorption of RCA of 3.3% , corresponding to an improvement of 18.51% . The cement mainly containing tricalcium silicate (C_3S) and dicalcium silicate (C_2S) reacts with sodium bicarbonate (NaHCO_3) through several stages. The hydration of cement consists of water reacting with cement to produce calcium hydroxide (Ca(OH)_2) and other hydration products, as illustrated in chemical Eqs. (1)-(2):



and



Subsequently, calcium hydroxide and sodium bicarbonate interact to produce calcium carbonate (CaCO_3), sodium carbonate (Na_2CO_3), and water as per the following chemical Eq. (3):



Calcium carbonate (CaCO_3) is therefore produced, which has the potential to occupy the voids of recycled aggregates, enhance their strength and durability and also decrease their porosity and water absorption.

The research was based on the only existing study on the treatment of recycled aggregates using a suspension of cement and sodium bicarbonate. We aimed to optimize the treatment of fine aggregates from ceramic waste using a combination of cement and sodium bicarbonate suspension, varying their concentrations along with the immersion durations, while carrying out an in-depth characterization of this type of aggregate. The physical, structural, and morphological analyses were performed to evaluate the impact of these parameters and to find the best conditions for enhancing the density of aggregates with minimum porosity and water absorption. To uncover the quantitative relationship among the treatment variables and the WCFA properties, a statistical modeling strategy was employed, which is based on response surface modeling (RSM) and Box-Bennkin design of experiments (BBD).

2. Methods

2.1 Materials

2.1.1 Waste Ceramic Fine Aggregates (WCFA)

The fine ceramic aggregate waste (WCFA) was produced by grinding ceramic floor tiles from the SARL CONFORT CERAM factory located in El Hadjeb, Biskra, Algeria. This waste is formed by red ceramics that are either from defective products or manufacturing residues. It is a very durable material that offers the highest resistance against both chemical and physical degradation [16,25].

Only unglazed tiles, which are purely fired biscuits, were taken for the study so that the physical properties such as porosity, water absorption, and density could be examined. The lack of the glaze layer (which is an impermeable layer) is taken as the most cautious case in this research.



Fig. 1. Toothed jaw crusher used for crushing aggregates

Waste ceramics were subjected to crushing by a toothed jaw crusher calibrated with the aim of fine aggregate production Fig. 1. This type of crusher, for the most part, is utilized to produce end products for aesthetic applications in the construction of marble powder. The machine, in this case, works by making the materials go through the two jaws, one of which is stationary while the other is free to move. The materials are pressed, and the teeth of the jaws enhance the compressive force and create more pressure points which result in more efficient crushing and also controlled particle size reduction. This kind of crusher is mostly used in the mining industry to deal with hard and rough materials like rocks, ores, and even building materials. The larger aggregates were then processed through a 5 mm sieve to get Waste Ceramic Fine Aggregates (WCFA). The measuring of the sieve was done according to the NF EN 933-1 standard [74]. A histogram showing the size

distribution of the aggregate obtained is presented in Fig. 2. The high value of the Coefficient of Uniformity ($C_u = 12.32$) implies that there was a significant variation in the sizes of the particles, and the Coefficient of Curvature ($C_c = 1.59$) is supportive of a fairly high and good quality distribution of particles. The fineness modulus, being equal to 3.51, shows that WCFA is coarser than standard sand used in construction with regard to the latter's finer particle size distribution. But then, the uniformity of the particle size distribution may help in the incorporation of WCFA into particular formulations of building materials with the possibility of alterations made to the performance of the product to be optimized.

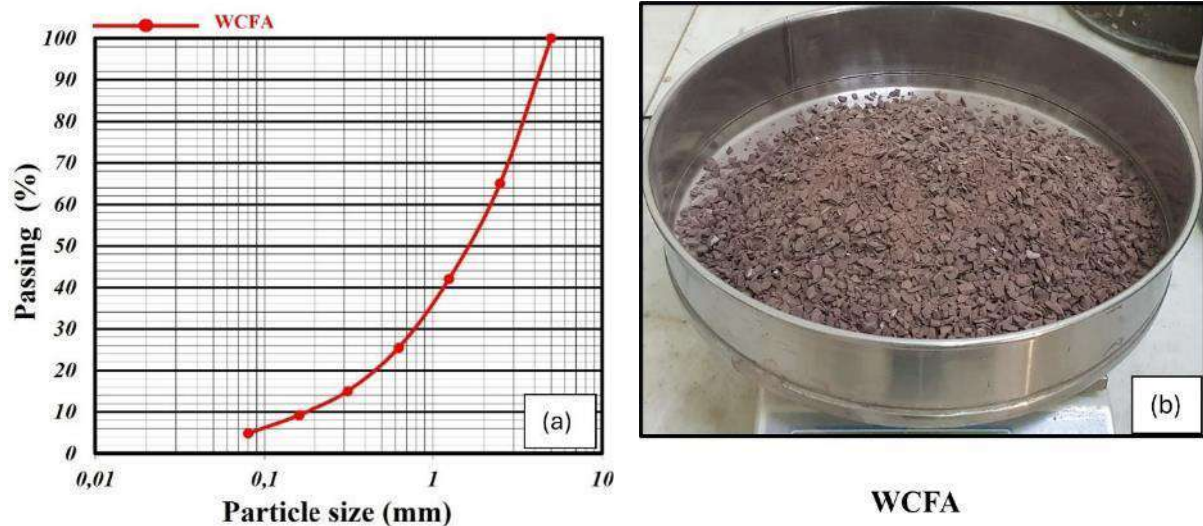


Fig. 2. Toothed jaw crusher used for crushing aggregates

2.1.2 Cement (C) and sodium bicarbonate (SB)

The cement that was utilized is CEM I - 42.5N SR3 LH (sulfate-resistant with low heat of hydration), which complied with the Algerian standard NA442-2013, having a clinker content of 95%-100% and containing 0-5% of additional components. Supplied by Biskria Cement. The main phases of clinker, namely dicalcium silicate (C_2S) and tricalcium silicate (C_3S), react with water according to Eqs. (1)-(2), producing calcium hydroxide ($Ca(OH)_2$). The latter then reacts with sodium bicarbonate ($NaHCO_3$) to form calcium carbonate ($CaCO_3$), as illustrated in Eq. (3).

Sodium bicarbonate ($NaHCO_3$) purity 100.3% min was supplied by EDEN-LABO SARL.

2.2 Sample Preparation

WCFA were crushed and sieved to obtain a uniform granulometry, with particle sizes ranging from 0.063 mm to 4 mm. Aggregates smaller than 4 mm were washed through a 0.063 mm sieve to remove irrelevant fine particles [58,74,75]. The aggregates were then air-dried under controlled laboratory conditions (20 ± 2 °C and $50 \pm 5\%$ relative humidity), for 14 days until a constant mass was obtained [65,70,73,74,75]. The basic properties of the fine ceramic waste aggregates (WCFA) before treatment were evaluated to establish a comparative basis for the processed samples. The results obtained show an absolute density of 2.3 g/cm^3 , a bulk density of 1.103 g/cm^3 , a water absorption of 6.046% and an apparent porosity of 14.32%. These data indicate that the aggregates studied have a high porosity and significant absorption capacity. Previous studies have reported variable characterizations of crushed ceramic waste [29,43,58–65,66–68,73].

2.3 Sample Treatment

After crushing, treatment solutions were prepared by dissolving cement and sodium bicarbonate in distilled water under magnetic stirring for 5 minutes. During treatment, samples were kept under gentle agitation to prevent sedimentation and ensure uniform exposure to the solution.

WCFA were treated to examine the effect of cement and $NaHCO_3$ suspensions on their porosity and water absorption. Samples of 1 kg were prepared for each treatment [73]. WCFA were immersed

in a solution comprising various amounts of cement and NaHCO_3 (Fig. 3.), with varying immersion durations, as shown in Table 1.



Fig. 3. Photos illustrating WCFA treatment process

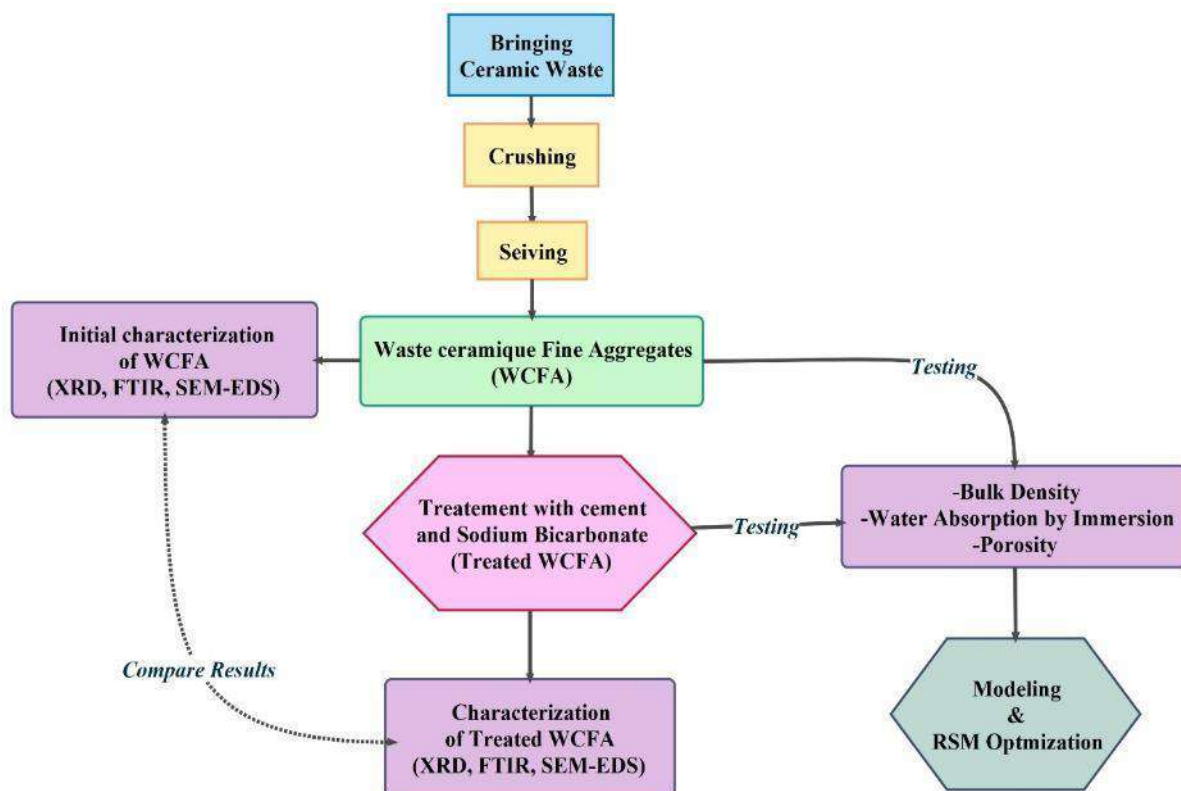


Fig. 4. Experimental procedure

Table 1. Mix composition and experimental conditions for processing recycled aggregates

Symbol	Slurry concentration		Components				Immersion duration (hours)
	C (%)	NaHCO ₃ (%)	C (%)	NaHCO ₃ (g)	W (g)	RCA (g)	
0C0SB0H	0	0	0	0	0	0	0
2C2SB12H	2	2	20	20	1000	1000	12
2C4SB12H	2	4	20	40	1000	1000	12
2C6SB12H	2	6	20	60	1000	1000	12
2C8SB12H	2	8	20	80	1000	1000	12
4C2SB12H	4	2	40	20	1000	1000	12
4C4SB12H	4	4	40	40	1000	1000	12
4C6SB12H	4	6	40	60	1000	1000	12
4C8SB12H	4	8	40	80	1000	1000	12
2C2SB24H	2	2	20	20	1000	1000	24
2C4SB24H	2	4	20	40	1000	1000	24
2C6SB24H	2	6	20	60	1000	1000	24
2C8SB24H	2	8	20	80	1000	1000	24
4C2SB24H	4	2	40	20	1000	1000	24
4C4SB24H	4	4	40	40	1000	1000	24
4C6SB24H	4	6	40	60	1000	1000	24
4C8SB24H	4	8	40	80	1000	1000	24
2C2SB48H	2	2	20	20	1000	1000	48
2C4SB48H	2	4	20	40	1000	1000	48
2C6SB48H	2	6	20	60	1000	1000	48
2C8SB48H	2	8	20	80	1000	1000	48
4C2SB48H	4	2	40	20	1000	1000	48
4C4SB48H	4	4	40	40	1000	1000	48
4C6SB48H	4	6	40	60	1000	1000	48
4C8SB48H	4	8	40	80	1000	1000	48
2C2SB72H	2	2	20	20	1000	1000	72
2C4SB72H	2	4	20	40	1000	1000	72
2C6SB72H	2	6	20	60	1000	1000	72
2C8SB72H	2	8	20	80	1000	1000	72
4C2SB72H	4	2	40	20	1000	1000	72
4C4SB72H	4	4	40	40	1000	1000	72
4C6SB72H	4	6	40	60	1000	1000	72
4C8SB72H	4	8	40	80	1000	1000	72

2.4 Experimental procedure

Physical testing of fine ceramic aggregates follows the methodology shown in Fig. 4. illustrates the sequence of experimental steps, from sample preparation to evaluation of the physical properties of the aggregates and their modeling using the Response Surface Method (RSM).

2.4.1 Bulk Density and Absolute Density

The WCFA bulk density (ρ_a) was measured as per ASTM C29/C29M [76] standard method using the shoveling procedure. The experiment was carried out in three repetitions for each test to confirm the accuracy and reproducibility of results. The total mass of the test sample was reduced at the temperature of $110 \pm 5^\circ\text{C}$ until no further loss of weight was observed to eliminate the effect of moisture left in the sample. A test mold, which was of known volume, was used for the test and the volume was previously validated according to the standard requirements to assure the accuracy of the calculation. The aggregate was then filled in the container without compacting it by pouring gently using a shovel from a height of no more than 50 mm above the edge of the container. To prevent segregation of the particles care was taken. The excess was then cut off using a ruler or

fingers, to equalize the heights of the surface protrusions and fill up the voids. Thereafter, the bulk density was computed using the following Eq. (4):

$$\rho_a = \frac{(G - T)}{V} \quad (4)$$

where, G is the mass of the filled container (g), T is the mass of the empty container (g), V is the volume of the container (cm³) [76]. All measurements were recorded to ± 0.01 g precision using a calibrated analytical balance. The absolute density (ρ_s) corresponds to the mass per unit volume of the material constituting the processed WCFA, without considering internal or intergranular voids. It is determined in accordance with ASTM C128 [72] with three repetitions of each test and by the following method: a graduated cylinder is filled with an initial volume of water (V_1), then a dry sample of mass (M) is added. After removing air bubbles, the new volume (V_2) is measured. The absolute density is obtained by the Eq. (5):

$$\rho_s = \frac{M}{(V_2 - V_1)} \quad (5)$$

where, M is the mass of dry aggregate (g), V1 is the initial volume of water (cm³), V2 is the final volume after immersion of aggregate (cm³) [75].

2.4.2 Water Absorption by Immersion

The water absorption test by immersion was conducted according to ASTM C128 [72], , with each measurement carried out three times. The samples underwent drying in an oven at $110 \pm 5^\circ\text{C}$ to obtain a constant weight (weight variation $\Delta m < 0.1\%$). The next step was to submerge the samples in water for 24 hours. Later on, the samples were allowed to drain, and then placed on a wet cloth for 15 to 20 minutes to reduce the water loss. The samples were then in their saturated surface dry (SSD) condition, and their weight was measured and recorded as S. After that, the samples were dried again in an oven at $110 \pm 5^\circ\text{C}$ until constant weight was reached, and this weight was referred to as dry weight A. Water absorption was calculated to an accuracy of 10^{-3} g by applying Eq. (6):

$$\text{Absorption}(\%) = 100 \times \frac{(S - A)}{A} \quad (6)$$

where, S is the mass of the saturated surface dry (SSD), A is the mass of the dry sample after oven drying [75].

2.4.3 Apparent Porosity

The ultrafine particles in each WCFA sample with an initial mass of 1 kg are removed by washing through a 0.063 mm sieve. The method for measuring apparent porosity ASTM C20 [73] is utilized for the tests, which are repeated three times each. To start, the sample is placed in an oven at $110 \pm 5^\circ\text{C}$ for drying until its weight becomes constant, then it is weighed (D) in order to exclude moisture from the material's pores. Subsequently to the drying process, the sample is placed in boiling water for two hours to ensure complete saturation of the pores. Following this, the sample is left in water for a minimum of 12 hours, then it gets removed and it gets wiped for getting a saturated dry surface (SSD). Now the saturated weight (W) is measured. The volume of the sample is obtained by using Eq. (7):

$$V(\text{cm}^3) = W - S \quad (7)$$

where, W stands for the weight of the specimen at the time of the saturated surface-dry (SSD) state, S hydrostatic weight represents mass of the sample, determined by suspending the sample in water from a thin wire operated by metal wire [77]. It is from the value of W and D that the calculated value of apparent porosity is evaluated by the result of the analysis as outlined in Eq. (8):

$$\text{Porosity}(\%) = 100 \times \frac{(W - D)}{V} \quad (8)$$

where, W is the mass of sample saturated surface dry (SSD) (g), D is the mass of sample dry (g), V is the volume of the test specimens (cm³), as defined in Eq. (7). [77].

2.5 Microstructural Analyses

2.5.1 Fourier Transform Infrared Spectroscopy (FTIR) Analysis: Identification of mineral phases

An FTIR analysis for determining the mineral phases of the recycled fine ceramic aggregate was executed in order to examine the impact of chemical treatment on the degree of crystallinity of calcite (CaCO₃). The recordings of the spectra were made in the range of 4000–400 cm⁻¹, thus making it possible to observe the progress of the carbonate aggregates and simultaneously keep track of the treatment-induced structural changes.

2.5.2 X-ray Diffraction (XRD) Analysis: Evolution of Mineral Phases after Treatment

The fine ceramic aggregate (WCFA) was characterized for their mineralogical phases using x-ray diffraction (XRD) as well as observing the evolution of the vesicles before and after treatment with a suspension of cement and sodium bicarbonate. Major minerals can be identified using non-destructive techniques and relative estimates of their abundances, as a result, the study provided a comprehensive picture of the chemical reactions taking place during the treatment process and their influence on the microstructure as well as the properties of the aggregate being used.

2.5.3 Scanning Electron Microscopy (SEM) and Energy Dispersive Spectroscopy (EDS)

The morphology of the treated aggregate was scrutinized through scanning electron microscopy (SEM), which allowed changing in surface structure and the deposition, especially calcium carbonate (CaCO₃), which is a crucial raw material for enhancing the aggregate's physical properties, to be observed.

Moreover, EDS helped in confirming the elemental composition of the sample surfaces as well as the presence of Ca²⁺ through the technique that was excellent in this study which enabled the ascertainment of the proportions of chemical elements that were present and validation of the occurrence of calcium ions (Ca²⁺), which also play an essential role in the crystallization of calcium carbonate (CaCO₃).

2.6 Response Surface Analysis and Experimental Design

The software Design-Expert (ver. 13.0.5.0) was used to improve the precision of experimental results and to create graphs showing the treatment effects. The different concentrations of cement and sodium bicarbonate were tested to determine the best treatment conditions.

The Box-Behnken design (BBD), the most popular method for studying the influence of three independent factors, namely cement content (%C), sodium bicarbonate content (%SB), and immersion duration (ID in hours), was employed for the layout of the experiments. We chose the Box-Behnken model as it is very productive in studying the linear, quadratic and interaction effects among factors, to reduce the number of experiments and eliminate limit cases and thereby provide a viable tradeoff between statistical precision and experimental economy.

Response surface model (RSM) was applied to analyze data with focus on main variables as bulk density, water absorption, and apparent porosity. By utilizing a modified second-order polynomial (Eq. (9)), [78–82], the connection between the factors and the responses was improved. The resulting equation is as follows:

$$Y = \beta_0 + \sum_{i=1}^k \beta_i X_i + \sum_{i < j}^k \beta_{ij} X_i X_j + \sum_{i=1}^k \beta_{ii} X_i^2 \quad (9)$$

where, Y is the response, Xi represents the independent variables (experimental factors), β₀ is the intercept, β_i, β_{ij}, and β_{ii} are the coefficients of the linear, interactive, and quadratic terms respectively [78–82].

Table 2 shows the factors and levels used to design the experiment. The variables were encoded in three categories: the minimum value was represented by (-1), the median value was represented by (0), and the maximum value was represented by (+1). Different combinations of variables (total of 33 experiments) were tested with this methodology, as listed in Table 3.

Table 2. Factors and code levels of RSM

Factors	Coded	Levels of code		
		Min (-1)	Medium (0)	Max (1)
Cement (%)	A	0	2	4
Sodium Bicarbonate (%)	B	0	4	8
Immersion Duration (h)	C	0	24	72

3. Results and Discussion

3.1 Effects of Treatments on Physical Properties

The results, which are illustrated in Table 3, are indicative of the significant impacts both of cement slurry and NaHCO_3 treatments on physical properties of WCFA. The physical properties of the samples, such as bulk density, water absorption, and apparent porosity, were used to assess the impact of each independent variable (cement composition, sodium bicarbonate concentration, and immersion duration). The assessment results have been compiled and charted in Fig. 5.

Table 3. Physical and chemical properties of treated samples

Sample No.	Notation	Cement (%)	Sodium Bicarbonate (%)	Immersion Duration (h)	Absolute Density (g/cm^3)	Bulk Density (g/cm^3)	Absorption by Immersion (%)	Apparent porosity (%)
1	0C0SB0H	0	0	0	2.3	1.1030 ± 0.0016	6.046 ± 0.352	14.32 ± 0.79
2	2C2SB12H	2	2	12	2.3	1.1611 ± 0.0049	4.783 ± 0.132	10.72 ± 0.41
3	2C4SB12H	2	4	12	2.3	1.1767 ± 0.0080	4.047 ± 0.119	9.93 ± 0.34
4	2C6SB12H	2	6	12	2.3	1.1894 ± 0.0031	3.833 ± 0.117	9.61 ± 0.33
5	2C8SB12H	2	8	12	2.3	1.1896 ± 0.0061	4.243 ± 0.095	9.72 ± 0.31
6	4C2SB12H	4	2	12	2.3	1.1787 ± 0.0065	4.495 ± 0.147	10.74 ± 0.41
7	4C4SB12H	4	4	12	2.3	1.1946 ± 0.0025	3.906 ± 0.133	9.75 ± 0.40
8	4C6SB12H	4	6	12	2.3	1.2046 ± 0.0030	3.817 ± 0.124	9.67 ± 0.37
9	4C8SB12H	4	8	12	2.3	1.2053 ± 0.0065	3.774 ± 0.126	9.59 ± 0.39
10	2C2SB24H	2	2	24	2.3	1.1978 ± 0.0049	4.793 ± 0.136	9.55 ± 0.32
11	2C4SB24H	2	4	24	2.3	1.2011 ± 0.0040	4.283 ± 0.132	8.57 ± 0.31
12	2C6SB24H	2	6	24	2.3	1.2158 ± 0.0053	4.163 ± 0.137	8.8 ± 0.33
13	2C8SB24H	2	8	24	2.3	1.2211 ± 0.0029	4.443 ± 0.142	8.89 ± 0.34
14	4C2SB24H	4	2	24	2.3	1.2040 ± 0.0066	4.357 ± 0.124	9.65 ± 0.41
15	4C4SB24H	4	4	24	2.3	1.2303 ± 0.0044	3.549 ± 0.092	7.45 ± 0.22
16	4C6SB24H	4	6	24	2.3	1.2307 ± 0.0051	3.572 ± 0.098	8.45 ± 0.39
17	4C8SB24H	4	8	24	2.3	1.2291 ± 0.0010	3.712 ± 0.112	8.49 ± 0.36
18	2C2SB48H	2	2	48	2.3	1.1891 ± 0.0014	4.216 ± 0.096	7.51 ± 0.32
19	2C4SB48H	2	4	48	2.3	1.2055 ± 0.0068	4.309 ± 0.127	7.75 ± 0.31
20	2C6SB48H	2	6	48	2.3	1.2091 ± 0.0022	4.362 ± 0.109	7.66 ± 0.33
21	2C8SB48H	2	8	48	2.3	1.2093 ± 0.0038	4.497 ± 0.124	8.98 ± 0.34
22	4C2SB48H	4	2	48	2.3	1.1870 ± 0.0071	4.431 ± 0.141	8.49 ± 0.36
23	4C4SB48H	4	4	48	2.3	1.2029 ± 0.0069	4.223 ± 0.111	7.96 ± 0.28
24	4C6SB48H	4	6	48	2.3	1.2111 ± 0.0062	4.217 ± 0.126	7.82 ± 0.34
25	4C8SB48H	4	8	48	2.3	1.2037 ± 0.0031	4.219 ± 0.126	9.19 ± 0.36
26	2C2SB72H	2	2	72	2.3	1.1513 ± 0.0052	3.702 ± 0.109	8.49 ± 0.33
27	2C4SB72H	2	4	72	2.3	1.1729 ± 0.0011	3.717 ± 0.126	8.23 ± 0.35
28	2C6SB72H	2	6	72	2.3	1.1741 ± 0.0042	3.695 ± 0.109	8.47 ± 0.31
29	2C8SB72H	2	8	72	2.3	1.1754 ± 0.0051	3.949 ± 0.122	9.54 ± 0.40
30	4C2SB72H	4	2	72	2.3	1.1469 ± 0.0069	3.755 ± 0.117	8.64 ± 0.37
31	4C4SB72H	4	4	72	2.3	1.1602 ± 0.0071	3.567 ± 0.103	7.74 ± 0.33
32	4C6SB72H	4	6	72	2.3	1.1592 ± 0.0042	3.551 ± 0.096	7.54 ± 0.31
33	4C8SB72H	4	8	72	2.3	1.1496 ± 0.0055	3.932 ± 0.120	9.01 ± 0.38

As shown in Table 3, the absolute density of all specimens remained steady at $2.30 \text{ g}/\text{cm}^3$, with low variability as the standard deviation is $\pm 0.001 \text{ g}/\text{cm}^3$, the remarkable degree of density homogeneity ascertains that crushing and sieving of WCFA materials generated almost homogenous granules. On the contrary, using cement suspension together with sodium

bicarbonate (NaHCO_3) caused remarkable changes in bulk density, water absorption, and apparent porosity. These changes are indicative of the fine aggregate's structural and chemical alterations that have been made without affecting their absolute density to a large extent.

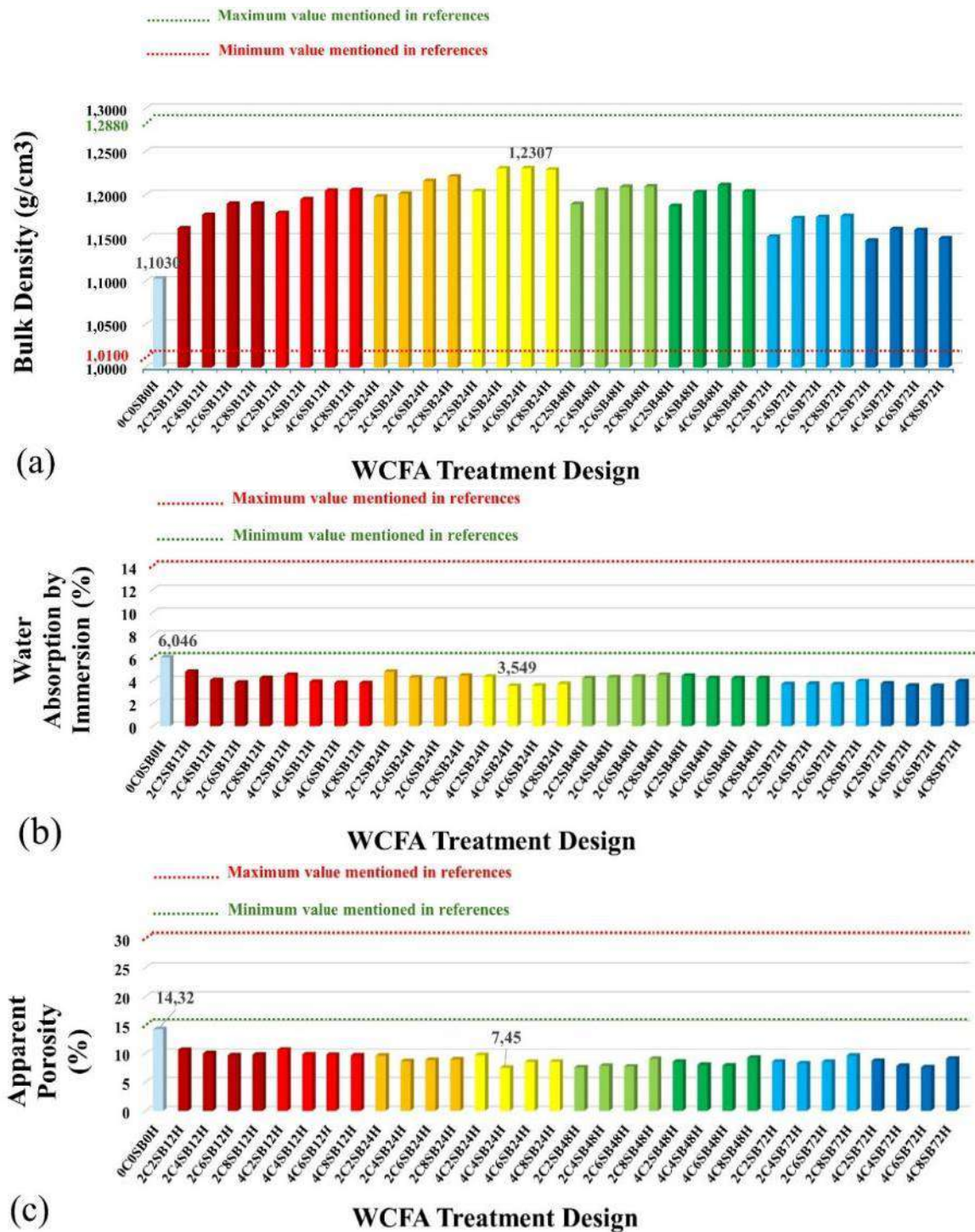


Fig. 5. Characteristics of the treated fine aggregate: (a) bulk density, (b) water absorption, and (c) apparent porosity

The samples that underwent treatment and were immersed for 12 hours exhibited a significant enhancement in their performance when compared to the control sample (0C0SB0H), signified by a bulk density of 1.1030 g/cm^3 , an immersion water absorption rate of 6.046%, and an apparent porosity of 14.32%. Literature data shows that fine aggregates obtained from crushing floor ceramic waste have bulk density ranging from 1.0100 to 1.2880 g/cm^3 , water absorption between 6.00% and 14.00%, and apparent porosity from 15.00% to 30.00% [18,42,58,60,62,67,83,84] Fig. 5.

Sample 2C8SB12H attains a maximum bulk density of 1.1896 g/cm³, a water absorption of 4.243%, and an apparent porosity of 9.72%. The figures present an enhancement of 7.85% in density, 29.8% decrease in water absorption, and 32.1% decline in apparent porosity when compared to the baseline sample 0C0SB0H, thus indicating the efficacy of the treatment from the start of immersion.

The effects of physical properties improvement by the dissolution of cement and NaHCO₃ in the mixtures or the prolongation of the immersion time were actually seen with increasing concentration. The 4C6SB24H specimen exhibited noteworthy characteristics such as a density of 1.2307 g/cm³, a water absorption percentage of 3.572%, and an apparent porosity of 8.45% following a 24-hour immersion period. When the results were compared to those of the control sample (0C0SB0H), the differences were the following: 11.6% increase in bulk density, 40.9% decrease in water absorption, and 41.0% reduction in apparent porosity corresponding to a considerable densification of the material and significant minimization of the internal voids all patterns of the same.

Sample 4C4SB24H exhibited a performance that was impressive, as it managed to achieve a bulk density of 1.2303 g/cm³, water absorption of 3.549%, and apparent porosity of 7.45%. With 11.5% higher bulk density, 41.3% less water absorption, and 47.9% less apparent porosity. The mentioned improved properties apply not only to the durability of fine aggregates but also to their adaptability to the various environmental situations existing in construction uses, which make them very prominent characteristics. The results support the ones presented by Phan et al. [73] who also communicated a similar density variation and porosity reduction while dealing with cementitious mixtures supplemented with sodium bicarbonate. The mixtures containing 4% of cement, 4% of sodium bicarbonate.

3.2 Structural modification of Treated WCFA

3.2.1 Analysis by Fourier Transform Infrared (FTIR) Spectroscopy: Mineral Phases

Fig. 6. shows the mineral phases identified by FT-IR spectroscopy, in a wavenumber range from 4000 to 400 cm⁻¹. Characteristic carbonate bands, particularly for calcite (CaCO₃), are observed at ~711, 743-776, and 792-793 cm⁻¹. These correspond to the out-of-plane deformation vibrations (ν_4) of the carbonate group (CO₃²⁻), indicating stable crystallization of CaCO₃ after treatment [30,85,86]. Silicates, mostly in quartz form (SiO₂), were identified by peaks at 553, 562, 569, and 580 cm⁻¹, which are attributed to bending vibrations Si-O. Also, peak at 1008 cm⁻¹ corresponds to the asymmetric Si-O-Si stretching vibration, confirming the presence of quartz under a well-crystallized structure.

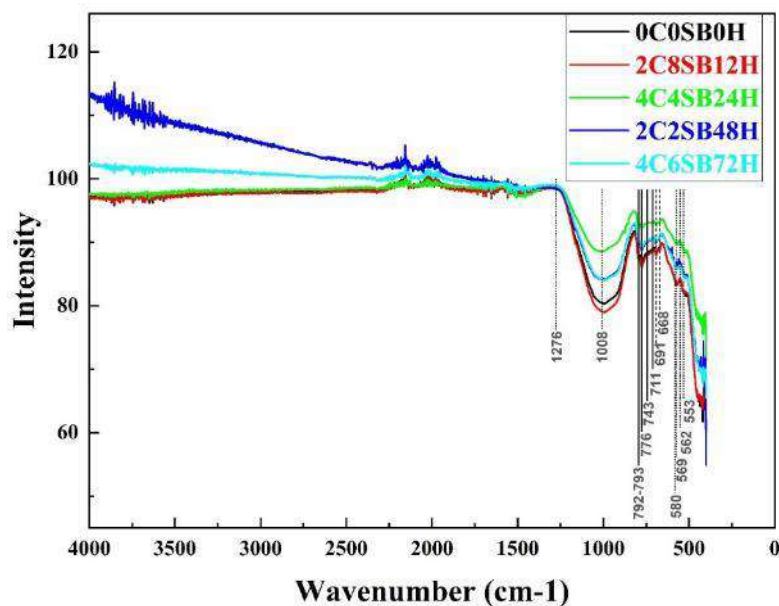


Fig. 6. FTIR analysis of mineral phases

A prominent band at 1276 cm^{-1} is likely to be due to Si-O bond deformation vibrations from secondary silicates which may point to the existence of either phyllosilicates or other silicate minerals. The bands at 668 and 691 cm^{-1} might be due to the composite of waste rock and carbonate phases of silicates, which would indicate mineral interactions. The results corroborate the hypothesis that the WCFA chemical processing gave rise to minerals mainly composed of calcite, which, in turn, improved the physical properties of the aggregate and reinforced its structure.

3.2.2 X-ray Diffraction (XRD) Analysis: Evolution of Mineral Phases After Treatment

X-ray diffraction analysis showed the mineral concepts presented in the WCFA sample before and after treatment [30] (Fig. 7). The untreated sample, identified as 0C0SB0H, was predominantly made up of quartz (SiO_2) and mullite ($3\text{Al}_2\text{O}_3 \cdot 2\text{SiO}_2$) based on the results which were in agreement with earlier studies [26,57]. The presence of calcite (CaCO_3) that wasn't detected in the initial sample at all, was manifested in the samples 2C8SB12H and 4C4SB24H, indicating the occurrence of a chemical interaction between the cement and sodium bicarbonate in the treated samples. The calcite peaks which were additional and distinct appeared in the case of sample 2C2SB48H, whereas the calcite peaks of the sample 4C6SB72H were a lot in number and showed the highest peaks among the others thus indicating that the calcite phase had a higher rate of the formation due to the long period of immersion. The important function of immersion duration in calcium carbonate deposition has been revealed through these observations since it extends the interaction period between the cement and NaHCO_3 .

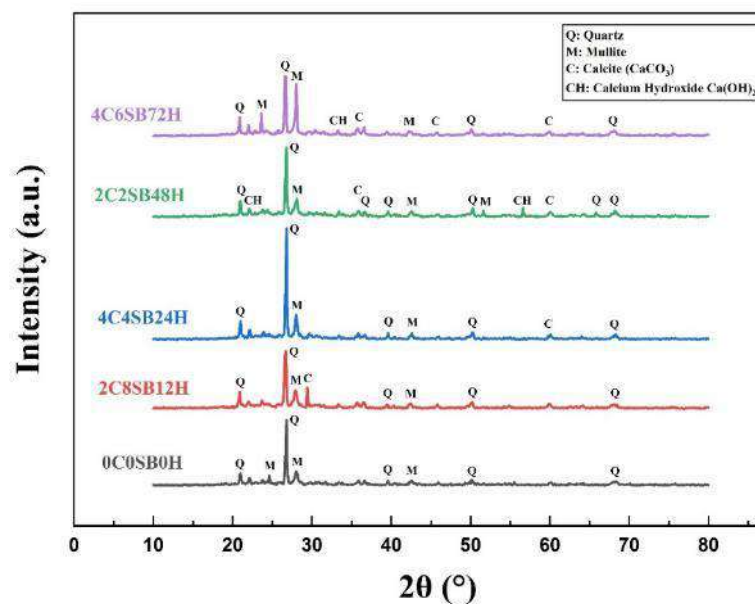


Fig. 7. X-ray diffraction analysis of mineral phase development in WCFA after chemical treatment

After treatment, calcium hydroxide (Ca(OH)_2) was also detected with a higher concentration for sample 2C2SB48H and sample 4C6SB72H. This shows that the initial product of cement hydration, Ca(OH)_2 , is slowly transformed to calcium carbonate when sodium bicarbonate is present, according to Eq. (3), thus rendering calcium hydroxide an important intermediate phase in the crystallization of calcite, the ultimate product.

The quartz and mullite peak densities in all samples did not significantly change, indicating that their chemical properties were not affected by the treatment they went through. The slow evolution of calcite, promoted by the extension of time under water, reveals that the application of a mixture of cement and sodium bicarbonate results in the perfect habitat for the deposition of new carbonaceous materials, especially calcium carbonate. The observed physical improvements are due to the structural changes, which resulted in the filling of the internal voids by crystalline calcite, wherein apparent porosity and water absorption were significantly reduced, and thus, the material's density and structural strength were enhanced.

3.2.3 Morphological Analysis (SEM – Scanning Electron Microscopy) and Compositional Analysis (EDS – Energy Dispersive Spectroscopy)

The scanning electron microscopy (SEM) photographs also compare the surface morphology, while offering an insider view on the effects of chemical treatments on WCFA. The images present a substantial alteration in the porous structure and the amount of calcium carbonate (CaCO_3) that was precipitated influenced by the cement concentration, the sodium bicarbonate concentration, and the immersion duration. The porosity of the control sample (0C0SB0H) is quite high and the pores are quite large and open, which indicates the structure of the untreated material at the beginning (Fig. 8).

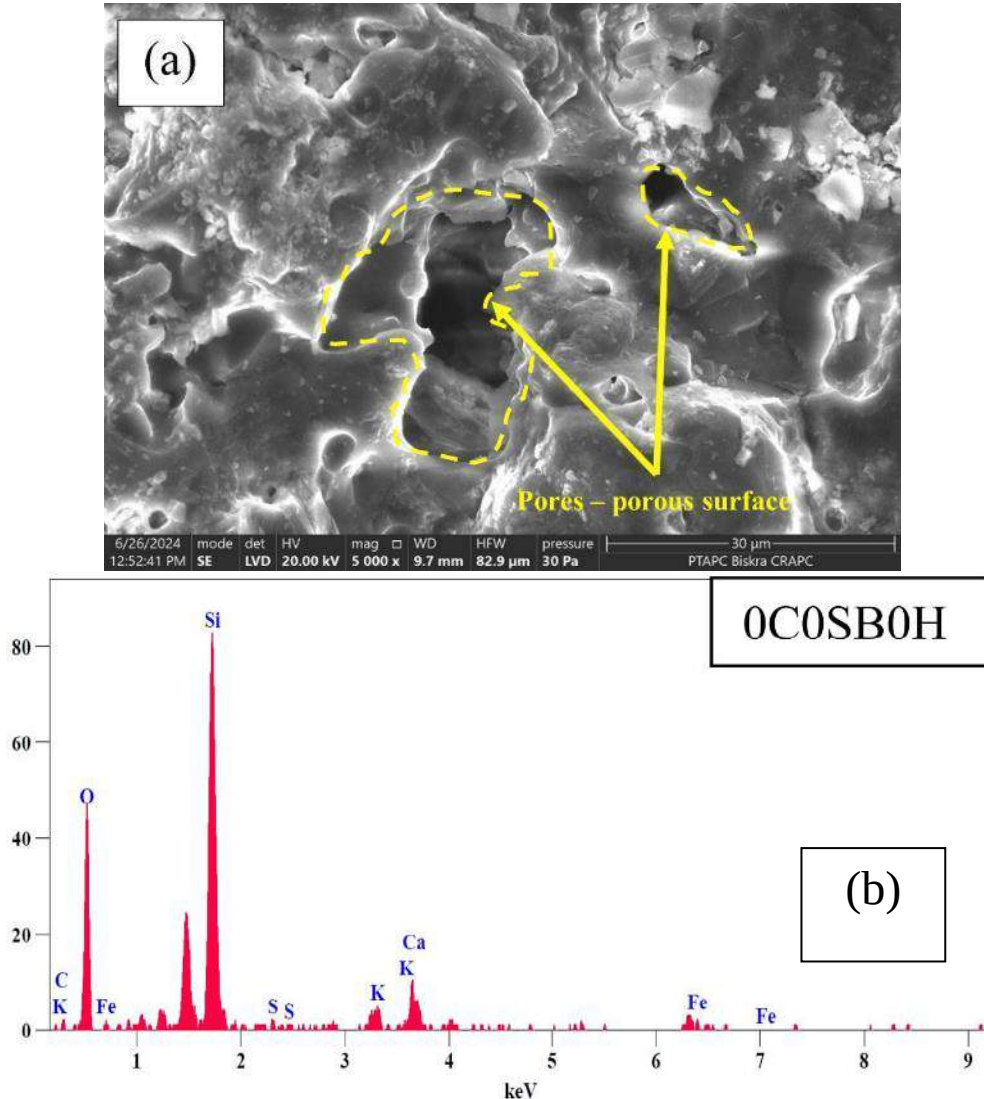


Fig. 8. Morphological analysis (a) and Compositional analysis (b) of sample 0C-0SB-0H using SEM-EDS

The analysis through energy dispersive spectroscopy (EDS) determined the composition of the samples. The control sample (0C0SB0H) did not exhibit any calcium carbonate presence and had a low calcium level (7.80%) and a negligible carbon level (4.47%) which verified the very low amounts of carbonaceous phases in the base material. On the other hand, sample 2C8SB12H experienced a dramatic reduction in open porosity, along with massive deposition of CaCO_3 on the grain surfaces and in the pores (Fig. 9). EDS analysis reveals a high calcium content of 9.71% (by weight) and a carbon content of 1.38 (Fig. 9), confirming the initial formation of calcium carbonate (CaCO_3) on the surface of the grains and in the pores. Samples 2C2SB48H and 4C6SB72H show intensified CaCO_3 precipitation (Fig. 10 and Fig. 11). This structural change is marked by more pore filling and densification of the aggregate surface. However, excessive packing of deposits on

4C6SB72H can be counterproductive, compromising aggregate integrity and limiting their homogeneous integration into cementitious matrices.

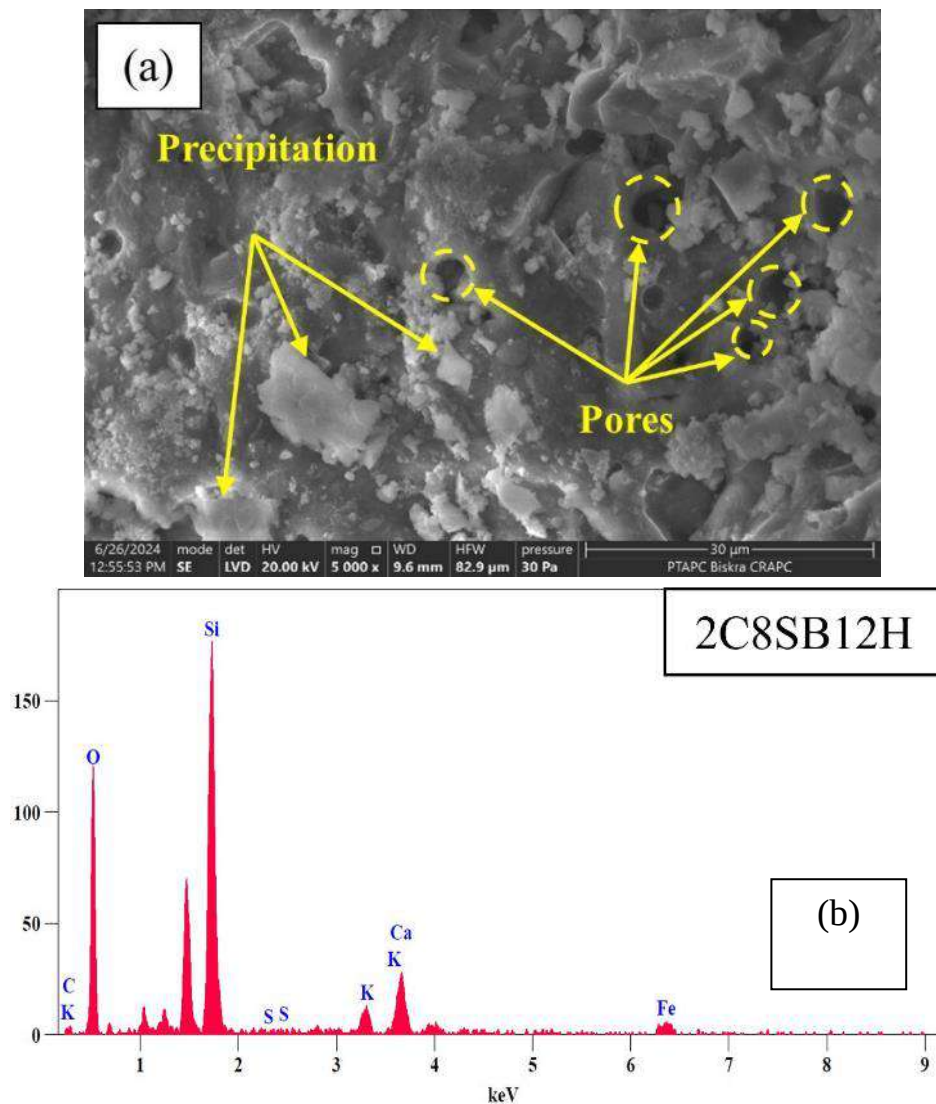


Fig. 9. Morphological (a) and elemental (b) analysis of sample 2C8SB12H by SEM -EDS

Morphological analysis (a) and Compositional analysis (b) of sample 4C6SB72H using SEM -EDS. For samples 2C2SB48H and 4C6SB72H (Fig. 10 and Fig. 11), EDS analyses reveal a significant increase in calcium (10.49% and 20.89%, respectively) and in carbon (5.31% and 20.62%).

The most promising sample, of all, is 4C4SB24H. It features a densified surface, a significantly reduced porosity and a homogeneous precipitation distribution, as revealed by the SEM observations in Fig. 12. The results of the scanning electron microscope (SEM) morphology together with the previous studies of Grabiec et al. [54], Phan et al. [73] and García-González et al. [69], corroborate the increase in density, decrease in porosity and water absorption. They also supported the Fourier-transform infrared spectroscopy (FTIR) and X-ray diffraction (XRD) methods by uncovering the mineral phase's existence and the treatment's efficacy. For the sample 4C4SB24H (Fig. 12), the EDS analysis supports the compositional change with a consistent level of calcium (12.82%) and carbon (16.58%), thus indicating the even production of CaCO_3 without the extra buildup.

Energy dispersive spectroscopy (EDS) analysis results agree with the morphological observations made by scanning electron microscopy (SEM) and confirm the changes in the structure of the treated aggregate that have taken place. The calcium to carbon ratio has significantly increased which means that calcium carbonate (CaCO_3) is being precipitated very slowly. This statement is

supported by the substantial reduction in surface porosity and pore density which has been measured with the help of microscopic images.

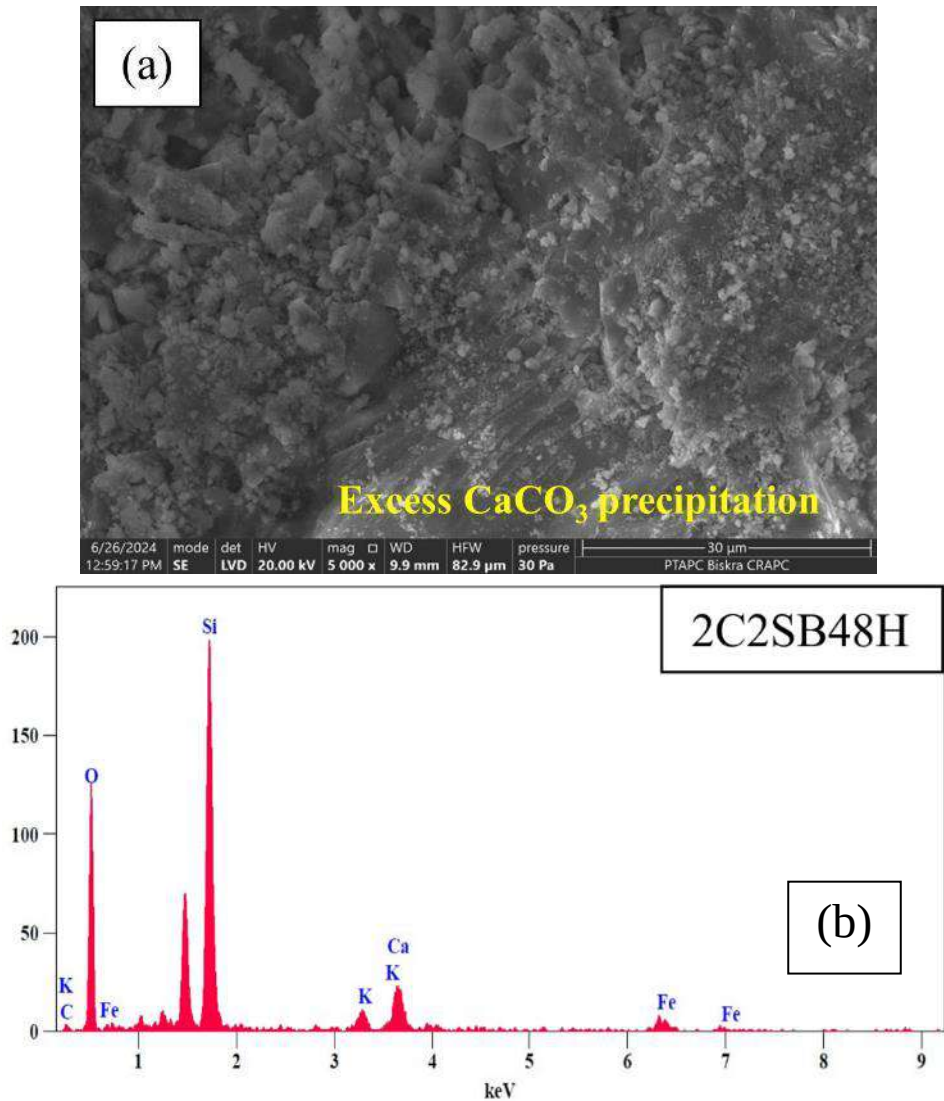
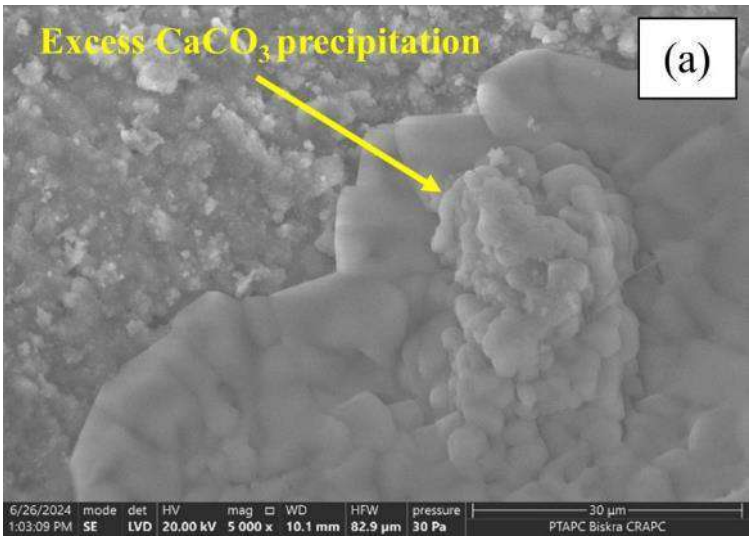


Fig. 10. Morphological analysis (a) and Compositional analysis (b) of sample 2C2SB48H using SEM -EDS.



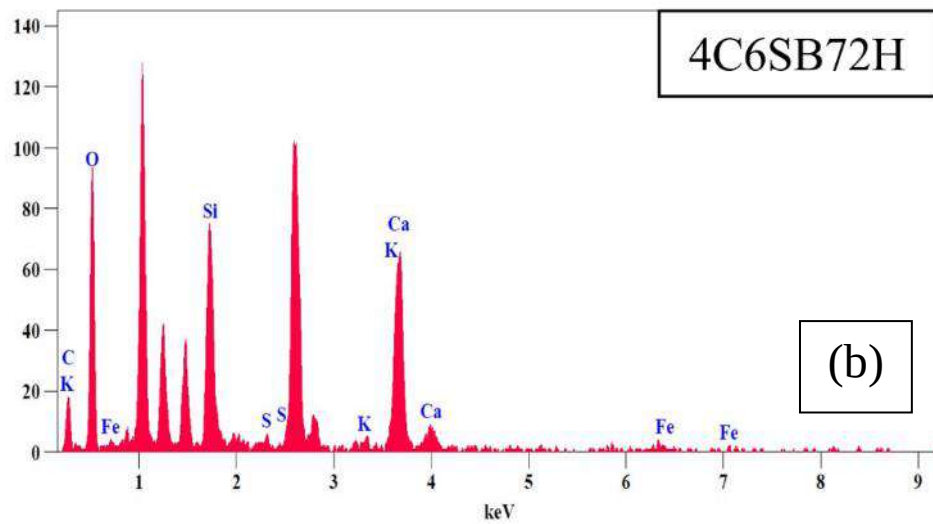


Fig. 11. Morphological analysis (a) and Compositional analysis (b) of sample 4C6SB72H using SEM -EDS

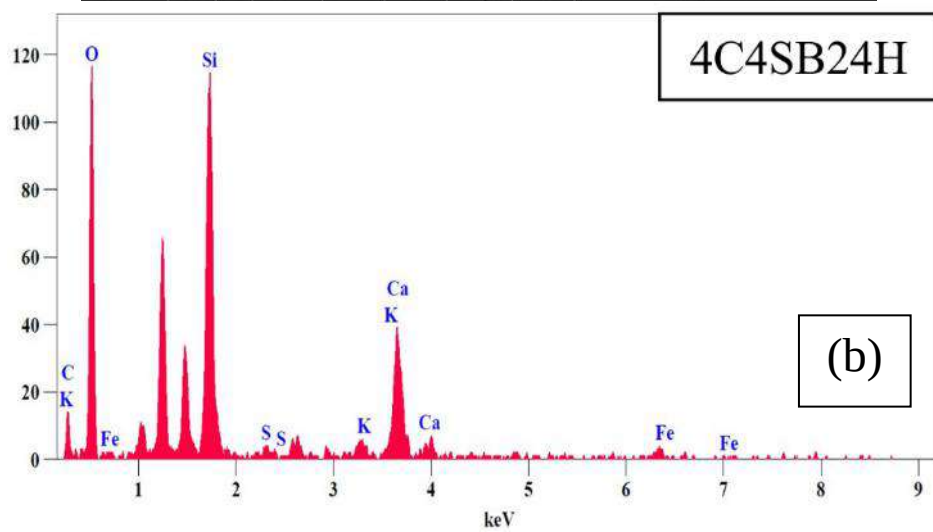
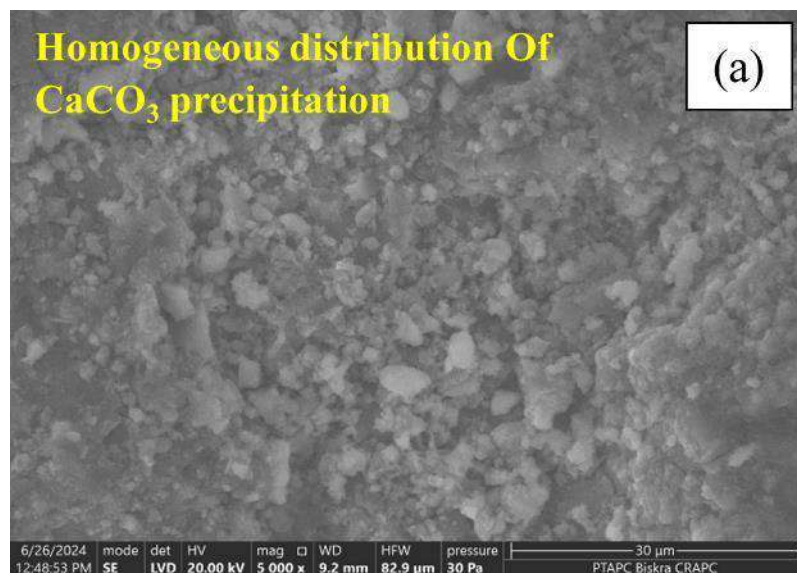


Fig. 12. Morphological analysis (a) and Compositional analysis (b) of sample 4C4SB24H using SEM -EDS

3.3 Statistical Analysis of RSM Results

Quadratic Regression analysis of data from Table 1 was performed using Design-Expert software. Relationships between response variables Y and influencing factors X1 (% C), X2 (% SB), and X3 (% H) are described by second-order regression equations [78,80,82], presented in Table 4, Table 5, and Table 6.

3.3.1 ANOVA for Quadratic Model

Table 4. Response 1 – Bulk Density

Source	Sum of Squares	df	Mean Square	F-value	p-value	
Model	0.0240	8	0.0030	52.53	< 0.0001	significant
A-Cement	0.0001	1	0.0001	2.15	0.1560	
B-Sodium Bicarbonate	0.0021	1	0.0021	36.06	< 0.0001	
C-Immersion Duration	0.0003	1	0.0003	4.47	0.0450	
AB	1.347×10^{-7}	1	1.347×10^{-7}	0.0024	0.9616	
AC	0.0011	1	0.0011	18.66	0.0002	
BC	0.0001	1	0.0001	1.41	0.2468	
B ²	0.0004	1	0.0004	7.48	0.0115	
C ²	0.0072	1	0.0072	125.54	< 0.0001	
Residual	0.0014	24	0.0001			
Cor Total	0.0253	32				
Fit Statistics						
Std. Dev.	0.0076		R ²		0.9460	
Mean	1.19		Adjusted R ²		0.9280	
C.V. %	0.6352		Predicted R ²		0.88646	
			Adeq Precision		31.5087	

$$\begin{aligned}
 \rho_b = & 1.09676 + 0.010118(A) + 0.012763(B) + 0.003542(C) \\
 & + 0.000025(A \times B) - 0.000231(A \times C) - 0.000029(B \times C) \\
 & - 0.000844(B^2) - 0.000039(C^2)
 \end{aligned} \quad (10)$$

where, P_b = bulk density (g/cm³), A = cement concentration (%), B = sodium bicarbonate concentration (%), C= immersion duration (hours).

Table 5. Response 2 – Absorption by Immersion

Source	Sum of Squares	df	Mean Square	F-value	p-value	
Model	6.94	8	0.8672	24.13	< 0.0001	significant
A-Cement	0.6032	1	0.6032	16.78	0.0004	
B-Sodium Bicarbonate	1.08	1	1.08	30.01	< 0.0001	
C-Immersion Duration	0.3647	1	0.3647	10.15	0.0040	
AB	0.0184	1	0.0184	0.5132	0.4807	
AC	0.2667	1	0.2667	7.42	0.0118	
BC	0.6112	1	0.6112	17.00	0.0004	
B ²	0.9026	1	0.9026	25.11	< 0.0001	
C ²	0.7119	1	0.7119	19.80	0.0002	
Residual	0.8627	24	0.0359			
Cor Total	7.80	32				

Fit Statistics			
Std. Dev.	0.1896	R ²	0.8894
Mean	4.13	Adjusted R ²	0.8525
C.V. %	4.60	Predicted R ²	0.7849
		Adeq Precision	24.1689

$$W_a = 5.95084 - 0.228902(A) - 0.498902(B) + 0.003528(C) - 0.009216(A \times B) + 0.003651(A \times C) + 0.002567(B \times C) + 0.038795(B^2) - 0.000386(C^2) \quad (11)$$

where, W_a = absorption by immersion (%), A = cement concentration (%), B = sodium bicarbonate concentration (%), C = immersion duration (hours).

Table 6. Response 3 – Apparent porosity

Source	Sum of Squares	df	Mean Square	F-value	p-value	
Model	50.68	8	6.33	35.54	< 0.0001	significant
A-Cement	0.1184	1	0.1184	0.6643	0.4231	
B-Sodium Bicarbonate	3.13	1	3.13	17.59	0.0003	
C-Immersion Duration	15.28	1	15.28	85.74	< 0.0001	
AB	0.1207	1	0.1207	0.6772	0.4186	
AC	0.0007	1	0.0007	0.0036	0.9523	
BC	2.87	1	2.87	16.12	0.0005	
B ²	5.69	1	5.69	31.93	< 0.0001	
C ²	6.84	1	6.84	38.37	< 0.0001	
Residual	4.28	24	0.1782			
Cor Total	54.95	32				

Fit Statistics			
Std. Dev.	0.4222	R ²	0.9222
Mean	9.00	Adjusted R ²	0.8962
C.V. %	4.69	Predicted R ²	0.8591
		Adeq Precision	30.1690

$$\varepsilon = 14.18613 + 0.028295(A) - 1.12784(B) - 0.151927(C) - 0.023575(A \times B) + 0.000180(A \times C) + 0.005566(B \times C) + 0.097417(B^2) + 0.001196(C^2) \quad (12)$$

where ε = apparent porosity (%), A = cement concentration (%), B = sodium bicarbonate concentration (%), C = immersion duration (hours).

3.3.2 ANOVA and Interaction Analysis

Analysis of variance shows R^2 values for bulk density, water absorption, and apparent porosity of 0.9460, 0.8894, and 0.9222 respectively, reflecting a strong correlation between predicted and actual values. This confirms that the regression model has a good fit with different tests on the data [79,81,87]. Moreover, the corresponding adjusted R^2 values were found to be 0.9280, 0.8525, and 0.8962 for p_b , W_a , and ε apparent porosity in that order. Thus, the quadratic model clarifies the variance in the response variables up to 92.80%, 85.25%, and 89.62% respectively, with only 7.20%, 14.75%, and 10.38% of the total variance remaining unexplained. Fig. 13 shows the correlation between the predicted and actual values, which closely align with a straight line, indicating a strong fit between the experimental and predicted data. Fig. 14 shows the normal distribution of residuals. An alignment of points along a straight line indicates that the errors follow a normal distribution and that no variance deviation is observed for all responses. In Fig. 15, the residuals are plotted against the predicted values. The random dispersion of the points indicates

that the assumptions of independence of errors and constant variance are satisfied for all response variables.

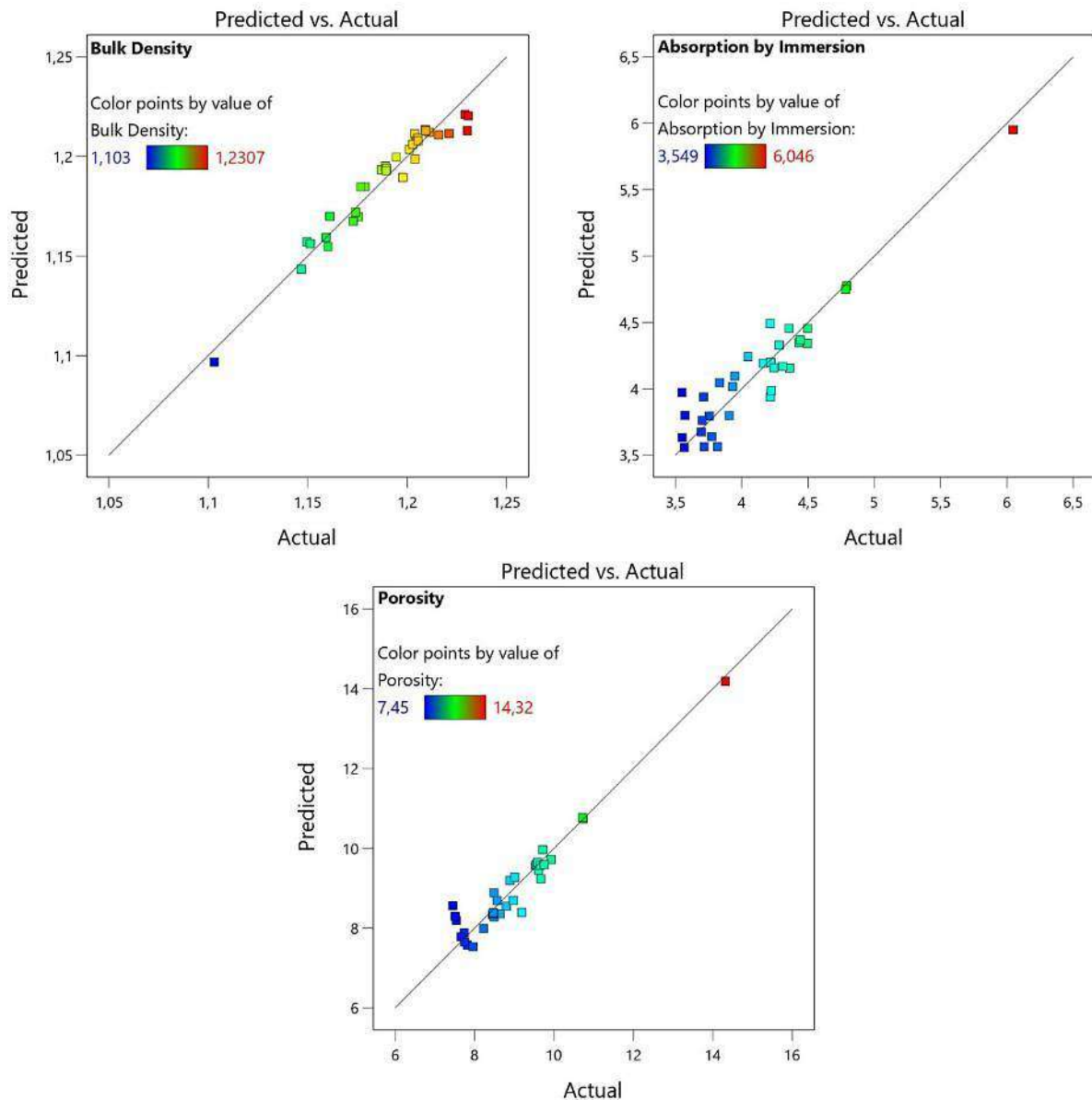


Fig. 13. Relationship between predicted and actual values for WCFA

All the above results confirm that the presented regression equations are adapted to the prediction of bulk density, water absorption and apparent porosity. In addition, the model works effectively to optimize the preparation parameters of WCFA treated with C and SB solution.

The data shown in Table 4, Table 5, and Table 6 for the analysis of variance (ANOVA) confirm the correctness as well as the dependability of the regression models that were developed for the purpose of predicting the bulk density, water absorption, and apparent porosity of WCFA. The bulk density model produced an F value of 52.53 which means a very high significance level ($p < 0.01\%$). The factors B, C, AC, B^2 , and C^2 are the most influential ones on the response. In the same way, the water absorption model was also very significantly validated through the statistical test ($F = 24.13$; $p < 0.01$), where the coefficients A, B, C, AC, BC, B^2 , and C^2 had the most pronounced impacts. The apparent porosity model at the same time indicated a very high statistical significance as well ($F = 35.54$; $p < 0.01$), with the factors being B, C, BC, B^2 , and C^2 .

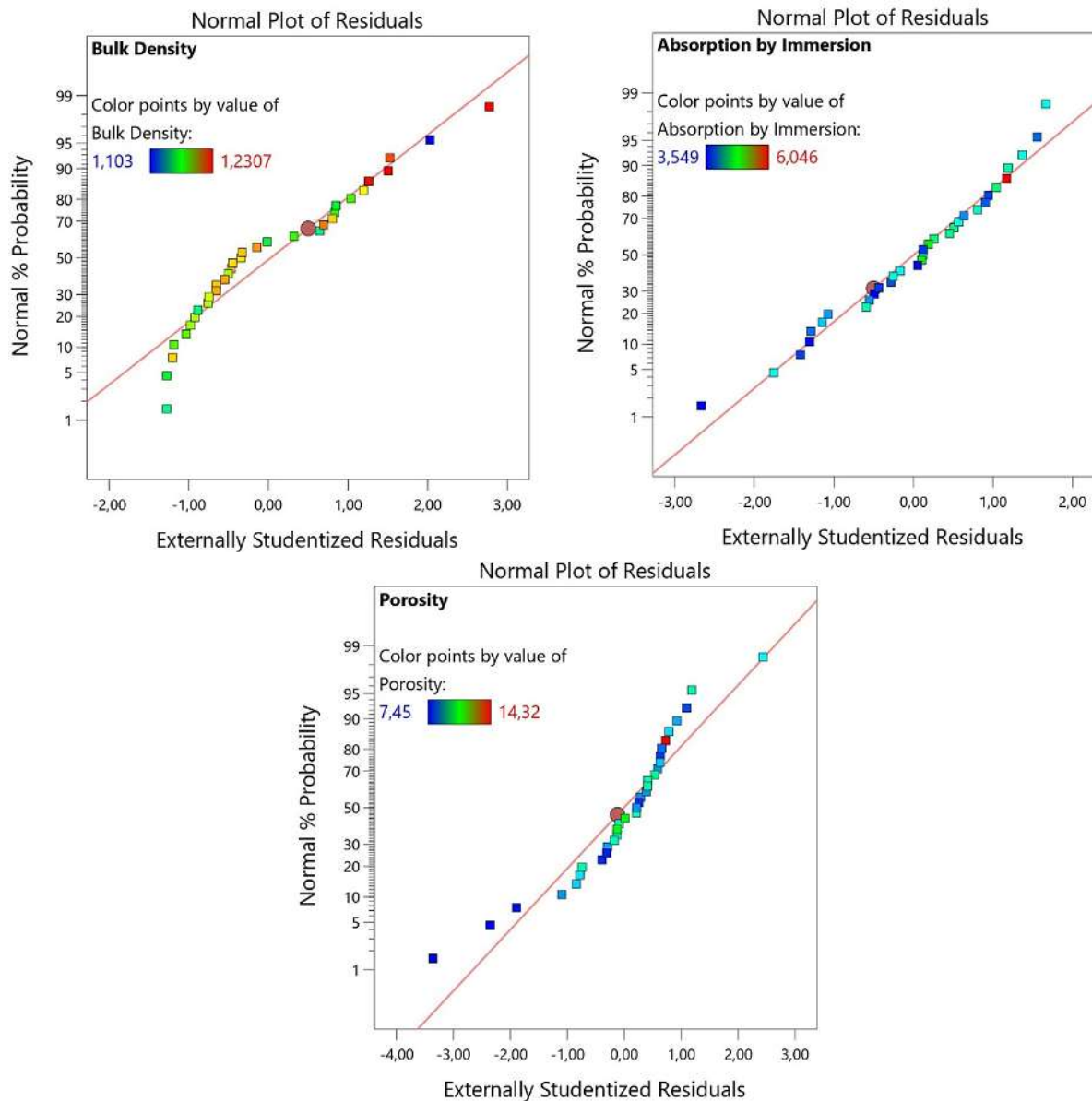


Fig. 14. Normal probability plots of residuals for WCFA

The proposed models are characterized by good predictive accuracy as evidenced by the predicted R^2 values of 0.8646 for bulk density, 0.7849 for water absorption, and 0.8591 for apparent porosity. So, the empirical Eqs. (10)-(11)-(12) can represent the link between treatment parameters and physical properties, hence allowing WCFA treatment to be optimized systematically and efficiently for its application in sustainable construction to the fullest.

The conformity of the second-order polynomial regression models developed for bulk density, water absorption, and apparent porosity to variance analysis (ANOVA) was demonstrated, with excellent statistical performance displayed by all the models. The predicted R^2 and adjusted R^2 of each model are closely interconnected as their greatest disparity does not go beyond 0.2 (0.8646 vs. 0.9280; 0.7849 vs. 0.8525; 0.8591 vs. 0.8962) which is a strong confirmation of the models' validity and high predictive power without any overfitting.

The values of adequate precision, which serve as an indication of the signal-to-noise ratio, were significantly higher than the recommended minimum (4) [88,89], and were found to be 31.509, 24.169, and 30.169 for bulk density, water absorption, and apparent porosity, respectively. This refers to a very strong and reliable signal while allowing the model fully and faithfully to explore the entire space of the experiment.

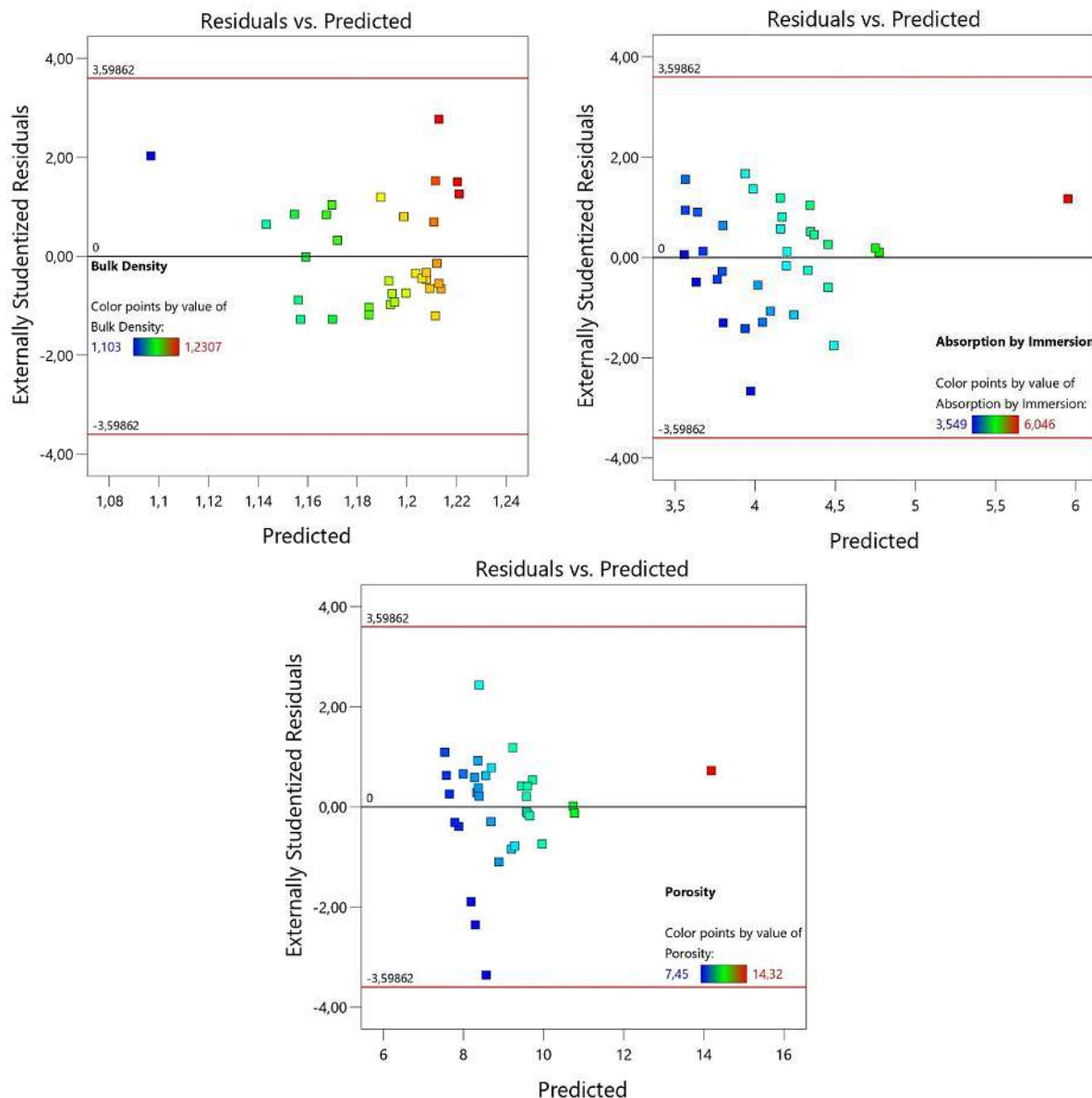


Fig. 15. Residuals vs. Predicted values for bulk density, water absorption, and apparent porosity of WCFA

Consequently, the studied response variables and the best conditions for WCFA treatment were predicted accurately by second-degree polynomial equations which were derived from the Box-Behnken experimental design. The models' accuracy and statistical validity are presented through Fig. 13, Fig. 14, and Fig. 15 which all exhibit a random distribution of residuals devoid of any systematic pattern, thus confirming the models' accuracy and statistical validity. These results highlight the models' ability to accurately unravel the complex interrelation between process variables and, on the other hand, to allow the structured optimization of the WCFA performance in sustainable construction applications.

The surfaces of responses along with the curves of contours (Fig. 16 and Fig. 17) clearly show how the factors of C, SB, and H interrelate and the physical properties of the treated aggregates that result. The experiments conducted evidently indicate that, through prolonged immersion, substantial and consistent enhancement of performance is achieved by the simultaneous raising of both cement and sodium bicarbonate concentrations, these factors result in increased bulk density, reduced water absorption, and decreased apparent porosity.

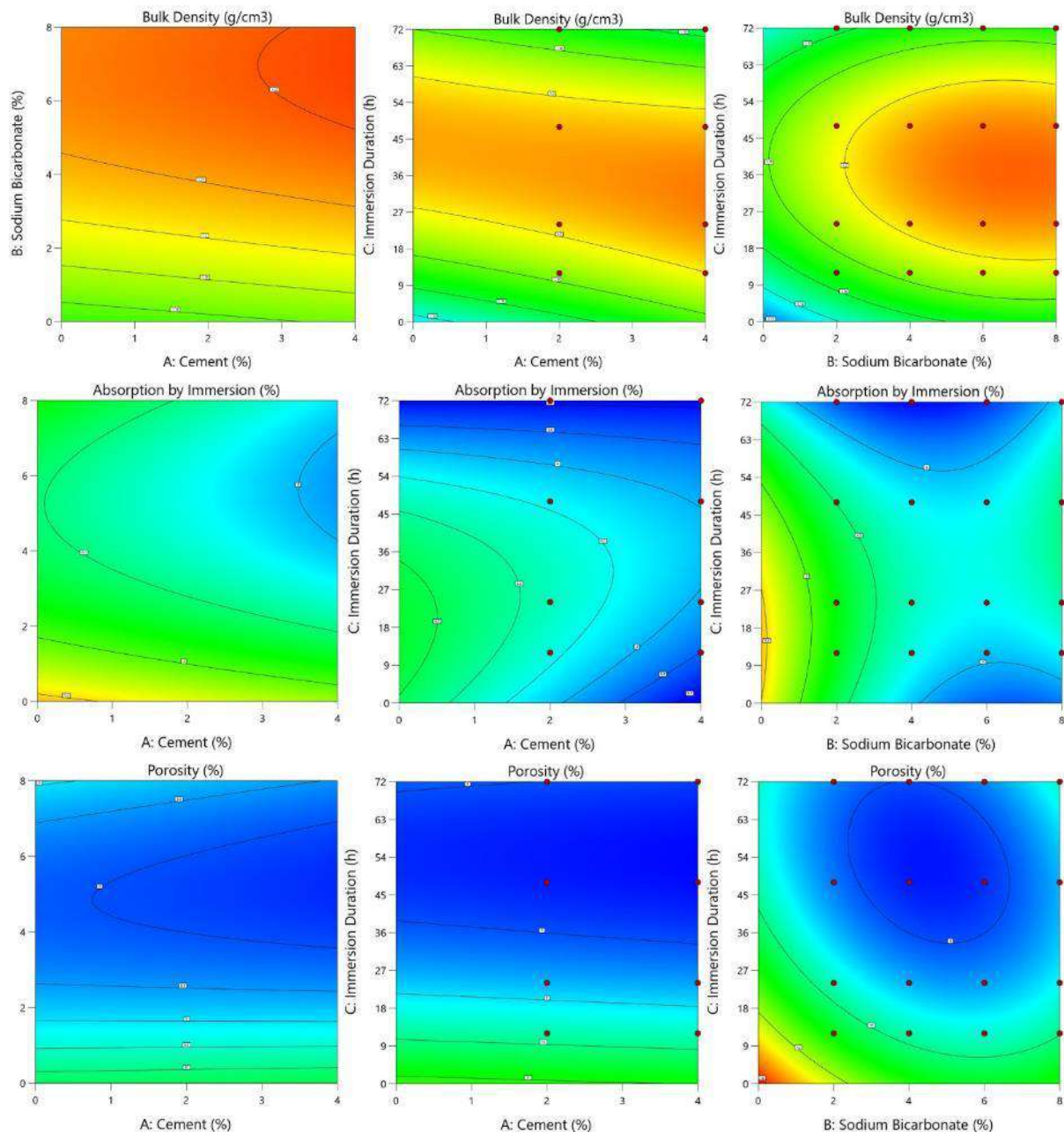


Fig. 16. 2D contour curves representing bulk density, water absorption and apparent porosity as a function of (C/SB), (C/ID), and (SB/ID) interactions for treated WCFA

After analyzing the influence of experimental parameters on bulk density, water absorption and apparent porosity of WCFA treated with cement (C) and sodium bicarbonate (SB), the ranges of these variables and the optimization approach (goal and importance) leading to the optimum values are presented in Table 7. The models were optimized using the desirability function of Design-Expert software. This functionality enables determining of the best input and output parameters (target responses) at the same time according to the given input conditions, as depicted in Fig. 18. The model forecasts optimal values like 1.213 g/cm³ for bulk density, 3.972% for water absorption, and 8.564% for apparent porosity. The mentioned results correlate to the optimal experimental conditions of 4% cement, 4% sodium bicarbonate and 24 hours of immersion, which are presented in Table 8.

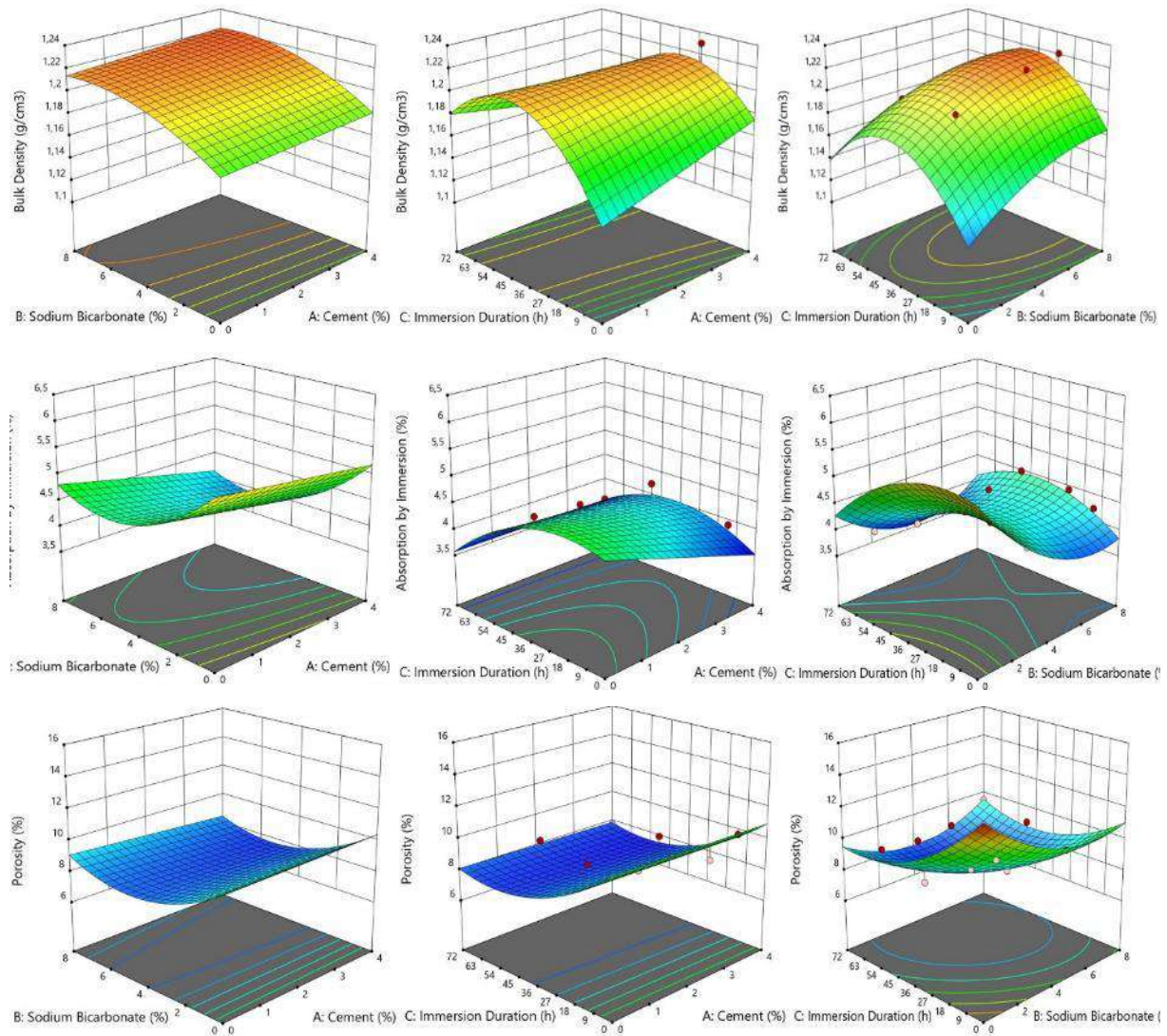


Fig. 17. 3D response graphs RSM models for bulk density, water absorption, and apparent porosity as a function of (C/SB), (C/ID), and (SB/ID) interactions of treated WCFA

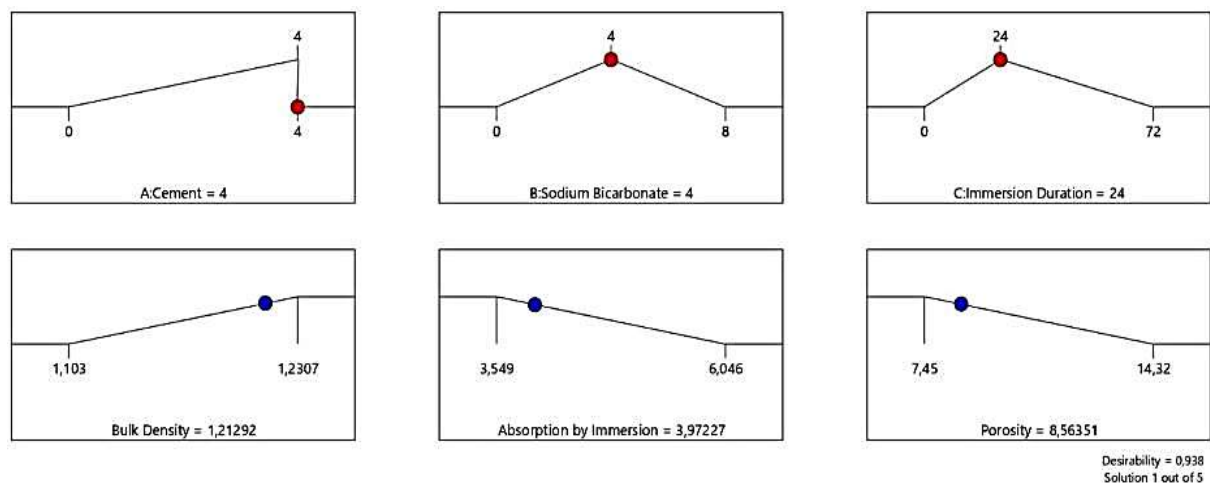


Fig. 18. Desirability ramps for optimum values for ρ_b , W_a , and ϵ of processed WCFA

Table 7. Treated WCFA stress factors and their responses

Name	Goal	Lower Limit	Upper Limit	Importance
A: Cement (%)	is target = 4	0	4	5
B: Sodium Bicarbonate (%)	is target = 4	0	8	5
C: Immersion Duration (hours)	is target = 24	0	72	5
Bulk Density (g/cm ³)	maximize	1.103	1.2307	3
Absorption by Immersion (%)	minimize	3.549	6.046	3
Apparent porosity (%)	minimize	7.45	14.32	3

Table 8. The five best solutions for WCFA treatment parameters and their corresponding responses

Number	Cement (%)	Sodium Bicarbonate (%)	Immersion Duration (hours)	Bulk Density (g/cm ³)	Absorption by Immersion (%)	Apparent porosity (%)	Coefficient of Desirability	
1	4.000	4.000	24.000	1.213	3.972	8.564	0.938	Selected
2	4.000	4.000	24.288	1.213	3.975	8.543	0.937	
3	4.000	4.000	24.580	1.213	3.978	8.522	0.936	
4	4.000	4.000	25.181	1.214	3.983	8.481	0.935	
5	4.000	4.000	21.388	1.211	3.944	8.758	0.911	

4. Conclusion

This study explored the impact of using a suspension of cement and sodium bicarbonate on the properties of recycled fine aggregate from ceramic waste (WCFA). The main results are as follows:

- Treatment of WCFA with the cement and sodium bicarbonate combination improved bulk density, apparent porosity and water absorption by immersion of all treated samples.
- The highest performance was shown by the sample treated with a solution containing 4% cement, 4% NaHCO₃, and 24 hours of immersion, which showed an 11.5% increase in density, a 41.3% decrease in water absorption, and a 47.9% reduction in apparent porosity compared to the untreated sample.
- FTIR and XRD techniques provided the evidence for the presence of calcium carbonate (CaCO₃) in the treated aggregates, which helped to lessen the porosity.
- The results of SEM imaging also supported the fact that there was a considerable improvement in the structure of WCFA due to chemical treatment, which was marked by the pores being filled with calcium carbonate (CaCO₃) precipitates.
- It has been observed that increasing the concentration of cement and sodium bicarbonate, as well as extending the immersion duration, generally improves the physical properties of fine ceramic aggregates (WCFA). But excessive immersion duration causes over-precipitation of calcium carbonate (CaCO₃), which may impair the internal cohesion of the aggregates with no significant physical property improvement.
- The output of the statistical models through analysis of variance (ANOVA) revealed the physical properties of the treated ceramic aggregates to be statistically significant at a very high level, with an error rate lower than 7%.
- It was found that the best physical performance could be achieved in treating fine ceramic particles with 4% cement and sodium bicarbonate each for 24 h of immersion. Using these optimal ratios, the modified model predicted a density of 1.213 g/cm³, a water absorption by immersion of 3.972%, and an apparent porosity of 8.564%. These predictive results were confirmed by experimental results, with the aim of attesting to the reliability of the optimized model developed by RSM.

In order to carry on with this research, we propose the addition of the treated WCFA at its best conditions (C: 4%, SB:4%, and ID: 24 hours) into a cement mix with the sand of dunes. Their impact on the mechanical properties, durability, and microstructure of ecological mortars will be studied for the purpose of discovering the best and the most sustainable formulation.

Nomenclature

WCFA	Waste Ceramic Fine Aggregates.	C ₃ S	tricalcium silicate.
C	Cement content (%).	H ₂ O	Water.
SB	Sodium bicarbonate (%).	CSH	Calcium silicate hydrate
ID	Immersion Duration (h).	Na ₂ CO ₃	Sodium carbonate.
XRD	X-ray diffraction.	Cu	Coefficient of Uniformity.
FTIR	Fourier Transform Infrared Spectroscopy.	Cc	Coefficient of Curvature.
SEM:	Scanning Electron Microscopy.	ρ _a	Bulk Density (g/cm ³).
EDS	Energy Dispersive Spectroscopy.	ρ _s	Absolute Density (g/cm ³).
BBD	Box-Behnken Design	ε	Apparent Porosity (%).
RSM	Response Surface Model	Wa	water absorption by immersion (%).
CDW	Construction and Demolition Waste	SSD	Saturated Surface Dry.
RCA	Recycled Concrete Aggregates.	ASTM	American Society for Testing and Materials
HCl	Hydrochloric acid	EN	European standard
C ₂ H ₄ O ₂	Acetic acid.	ANOVA	Analysis of Variance.
CaCO ₃	Calcium carbonate.	R ²	Coefficient of determination.
NaHCO ₃	Sodium bicarbonate.	F	F-value in ANOVA.
C ₂ S	dicalcium silicate.	P	P-value in ANOVA.
Ca(OH) ₂	Calcium hydroxide.		

(i=0,2,4)C(j=0,2,4)SB(k=0,12,24,48,75)H

Sample code used to denote treatment conditions

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References

- [1] Alanyali H, Çöl M, Yilmaz M, Karagöz Ş. Concrete produced by steel-making slag (basic oxygen furnace) addition in portland cement. *Int J Appl Ceram Technol* 2009;6:736–48. <https://doi.org/10.1111/j.1744-7402.2008.02317.x>.
- [2] Hebhoub H, Aoun H, Belachia M, Houari H, Ghorbel E. Use of waste marble aggregates in concrete. *Constr Build Mater* 2011;25:1167–71. <https://doi.org/10.1016/j.conbuildmat.2010.09.037>.
- [3] Comisión Permanente del Hormigón. Code on Structural Concrete (EHE-08). Madrid: Ministerio de Fomento; 2010.
- [4] British Standards Institution. Concrete – Complementary British Standard to BS EN 206. London: BSI; 2016.
- [5] Works Bureau Technical Circular No. 12/2002. Hong Kong: Works Bureau; 2002.
- [6] DIN 4226-100. Aggregates for mortar and concrete – Part 100: Recycled aggregates. Berlin: Deutsches Institut für Normung; 2002.
- [7] NEN-EN 12620+A1. Aggregates for concrete. Delft: Nederlands Normalisatie-instituut; 2008.
- [8] Li L, Liu Z, Liu W, Wang Z, Zhou C, Zeng Q. Microstructure and transport properties of cement mortar made with recycled fine ceramic aggregates. *Dev Built Environ* 2025;22:100643. <https://doi.org/10.1016/j.dibe.2025.100643>.
- [9] Silva J, De Brito J, Veiga R. Partial replacement of cement with fine ceramics in rendering mortars. *Mater Struct* 2008;41:1333–44. <https://doi.org/10.1617/s11527-007-9332-z>.

- [10] Frías M, Rodríguez O, Vegas I, Vigil R. Properties of calcined clay waste and its influence on blended cement behavior. *J Am Ceram Soc* 2008;91:1226–30. <https://doi.org/10.1111/j.1551-2916.2008.02289.x>.
- [11] Sánchez De Rojas MI, Marín FP, Frías M, Rivera J. Properties and performances of concrete tiles containing waste fired clay materials. *J Am Ceram Soc* 2007;90:3559–65. <https://doi.org/10.1111/j.1551-2916.2007.01944.x>.
- [12] Sánchez De Rojas MI, Marín F, Rivera J, Frías M. Morphology and properties in blended cements with ceramic wastes as a pozzolanic material. *J Am Ceram Soc* 2006;89:3701–5. <https://doi.org/10.1111/j.1551-2916.2006.01279.x>.
- [13] Sánchez De Rojas MI, Frías M, Rivera J, Escorihuela MJ, Marín FP. Research on the pozzolanic activity of waste materials from calcined clay. *Mater Constr* 2001;51:45–52. <https://doi.org/10.3989/mc.2001.v51.i261.379>.
- [14] Puertas F, García-Díaz I, Barba A, Gazulla MF, Palacios M, Gómez MP, et al. Ceramic wastes as alternative raw materials for Portland cement clinker production. *Cem Concr Compos* 2008;30:798–805. <https://doi.org/10.1016/j.cemconcomp.2008.06.003>.
- [15] Binici H. Effect of crushed ceramic and basaltic pumice as fine aggregates on concrete mortars properties. *Constr Build Mater* 2007;21:1191–7. <https://doi.org/10.1016/j.conbuildmat.2006.06.002>.
- [16] Pereira J, Brito J, Correia JR. Effects on concrete durability of using recycled ceramic aggregates. *Mater Struct* 2005;38:157–64. <https://doi.org/10.1617/s11527-005-9014-7>.
- [17] Brito J, Pereira AS, Correia JR. Mechanical behaviour of non-structural concrete made with recycled ceramic aggregates. *Cem Concr Compos* 2005;27:429–33. <https://doi.org/10.1016/j.cemconcomp.2004.07.005>.
- [18] Debieb F, Kenai S. The use of coarse and fine crushed bricks as aggregate in concrete. *Constr Build Mater* 2008;22:886–93. <https://doi.org/10.1016/j.conbuildmat.2006.12.013>.
- [19] De Brito J. Structural concrete with incorporation of coarse recycled concrete and ceramic aggregates: Durability performance. *Mater Struct* 2009;42:663–75. <https://doi.org/10.1617/s11527-008-9411-9>.
- [20] Guerra I, Vivar I, Llamas B, Juan A, Moran J. Eco-efficient concretes: The effects of using recycled ceramic material from sanitary installations on the mechanical properties of concrete. *Waste Manag* 2009;29:643–6. <https://doi.org/10.1016/j.wasman.2008.06.018>.
- [21] Juan-Valdés A, Llamas B, Morán J. Re-use of construction and demolition residues and industrial wastes for the elaboration of recycled eco-efficient concretes. *Span J Agric Res* 2010;8:34–44. <https://doi.org/10.5424/sjar/2010081-1140>.
- [22] Koyuncu H, Guney Y, Yilmaz G, Koyuncu S, Bakis R. Utilization of ceramic wastes in the construction sector. *Key Eng Mater* 2004;264–268:2509–12. <https://doi.org/10.4028/www.scientific.net/KEM.264-268.2509>.
- [23] Torkittikul P, Chaipanich A. Utilization of ceramic waste as fine aggregate within Portland cement and fly ash concretes. *Cem Concr Compos* 2010;32:440–9. <https://doi.org/10.1016/j.cemconcomp.2010.02.004>.
- [24] Andreola F, Barbieri L, Lancellotti I, Leonelli C. Recycling of industrial wastes in ceramic manufacturing: State of the art and glass case studies. *Ceram Int* 2016;42:13333–8. <https://doi.org/10.1016/j.ceramint.2016.05.205>.
- [25] Siddesha H. Experimental studies on the effect of ceramic fine aggregate on the strength properties of concrete. *Int J Eng Sci Technol* 2011;3:71–6.
- [26] Tabak Y, Kara M, Günay E, Yildirim ST, Yilmaz Ş. Ceramic tile waste as a waste management solution for concrete. In: *Proceedings of the 3rd International Conference on Industrial and Hazardous Waste Management*; September 2012; Chania, Greece. 2012. p. 1–8.
- [27] Coelho A, De Brito J. Economic viability analysis of a construction and demolition waste recycling plant in Portugal – Part I: Location, materials, technology and economic analysis. *J Clean Prod* 2013;39:338–52. <https://doi.org/10.1016/j.jclepro.2012.08.024>.
- [28] Khatib JM. Properties of concrete incorporating fine recycled aggregate. *Cem Concr Res* 2005;35:763–9. <https://doi.org/10.1016/j.cemconres.2004.06.017>.
- [29] Djedid T, Mani M, Ouakouak A, Guettala A. Effect of varying silica-limestone sand fines on the physical-mechanical performance of concrete. *Frattura ed Integrità Strutturale* 2022;16:580–91. <https://doi.org/10.3221/IGF-ESIS.59.38>.
- [30] Djedid T, Mani M, Guettala A, Hima A. Analysis of workability, mechanical strength and durability by the FT-IR method of concrete based on silica-limestone sand preserved in aggressive environments. *Frattura ed Integrità Strutturale* 2021;15:566–79. <https://doi.org/10.3221/IGF-ESIS.59.37>.
- [31] Agrela F, Sánchez De Juan M, Ayuso J, Geraldés VL, Jiménez JR. Limiting properties in the characterisation of mixed recycled aggregates for use in the manufacture of concrete. *Constr Build Mater* 2011;25:3950–5. <https://doi.org/10.1016/j.conbuildmat.2011.04.027>.

- [32] Evangelista L, De Brito J. Durability performance of concrete made with fine recycled concrete aggregates. *Cem Concr Compos* 2010;32:9–14. <https://doi.org/10.1016/j.cemconcomp.2009.09.005>.
- [33] Medina C, Sánchez De Rojas MI, Frías M. Freeze-thaw durability of recycled concrete containing ceramic aggregate. *J Clean Prod* 2013;40:151–60. <https://doi.org/10.1016/j.jclepro.2012.08.042>.
- [34] Richardson A, Coventry K, Bacon J. Freeze/thaw durability of concrete with recycled demolition aggregate compared to virgin aggregate concrete. *J Clean Prod* 2011;19:272–7. <https://doi.org/10.1016/j.jclepro.2010.09.014>.
- [35] Rodrigues F, Evangelista L, De Brito J. Physical, chemical and mineralogical characterization of fine aggregates from construction and demolition waste recycling plants. *J Clean Prod* 2013;52:438–45. <https://doi.org/10.1016/j.jclepro.2013.02.023>.
- [36] De Brito J. Influence of the pre-saturation of recycled coarse concrete aggregates on concrete properties. *Mag Concr Res* 2011;63:617–27. <https://doi.org/10.1680/mac.2011.63.8.617>.
- [37] Wattanasiriwech D, Saiton A, Wattanasiriwech S. Paving blocks from ceramic tile production waste. *J Clean Prod* 2010;17:1663–8. <https://doi.org/10.1016/j.jclepro.2009.08.008>.
- [38] Kou SC, Poon CS, Agrela F. Comparisons of natural and recycled aggregate concretes prepared with the addition of different mineral admixtures. *Cem Concr Compos* 2011;33:788–95. <https://doi.org/10.1016/j.cemconcomp.2011.05.009>.
- [39] Halicka A, Ogrodnik P, Zegardlo B. Using ceramic sanitary ware waste as concrete aggregate. *Constr Build Mater* 2013;48:295–305. <https://doi.org/10.1016/j.conbuildmat.2013.06.063>.
- [40] Medina C, Sánchez De Rojas MI, Frías M. Reuse of sanitary ceramic wastes as coarse aggregate in eco-efficient concretes. *Cem Concr Compos* 2012;34:48–54. <https://doi.org/10.1016/j.cemconcomp.2011.08.015>.
- [41] Profile SEE. The use of waste ceramic tile in cement production. *Cem Concr Res* 2000;30:0–3. [https://doi.org/10.1016/S0008-8846\(00\)00202-7](https://doi.org/10.1016/S0008-8846(00)00202-7).
- [42] Anderson DJ, Smith ST, Au FTK. Mechanical properties of concrete utilising waste ceramic as coarse aggregate. *Constr Build Mater* 2016;117:20–8. <https://doi.org/10.1016/j.conbuildmat.2016.04.153>.
- [43] Alves AV, Vieira TF, De Brito J, Correia JR. Mechanical properties of structural concrete with fine recycled ceramic aggregates. *Constr Build Mater* 2014;64:103–13. <https://doi.org/10.1016/j.conbuildmat.2014.04.037>.
- [44] Medina C, Frías M, Sánchez De Rojas MI, Thomas C, Polanco JA. Gas permeability in concrete containing recycled ceramic sanitary ware aggregate. *Constr Build Mater* 2012;37:597–605. <https://doi.org/10.1016/j.conbuildmat.2012.08.023>.
- [45] Pereira-de-Oliveira LA, Castro-Gomes JP, Santos PMS. The potential pozzolanic activity of glass and red-clay ceramic waste as cement mortars components. *Constr Build Mater* 2012;31:197–203. <https://doi.org/10.1016/j.conbuildmat.2011.12.110>.
- [46] O'Farrell M, Sabir BB, Wild S. Strength and chemical resistance of mortars containing brick manufacturing clays subjected to different treatments. *Cem Concr Compos* 2006;28:790–9. <https://doi.org/10.1016/j.cemconcomp.2006.05.014>.
- [47] Silva J, De Brito J, Veiga R. Incorporation of fine ceramics in mortars. *Constr Build Mater* 2009;23:556–64. <https://doi.org/10.1016/j.conbuildmat.2007.10.014>.
- [48] Barbudo A, De Brito J, Evangelista L, Bravo M, Agrela F. Influence of water-reducing admixtures on the mechanical performance of recycled concrete. *J Clean Prod* 2013;59:93–8. <https://doi.org/10.1016/j.jclepro.2013.06.022>.
- [49] Matias D, De Brito J, Rosa A, Pedro D. Mechanical properties of concrete produced with recycled coarse aggregates – Influence of the use of superplasticizers. *Constr Build Mater* 2013;44:101–9. <https://doi.org/10.1016/j.conbuildmat.2013.03.011>.
- [50] Brown T, Taylor AH. Performance of concrete made with commercially produced coarse recycled concrete aggregate. *Cem Concr Res* 2001;31:0–0.
- [51] Juan-Valdés A, Pozo J, Guerra-Romero MI. Pre-saturation technique of recycled aggregates: solution to the water absorption drawback in recycled concrete manufacture. *Materials (Basel)* 2014;7:6224–42. <https://doi.org/10.3390/ma7096224>.
- [52] Ismail S, Ramli M. Mechanical strength and drying shrinkage properties of concrete containing treated coarse recycled concrete aggregates. *Constr Build Mater* 2014;68:726–39. <https://doi.org/10.1016/j.conbuildmat.2014.06.058>.
- [53] Spaeth V, Tegguer AD. Improvement of recycled concrete aggregate properties by polymer treatments. *Int J Sustain Built Environ* 2014;2:143–52. <https://doi.org/10.1016/j.ijbsbe.2014.03.003>.
- [54] Grabiec AM, Klama J, Zawal D, Krupa D. Modification of recycled concrete aggregate by calcium carbonate biodeposition. *Constr Build Mater* 2012;34:145–50. <https://doi.org/10.1016/j.conbuildmat.2012.02.027>.

- [55] Mukherjee A, Basu PC, Reddy MS. Lactose mother liquor as an alternative nutrient source for microbial concrete production by *Sporosarcina pasteurii*. J Ind Microbiol Biotechnol 2009;36:433–8. <https://doi.org/10.1007/s10295-008-0514-7>.
- [56] Chunxiang Q, Jianyun W, Ruixing W, Liang C. Corrosion protection of cement-based building materials by surface deposition of CaCO_3 by *Bacillus pasteurii*. Mater Sci Eng C 2009;29:1273–80. <https://doi.org/10.1016/j.msec.2008.10.025>.
- [57] Shah KW, Huseien GF. Recycled ceramics in sustainable concrete: properties and performance. 1st ed. Boca Raton (FL): CRC Press; 2021.
- [58] Yacine A, Ratiba M, Abderrahmane G. Ceramic waste influence on dune sand mortar performance. Constr Build Mater 2016;125:703–13. <https://doi.org/10.1016/j.conbuildmat.2016.08.083>.
- [59] Senthamarai RM, Manoharan PD. Concrete with ceramic waste aggregate. Cem Concr Compos 2005;27:910–3. <https://doi.org/10.1016/j.cemconcomp.2005.04.003>.
- [60] Wang J, Zhang R, Hou F, Ye G. Influence of pore property and alkalinity on the bio-deposition treatment efficiency of recycled aggregates. J Build Eng 2025;89:109381. <https://doi.org/10.1016/j.jobe.2024.109381>.
- [61] Letelier V, Hott F, Bustamante M, Wenzel B. Effect of recycled coarse aggregate treated with recycled binder paste coating and accelerated carbonation on mechanical and physical properties of concrete. J Build Eng 2024;82:108311. <https://doi.org/10.1016/j.jobe.2023.108311>.
- [62] Ray S, Haque M, Sakib MN, Mita AF, Rahman MDM. Use of ceramic wastes as aggregates in concrete production: a review. J Build Eng 2021;43:102567. <https://doi.org/10.1016/j.jobe.2021.102567>.
- [63] Higashiyama H, Yagishita F, Sano M, Takahashi O. Compressive strength and resistance to chloride penetration of mortars using ceramic waste as fine aggregate. Constr Build Mater 2012;26:96–101. <https://doi.org/10.1016/j.conbuildmat.2011.05.008>.
- [64] Manikandan KP, Nanthakumar P, Balachandar M, Shankar DG, Vijayakumari G. Partial replacement of aggregate with ceramic tile in concrete. Mater Today Proc 2023. <https://doi.org/10.1016/j.matpr.2023.05.214>.
- [65] Mistri A, Kumar S, Dhami N, Mukherjee A, Barai SV. A review on different treatment methods for enhancing the properties of recycled aggregates for sustainable construction materials. Constr Build Mater 2020;233:117894. <https://doi.org/10.1016/j.conbuildmat.2019.117894>.
- [66] Varshney H, Khan RA, Khan IK. Quality improvement techniques for recycled aggregate concrete incorporating fibers and *Bacillus* species bacteria: a review. Mater Today Proc 2024. <https://doi.org/10.1016/j.matpr.2023.03.223>.
- [67] Tavakoli D, Heidari A, Karimian M. Properties of concretes produced with waste ceramic aggregate. Asian J Civ Eng 2013;14:369–82.
- [68] Reusing ceramic waste as fine aggregate and supplemental cementitious material in the manufacture of sustainable concrete. Constr Build Mater 2023.
- [69] Phan QHN, Vo HT, Bui PT, Nguyen NT. Effect of Recycled Coarse Aggregate Treatment Using Cement – Sodium Bicarbonate Slurry on Compressive Strength of Hardened Concrete. Lecture Notes in Civil Engineering. 2022 Sep 21;449–57. https://doi.org/10.1007/978-981-19-3303-5_38
- [70] Andal MN, Thenmozhi R, Bahurudeen A. Strength enhancement of recycled fine aggregate beam through chemical treatment – A waste to wealth approach. J Mater Res Technol. 2024;30:7340–7351. <https://doi.org/10.1016/j.jmrt.2024.05.069>
- [71] Grabiec AM, Klama J, Zawal D, Krupa D. Modification of recycled concrete aggregate by calcium carbonate biodeposition. Construction and Building Materials. 2012;34:145–150. doi:10.1016/j.conbuildmat.2012.02.027
- [72] Hamada HM, Shi J, Abed F, Al Jawahery MS, Majdi A, Yousif ST. Recycling solid waste to produce eco-friendly ultra-high performance concrete: A review of durability, microstructure and environment characteristics. Sci Total Environ. 2023;876:162804
- [73] AFNOR. NF EN 933-1. Essais pour déterminer les caractéristiques géométriques des granulats – Partie 1. Paris: AFNOR; n.d.
- [74] ASTM C29/C29M-07. Standard test method for bulk density (unit weight) and voids in aggregate. West Conshohocken (PA): ASTM International; 2007. https://doi.org/10.1520/C0029_C0029M-07.
- [75] ASTM C128-22. Standard test method for relative density (specific gravity) and absorption of fine aggregate. West Conshohocken (PA): ASTM International; 2022.
- [76] ASTM C20-00(2022). Standard test methods for apparent porosity, water absorption, apparent specific gravity, and bulk density of burned refractory brick and shapes by boiling water. West Conshohocken (PA): ASTM International; 2022. <https://doi.org/10.1520/C0020-00R22>.
- [77] Tuan NH, Huynh LX, Anh PH. A fuzzy finite element algorithm based on response surface method for free vibration analysis of structure. Vietnam J Mech 2015;37:17–27. <https://doi.org/10.15625/0866-7136/37/1/3923>.

- [78] Sun Q, Zhu H, Li H, Gao M. Application of response surface methodology in the optimization of fly ash geopolymer concrete. *ISA Trans* 2018;48:45–52.
- [79] Arshad HM, Akhtar M, Gilmour SG. Augmented Box–Behnken designs for fitting third-order response surfaces. *Commun Stat Theory Methods* 2012;41:4225–39. <https://doi.org/10.1080/03610926.2011.568154>.
- [80] Zhang L, Yue Y. Influence of waste glass powder usage on the properties of alkali-activated slag mortars based on response surface methodology. *Constr Build Mater* 2018;181:527–34. <https://doi.org/10.1016/j.conbuildmat.2018.06.040>.
- [81] Myers RH, Montgomery DC, Anderson-Cook CM. Response surface methodology: process and product optimization using designed experiments. 3rd ed. Hoboken (NJ): John Wiley & Sons; 2009.
- [82] Senthamarai RM, Manoharan PD. Concrete with ceramic waste aggregate. *Cem Concr Compos* 2005;27:910–3. <https://doi.org/10.1016/j.cemconcomp.2005.04.003>.
- [83] Liu F, Liu J, Ma B, Huang J, Li H. Basic properties of concrete incorporating recycled ceramic aggregate and ultra-fine sand. *J Wuhan Univ Technol Mater Sci Ed* 2015;30:352–60. <https://doi.org/10.1007/s11595-015-1152-3>.
- [84] Brunello V, Corti C, Sansonetti A, Tedeschi C, Rampazzi L. Non-invasive FTIR study of mortar model samples: comparison among innovative and traditional techniques. *Eur Phys J Plus* 2019;134:1–17. <https://doi.org/10.1140/epjp/i2019-12667-1>.
- [85] Wolpert M, Hellwig P. Infrared spectra and molar absorption coefficients of the 20 alpha amino acids in aqueous solutions in the spectral range from 1800 to 500 cm⁻¹. *Spectrochim Acta A Mol Biomol Spectrosc* 2006;64:987–1001. <https://doi.org/10.1016/j.saa.2005.08.025>.
- [86] García-González J, Rodríguez-Robles D, Wang J, De Belie N, Morán-del Pozo JM, Guerra-Romero MI, et al. Quality improvement of mixed and ceramic recycled aggregates by biodeposition of calcium carbonate. *Constr Build Mater* 2017;154:1015–23. <https://doi.org/10.1016/j.conbuildmat.2017.08.039>.
- [87] Zerig T, Aidoud A, Belachia M, Djedid T, Abbas M. Combined sand eco-mortar reinforced with polyethylene terephthalate: behavior and optimization using RSM method. *Constr Build Mater* 2023;404:133160. <https://doi.org/10.1016/j.conbuildmat.2023.133160>.
- [88] Boumaaza M, Belaadi A, Alshahrani H, Bourchak M, Jawaid M. Response surface methodology optimization of palm rachis biochar content and temperature effects on predicting bio-mortar compressive strength, porosity and thermal conductivity. *J Nat Fibers* 2023;20. <https://doi.org/10.1080/15440478.2022.2162184>.
- [89] Zerig T, Belachia M, Aidoud A, Meftah N, Djedid T, Abbas M. Statistical analysis using the RSM approach of the physical behavior of green polymerized eco-mortar. *J Clean Prod* 2024;450:141858. <https://doi.org/10.1016/j.jclepro.2024.141858>.

Recycling plastic waste into bricks: A sustainable approach using HDPE, PET, RFA, and PP

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Plastic waste poses a significant environmental challenge due to its resistance to degradation. One promising solution is its utilization as a construction material, such as lightweight or floating bricks. This study investigates the feasibility of using High-Density Polyethylene (HDPE), Polyethylene Terephthalate (PET), and Polypropylene (PP) combined with Recycled Fine Aggregate (RFA) to produce sustainable bricks. A total of 39 specimens and 3 control samples (100% HDPE) were tested through three experimental series. The first series examined HDPE-RFA mixtures (100–70% HDPE; 0–30% RFA). Increasing RFA content significantly improved the compressive and shear strengths, reaching maximum values of 47.43 MPa and 0.700 MPa, respectively. The second series evaluated HDPE-PET blends (90:10 to 70:30). Although pure HDPE showed the highest compressive strength (30.43 MPa), the 80:20 ratio provided an optimal balance between performance and efficiency. The third series analyzed HDPE-PP mixtures, with the 90:10 ratio yielding the highest strength (30.13 MPa), while the 80:20 ratio demonstrated more stable overall performance. All specimens met lightweight brick criteria and exhibited low water absorption. Increased PET content improved moisture resistance, making HDPE-PET blends suitable for water-sensitive applications. Density values ranged from 0.088 to 0.101 g/cm³, with HDPE-RFA mixtures showing the highest density. Overall, the results indicate that incorporating PET or RFA enhances mechanical strength, while PP improves stability. These findings demonstrate the strong potential of HDPE-based composite bricks as sustainable and eco-friendly alternatives for construction and floating applications.

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1. Introduction

Plastic is a very dangerous substance that is incredibly difficult to degrade, making it a major concern worldwide. Plastic is widely used in our everyday lives for items like polythene bags, disposables, furniture, food packaging, and other accessories. Plastic varies greatly in kind and composition based on their chemical makeup. Separating plastic garbage is a significant challenge [1,2]. The UN said that over 11.2 billion tons of solid trash are collected globally, mostly from human activities [3,4]. Plastic is a major contributor to the world's solid waste, with its usage steadily rising each year, leading to a corresponding growth in plastic garbage [5]. It is important to highlight that

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over 300 million metric tons of plastic are generated yearly, with around 20% of this garbage not properly handled. Additionally, most plastic is discarded within a year of production [6,7]. According to data from the Indonesian Plastics Industry Association (INAPLAS) and the Central Statistics Agency (BPS), Indonesia produces 64 million tons of plastic garbage annually, with 3.2 million tons of it ending up in the sea [8]. Plastic waste is a significant global issue due to its non-biodegradable nature, taking hundreds of years to degrade and being long-lasting, as studies have shown that plastic can persist for up to 4,500 years without breaking down. So, the issue of plastic trash must be resolved and handled more sensibly. Trash recycling is a beneficial approach that helps safeguard the ecosystem from the negative impacts of trash. Issues with waste management are connected to overproduction [9,10].

Symbol	Plastic Type	Example
	PET polyethylene Terephthalate	 Bottles of water and soft drinks
	HDPE High-Density polyethylene	 Shampoo bottles and helmet
	PVC Polyvinyl Chloride	 Water pipes and plastic foil
	LDPE LOW-Density polyethylene	 Shopping bags and trash bags
	PP polypropylene	 Surgical tools, bumpers, lining and clothing
	PS Polystyrene	 cassettes and CD cases
	Others	 Acrylic polycarbonate, Polyactic fibers, nylon and fibre glass baby feeding bottles

Fig. 1. Type of plastic

Based on reports published by various sources and experts related to climate change and global warming, one of which is the United Nations (UN) UN-Habitat's New Urban Agenda, published in 2020, predicts that 90% of the largest cities in the world will be exposed to rising sea levels in 2050. Most towns and settlements in coastal areas will be affected by erosion and flooding, which can damage homes and infrastructure, so millions of residents must be displaced [11,12]. Plastic waste can be categorized into several types based on their properties and recyclability. PET (Polyethylene Terephthalate), commonly used in beverage bottles and food containers, is lightweight and highly recyclable [13]. HDPE (High-Density Polyethylene), found in items like milk jugs, detergent bottles, and toys, is strong, durable, and widely recycled [14]. PVC (Polyvinyl Chloride), used in pipes, window frames, and vinyl flooring, is resistant to chemicals but challenging to recycle [15]. LDPE (Low-Density Polyethylene), often used in plastic bags, cling wraps, and squeezable bottles, is moderately recyclable. PP (Polypropylene), present in food containers, bottle caps, and automotive parts, is tough, heat-resistant, and recyclable [16]. PS (Polystyrene), which includes Styrofoam cups, food trays, and packing peanuts, is lightweight but difficult to recycle [17]. Multilayer plastics, such as snack wrappers and Tetra Pak, are hard to recycle due to their mixed materials [18]. Microplastics, tiny particles from degraded plastics or synthetic fibers, are not recyclable and pose significant environmental risks [19]. Bioplastics, made from renewable resources like corn starch, are sometimes compostable under specific conditions. Lastly, mixed plastics (Code 7) include diverse materials like polycarbonate and acrylic, with varying recyclability depending on the specific type. Understanding these categories is crucial for effective waste management and recycling efforts [20].

This project paper focuses on utilizing PET and HDPE [21] to produce plastic bricks, as these types of plastic are widely available and commonly discarded as waste [22]. PET, often used in water bottles and food packaging, is lightweight and highly recyclable, making it suitable for reuse in construction materials. Similarly, HDPE [13,23], found in containers such as milk jugs and

detergent bottles, is durable, resistant to moisture and chemicals, and easily recyclable [24]. By targeting these abundant and recyclable plastic types, the research seeks to reduce environmental waste while providing an innovative solution for sustainable construction [25]. The Ministry of Environment and Forestry reported that of Indonesia's daily waste production of 189 thousand tons, plastic waste reached 15% (28.4 thousand tons per day) [26]. Of the waste products produced daily, only 10-15% of worldwide plastic waste is recycled, while 60-70% is disposed of in landfills. The remaining 15-30% are disposed carelessly of the environment, especially in water areas such as rivers, lakes, beaches, and oceans. This is exacerbated by the increasing number of packaging products circulating in the community [8]. Therefore, plastic recycling becomes very important and a solution that can be implemented [26].

Plastic waste, especially PET plastic, has become rampant as a dangerous global emergency challenge. PET plastic materials, such as drinking water bottles, food packaging, and other products, are often found in the environment [27,28]. The specialty of PET lies in its light, sturdy, flexible, and easy-to-color nature. However, despite its advantages, PET still dominates the waste piles, disturbing the environment. PET is also a type of plastic that is difficult to decompose, and the decomposition period of this type of plastic can take up to 1,000 years. Polyethylene Terephthalate (PET) is a type of food-grade plastic that makes it easy to recycle. But unknowingly, the more often PET is used, the lower its quality. This can affect the quality of the recycling results if you want to reuse it for food packaging [12,29,30].

A series of experimental tests were conducted to evaluate the physical and mechanical properties of the plastic brick specimens, including compressive strength, water absorption, and thermal conductivity. The results indicated that the plastic bricks exhibited significantly lower water absorption rates (ranging from 0 to 0.35%) compared to conventional clay bricks, thereby enhancing their durability. The compressive strength also varied considerably among samples, with the PET/sand composite achieving a maximum strength of 76.85 MPa—substantially exceeding that of traditional bricks. It has been reported that plastic bricks manufactured from recycled materials can effectively replace conventional bricks, thereby reducing dependence on natural resources [27]. The casting process involved melting the plastic waste and pouring it into moulds measuring 190 × 90 × 90 mm, in accordance with the IS 1077 standard. The physical properties assessed included soundness, hardness, efflorescence, and impact resistance. Compression tests on brick masonry models were conducted following the IS 1905 standard, using wall specimens with dimensions of 50×11×50 cm. The results demonstrated that plastic bricks possess advantageous characteristics—such as reduced weight, lower water absorption, and higher compressive strength—rendering them suitable for load-bearing structural applications [31,32].

Numerous studies have explored the incorporation of plastic waste as a binder, aggregate, or as a partial replacement for cement and sand in brick production [30]. Research has shown that the inclusion of recycled HDPE waste in construction materials enhances mechanical performance [31], improves compressive strength [32], and decreases water absorption [33]. Several investigations have further highlighted the potential of using various types of plastic waste, particularly HDPE, in sustainable brick manufacturing [34,35].

The proposed research addresses the problem of plastic waste accumulation by utilizing it in the production of floating house bricks. This study aims to establish a theoretical foundation for sustainable development by exploring innovative recycling techniques that convert Polyethylene Terephthalate (PET), Low-Density Polyethylene (LDPE), and High-Density Polyethylene (HDPE) waste into construction materials [7]. The problem-solving approach involves identifying the main challenges associated with using plastic waste for floating house applications—specifically, material quality, production cost, and environmental impact.

In parallel, investigations have examined the influence of temperature and humidity on salt crystallization in fired clay bricks exposed to saline environments. Research indicates that moisture facilitates the transport of salts into the pores of masonry materials, with the crystallization process being strongly dependent on ambient temperature and humidity. Experimental studies on uniaxial salt crystallization in burned clay bricks conducted at various temperatures (20 °C, 30 °C, and 40

°C) and humidity levels (70%, 80%, and 90%)—while keeping other variables constant—led to the derivation of two predictive equations describing the effects of temperature and humidity, respectively. The findings revealed that salt crystallization intensity increases with higher temperatures of both the brick samples and the sodium sulphate (Na_2SO_4) solution, but decreases with lower relative humidity [36].

To enhance the performance of recycled plastic bricks, additives can be incorporated to improve material properties, optimize the production process, and support recycling programs as a sustainable plastic waste source. Continuous evaluation and improvement of the plastic brick manufacturing process are essential to ensure that the final product meets both structural and environmental standards. Plastic waste, particularly HDPE and PET, poses a major environmental threat due to its persistence and particle size (below 50 μm), which adversely affects soil fertility and marine ecosystems. Therefore, effective and sustainable solutions are urgently required.

This research proposes the utilization of HDPE and PET waste to produce plastic bricks designed as modular components for floating structures. In addition to mitigating plastic pollution, the approach reduces construction costs by minimizing the use of natural materials such as cement and clay. The use of plastic bricks for floating houses offers an innovative and sustainable response to recurrent flooding and tidal inundation in several Indonesian regions—combining environmental responsibility with practicality and economic efficiency.

The proposed modular design employs interlocking plastic bricks, similar to LEGO-like systems, where individual bricks act as both structural and binding elements. HDPE- and PET-based plastic bricks were fabricated in customized shapes and bonded to form modular wall units. These were subsequently evaluated through diagonal shear testing and numerical simulations to assess structural performance. Data analysis was performed using OriginPro 2023 for both statistical and graphical evaluations. The software enabled the visualization and correlation of multiple parameters, including density, compressive strength, water absorption, and shear strength, to identify trends and assess the influence of varying plastic waste ratios on the overall performance of interlocking plastic bricks [37].

This study forms part of a long-term research initiative aimed at developing environmentally friendly and sustainable floating houses within the next five years. The novelty of this work lies in its focus on the performance and interconnectivity of plastic bricks for enhanced strength and structural efficiency in floating house applications—an area with limited prior research. The key contributions of this study include: (1) advancing plastic recycling applications in construction engineering, and (2) developing lightweight, high-strength, low-absorption plastic bricks suitable for floating structures. These bricks exhibit high compressive strength, reduced water absorption compared to conventional bricks, and low density, enabling buoyancy and structural adaptability for coastal and flood-prone regions.

2. Materials and Methods

2.1 Materials

2.1.1 Recycled Fine Aggregate (RFA)

Recycled Fine Aggregate (RFA) is a material obtained from the processing and crushing construction and demolition waste, such as concrete, bricks, tiles, and asphalt. It is an eco-friendly alternative to natural sand, reducing the demand for natural resources and promoting sustainable construction practices [33], [34]. Typically, finer than 4.75 mm, RFAs are well-suited for use as fine aggregates in construction. Due to the crushing process, they have angular particle shapes and rough surface textures [35]. RFA is shown in Figure 2. Recycled Fine Aggregate (RFA), which consists of crushed fine particles obtained from processed construction and demolition waste. To further understand the material behavior of RFA when mixed with HDPE in plastic bricks.



Fig. 2. Recycled Fine Aggregate (RFA) Waste

2.1.2 Waste Plastic HDPE and PET

Waste Plastic PET and HDPE are three of the most recycled plastics, offering significant potential for reuse in construction and other applications. Polyethylene Terephthalate (PET) is a lightweight, strong, and transparent thermoplastic widely used in beverage bottles, food containers, and packaging. Recycling PET and HDPE involves collecting, cleaning, shredding, and melting the waste plastic into pellets or flakes [36]. These recycled plastics can be used as additives or substitutes in construction materials to improve properties like durability and water resistance. Due to its high tensile strength and thermal stability, PET is often incorporated into lightweight composites or reinforcing materials and is commonly used in structural elements or as a filler material in SNI 7974:2013 [37]. PET (Polyethylene Terephthalate) waste is shown in Figure 3(a), and HDPE (High-Density Polyethylene) waste is shown in Figure 3(b).



(a)



(b)

Fig. 3. (a) PET Plastic Waste, (b) HDPE Plastic waste

PET plastic waste typically originates from post-consumer products such as beverage bottles, food containers, and packaging materials. The waste is usually processed through cleaning, shredding, and flake production, resulting in angular and glossy flakes with smooth surfaces. HDPE plastic waste is commonly sourced from items such as detergent bottles, milk jugs, plastic crates, and industrial containers. After collecting and cleaning, the waste is granulated into coarse chips or flakes with a dull and waxy appearance [38,39]. HDPE has a lower melting point (approximately 130°C).

2.1.3 Waste Plastic PP

PP (Polypropylene) plastic waste used in this study was obtained from food and detergent packaging waste, which had been shredded before use. PP (Polypropylene) plastic waste can be seen in Figure 4. Polypropylene (PP) plastic waste is widely sourced from household and industrial items such as food containers, automotive components, packaging films, and woven

bags. After collecting, PP waste is cleaned and shredded into flakes or granules, often exhibiting a semi-glossy surface with relatively smooth and rounded edges compared to HDPE.



Fig. 4. PP Plastic Waste

2.2 Material Testing

Performing tests on the materials utilized in manufacturing test items is a prerequisite for the procedure. Activities such as the following are included in the material testing that is carried out: Density measures the mass of each unit volume of an object. Normal bricks have a density of around 2200-2400 kg/m³. Measuring the density of bricks using the Archimedes method refers to the ASTM C134-95 standard (ASTM, 2013) and is calculated using Eq 1.

$$\rho_{pc} = \frac{mb - ms}{mb} \times 100 \quad (1)$$

where, ρ_{pc} = density (kg/cm³), ms = mass of dry sample (kg), mb = mass of wet sample (kg).

Evaluation of specific gravity and water absorption of fine aggregates in accordance with SNI 1970:2008 (BSN, 2008a) for fine aggregates. The definition of dry bulk specific gravity refers to the comparison of the condition of the aggregate dried in an oven. The surface dry bulk specific gravity (SSD) compares the aggregate's condition in a dry saturated state [41]. Meanwhile, apparent specific gravity refers to comparing the aggregate's condition when saturated with water. Calculations of specific gravity and water absorption of Sand and fine aggregate are in Eq (2) to Eq (5).

$$\text{Dry bulk density} = \frac{Wk}{Wb + SSD - Wt} \quad (2)$$

$$\text{Surface dry bulk density} = \frac{SSD}{Wb + SSD - Wt} \quad (3)$$

$$\text{Apparent specific gravity} = \frac{Wk}{Wb + Wk - Wt} \quad (4)$$

$$\text{Water absorption} = \frac{SSD - Wk}{Wk} \quad (5)$$

where, WK = oven dry weight (grams), Wb = weight of Erlenmeyer flask containing water (grams), Wt = weight of Erlenmeyer flask containing water-sand (grams), SSD = weight of surface dry fine aggregate.

Based on SNI ASTM C117:2012, mud is a lump or layer that covers the surface of the aggregate and passes through sieve No. 200. Mud and fine dust from rock breaking are particles measuring between 0.002 mm and 0.006 mm (2 to 6 microns). The mud content can be calculated using the following Eq (6).

$$\text{Mud Level} = \frac{W_1 - W_2}{W_1} \times 100\% \quad (6)$$

where, W1= initial oven dries aggregate weight (grams), W2= oven dries aggregate weight after washing (grams).

Based on SNI ASTM C136:2012, Fine aggregate gradation testing also functions to determine the Fine Grain Modulus (FGM) value. The MG value indicates the fineness or coarseness index of aggregate grains, which ranges from 1.5 to 3.8. The greater the MG value, the MG value can be calculated using Eq (7).

$$FGM = \frac{\sum A}{100} \quad (7)$$

where, A= cumulative retained percentage (%).

Compressive strength testing on bricks made using plastic waste can be done by referring to the concrete brick compressive strength testing method according to the SNI 1974:2011 standard. The compressive strength test can be calculated using the following Eq (8).

$$Compressive\ Strength = \frac{Maximum\ Load\ (N)}{Cross-sectional\ Area(mm^2)} \quad (8)$$

Measure the dimensions of the specimen to calculate its cross-sectional area. where = the highest load the material can withstand before failure, measured in Newtons (N), Cross-sectional Area (mm²) = surface area under compression. The diagonal shear strength test assesses the ability of plastic bricks and mortar to withstand maximum shear force. Based on ASTM E519/E519M-22 on Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages, there are several requirements for shear strength testing, including the size of the test specimen. In addition, this study also refers to SNI 03-4166-1996 on Shear Strength Testing Method for Red Brick Walls. The minimum size of the test object is 1.2 m x 1.2 m, with a permissible dimension measurement limit of 6 mm. However, this study used a test object size of 30 cm x 30. specimens with dimensions of 300 x 300 mm². The calculation formula for the shear strength of brick walls can be seen in Eq (9).

$$Shear\ Stress = \frac{0,707\ P}{A_n} \quad (9)$$

where, Ss= diagonal shear strength of brick (MPa), P =load maximum test (kg), A= cross-sectional area pressure (cm²). Where A_n is calculated using the formula in Eq (10).

$$A = \left(\frac{(b + h)}{2} \right) t n \quad (10)$$

where, B = specimen width, mm (in), h = specimen height, mm (in), t = total thickness of specimen, mm (in), n Percentage of gross area of solid units. If necessary, the shear strain is calculated using the formula in Eq (11).

$$\gamma = \left(\frac{(\Delta V + \Delta H)}{g} \right) \quad (11)$$

where, γ = shear strain, or (mm/mm), ΔV=vertical shortening (mm), ΔH =horizontal extension (mm), g =vertical measuring length (mm). The Stiffness modulus can be calculated as in Eq (12).

$$G = \frac{Ss}{\gamma} \quad (12)$$

where, G=stiffness modulus (MPa), Ss =shear stress on area, MPa, γ =shear strain, or (mm/mm).

3. Methodology

A control variable is required to be used to conduct this study [35], [42]. In addition, experimental research is carried out to test hypotheses that are prepared in a strict manner and are typically carried out for specific fields [43], [44], [45]. This study uses HDPE, PET, and PP plastic waste recycling techniques as the main basis, which are combined with secondary materials such as Recycled Fine Aggregate Waste (RFA). All waste was collected from urban waste around

Yogyakarta and laboratory waste. After collecting, all materials were washed to remove contamination from other particles, and RFA and GWP were first screened to ensure particle sizes were in the range of 4.75-0.075 mm. Solid plastic waste was shredded to a size of ± 30 mm and categorized based on the type of plastic. A series of physical tests was applied to the materials using ASTM standards (C128, C117, C136, and F1312). Since there is no official mix design for plastic bricks, the calculation of brick proportions is based on the weight ratio to 1 volume of the brick specimen. The control specimens in this study were 100% HDPE and 100% PET. There were 21 variations, including the control specimens that would be tested for the effect of each material, as stated by the experimental method. according to SNI 15-2094-2000 (BSN, 2000), For Plastic bricks dimensions 23 x 11 x 5 cm with a certain thickness tolerance. According to SNI, Compressive strength is one of the main parameters that determines the ability of bricks to withstand loads.

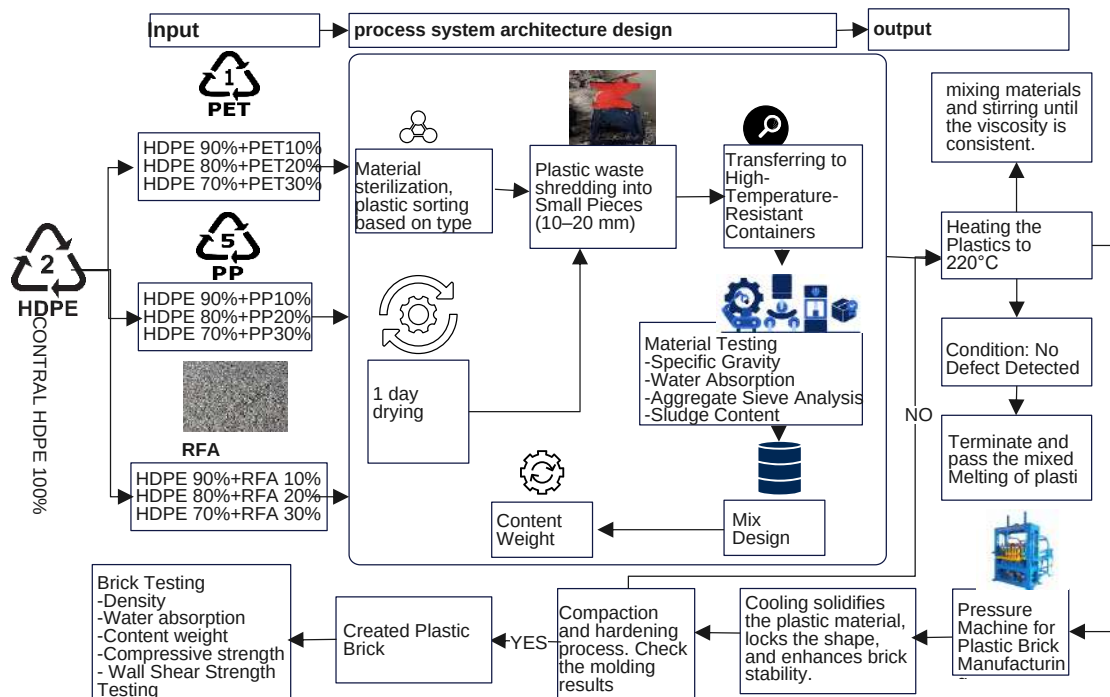


Fig. 5. Experimental methods

The test was conducted to analyse the specifications of the materials used. The test specimen was conducted as follows:

3.1 Density or Solidity Test

The first step to test the density of plastic bricks according to SNI 15-2094-2000 is to prepare measuring instruments and scales. Next, the dimensions of the test object are measured (length, width, and height). Then, the test object is weighed using a digital scale to find the dry mass. After that, the volume of the test object and density were calculated using the specified formula [46].

3.2 Water Absorption Test

The first step taken to test the water resistance of bricks, according to SNI 15-2094-2000, is to prepare the necessary tools and materials, such as scales and containers filled with water. Then, the test object is weighed in a dry state to obtain the dry weight. Furthermore, the test object is soaked for 24 hours in a container filled with water. After soaking, the brick is dried and weighed using a cloth on its surface. The data obtained is then processed with a predetermined formula. Based on SNI, the maximum water absorption in bricks must be 20% [47], [48].

3.3 Mix Design

The data generated from the density and water absorption tests are then used to plan the plastic brick mixture [49,50]. The control specimen used is the HDPE Control Brick, with a ratio of 100%. In addition, there are several types of mixed bricks consisting of HPE1, using a mixture of 90%

HDPE and 10% PET, HPE2 utilizing a mixture of 80% HDPE and 20% PET, and HPE3, using a mixture of 70% HDPE and 30% PET. The planned ratio is used to determine the plastic waste mixture's effect on the brick's characteristics and properties. HPP1 uses a mixture of 90% HDPE and 10% PP, HPP2 uses a mixture of 80% HDPE and 20% PP, and HPP3 uses a mixture of 70% HDPE and 30% PP. HR1 uses a mixture of 90% HDPE and 10% RFA, HR2 uses a mixture of 80% HDPE and 20% RFA, and HR3 uses a mixture of 70% HDPE and 30% RFA. The mix design can be seen in Table 3.

Table 1. Brick specimens mix ratio

Test Object Code	HDPE (%)	PET (%)	PP (%)	RFA (%)
HDPE Control	100	-	-	-
HPE1	90	10	-	-
HPE2	80	20	-	-
HPE3	70	30	-	-
HPP1	90	-	10	-
HPP2	80	-	20	-
HPP3	70	-	30	-
HR1	90	-	-	10
HR2	80	-	-	20
HR3	70	-	-	30

3.4 Preparation of Test Objects

The making of test objects begins with preparing the tools and materials. The shredded plastic waste is then put into a melting machine at a temperature of 250°-300° based on the planned ratio. After melting, the plastic is weighed at high temperature and moulded using a hydraulic press machine with a rectangular block mould measuring 23 cm x 11 cm x 5 cm for approximately 10 minutes so the plastic does not shrink [51]. Furthermore, the plastic bricks shaped like rectangular blocks are soaked in a container filled with water to lower the temperature. The shape of the mould can be seen in Figure 6.

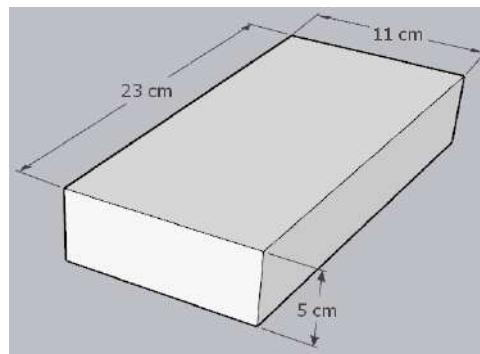


Fig. 6. Rectangular block-shaped mould

4. Results and Discussion

This study was performed at many Universitas Muhammadiyah Yogyakarta (UMY) Labs, namely the Lab Structure, and the UMY Computing Lab. The facilities provide the requisite equipment and expertise for experimental procedures and analysis. The research sought to assess the density, compressive strength, and water absorption characteristics of plastic bricks formulated with diverse compositions and supplementary materials, including plastic waste like HDPE (High-Density Polyethylene), PET (Polyethylene Terephthalate), PP, and recycled fine aggregate (RFA).

4.1 Gradation of Fine Aggregate

Fine aggregate gradation testing also functions to determine the Fine Grain Modulus (FGM) value. The MG value indicates the fineness or coarseness index of aggregate grains which range from 1.5

to 3.8. The greater the MG value, the coarser and larger the gradation, and if the FGM is smaller, the finer the grain gradation. The FGM value can be calculated using Equation (7).

Table 2. Gradation Analysis of Fine Aggregate

Filter Number	Size	Retained Weight (grams)	Retained Weight (%)	Cumulative Retained Weight (%)	Cumulative Pass Weight (%)
No. 4	4.8	16	1.6	1.6	98.4
No. 8	2.4	56	5.6	7.2	92.8
No. 16	1.2	151	15.1	22.3	77.7
No. 30	0.6	158	15.8	38.1	61.9
No. 50	0.3	279	27.9	66	34
No. 100	0.15	277	27.7	93.7	6.3
Pan		63	6.3	100	0
Total		1000	100		371.1
Cumulative Retained Weight (%)				228.9	
Fineness Modulus (FM)				FM= 228.9/100= 2.289	

The gradation study of the fine aggregate indicated a cumulative retained weight of 228.9%, yielding a fineness modulus (FM) of 2.289. As per ASTM C136, this number signifies that the fine aggregate utilized in this study is categorized as medium fine sand, offering appropriate workability and adhesion properties for plastic brick manufacturing. The gradation analysis of the fine aggregates (D1, D2, D3, and D4) was conducted in accordance with ASTM C136 to ascertain the particle size distribution within the samples.

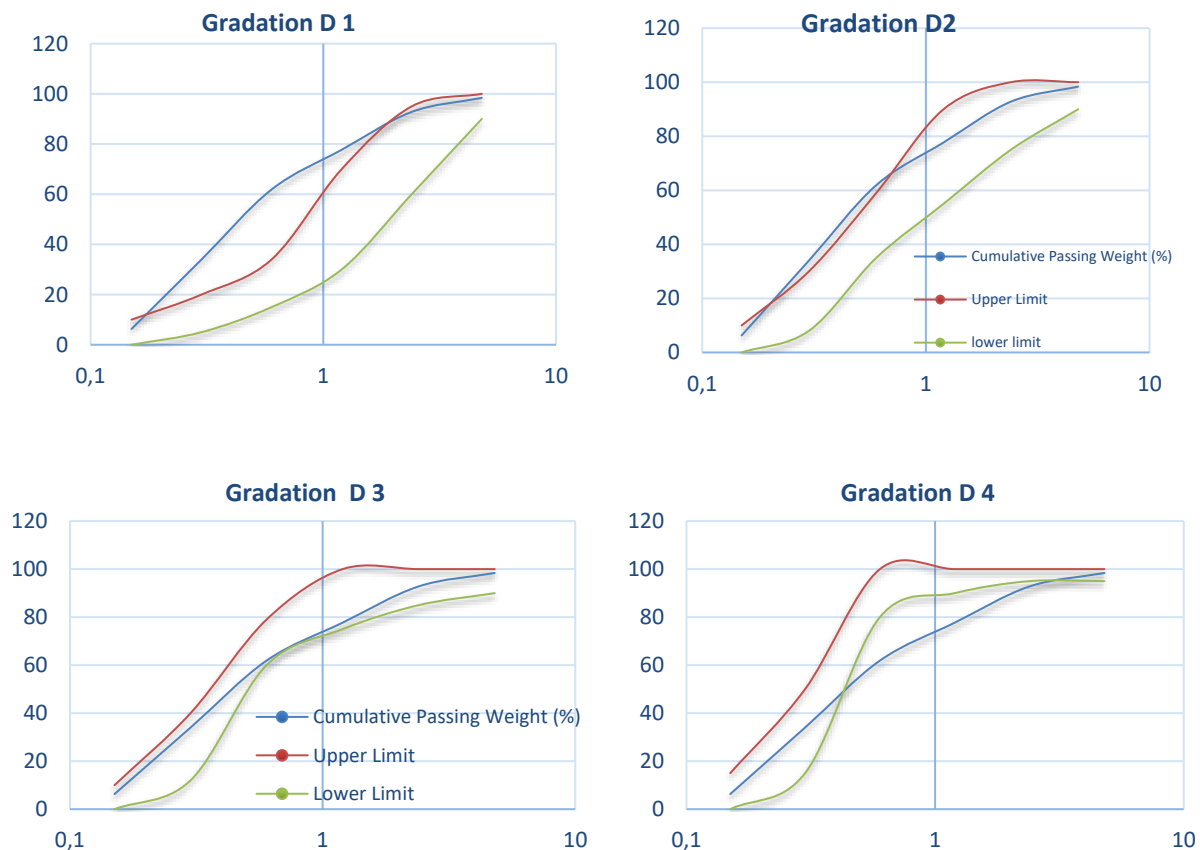


Fig. 7. Analysis of fine aggregate granules gradation

The outcomes were illustrated as cumulative % passing against sieve size on a semi-logarithmic scale, as depicted in Fig. 7. Each gradation curve was evaluated against the established upper and lower limits of fine aggregate gradation to determine compliance and appropriateness for brick

manufacture. The Gradation D1 curve approaches the upper limit within the fine particle spectrum, signifying a comparatively finer sand. This gradient enhances the workability and surface texture of the plastic brick mixture. Excessive fine content may marginally diminish permeability and elevate plastic requirements while mixing. Gradation D2 exhibits a balanced particle distribution that lies within the top and lower thresholds, indicating well-graded sand. This gradation improves density, interlocking, and adhesive strength between sand and plastic materials. The fineness modulus (FM = 2.289) indicates that D2 is classified as medium fine sand, optimal for attaining homogeneous compaction and an excellent surface finish in the product. The Gradation D3 curve exhibits a minor tilt towards the coarser end while remaining within acceptable parameters. This signifies a relatively gritty sand that can enhance the drainage and structural integrity of the interlocking plastic bricks. Nonetheless, when utilized in excessive amounts, it may diminish surface cohesiveness during the melding process. The D4 curve nears the lower threshold at smaller filter sizes, indicating a coarser aggregate composition. This gradation improves compressive strength and dimensional stability, while it may somewhat diminish workability.

4.2 Plastic Bricks Measurements

Upon completion of the curing time, the dimensions of the bricks were assessed using calibrated measuring instruments to identify any discrepancies. Variations in size may signify irregularities in the composition of the mixture, compaction force, or curing conditions.

Table 3. Plastic bricks measurements

No.	Code Test Object	Ratio	Length (cm)	Width (cm)	Height (cm)	Volume (cm ³)	Initial Weight (grams)
1	HDPE1	100%; 0%	22.26	10.79	5.79	1390.673466	1.22
2	HDPE2		22.375	10.845	5.95	1443.808406	1.2
3	HDPE3		22.25	10.7	5.9	1404.6425	1.27
Average	HDPE		22.295	10.778	5.88	1413.041457	1.23
4	HP A1	90%; 10%	22.41	10.936	5.61	1374.875014	1.27
5	HP A2		22.47	10.83	5.57	1355.460057	1.255
6	HP A3		22.45	10.91	5.5	1347.11225	1.28
Average	HP A		22.443	10.892	5.56	1359.149107	1.268
7	HP B1	80%; 20%	22.235	10.855	5.66	1366.102836	1.300
8	HP B2		22.36	10.05	5.64	1267.40952	1.265
9	HP B3		22.33	10.75	5.76	1382.6736	1.205
Average	HP B		22.308	10.552	5.687	1338.728652	1.257
10	HP C1	70%; 30%	22.24	10.73	5.37	1281.471024	1.265
11	HP C2		22.25	10.97	5.315	1297.298488	1.285
12	HP C3		22.245	10.86	5.5	1328.69385	1.195
Average	HP C		22.245	10.85	5.395	1302.487787	1.248

Compliance with standardized brick dimensions facilitates comparison testing, guaranteeing that mechanical parameters like compressive strength, density, and water absorption are assessed under consistent circumstances. In conclusion, ensuring accurate measurements in plastic brick production is crucial for quality assurance, structural integrity, and adherence to ASTM C67-05 and SNI 15-2094-2000 requirements. It drawing attention to their dimensions, volume, and weight. The bricks are 100% HDPE and 90:10, 80:20, and 70:30 HDPE-PET blends. As PET content rises, brick volume and mass vary somewhat. In the HP B1 sample, the 80:20 HDPE to PET ratio resulted in the greatest starting weight (1.300 grams) while having a smaller volume (1366.10 cm³), suggesting a denser and more compact structure. This shows that the 80:20 ratio may be best for lasting plastic bricks since it balances material density and structural integrity.

4.3 Test of Plastic Bricks

In this investigation, three significant laboratory tests were conducted. For every test, three specimens were made. To ascertain the specimens' density, compressive strength, and water

absorption, a brick compression test was conducted according to ASTM C67-05, and the Indonesian Standard SNI 15-2094-2000 was followed when conducting the testing procedure. To determine the flexural characteristics, the three-point bending flexural test was conducted in accordance with Indonesian Standard SNI 03-6458-2000. The testing procedure is shown in Figure 8. The plastic bricks were cast and cured for hours after the completion of the curing period, and the bricks were further tested. The following are a few tests that were performed on plastic bricks.

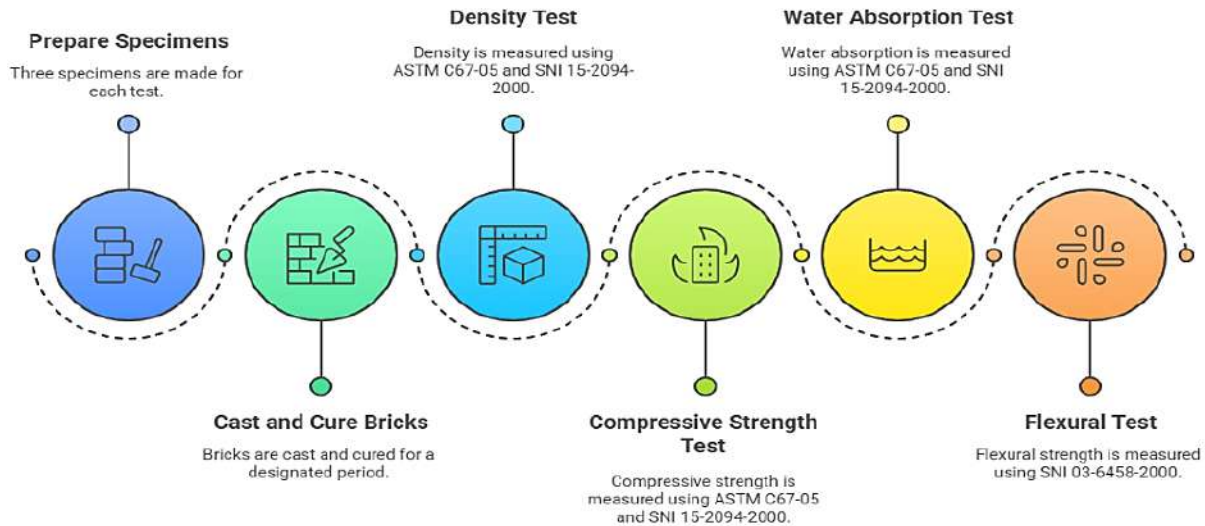


Fig. 8. Testing process plastic bricks

4.3.1 Compressive Strength Test

Compressive strength testing is carried out on cube-shaped test specimens that a curing period of 7 days. This test refers to SNI 15-2094-2000 concerning Solid Red Bricks for Wall Pairs. It was conducted using the Compression Testing Machine in the Structure and Construction Materials Laboratory, Civil Engineering Study Program, Muhammadiyah University of Yogyakarta. The compressive strength test results are shown in Figure 9.

From the results of the compressive strength test in Figure 8, there is an increase in the compressive strength value of the plastic bricks with 90% HDPE and 10% PP variations compared to the plastic bricks with 100% HDPE variations, with a difference in increase of 5.81 MPa. However, there is a decrease in the compressive strength value as the variation of the PP plastic mixture increases in the plastic bricks. This can be caused by the two materials being inhomogeneous between HDPE and PP, which reduces the bonds between molecules in the mixture. In addition, the difference in melting temperature between the two materials can cause non-uniformity in the brick structure, thus affecting the stress distribution during the compression test.

Compressive strength that controls the brick variation of 100% HDPE produces an average compressive strength of 30.43 MPa. Meanwhile, bricks with a variation of 10% PET mixture produce an average compressive strength of 26.7 MPa, bricks with a mixture of 20% PET produce an average of 18.1 MPa, and bricks with a mixture of 30% PET produce an average of 14.5 MPa. The difference in compressive strength results in each variation due to the large number of PET proportion levels that can affect the ability of the brick to withstand loads. The graph of the compressive strength test results of plastic bricks is shown in Figure 10. From the results of the compressive strength test of plastic bricks shown in Figure 10, it is known that bricks with a proportion of HDPE have the highest average compressive strength compared to bricks with a mixture of 10%, 20%, and 30% PET. This is due to the flexible nature of HDPE which has high mechanical resistance. HDPE can form a solid structure when melted and molded using a mold to increase durability. The addition of 10% PET causes a decrease in compressive strength due to the difference in physical properties of HDPE and PET, where PET is more fragile when compared to HDPE.

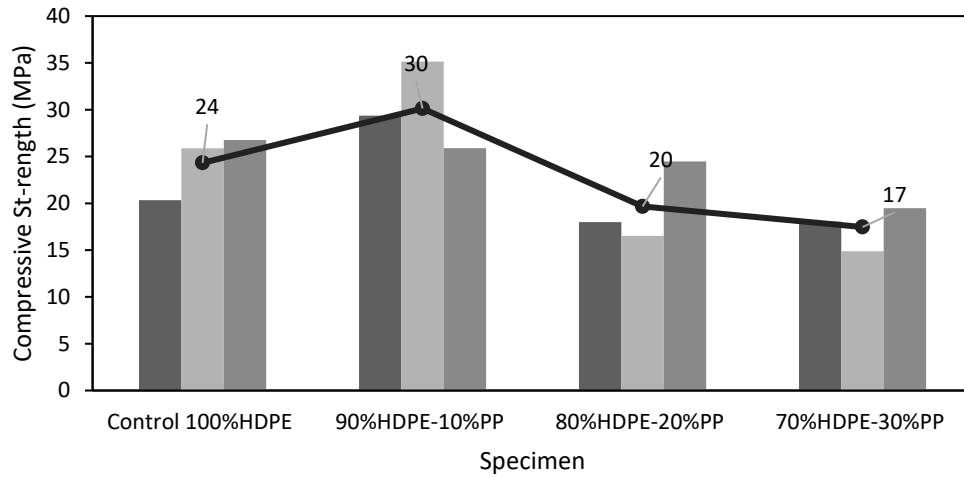


Fig. 9. Brick Compressive strength

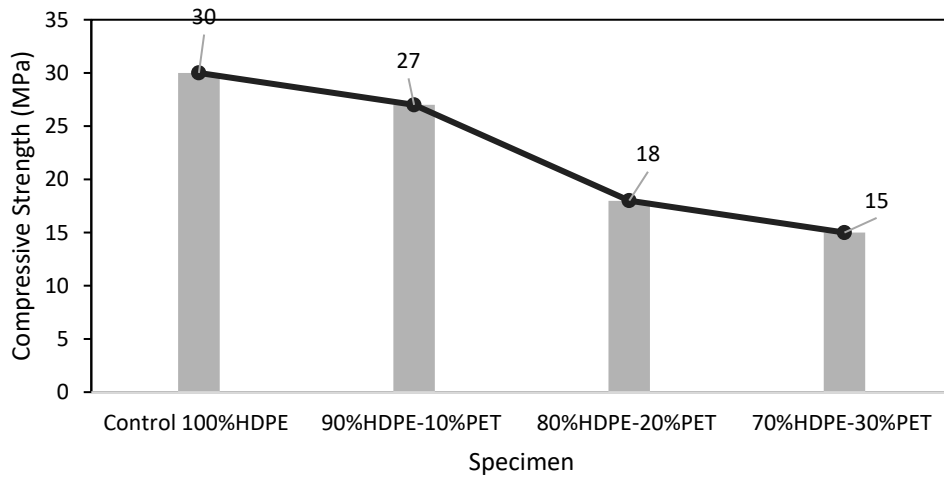


Fig. 10. HDPE-PET plastic brick compressive strength test result graph

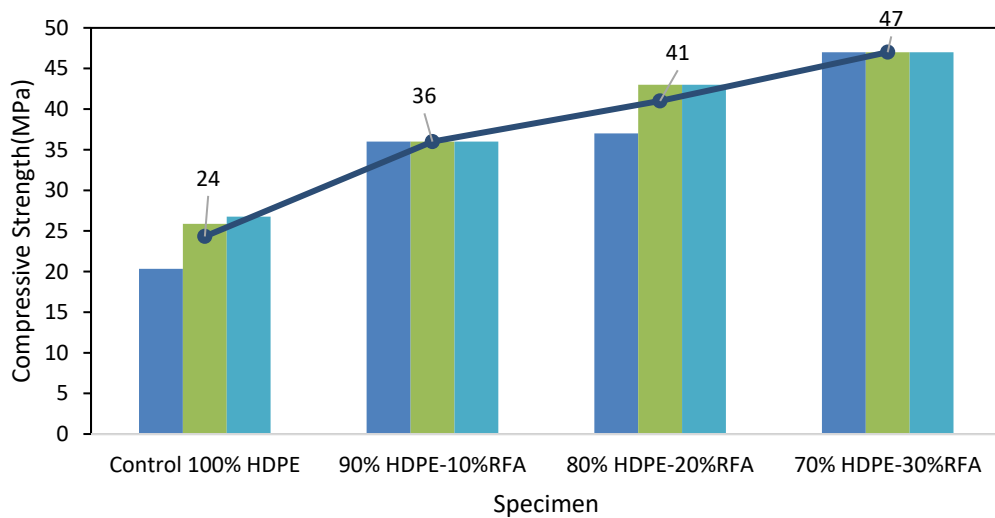


Fig. 11. Brick compressive strength diagram

Brick compressive strength testing refers to the provisions of SNI 15-2094– 2000b [46] about solid red bricks for wall masonry. Compressive strength testing using a Compression Testing Machine at the Structure and Construction Materials Laboratory, Civil Engineering, Muhammadiyah University

of Yogyakarta. Brick compressive strength testing is carried out when the mortar used as capping is 7 days old [52]. The results of the red brick compressive strength test can be seen in Figure 11. The maximum compressive strength results on the SPW HDPE D specimen, with variations of 70% HDPE and 30% RFA, with an average of 47.43 MPa, and shows the compressive strength of PW HDPE A brick with 100% HDPE variation, which is lower than SPW HDPE with additional RFA content.

4.3.2 Relationship between Characteristics and Compressive Strength

The relationship between brick characteristic testing and brick compressive strength for quality evaluation. Using polynomial regression, the relationship between brick characteristic testing and brick compressive strength is represented in Figures 12 and 13. The results of the linear regression analysis in Figure 12 show a coefficient correlation value of 0.9999. There is a positive correlation between the percentage of water absorption and compressive strength. The results of the analysis show that the higher the compressive strength, the greater the water absorption. In a study with HDPE as a substitute for sand in bricks, the compressive strength was observed to decrease with a higher percentage of HDPE, indicating that the material's compressive strength was not increased by water absorption. Research results showed that HDPE bricks exhibit superior mechanical properties, including higher compressive strength and lower water absorption compared to traditional clay bricks.

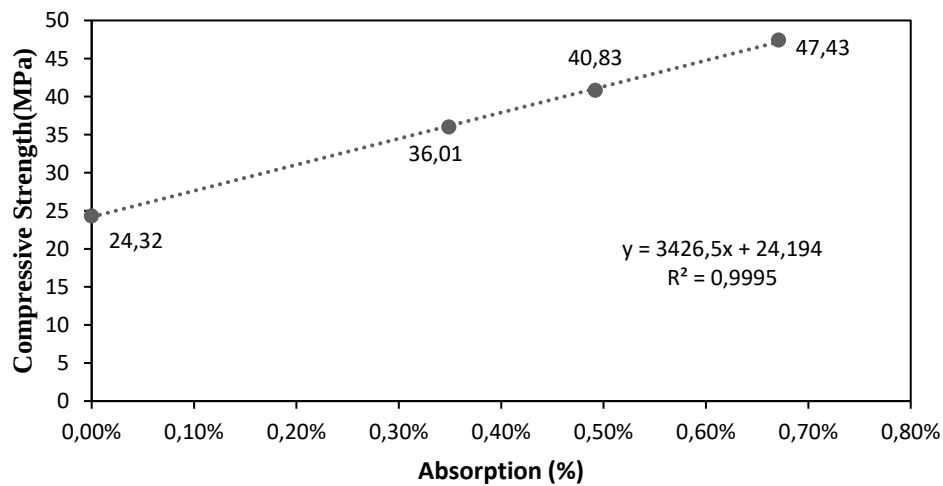


Fig. 12. Graph of the relationship between brick absorption and brick compressive strength

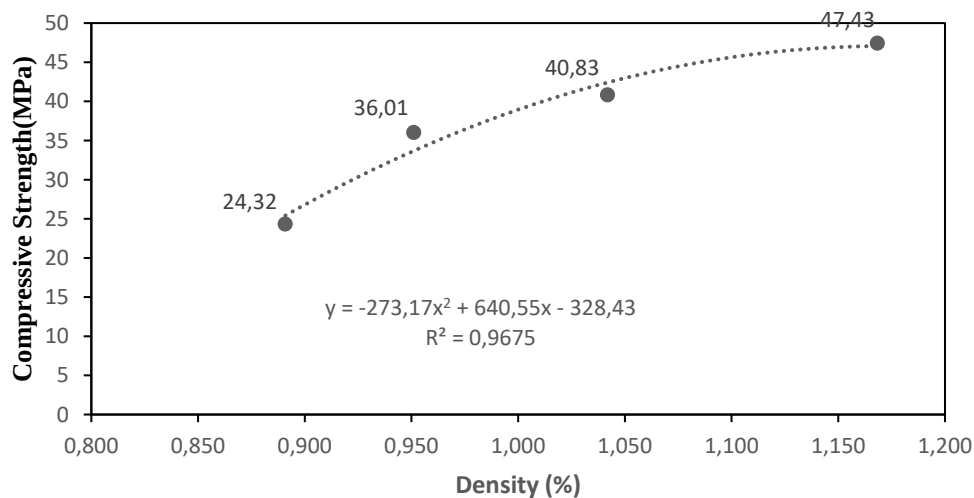


Fig. 13. Graph of the relationship between brick density and brick compressive strength

The results of the polynomial regression analysis in Figure 13 show a correlation coefficient value of 0.9675, and there is a positive correlation between the percentage of density and compressive strength. The relationship between the apparent density of HDPE-based bricks with RFA mixtures with RFA mixtures shows that these two tests provide parameters in evaluating the quality of bricks that are close to the same. The inclusion of HDPE in construction materials generally results in a reduction in density. HDPE is less dense than traditional aggregates such as sand and gravel. For example, when HDPE is used as a partial replacement for natural coarse aggregate, the resulting concrete is lighter, as evidenced by a 2.97% reduction in density compared to the control sample without HDPE.

4.3.3 Wall Shear Strength Testing

Shear strength testing of brick walls refers to ASTM E519/E519M-10(ASTM, 2002) about the standard test method for diagonal compressive strength (shear strength) on brick masonry walls. The brick wall shear strength test was taken as much as 1 test sample in each variation. The brick composition material consists of different HDPE, namely 100%, 90%, 80%, and 70%, with different RFA variations, namely 0%, 10%, 20% and 35%. The brick masonry wall shear strength test results can be seen in Figure 14. The polynomial regression analysis in the Figure shows the compressive strength coefficient and RFA content value of 0.9232. This indicates that the value of the shear strength coefficient and RFA content is close. The optimal compressive strength results on the SPW HDPE C specimen with a variation of 80% HDPE and 20% RFA are 0.700 MPa.

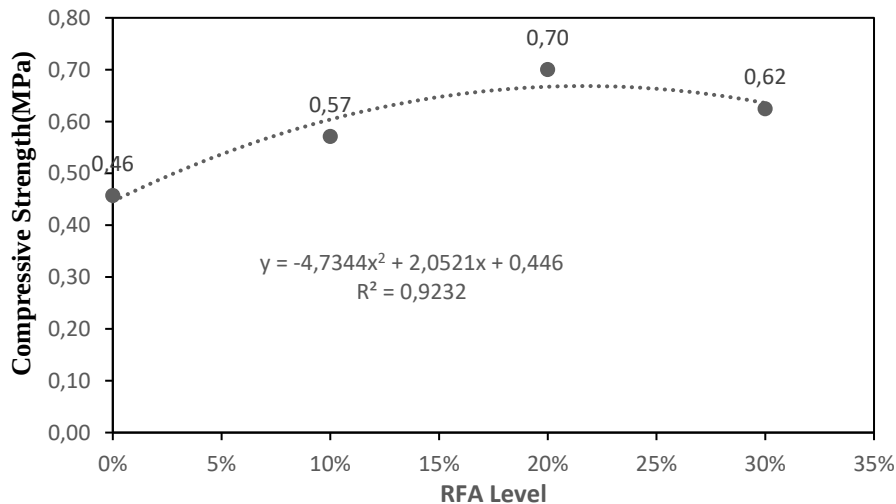


Fig. 14. Comparison graph of shear strength results of brick walls with varying RFA content

4.3.4 Wall Shear Strength Crack Pattern

The specimens tested for wall shear strength are then monitored for their crack patterns to determine how each pair of bricks with different mixtures behave when given an axial load. The crack pattern of the plastic brick's compressive strength usually differs. This occurs because of differences in the texture and characteristics of the materials used or the material content of the test object. According to [54], there are five types of wall shear strength failure modes, namely, horizontal sliding, steps (Step-Stairs Sliding), diagonal cracks (Diagonal Crack), multiple cracks (Multi Crack), and detachment. The shear strength test process for brick wall pairs can be seen in Figure 15. Based on the crack patterns of the test specimens in Table 4, it can be concluded that the visually visible crack patterns are the step-stairs sliding failure mode and the horizontal sliding failure mode. Crack patterns can occur because the mechanical properties of the materials used in masonry, such as mortar and plaster, significantly affect the shear strength and internal friction coefficient.

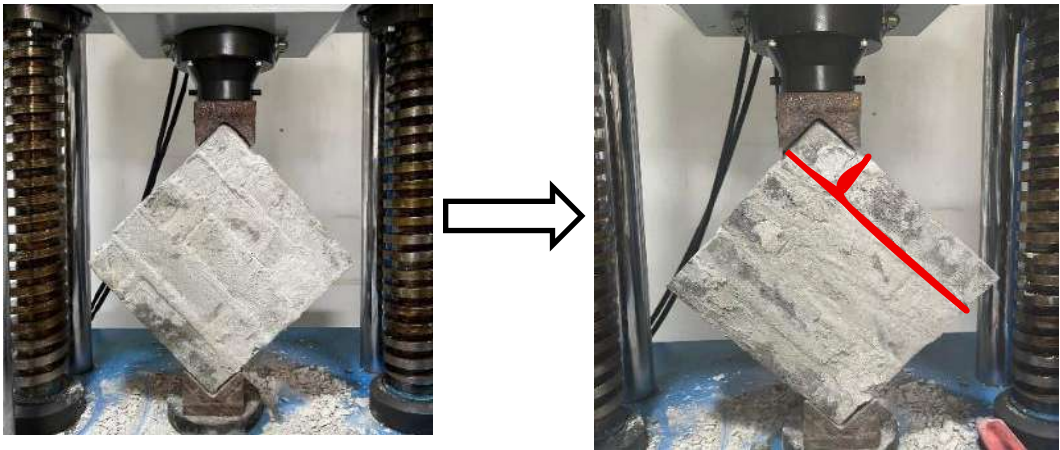
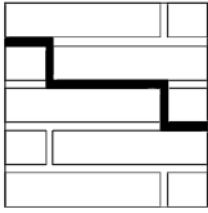
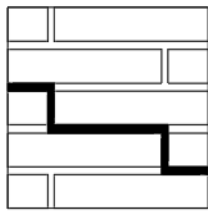
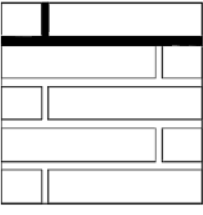
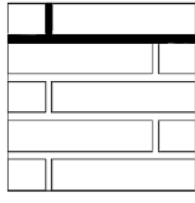


Fig. 15. The process of testing the shear strength of brick wall pairs

Table 4. Crack pattern of shear strength test of brick wall pair

Specimen	Failure mode image	Failure Mode Sketch
PW HDPE A		
SPW HDPE B		
SPW HDPE C		
SPW HDPE D		

The failure modes seen in plastic bricks made of HDPE and different amounts of Recycled Fine Aggregate (RFA) show that the matrix composition has a big effect on how well they hold up. When 100% HDPE specimens break, they do so in a brittle way, with wide fracture planes and abrupt separation under shear stresses. This is because there are no particles to slow down the spread of cracks. The addition of RFA (10% to 30%) changes how failure happens by making the material stiffer and more resistant to internal friction. As the amount of RFA in bricks goes up, they go from being brittle to being more ductile. The cracks become more confined, spread out, and eventually turn into stepwise crushing collapses. This matrix effect is like masonry systems, where (HDPE)

layers add shear strength and (RFA) layers make things more ductile by letting more energy escape. RFA works like these stiff inclusions in plastic bricks by making interlocking frictional channels in the HDPE matrix, which makes shear capacity better. If reinforcing fibers were included, such in Fabric Reinforced Cementitious Matrix systems, the shear capacity and ductility may be much better. As the amount of RFA increases, the shear strength and ductility should gradually climb. This will make plastic bricks a more reliable and long-lasting building material that can better resist shear deformation.

4.3.5 Relationship Between Compressive Strength and Shear Strength of Walls

Based on Figure 16, it can be concluded that the addition of RFA to HDPE bricks has been proven to increase the compressive strength value against HDPE bricks at certain levels and increase the shear strength value at but in the SPW HDPE D variation, the shear strength value decreases when applied to brick walls. This can happen because it is less effective in terms of bonding between mortar and bricks on brick walls, even though it has been proven effective for bonding between particles and pores in mortar bricks. In addition, HDPE bricks with RFA mixtures have different properties depending on where the HDPE and RFA come from. RFA in this test has properties that easily absorb water, and HDPE has properties that do not easily absorb water. However, the use of HDPE and RFA with the appropriate levels will still be able to produce quality construction products. From the polynomial regression calculations, it can be concluded that the optimal specimen recommended in this study is SPW HDPE D. At this level, the compressive strength value obtained was 47.43 MPa, and the shear strength value was 47.43 MPa, 0.625 MPa. The relationship graph of the compressive strength of bricks with the shear strength of brick masonry walls can be seen in Figure 16. This shows the values of the shear strength coefficient and shear strength that are close to.

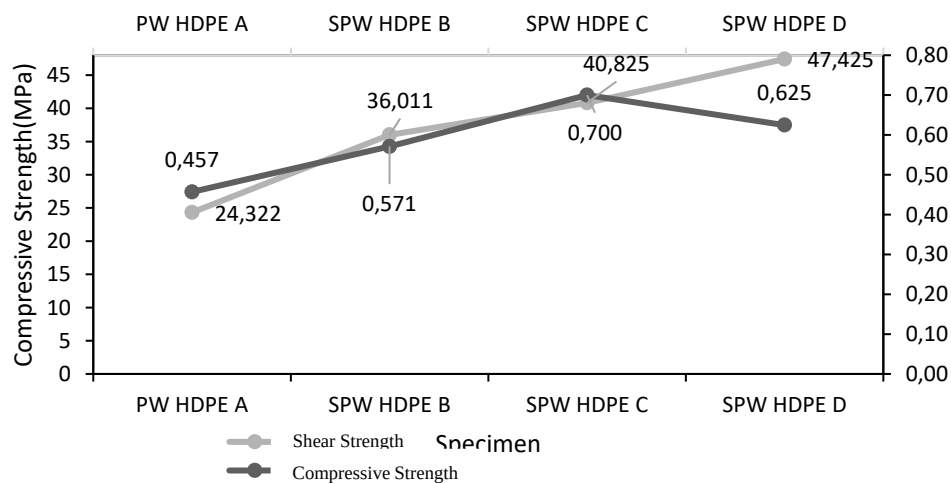


Fig. 16. Graph of the relationship between the compressive strength of bricks and the shear strength of brick wall pairs

Depicts the correlation between the compressive strength of individual plastic bricks and the shear strength of their respective wall pairs. The graph shows a mostly positive connection, suggesting that a rise in the compressive strength of the bricks corresponds with an enhancement in the shear strength of the wall pairs. This tendency indicates that more robust bricks improve interlocking and bonding efficacy within the wall structure, hence augmenting its resistance to lateral stress. The augmentation of shear strength is not always immediately proportionate to the escalation of compressive strength, presumably attributable to other influencing elements like material compatibility, surface roughness, and brick morphology, which impact the cohesion among bricks in a wall assembly. The graph underscores the need to maximize compressive strength to enhance structural performance in brick wall systems.

5. Conclusion

Based on the results of experimental testing and comprehensive data analysis from three investigative studies, plastic waste-based composite bricks have demonstrated considerable potential as lightweight, durable, and sustainable construction materials. These properties make them particularly suitable for floating house applications, where low density and high water resistance are critical performance criteria. The integration of High-Density Polyethylene (HDPE) with other plastic wastes and recycled aggregates presents an innovative, eco-friendly alternative to conventional masonry materials, effectively addressing the dual challenges of construction material demand and plastic waste management.

- Bricks composed entirely of HDPE achieved a compressive strength of 30.43 MPa with zero water absorption, indicating outstanding durability. The optimal blend, designated BP2 (80% HDPE and 20% PET), achieved the best balance between mechanical strength and waste utilization, with the lowest recorded water absorption of 0.3982%. HDPE–PET composites also exhibited the lowest densities among all tested mixtures, rendering them ideal for lightweight and moisture-sensitive applications.
- The incorporation of Recycled Fine Aggregate (RFA) into HDPE matrices significantly enhanced compressive strength, with a maximum value of 47.43 MPa achieved at a 70:30 HDPE–RFA ratio (sample SPW HDPE D). Water absorption decreased as RFA content increased, reaching 0.7508% at 20% RFA. Among all composite types, HDPE–RFA mixtures exhibited the highest densities, thereby improving structural mass and stability—an advantageous property for load-bearing applications.
- The addition of 10% Polypropylene (PP) improved compressive strength to 30.13 MPa; however, higher PP proportions led to a decline in strength due to partial incompatibility between the polymers. Water absorption increased with higher PP content, reaching 1.631% at 20% PP, which limits the suitability of these blends for moisture-exposed environments. The density of HDPE–PP composites rose progressively with PP content, attaining 0.1039 g/cm³ at 30% PP, situating them as intermediate materials between the HDPE–PET and HDPE–RFA groups.
- Furthermore, shear strength assessments revealed that plastic composite bricks outperformed conventional red clay bricks, achieving shear values of up to 4.3 MPa. Overall, these findings affirm the feasibility of HDPE-based composite bricks as sustainable, high-performance alternatives to traditional construction materials. By transforming plastic waste into structurally reliable and environmentally responsible building units, this research supports the advancement of circular economy principles in the construction industry.

Acknowledgements

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References

- [1] Dey S, Veerendra GTN, Babu PSSA, Manoj AVP, Nagarjuna K. Degradation of Plastics Waste and Its Effects on Biological Ecosystems: A Scientific Analysis and Comprehensive Review. Vol. 2. Springer US; 2024. <https://doi.org/10.1007/s44174-023-00085-w>
- [2] Ragaert K, Delva L, Van Geem K. Mechanical and chemical recycling of solid plastic waste. *Waste Manag.* 2017;69:24-58. <https://doi.org/10.1016/j.wasman.2017.07.044>
- [3] Rodic L. Comparing Solid Waste Management in the World's Cities. 2010;(November):15-18.
- [4] Moshkal M, Akhapov Y, Ogihara A. Sustainable Waste Management in Japan: Challenges, Achievements, and Future Prospects: A Review. *Sustainability.* 2024;16(17):7347. <https://doi.org/10.3390/su16177347>
- [5] Sina DAT, Udiana IM, Da Costa BD. Pengaruh Penambahan Cacahan Limbah Plastik Jenis HDPE pada Kuat Lentur Beton. *J Tek Sipil.* 2012;1(4):14.
- [6] Kibria MG, Masuk NI, Safayet R, Nguyen HQ, Mourshed M. Plastic Waste: Challenges and Opportunities to Mitigate Pollution and Effective Management. Vol. 17. Springer International Publishing; 2023. <https://doi.org/10.1007/s41742-023-00507-z>

- [7] Kumar R, et al. Impacts of plastic pollution on ecosystem services, sustainable development goals, and need to focus on circular economy and policy interventions. *Sustain.* 2021;13(17):1-40. <https://doi.org/10.3390/su13179963>
- [8] Ismawati Y, Septiono MA, Proboretno N. Plastic Waste Trade. 2024.
- [9] Pilapitiya PGCNT, Ratnayake AS. The world of plastic waste: A review. *Clean Mater.* 2024;11:100220:e1700782. <https://doi.org/10.1016/j.clema.2024.100220>
- [10] Geyer R, Jambeck JR, Law KL. Production, use, and fate of all plastics ever made. *Sci Adv.* 2017;3(7). <https://doi.org/10.1126/sciadv.1700782>
- [11] UNFCCC. United Nations Climate Change, 2022. 2022.
- [12] McMichael C, Dasgupta S, Ayeb-Karlsson S, Kelman I. A review of estimating population exposure to sea-level rise and the relevance for migration. *Environ Res Lett.* 2020;15(12). <https://doi.org/10.1088/1748-9326/abb398>
- [13] Alake Olaniyi MSA, Adeola OMO. Evaluation of Bottle Bricks Walls Using Polyethylene Terephthalate (pet) Bottle as Bricks. *Sustain Environ.* 2023;8(3). <https://doi.org/10.22158/se.v8n3p50>
- [14] Vera P, Canellas E, Nerín C. Designing safe recycled high-density polyethylene (HDPE) for child toys. *J Hazard Mater.* 2024;478:135482. <https://doi.org/10.1016/j.jhazmat.2024.135482>
- [15] Miliute-Plepiene J, Fråne A, Almasi AM. Overview of polyvinyl chloride (PVC) waste management practices in the Nordic countries. *Clean Eng Technol.* 2021;4:100246. <https://doi.org/10.1016/j.clet.2021.100246>
- [16] Sarker M. Conversion of Low Density Polyethylene (LDPE) and Polypropylene (PP) Waste Plastics into Liquid Fuel Using Thermal Cracking Process. *Br J Environ Clim Chang.* 2012;2(1):1-11. <https://doi.org/10.9734/BJECC/2012/994>
- [17] Chan HHS, Not C. Variations in the spatial distribution of expanded polystyrene marine debris: Are Asian's coastlines more affected? *Environ Adv.* 2023;11:100342. <https://doi.org/10.1016/j.envadv.2023.100342>
- [18] Pučnik R, et al. A waste separation system based on sensor technology and deep learning: A simple approach applied to a case study of plastic packaging waste. *J Clean Prod.* 2024;450. <https://doi.org/10.1016/j.jclepro.2024.141762>
- [19] Ghosh S, Sinha JK, Ghosh S, Vashisth K, Han S, Bhaskar R. Microplastics as an Emerging Threat to the Global Environment and Human Health. *Sustain.* 2023;15(14). <https://doi.org/10.3390/su151410821>
- [20] Lackner M. Bioplastics - Biobased plastics as renewable and/or biodegradable alternatives to petroplastics. 2015.
- [21] Venkatesh G, Nanditha M. Production of Plastic Reinforced Concrete using High Density Polyethylene (HDPE) and Polyethylene Terephthalate (PET) Waste Plastics as Aggregate Replacement. 2023.
- [22] Uvarajan T, Gani P, Chuan NC, Zulkernain NH. Reusing plastic waste in the production of bricks and paving blocks: a review. *Eur J Environ Civ Eng.* 2022;26(14):6941-6974. <https://doi.org/10.1080/19648189.2021.1967201>
- [23] Alhussein MAA, Zaki A, Nursetiawan. Plastic Bricks as A Solution for Utilizing Plastic Waste for Green Materials. *J Phys Conf Ser.* 2025;2989(1). <https://doi.org/10.1088/1742-6596/2989/1/012023>
- [24] Coelho PM, Corona B, ten Klooster R, Worrell E. Sustainability of reusable packaging-Current situation and trends. *Resour Conserv Recycl X.* 2020;6:100037. <https://doi.org/10.1016/j.rcrx.2020.100037>
- [25] Utsev JT, Imoni S, Onuzulike C, Akande EO, Orseer AM, Tiza MT. Strategies for Sustainable Construction Waste Minimization in the Modern Era. *Bincang Sains dan Teknol.* 2024;3(1):1-10. <https://doi.org/10.56741/bst.v3i01.506>
- [26] Ratnawati S. Processing of Plastic Waste Into Alternative Fuels in The Form of Grounded (Pertalastic) Through Pyrolysis Process in Science Laboratory of MTsN 3 West Aceh. *Indones J Chem Sci Technol.* 2020;3(1):8. <https://doi.org/10.24114/ijcst.v3i1.18310>
- [27] Kulkarni P, Ravekar V, Rama Rao P, Waigokar S, Hingankar S. Recycling of waste HDPE and PP plastic in preparation of plastic brick and its mechanical properties. 2022. <https://doi.org/10.1016/j.clema.2022.100113>
- [28] Ashraf N, El-Monayeri OD, Hassan HA. Lego-like Bricks Manufacturing Using Recycled Polyethylene (PE) and Polyethylene Terephthalate (PET) Waste in Egypt. *Sustain.* 2024;16(19). <https://doi.org/10.3390/su16198567>
- [29] Radhakrishnan S. Environmental Implications of Reuse and Recycling of Packaging. *Environ Footprints Eco-Design Prod Process.* 2016:165-192. https://doi.org/10.1007/978-981-287-913-4_7
- [30] Licciardello F. Unexpected possible consequences of plastic packaging reuse. *Curr Opin Food Sci.* 2024;56:101131. <https://doi.org/10.1016/j.cofs.2024.101131>
- [31] Zaki A, Yadi S, Mahbubi K, Nindhita KW, Fauzi A. Empowering the community through plastic waste management and eco-friendly construction materials in Bangunjiwo Village. *BIO Web Conf.* 2024;137:1-6. <https://doi.org/10.1051/bioconf/202413703015>

- [32] Nindhita KW, Zaki A, Zaini M, Kusumawijaya AM. Corrosion analysis of concrete based on industrial waste and bacteria by non-destructive test methods. *Sci Rev Eng Environ Sci*. 2025;34(2):109-127. <https://doi.org/10.22630/srees.10299>
- [33] Fan CC, Huang R, Hwang H, Chao SJ. The effects of different fine recycled concrete aggregates on the properties of Mortar. *Materials (Basel)*. 2015;8(5):2658-2672. <https://doi.org/10.3390/ma8052658>
- [34] Zega CJ, Di Maio AA, Zerbino RL. Influence of Natural Coarse Aggregate Type on the Transport Properties of Recycled Concrete. *J Mater Civ Eng*. 2014;26(6). [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000910](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000910)
- [35] Moradi I, Irannajad M. Effect of wet ball milling on copper ore flotation by fractal geometry. *Sep Sci Technol*. 2023;58(8):1473-1490. <https://doi.org/10.1080/01496395.2023.2189063>
- [36] Ghani MU, et al. Mechanical and environmental evaluation of PET plastic-graphene nano platelets concrete mixes for sustainable construction. *Results Eng*. 2024;21:101825. <https://doi.org/10.1016/j.rineng.2024.101825>
- [37] Widyawati F. Pemanfaatan Serat Sisal (Agave Sisalana L.) Dan Limbah Plastik Pet Untuk Pembuatan Bata Ringan CLC (Cellular Lightweight Concrete). *J TAMBORA*. 2020;4(1):21-25. <https://doi.org/10.36761/jt.v4i1.566>
- [38] Sheyko S, Tsyganov V, Hrechanyi O, Vasilchenko T, Hrechana A. Determination of the optimal temperature regime of plastic deformation of micro alloyed automobile wheel steels. *Res Eng Struct Mater*. 2024;10(1):331-339. <http://dx.doi.org/10.17515/resm2023.49me0428tn>
- [39] Suvidha, Jain S, Kumar C. A. Studies on durability, thermal and morphological property with partial replacement of single use waste plastic and complete recycled aggregate in eco-friendly self-compacting concrete. *Res. Eng. Struct. Mater.*, 2025; 11(4): 1413-1426. <http://dx.doi.org/10.17515/resm2024.315ma0609rs>
- [40] BSN. Cara Uji Berat Jenis dan Penyerapan Air Agregat Halus. Badan Standarisasi Nasional Indonesia. 2008:7-18.
- [41] Ogundipe OB, Ikubanni PP, Adebayo RA, Abolarinwa BO, Dike C.P., Lawal SO, Ajewole JB, Oladimeji SO. Evaluation of bentonite suitability as a binder on the physico-mechanical and thermal properties of insulating bricks produced from quartz deposit. *Res. Eng. Struct. Mater.*, 2025; 11(2): 799-818. <http://dx.doi.org/10.17515/resm2024.333me0628rs>
- [42] Atinc G, Simmering MJ, Kroll MJ. Control variable use and reporting in macro and micro management research. *Organ Res Methods*. 2012;15(1):57-74. <https://doi.org/10.1177/1094428110397773>
- [43] Gile D. Experimental research. *Res Transl Interpret*. 2015:220-228.
- [44] Ramadhana MA, Allo DK. Experimental Research in English Language Teaching: A Peek from Undergraduate Students' Theses. *J Stud Guru dan Pembelajaran*. 2021;4(1):32-38. <https://doi.org/10.30605/jsgp.4.1.2021.474>
- [45] Owoeye SS, Lamidi YD, Ajayi OJ. Physico-mechanical and corrosion behavior of acid resistant bricks developed from quarry waste silicate residues using powder metallurgy. *Res. Eng. Struct. Mat.*, 2017; 3(1): 55-64. <http://dx.doi.org/10.17515/resm2016.42ma0518>
- [46] BSN. SNI-15-2094-2000 Bata merah pejal untuk pasangan dinding. Badan Standarisasi Nasional Indonesia. 2000.
- [47] Fabbri A, Soudani L, McGregor F, Morel JC. Analysis of the water absorption test to assess the intrinsic permeability of earthen materials. *Constr Build Mater*. 2019;199:154-162. <https://doi.org/10.1016/j.conbuildmat.2018.12.014>
- [48] 2nd International Conference on Applied Mechanics and Materials, ICAMM 2013. 2014.
- [49] Rodrigues F, Evangelista L, De Britoa J. A new method to determine the density and water absorption of fine recycled aggregates. *Mater Res*. 2013;16(5):1045-1051. <https://doi.org/10.1590/S1516-14392013005000074>
- [50] Atoyebi OD, Modupe AE, Aladegboye OJ, Odeyemi SV. Dataset of the density, water absorption and compressive strength of lateritic earth moist concrete. *Data Br*. 2018;19:2340-2343. <https://doi.org/10.1016/j.dib.2018.07.032>
- [51] Duan Z, Zhao W, Ye T, Zhang Y, Zhang C. Measurement of Water Absorption of Recycled Aggregate. *Materials (Basel)*. 2022;15(15). <https://doi.org/10.3390/ma15155141>
- [52] Elhusna E, Agustin R. Kuat Tekan Bata Merah Dengan Variasi Usia ve Kadar Air Adukan Tanah Liat. *Inersia J Tek Sipil*. 2016;8(2):49-54.
- [53] ASTM E 519-02. Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages. *Am Soc Test Mater*. 2002.
- [54] Longo F, Cascardi A, Lassandro P, Aiello MA. Thermal and seismic capacity improvements for masonry building heritage: A unified retrofitting system. *Sustain*. 2021;13(3):1-16. <https://doi.org/10.3390/su13031111>

Experimental investigations on tensile and fracture behaviors of steel fiber-reinforced self-compacting concrete

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Abstract

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To investigate steel fiber's influence on self-compacting concrete (SCC)'s tensile and fracture properties, Brazilian disc splitting and Notched Semi-Circular Bending (NSCB) tests were conducted. The effects of steel fiber length, diameter, and volume fraction on SCC's tensile strength and fracture toughness, as well as the correlation between the two properties in steel fiber-reinforced SCC (SFRSCC), were analyzed. Results show: tensile strength first increased then decreased with rising fiber length, diameter, and volume fraction; fracture toughness first rose then fell with length and diameter but increased continuously with volume fraction. Steel fiber volume fraction had a more significant impact on NSCB's secondary bearing stage than length or diameter. A significant linear relationship was found between the tensile strength and fracture toughness of SFRSCC.

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1. Introduction

Self-compacting Concrete (SCC) has emerged as one of the most extensively utilized materials in the construction industry in recent years, owing to its high workability and self-compacting characteristics that eliminate the need for external vibration [1,2]. However, despite its numerous advantages, SCC exhibits limitations in mechanical strength, formability, and crack resistance [3,4]. Consequently, the incorporation of steel fibers into self-compacting concrete (SCC) has been proposed to enhance its material properties. Steel fibers, known for their high stiffness and tensile strength, effectively bridge both micro-cracks and macro-cracks during the post-cracking phase of concrete. Consequently, incorporating appropriate amounts of steel fibers into concrete can enhance its mechanical properties [5]. Abbas et al. [6] investigated the influence of steel fiber length on the 28-day splitting tensile strength of concrete, demonstrating a superior enhancement with 8-mm fibers. Shi et al. [7] conducted uniaxial tensile tests on cylindrical specimens incorporating hooked-end and straight steel fibers, revealing that the tensile strength initially increased and then plateaued with rising volume fractions. Notably, straight fibers exhibited a greater strength increment than hooked-end fibers. However, with regard to the mechanical behavior, toughness, and flexural performance of concrete after cracking, end-hooked steel fibers have been widely demonstrated to provide better reinforcing efficiency than straight steel fibers due to their distinctive mechanical anchoring effect [8]. Ning et al. [9] reported that steel fiber volume fractions of 0.38% and 0.64% significantly reduced the maximum crack width and average

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crack spacing in SCC beams, with the fiber aspect ratio having a pronounced effect on crack dimensions. Liu [10] assessed the mechanical properties of steel fiber-reinforced concrete through axial compression, residual flexural tensile strength, and flexural beam tests, confirming significant improvements in compressive and tensile strength compared to plain concrete. Han et al. [11] demonstrated that steel fibers effectively retard crack propagation, enhance the peak strain of SCC, and significantly improve compressive toughness in a dosage-dependent manner, while concurrently reducing workability due to increased internal frictional resistance. Li et al. [12] demonstrated that incorporating steel fibers effectively enhances the mechanical properties of SCC while concurrently optimizing its meso- and micro-pore structure. Ola [13] observed that excessive volumes of steel fiber may reduce tensile and compressive strength but still enhance crack resistance. Studies have shown that steel fiber-reinforced self-compacting concrete (SFRSCC) exhibits superior tensile and flexural properties compared with conventional vibrated steel fiber-reinforced concrete. This is mainly attributed to the crack-bridging effect of steel fibers [14]. The splitting tensile strength of self-compacting concrete with a steel fiber volume fraction of 0.75% can reach 4.12 MPa, which is 1.67 times that of ordinary steel fiber-reinforced concrete (2.47 MPa) [15]. It is evident that existing research predominantly focuses on conventional steel fiber-reinforced concrete (SFRC), which is prone to issues such as insufficient compaction, fiber clustering, or sedimentation during construction, thereby compromising structural performance. In contrast, steel fiber-reinforced self-compacting concrete (SFRSCC) eliminates the need for vibration due to its superior fluidity and self-consolidating characteristics, offering significant advantages in both construction efficiency and performance. Current studies and applications of SFRSCC remain limited, particularly regarding the comprehensive characterization of its tensile behavior and fracture mechanisms, which warrants further in-depth investigation. Fracture toughness is a fundamental parameter for analyzing the fracture behavior of concrete [16]. Therefore, research on fracture toughness is essential for understanding concrete fracture mechanisms. Current methods for determining fracture toughness include the Notched Semi-Circular Bending (NSCB) test [17], which has gained wide acceptance due to its simplicity in specimen preparation and testing apparatus [18,19]. However, this method has not been systematically applied to examine the fracture toughness of SFRSCC.

In this study, the tensile strength of SFRSCC with varying steel fiber lengths, diameters, and volume fractions was determined via Brazilian disc splitting tests. The load-displacement curve characteristics and failure modes of specimens were analyzed during this process. Utilizing NSCB tests, the effects of fiber geometric parameters on SCC fracture toughness were examined, alongside the load-displacement curve features and failure characteristics under NSCB conditions. Furthermore, a functional relationship between the fracture toughness and the tensile strength in SFRSCC was established through fitting analysis, providing a reference for engineering applications of SFRSCC.

2 Experiment Materials and Methods

2.1 Raw Materials

The raw materials comprised cement, fly ash, slag powder, water, a superplasticizer, coarse aggregates (including fine stones), fine aggregates (including fine sand and river sand), and hooked-end steel fibers. Fig. 1 illustrates different steel fibers, and their specifications are detailed in Table 1.

Table 1. Mechanical and geometric characteristics of hooked-end steel fibers

Length (mm)	Diameter (mm)	Tensile strength (MPa)	Modulus of elasticity (GPa)
25	0.80	1200	200
30	0.70		
30	0.80		
35	0.55		
35	0.70		
35	0.80		



Fig. 1. Steel fibers with different characteristic parameters

2.2 Specimen Preparations

A C50 self-compacting concrete mix proportion was adopted as the baseline for preparing concrete specimens, with the proportion of mixture provided in Table 2. A total of 10 groups of specimens were designed in the test. The steel fiber parameters were selected in accordance with the Technical Specification for Application of Fiber Reinforced Concrete (JGJ/T 221-2010) and combined with actual engineering application conditions. The volume fractions of steel fibers were set as 0%, 0.3%, 0.6%, 0.9%, and 1.2%. Ten test groups were designed, with steel fiber parameters summarized in Table 3.

Mixings were performed using an HJW-30/60 single horizontal shaft forced-action concrete mixer. Initially, steel fibers and fine stones were added and dry-mixed for 30 seconds. Subsequently, fine sand, river sand, and cementitious materials were incorporated and dry-mixed for an additional 30 seconds. Half of the premixed water and superplasticizer solution was added and mixed for 30 seconds, followed by the gradual addition of the remaining half and mixing for 60 seconds until homogeneity was achieved. After pouring, the specimens were not vibrated. The molds were released after 24 hours at ambient temperature and transferred to a curing chamber for 28 days, after which mechanical tests were conducted.

Table 2. The proportion of mixture for C50 self-compacting concrete (unit: kg/m³)

Cement	Fine sand	River sand	Fine crushed stone	Water	Ground granulated blast furnace slag (GGBS)	Fly as
422	385	386	836	170	91	91

Table 3. Design parameters for specimen grouping

Volume fraction of steel fibers (%)	Length of steel fibers(mm)	Diameter of steel fibers(mm)
0	0	0
0.3	30	0.70
0.6	30	0.70
0.9	30	0.70
1.2	30	0.70
0.9	25	0.80
0.9	30	0.80
0.9	35	0.80
0.9	35	0.70
0.9	35	0.55

2.3 Test methods

2.3.1 Splitting Tensile Tests

The tensile strength was indirectly determined via the Brazilian splitting test. Test specimens were cylinders with a diameter of 100 mm and a height of 100 mm, four replicate tests were conducted for each condition. As depicted in Fig. 2, displacement-controlled loading was applied at a rate of 0.2 mm/min using an IRSM-RC-1500 rock/concrete uniaxial compression-shear apparatus.



Fig. 2. Brazilian disc specimen (BD) (a) Front and (b) Vertical

2.3.2 NSCB Tests

As shown in Fig. 3, NSCB semi-cylinder specimens were configured with a radius (R) of 50 mm and a thickness (B) of 50 mm [20,21]. The notch length (a) was 20 mm, and the width (c) was 1.5 mm. Displacement-controlled loading was applied at 0.2 mm/min using a universal testing machine, with a support span (s) of 80 mm, five replicate tests were conducted for each condition.

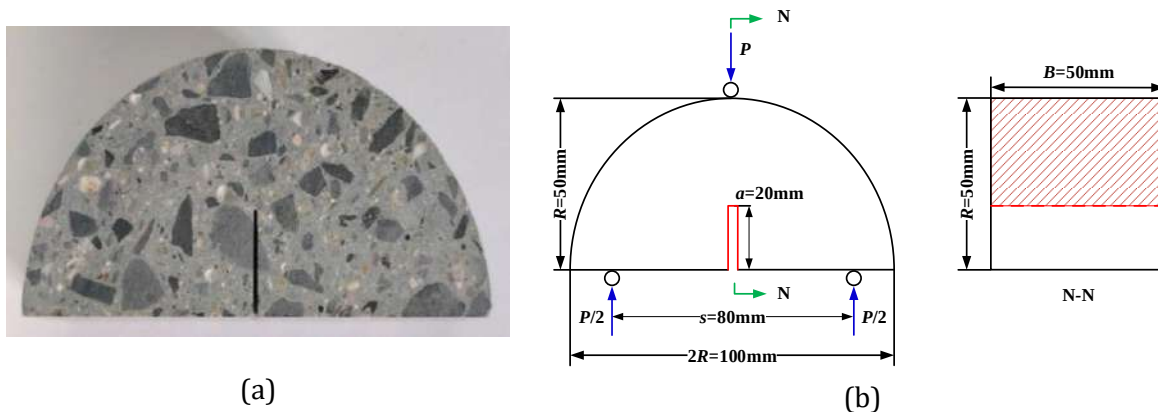


Fig. 3. Notched semi-circular bend specimen (NSCB) (a) Physical and (b) Diagram

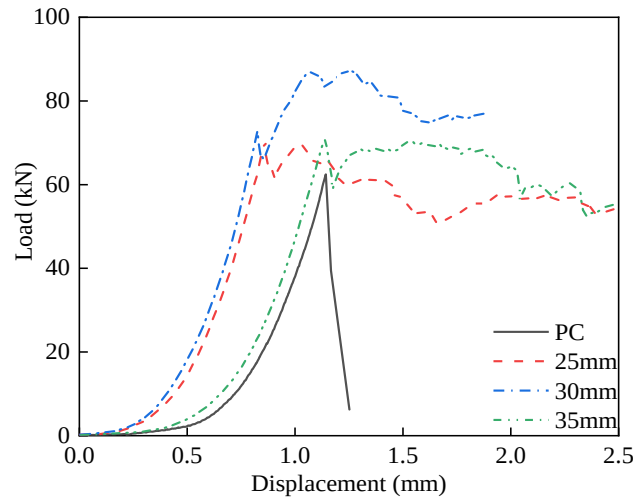
3. Results and Analysis

3.1 Brazilian Splitting Tests

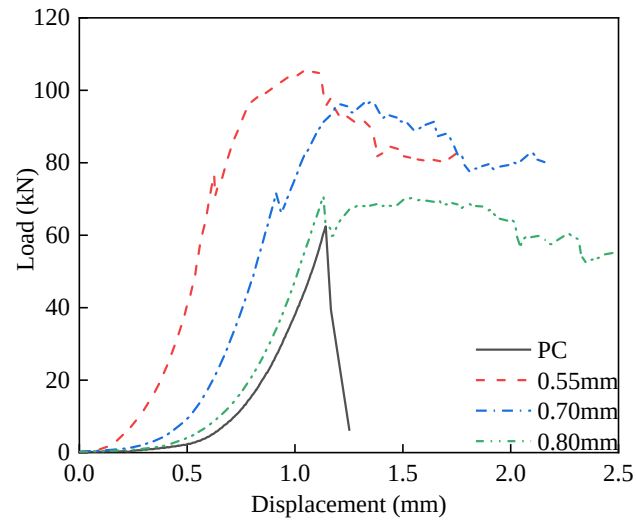
3.1.1 Experimental Results

Owing to space limitations, Fig. 4 presents partial load-displacement curves of SCC under Brazilian splitting tests with varying steel fiber lengths, diameters, and volume fractions. As illustrated in Fig. 4, plain concrete exhibits a rapid load drop after reaching the peak load, accompanied by instantaneous specimen fragmentation and complete loss of bearing capacity. In contrast, SFRSCC demonstrates a slight load reduction during loading, followed by a secondary ascending branch of the curve. The magnitude of this post-cracking strength recovery varies significantly with the

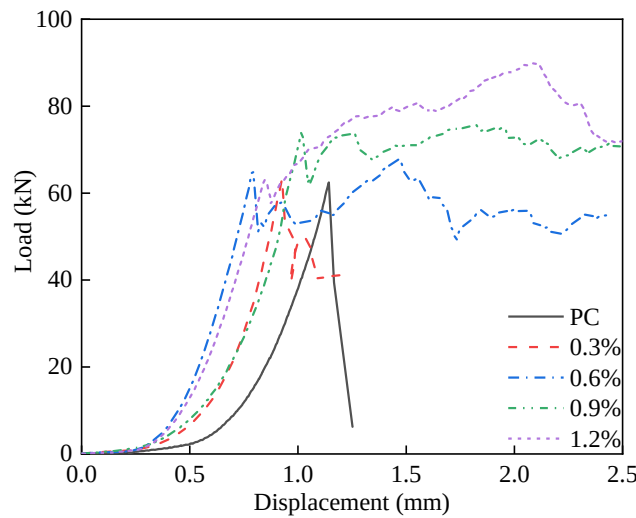
fiber length, the diameter, and the volume fraction. Correspondingly, SFRSCC specimens retain substantial residual bearing capacity after crack initiation without catastrophic failure.



(a)



(b)



(c)

Fig. 4. Load-displacement curves of SFRSCC in the Brazilian splitting test(a) Influence of lengths ($D=0.8\text{mm}$, $V_f=0.9\%$), (b) Influence of diameters ($L=35\text{mm}$, $V_f=0.9\%$) and (c) Influence of volume fractions ($L=30\text{mm}$, $D=0.7\text{mm}$)

Specifically, when the volume fraction and diameter of steel fibers remain constant, the secondary ascending amplitude of the load-displacement curve obtained from the Brazilian splitting test first increases and then decreases with the increase in steel fiber length. The maximum secondary ascending amplitude appears at a fiber length of 30 mm. Compared with 25 mm fibers, 30 mm fibers provide a longer anchorage length, ensuring a strong bond between fibers and the matrix. However, when the fiber length is increased to 35 mm, the excessive length tends to cause uneven dispersion in concrete. Many fibers cannot effectively participate in load bearing, and local fiber agglomeration weakens the strength of the matrix, resulting in a lower secondary ascending amplitude of the load-displacement curve. When the length and volume fraction of steel fibers are unchanged, the secondary ascending amplitude of the load-displacement curve decreases continuously with the increase in fiber diameter. The largest secondary ascending amplitude is achieved at a fiber diameter of 0.55 mm. Under the same volume fraction, smaller diameter corresponds to a greater number of fibers, which can form a denser and more uniform crack-bridging network across the crack surface and thus exhibit better cooperative performance. When the length and diameter of steel fibers are kept constant, the secondary ascending amplitude of the load-displacement curve increases continuously with the increase in fiber volume fraction. A higher volume fraction enhances the fiber density in the cracked zone and reduces the average spacing between fibers. More fibers contribute to crack resistance, provide higher pull-out resistance and greater fracture energy. The combined effect of these factors leads to a continuous increase in the secondary ascending amplitude of the load-displacement curve after cracking [22].

3.1.2 The tensile Strength

Owing to space limitations, Fig. 5 illustrates the effect of steel fiber length on the tensile strength of SCC. Under conditions of fiber diameter (0.8 mm) and volume fraction (0.9%), the tensile strength initially increased and then decreased with increasing fiber lengths. The maximum tensile strength was observed at the fiber length of 30 mm, with a mean tensile strength enhancement of 24.3% compared to plain concrete. When the fiber length was 25 mm, the shorter embedment length on both sides of the splitting cracks led to easier fiber pull-out during tensile loading. Conversely, excessive fiber length (35 mm) reduced the number of fibers across the fracture plane, diminishing crack-arresting efficiency.

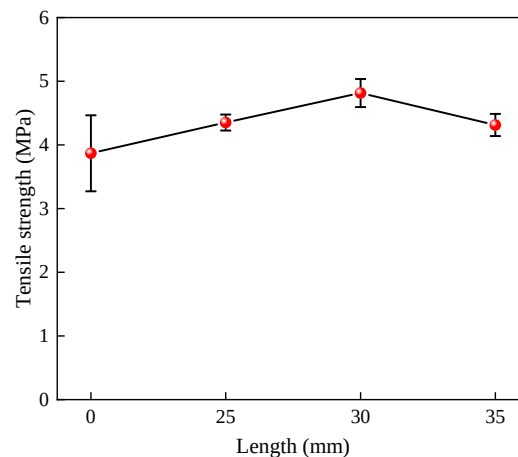


Fig. 5. Effects of steel fiber length on tensile strength of SFRSCC ($D=0.8\text{mm}$, $V_f=0.9\%$)

Fig. 6 demonstrates the influence of steel fiber diameter on SCC tensile strength. With fixed fiber length (35 mm) and volume fraction (0.9%), tensile strength exhibited a parabolic trend, peaking at a diameter of 0.70 mm (30.23% increase vs. plain concrete). This optimal behavior is attributed to the larger specific surface area of 0.70-mm fibers, enhancing mechanical interlocking with the matrix for superior stress transfer. However, when the fiber diameter increased to 0.80 mm, the tensile strength began to decrease. This may be attributed to the change in fiber number and bonding characteristic between fiber and matrix under the same volume fraction. Larger diameter

reduces the total number of fibers per unit volume, which weakens the crack arresting effect and stress distribution mechanism, thereby leading to the decline of tensile properties.

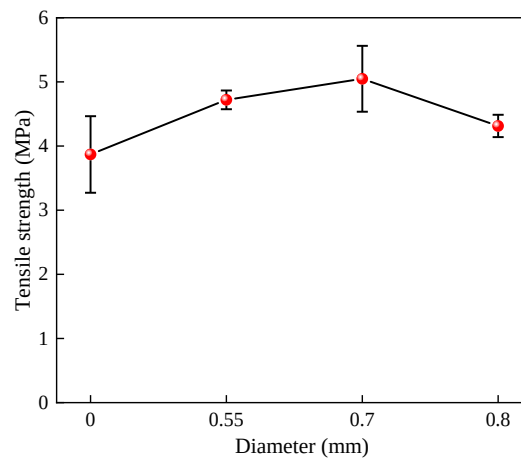


Fig. 6. Effects of steel fiber diameter on tensile strength of SFRSCC ($L=35\text{mm}$, $V_f=0.9\%$)

Fig. 7 presents the effect of steel fiber volume fraction (V_f) on tensile strength. The tensile strength initially increased with V_f but decreased beyond 0.9%, at which point a maximum tensile strength enhancement of 12.1% was achieved. At this optimal volume fraction, steel fibers were uniformly dispersed in the matrix, which effectively bridged microcracks and retarded the propagation of macrocracks without significantly impairing workability. In contrast, when V_f reached 1.2%, fiber agglomeration restricted the flow of cement paste, resulting in internal voids and weak interfacial zones, thereby reducing the tensile strength [23,24].

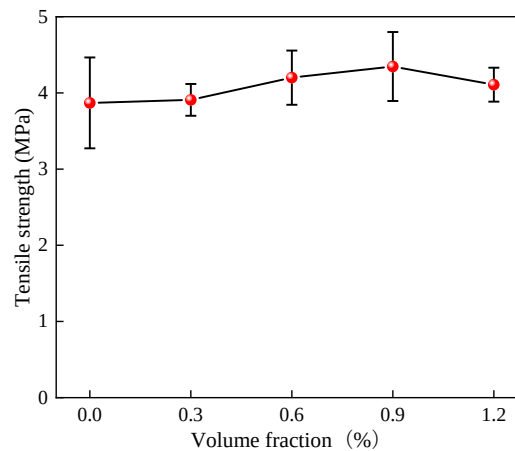
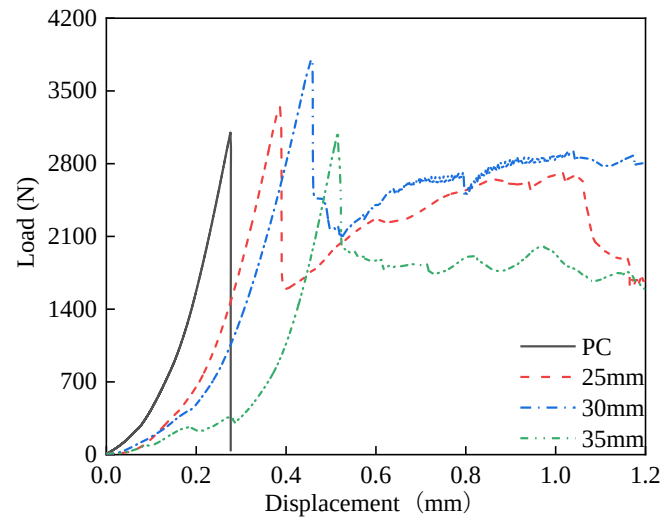


Fig. 7. Effects of steel fiber volume fraction on tensile strength in SFRSCC ($L=30\text{mm}$, $D=0.7\text{mm}$)

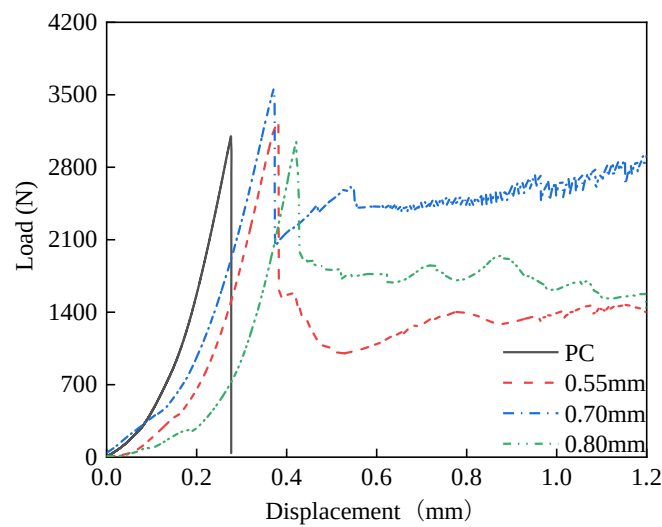
3.2 NSCB Tests

3.2.1 Experimental Results

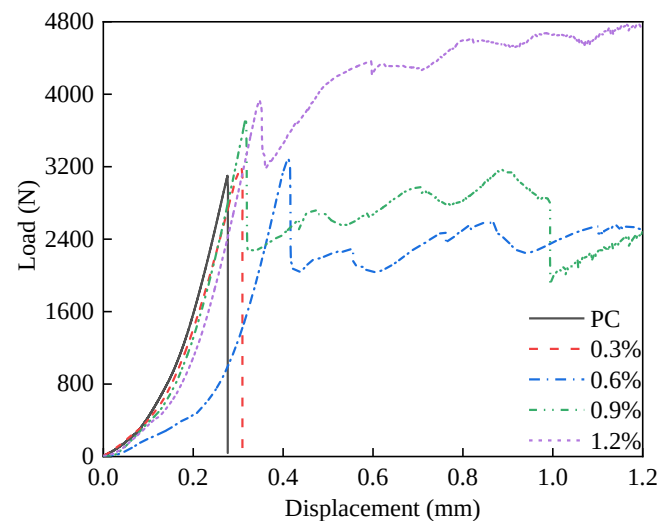
Fig. 8 depicts representative load-displacement curves from NSCB tests under varying fiber parameters. Plain concrete and low-fiber specimens ($V_f=0.3\%$) exhibited abrupt load drops post-peak, indicating a complete loss of bearing capacity. In contrast, higher V_f specimens displayed distinct post-cracking behavior.



(a)



(b)



(c)

Fig. 8. Load-displacement curves of NSCB specimens in SFRSCC (a) Influence of lengths ($D=0.80\text{mm}$, $V_f=0.9\%$), (b) Influence of diameters ($L=35\text{mm}$, $V_f=0.9\%$) and (c) Influence of volume fractions ($L=30\text{mm}$, $D=0.7\text{mm}$)

The stable bearing phase ($0.6\%-0.9\%V_f$): After the initial cracking, loads stabilized below the critical level with flattened curves. Notably, the curve morphology showed a minimal sensitivity

to the fiber geometry ($L=25\text{-}35\text{ mm}$, $D=0.55\text{-}0.80\text{ mm}$), confirming that the volume fraction dominates the post-cracking behavior over geometric parameters. The fiber bridging and stress redistribution delayed the failure [9], though $V_f=0.6\%$ provided only partial crack restraints. The strengthening phase ($1.2\% V_f$): Specimens demonstrated the post-cracking strength recovery, surpassing the critical load via 3D fiber networks that transferred the stress through pull-out mechanisms. Reducing fiber spacings dispersed crack-tip stresses [25], activating broader matrix participation, and macroscopic load resurgence.

For self-compacting concrete (SCC), the residual bearing capacity not only reflects the toughness and energy absorption capacity of the material, but also reveals the improvement effect of internal crack propagation, interface bonding, and fiber bridging on the mechanical properties after damage. Referring to Reference [26] and combining with the NSCB load-displacement curves of SFRSCC in Fig. 8, the load corresponding to a post-peak displacement of 0.9 mm was selected as the residual bearing capacity, and the statistical results are shown in Table (4~6). It can be seen from Table (4~6) that: when the steel fiber diameter is 0.70 mm and the length is 30 mm , the residual bearing capacity of SCC after the NSCB test increases continuously with the increase in steel fiber volume fraction; when the steel fiber diameter is 0.80 mm and the volume fraction is 0.90% , the residual bearing capacity of SCC after the NSCB test first increases and then decreases with the increase in steel fiber length; when the steel fiber length is 35 mm and the volume fraction is 0.90% , the residual bearing capacity of SCC after the NSCB test first increases and then decreases with the increase in steel fiber diameter.

Table 4. Effect of steel fiber volume fraction on residual bearing capacity of self-compacting concrete after NSCB test

Volume fraction of steel fibers(%)	Length of steel fibers(mm)	Diameter of steel fibers(mm)	residual bearing capacity(kN)
0.30	30	0.70	0
0.60	30	0.70	2.50
0.90	30	0.70	2.82
1.20	30	0.70	4.60

Table 5. Effect of steel fiber length on residual bearing capacity of self-compacting concrete after NSCB test

Volume fraction of steel fibers(%)	Length of steel fibers(mm)	Diameter of steel fibers(mm)	residual bearing capacity(kN)
0.90	25	0.80	2.61
0.90	30	0.80	2.79
0.90	35	0.80	1.80

Table 6. Effect of steel fiber volume fraction on residual bearing capacity of self-compacting concrete after NSCB test

Volume fraction of steel fibers(%)	Length of steel fibers(mm)	Diameter of steel fibers(mm)	residual bearing capacity(kN)
0.90	35	0.55	1.30
0.90	35	0.70	2.61
0.90	35	0.80	1.90

Fig. 9 quantifies residual bearing capacities. Plain and low-fiber specimens fractured catastrophically along the notch, reflecting the brittle failure. With increased fiber parameters, cracks propagated tortuously above the notch (Fig. 9), as aggregates and fibers altered crack paths. Longer/larger fibers intensified crack deflection, increasing the fracture surface irregularity due to the impeded crack advancement.



(a) PC



(b) $L=30\text{mm}$, $D=0.70\text{mm}$, $V_f=0.3\%$



(c) $L=30\text{mm}$, $D=0.70\text{mm}$, $V_f=0.6\%$



(d) $L=30\text{mm}$, $D=0.70\text{mm}$, $V_f=0.9\%$



(e) $L=30\text{mm}$, $D=0.70\text{mm}$, $V_f=1.2\%$



(f) $L=35\text{mm}$, $D=0.55\text{mm}$, $V_f=0.9\%$



(g) $L=35\text{mm}$, $D=0.70\text{mm}$, $V_f=0.9\%$



(h) $L=35\text{mm}$, $D=0.80\text{mm}$, $V_f=0.9\%$



(i) $L=25\text{mm}$, $D=0.80\text{mm}$, $V_f=0.9\%$



(j) $L=30\text{mm}$, $D=0.80\text{mm}$, $V_f=0.9\%$

Fig. 9 Residual load-bearing capacity of NSCB specimens in SFRSCC

3.2.2 The Fracture Toughness

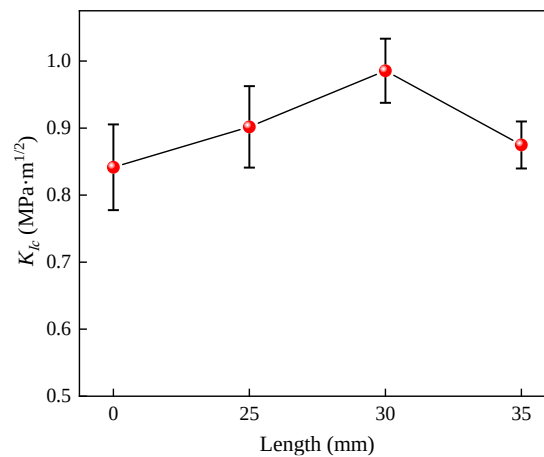
The stress intensity factor K serves as a metric for a material's resistance to crack propagation [20]. It is governed by factors such as applied stress, crack geometry, and dimensions. Research indicates that when K reaches a critical value, unstable crack propagation occurs. This critical value is termed the fracture toughness K_{IC} [21], which represents one of the fundamental fracture parameters in concrete fracture analysis and characterizes the material's crack resistance. According to the ISRM-recommended method, the Mode-I fracture toughness can be calculated using the formula proposed by Kuruppu et al [20]:

$$K_{IC} = Y' \frac{P_{max} \sqrt{\pi a}}{2RB} \quad (1)$$

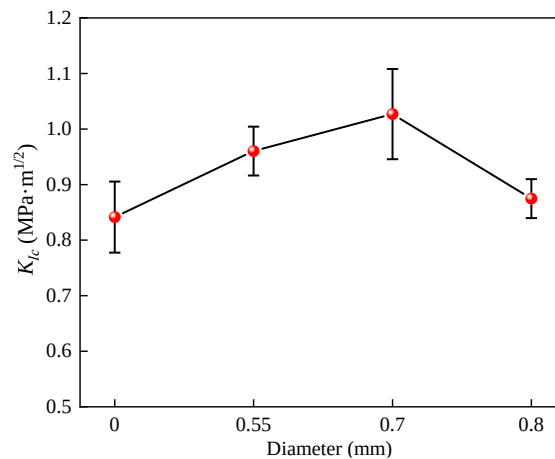
where K_{IC} is the stress intensity factor at crack initiation (i.e., fracture toughness), in $\text{MPa} \cdot \text{m}^{1/2}$; P_{max} is the critical load at crack propagation (hereafter termed "critical load"), in kN; a is the notch length (20 mm); B is the specimen thickness (50 mm); R is the specimen radius (50 mm); Y' is the dimensionless stress intensity factor for Mode-I loading, expressed as:

$$Y' = -1.297 + 9.516 \left(\frac{s}{2R} \right) - (0.47 + 16.457 \left(\frac{s}{2R} \right) \beta + (1.071 + 34.401 \left(\frac{s}{2R} \right) \beta^2) \quad (2)$$

where s is the span between the two supporting cylinders (80 mm); $\beta = a/R$, with $a = 20\text{mm}$, yielding $\beta=0.4$. Substituting the dimensions of the SCB specimen into Eq. (2) gives $Y'=5.436248$. Using Eq. (1), the fracture toughness K_{IC} of self-compacting concrete (SCC) NSCB specimens with varying steel fiber characteristic parameters was calculated, as illustrated in Fig. 10.



(a)



(b)

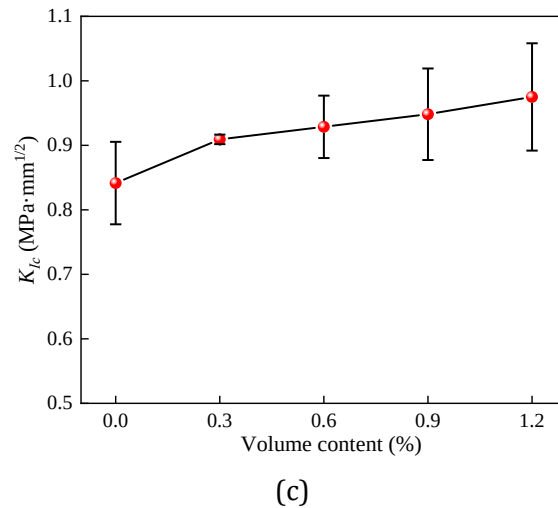


Fig. 10. Effects of steel fibers on the fracture toughness of SFRSCC (a) Influence of lengths ($D=0.8\text{mm}$, $V_f=0.9\%$), (b) Influence of diameters ($L=35\text{mm}$, $V_f=0.9\%$) and (c) Influence of volume fractions ($L=30\text{mm}$, $D=0.7\text{mm}$)

Fig. 10(a-b) reveals that the fracture toughness of SCC initially increases and then decreases with increasing steel fiber length and diameter. Specifically, when other fiber parameters are held constant, the maximum enhancement in fracture toughness (17.11%) relative to plain concrete occurs at a fiber length of 30 mm. The greatest improvement (22.03%) is achieved at a fiber diameter of 0.70 mm. Fig. 10(c) demonstrates a significant positive correlation between K_{IC} and V_f at $V_f=0.3\%$, $K_{IC}=0.91\text{MPa} \cdot \text{m}^{1/2}$, and an 8.33% increase over plain concrete ($0.84\text{MPa} \cdot \text{m}^{1/2}$). Further increases in V_f , K_{IC} continue to elevate, when $V_f=0.6\%$, $K_{IC}=0.93\text{MPa} \cdot \text{m}^{1/2}$ (+10.71%), when $V_f=0.9\%$, $K_{IC}=0.94\text{MPa} \cdot \text{m}^{1/2}$ (+12.00%), and when $V_f=1.2\%$, $K_{IC}=0.97\text{MPa} \cdot \text{m}^{1/2}$ (+15.48%). During the NSCB test of SFRSCC specimens, crack propagation leads to the pull-out of steel fibers from the matrix. The pull-out process requires overcoming the interfacial bond strength and friction between the fibers and the matrix. Owing to its excellent fluidity, self-compacting concrete forms a denser and more uniform interfacial transition zone with steel fibers, which improves the bond performance between fibers and the matrix [27]. This enables SFRSCC to dissipate more energy during the fracture process, thereby enhancing its fracture toughness.

It can be seen from the test results that the variation law of fracture toughness of steel fiber-reinforced self-compacting concrete (SFRSCC) with steel fiber volume fraction, length, and diameter is basically consistent with that of residual bearing capacity in Section 3.2.1. Specifically, both the fracture toughness and residual bearing capacity of SFRSCC increase continuously with the increase in steel fiber volume fraction. With the increase in steel fiber diameter, both increase first and then decrease. Similarly, with the increase in steel fiber length, both also show a trend of increasing first and then decreasing.

It is worth noting that the tensile strength and fracture toughness exhibit different variation trends at high fiber volume fractions. When the volume fraction exceeds 0.9%, the tensile strength decreases, whereas the fracture toughness continues to increase even at 1.2%. The main reason for this difference is as follows: Tensile strength is an overall mechanical index that is highly sensitive to the compactness, uniformity, and internal defects of concrete. Excessive fiber content tends to deteriorate workability and increase internal pores and weak interfaces, thereby reducing the tensile strength [28]. In contrast, fracture toughness mainly depends on the crack bridging, crack resistance, and energy dissipation of fibers after crack initiation. Even at a high volume fraction, the increased number of fibers can still provide sufficient bridging stress and toughening effect during crack propagation.

3.3 Correlation Analysis of The Tensile Strength and The Fracture Toughness

The failure of steel fiber-reinforced concrete (SFRC) originates from the propagation of microcracks under tensile stress. The fracture toughness quantifies the material's intrinsic resistance to crack propagation and correlates with the mechanical strength. Numerous studies [17, 29-34] have established a linear relationship between Mode-I fracture toughness (K_{Ic}) and tensile strength (σ_t) for materials like rock, clay, and concrete (Table 7).

Table 7. Compilation of empirical correlations between K_{Ic} and σ_t

Data source	Material category	Empirical formula
Su et al. [17]	Steel Fiber-Reinforced Reactive Powder Concrete	$\sigma_t = 1.76 K_{Ic} + 4.30$
Yang et al. [29]	Coal	$\sigma_t = 5.48 K_{Ic}$
Deng et al. [30]	Sandstone	$\sigma_t = 8.32 K_{Ic}$
Wang et al. [31]	Clay	$\sigma_t = 2.82 K_{Ic}$
Zhang [32]	Rock	$\sigma_t = 6.88 K_{Ic}$
Ding et al. [33]	Steel fiber-reinforced recycled concrete	$\sigma_t = 4.44 K_{Ic} + 0.86$
Liu et al. [34]	Plastic concrete	$\sigma_t = 2.28 K_{Ic}$

Given that concrete aggregates derive from crushed natural rocks, the relationship between K_{Ic} and σ_t for SFRC was investigated. Linear regression of experimental data (Fig. 11) yields.

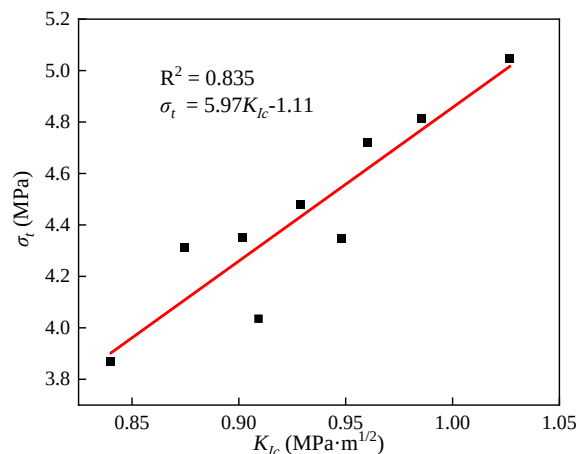


Fig. 11. Correlations between tensile strength (σ_t) and fracture toughness (K_{Ic}) of SCC with varied steel fiber parameters

σ_t is determined from the splitting tensile strength of cylindrical specimens, while K_{Ic} describes the resistance to rapid crack propagation in NSCB specimens. Both parameters reflect tensile performance but differ in focus: K_{Ic} emphasizes the resistance to crack propagation, whereas σ_t measures the global maximum load-bearing capacity. Given the simplicity of the Brazilian splitting test compared to fracture toughness tests, this linear correlation facilitates the prediction of K_{Ic} for SFRC.

4. Conclusions

- After incorporation of steel fibers, the concrete specimens do not experience complete fracture upon the occurrence of splitting cracks. The load-displacement curves show a slight stress drop, but the bearing capacity continues to increase with further loading. Such an increase is closely related to fiber length, diameter, and volume fraction.
- The tensile strength of self-compacting concrete (SCC) first increases and then decreases with increasing steel fiber length, with the maximum average tensile strength obtained at a fiber length of 30 mm. Similarly, the tensile strength increases initially and then decreases

with increasing fiber diameter, reaching the peak value at 0.70 mm. With increasing steel fiber volume fraction, the tensile strength rises first and then declines, and the optimal strengthening effect is achieved at a volume fraction of 0.9%.

- The load-displacement curves of steel fiber-reinforced self-compacting concrete (SFRSCC) obtained from NSCB tests exhibit a brief stress reduction during initial crack propagation, followed by a secondary load-bearing stage. The mechanical behavior in this stage is more significantly affected by the steel fiber volume fraction than by fiber length or diameter. At volume fractions of 0.6% and 0.9%, the specimens retain bearing capacity below the cracking load after initial cracking, and the curves gradually flatten. At a volume fraction of 1.2%, the bearing capacity not only resists degradation after initial cracking but also continues to increase beyond the critical load.
- The fracture toughness of SCC first increases and then decreases with increasing steel fiber length and diameter. Under the condition that other fiber parameters remain unchanged, the maximum improvement in fracture toughness (relative to plain concrete) is achieved at a fiber length of 30 mm and a fiber diameter of 0.70 mm. The fracture toughness shows a clear increasing trend with rising steel fiber volume fraction, reaching the optimal enhancement at 1.2%.
- Based on the experimental data for SFRSCC, a statistically robust linear relationship exists between the tensile strength (σ_t) and the mode-I fracture toughness (K_{IC}). The constitutive equation is formulated as: $\sigma_t = 5.97K_{IC} - 1.11$, with a coefficient of determination ($R^2 = 0.835$), confirming a strong correlation.

Acknowledgments

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References

- [1] Traver-Abella C, Bonet JL, Miguel PF, et al. Effect of steel fibres on the shear behaviour of SCC dry joints in precast segmental bridges. *Constr Build Mater*, 2024; 415: 134998. <https://doi.org/10.1016/j.conbuildmat.2024.134998>
- [2] Karuppasamy M, Balakrishnan B, Chandran K, et al. Analysis of mechanical properties in self-compacting concrete enhanced with steel fibers. *Micro Nano Lett*, 2024; 19(4): e12203. <https://doi.org/10.1049/mna2.12203>
- [3] Pour AK, Shirkhani A, Noroozinejad Farsangi E. Effect of high-range water reducer and W/C ratio on the fresh and mechanical properties of fiber-reinforced natural zeolite SCC. *Struct Concr*, 2024; 25(5): 4075 - 4102. <https://doi.org/10.1002/suco.202301071>
- [4] Mousavi SR, Afshoon I, Bayatpour MA, et al. Effect of waste glass and curing aging on fracture toughness of self-compacting mortars using ENDB specimen. *Constr Build Mater*, 2021; 282: 122711. <https://doi.org/10.1016/j.conbuildmat.2021.122711>
- [5] Zhao Y, Bi J, Sun Y, et al. Synergetic effect of ground granulated blast-furnace slag and hooked-end steel fibers on various properties of steel fiber reinforced self-compacting concrete. *Struct Concr*, 2022; 23(1): 268 - 284. <https://doi.org/10.1002/suco.202000722>
- [6] Abbas S, Soliman AM, Nehdi ML. Exploring mechanical and durability properties of ultra-high performance concrete incorporating various steel fiber lengths and dosages. *Constr Build Mater*, 2015; 75: 429 - 441. <https://doi.org/10.1016/j.conbuildmat.2014.11.017>
- [7] Shi X, Park P, Rew Y, et al. Constitutive behaviors of steel fiber reinforced concrete under uniaxial compression and tension. *Constr Build Mater*, 2020; 233: 117316. <https://doi.org/10.1016/j.conbuildmat.2019.117316>
- [8] Chen CC, Wu ZM, Hu X, et al. Influence of fiber shape and curing system on strength and toughness of UHPC. *Materials Reports*, 2024, 38(15): 139-149.
- [9] Ning XL, Ding YN. Experimental research on crack width of steel fibers reinforced self-consolidating concrete beams. *Eng Mech*, 2017; 34(4): 116 - 122.
- [10] Liu RY, Wang ZJ. Research on crack development of unreinforced steel fiber concrete compression-bending. *Build Struct*, 2020; 50(22): 93 - 93.

- [11] Han JP, Han WL, Wen XH, et al. Experimental investigation on the influence of steel fiber on the workability and compressive toughness of self-compacting concrete. *J Lanzhou Univ Technol*, 2024; 50(5): 112 - 117.
- [12] Li J, Hu HN. Mechanical properties and pore structure characteristics of fiber reinforced self-compacting concrete. *Acta Mater Compos Sin* (Online First). Accessed 2025-06-06.
- [13] Ola AQ. High temperature effect on shear transfer strength of steel fiber reinforced self-compacted concrete. *J Eng Appl Sci*, 2019; 14(10): 3158 - 3174. <https://doi.org/10.36478/jeasci.2019.3158.3174>
- [14] MUSA A M. Robustness of self consolidating steel fiber reinforced concrete[J]. 2009.
- [15] Hafez B, Glusac J, Aslani F. Re-centring and flexural toughness performance of novel steel fibre-reinforced self-compacting concrete. *International Journal of Concrete Structures and Materials*, 2025, 19(1): 90. <https://doi.org/10.1186/s40069-025-00836-7>
- [16] Ayatollahi MR, Mahdavi E, Alborzi MJ, et al. Stress intensity factors of semi-circular bend specimens with straight-through and chevron notches. *Rock Mech Rock Eng*, 2016; 49: 1161 - 1172. <https://doi.org/10.1007/s00603-015-0830-y>
- [17] Su C, Wu Q, Weng L, et al. Experimental investigation of mode I fracture features of steel fiber-reinforced reactive powder concrete using semi-circular bend test. *Eng Fract Mech*, 2019; 209: 187 - 199. <https://doi.org/10.1016/j.engfracmech.2019.01.029>
- [18] Razmi A, Mirsayar MM. On the mixed mode I/II fracture properties of jute fiber-reinforced concrete. *Constr Build Mater*, 2017; 148: 512 - 520. <https://doi.org/10.1016/j.conbuildmat.2017.05.034>
- [19] Zhang J, Sakhaeifar MS, Little DN, et al. Characterization of crack growth rate of sulfur-extended asphalt mixtures using cyclic semicircular bending test. *J Mater Civ Eng*, 2018; 30(12): 04018311. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0002517](https://doi.org/10.1061/(ASCE)MT.1943-5533.0002517)
- [20] Kuruppu MD, Obara Y, Ayatollahi MR, et al. ISRM-suggested method for determining the mode I static fracture toughness using semi-circular bend specimen. *Rock Mech Rock Eng*, 2014; 47: 267 - 274. <https://doi.org/10.1007/s00603-013-0422-7>
- [21] Aziminezhad M, Mardi S, Hajikarimi P, et al. Loading rate effect on fracture behavior of fiber reinforced high strength concrete using a semi-circular bending test. *Constr Build Mater*, 2020; 240: 117681. <https://doi.org/10.1016/j.conbuildmat.2019.117681>
- [22] Kosior-Kazberuk M, Krassowska J. Fracture behaviour of basalt and steel fibre reinforced concrete. *Budownictwo i Inżynieria Środowiska*, 2015, 6(2).
- [23] Jin QP, Chen C, Huang QL, et al. Review on the Effects of fiber action in ultra high-performance concrete. *J Water Resour Archit Eng*, 2024; 22(2): 55 - 65.
- [24] Ma QY, Zhang HP, Li LY, et al. Compressive-tensile properties and hybrid effect of fiber reinforced self-compacting concrete. *J Anhui Univ Technol Nat Sci*, 2022; 39(3): 317 - 326.
- [25] Ma YD, Ma KZ, Liu BQ. Analysis of shear bearing capacity of hybrid steel fiber reinforced concrete deep beams. *J Harbin Inst Technol*, 2022; 54(3): 95 - 107.
- [26] Oettel V, Schulz M, Haist M. Empirical approach for the residual flexural tensile strength of steel fiber-reinforced concrete based on notched three-point bending tests[J]. *Structural Concrete*, 2022, 23(2): 993-1004. <https://doi.org/10.1002/suco.202100565>
- [27] Cunha V M C F, Barros J A O, Sena-Cruz J M. Pullout behavior of steel fibers in self-compacting concrete. *Journal of Materials in Civil Engineering*, 2010, 22(1): 1-9. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000001](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000001)
- [28] Guo YD, Liu YZ, Wang WJ, et al. Influence of basalt fiber characteristic parameters on uniaxial tensile properties of concrete[J]. *Acta Materiae Compositae Sinica*, 2023, 40(5): 2897-2912.
- [29] Yang L, Li JZ, Lou JF, et al. Study on fracture characteristics of coal samples based on notched semi-circular bending tests. *J Min Saf Eng*, 2022; 39(5): 1041 - 1050.
- [30] Deng HF, Zhu M, Li JL, et al. Study of mode-I fracture toughness and its correlation with strength parameters of sandstone. *Rock Soil Mech*, 2012; 33(12): 3585 - 3591.
- [31] Wang JJ, Zhu JG, Chiu CF, et al. Experimental study on fracture toughness and tensile strength of a clay. *Eng Geol*, 2007; 94(1-2): 65 - 75. <https://doi.org/10.1016/j.enggeo.2007.06.005>
- [32] Zhang ZX. An empirical relation between mode I fracture toughness and the tensile strength of rock. *Int J Rock Mech Min Sci*, 2002; 39(3): 401 - 406. [https://doi.org/10.1016/S1365-1609\(02\)00032-1](https://doi.org/10.1016/S1365-1609(02)00032-1)
- [33] Ding YH, Zou CL, Guo M, et al. Mechanical and fracture properties of steel fiber reinforced concrete with large amount of recycled aggregate. *Bull Chin Ceram Soc*, 2023; 42(7): 2532 - 2540.
- [34] Liu GH, Gan L, Liu Y, et al. Effect of different sand ratio and aggregate sizes on fracture toughness of plastic concrete. *Adv Sci Technol Water Resour*, 2024; 44(3): 39 - 44 + 87.

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Strengthening of fire-damaged reinforced concrete columns under pre loading

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Article Info	Abstract
Article History: Received 26 Dec 2025 Accepted 31 Mar 2026 Keywords: Fire exposure; Fire duration; Post-fire rehabilitation; Ultra high performance concrete; Reinforced concrete columns; Sustained eccentric loading	This paper presents an experimental investigation on the rehabilitation of fire-damaged reinforced concrete (RC) columns subjected to sustained eccentric loading using Ultra-High-Performance Concrete (UHPC) jacketing. The experimental program consisted of nine small-scale RC column specimens, including one unexposed control specimen and eight fire-exposed specimens subjected to temperatures of 500 °C and 700 °C for durations of 60 and 120 minutes. During fire exposure, a sustained eccentric axial load corresponding to 50% of the ultimate capacity was maintained to simulate service conditions. The results indicated that fire exposure significantly reduced the structural capacity of the columns. Compared with the control specimen, ultimate load reductions reached 11.65% and 14.11% at 500 °C, and 20.85% and 36.8% at 700 °C for 60 and 120 minutes, respectively. Following removal of the damaged concrete cover, the original cross-section was restored using a 20 mm UHPC jacket without increasing column dimensions. Strengthening resulted in substantial recovery and enhancement of load capacity, with recovery ratios ranging from 104.86% to 157.28% relative to the corresponding fire-damaged specimens. In addition to strength improvement, UHPC confinement significantly enhanced stiffness, ductility, and crack control performance. A clear transition in failure mode was observed after strengthening, shifting from compression-controlled crushing in fire-damaged columns to localized shear behavior in the corbel region under eccentric loading. The strengthening efficiency was more pronounced in severely fire-damaged specimens, highlighting the interaction between fire severity and confinement effectiveness. The findings demonstrate that UHPC jacketing is a highly effective rehabilitation technique for pre-loaded fire-exposed RC columns, providing both recovery of fire-induced degradation and structural enhancement beyond the original reference state.

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1. Introduction

Reinforced concrete (RC) columns are fundamental structural members designed primarily to resist compressive loads while sustaining bending moments caused by unavoidable load eccentricities in practical structural systems. The ultimate capacity of RC columns depends on the mechanical properties of concrete and reinforcing steel, cross-sectional geometry, and reinforcement configuration [1–6]. Under fire exposure, these governing parameters undergo severe degradation due to elevated temperatures, producing reductions in stiffness, cracking, bond deterioration, and possible spalling of the concrete cover.

Fire is one of the most critical accidental actions affecting structural systems. Elevated temperatures cause dehydration of cement hydrates, thermal incompatibility between aggregates and cement paste, and reduction in the yield strength and elastic modulus of reinforcing steel — all of which reduce the load-carrying capacity and serviceability of RC columns [7–10]. Recent studies

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have further shown that the residual response of fire-exposed RC columns is influenced not only by the peak temperature reached during fire exposure, but also by exposure duration, cooling regime, loading history, and the interaction between thermal and mechanical actions during the fire event [11,12].

In actual structural fire events, RC members generally remain subjected to sustained service loads while experiencing fire exposure. This combined action can intensify cracking, stiffness degradation, and residual strength loss, particularly when axial load is accompanied by load eccentricity [13]. Several studies have demonstrated that sustained axial pre-loading accelerates crack propagation and stiffness degradation under fire conditions, resulting in more pronounced residual strength losses compared with unloaded specimens [13]. However, most available experimental investigations have focused either on unloaded columns during fire exposure or on repaired members that were not subjected to sustained mechanical loading during fire exposure. As a result, the available experimental evidence remains limited for pre-loaded short RC columns exposed to fire under eccentric compression [12,13].

Post-fire rehabilitation of RC columns is a practical alternative to demolition when residual capacity is adequate and repair is economically favorable. Different strengthening methods have been proposed, including FRP wrapping, FRCM, steel jacketing, and concrete jacketing, but their effectiveness strongly depends on the severity of thermal damage, the geometry of the repaired cross-section, and the compatibility of the repair material with the existing concrete [14–16]. Contemporary research has increasingly focused on the use of advanced cementitious and composite materials to restore or enhance the behavior of damaged RC members after fire exposure, including FRCM-based systems, textile-reinforced mortar jackets, and ultra-high-performance repair materials [17,18]. Ultra-High-Performance Concrete (UHPC) has recently emerged as a highly promising repair material because of its dense matrix, very high compressive strength, improved tensile resistance through steel fibers, and excellent durability. For repair applications in compression-dominated members, UHPC provides not only high compressive strength but also improved tensile resistance and crack-bridging capability, which are important for limiting crack propagation and reducing deformation after fire-induced damage [17].

Nevertheless, a clear experimental gap exists in the literature regarding short RC columns subjected to sustained eccentric pre-loading during fire exposure followed by rehabilitation with a thin UHPC jacket. In particular, the combined influence of fire temperature (severity), exposure duration, and sustained eccentric loading on residual strength degradation, stiffness loss, failure mode transition, and the quantitative efficiency of UHPC confinement remains insufficiently documented [18,19]. Moreover, the distinction between post-fire strength recovery and capacity enhancement after rehabilitation has not always been clearly addressed in previous studies, despite its importance for practical repair assessment and decision-making [17–19]. Addressing this gap is essential to provide practical guidelines for repair versus replacement decisions and to calibrate design/assessment methods for post-fire rehabilitation.

Accordingly, this study presents a systematic experimental program on pre-loaded short RC columns subjected to two fire severities (500 °C and 700 °C) and two exposure durations (60 and 120 minutes) under sustained eccentric loading. After controlled fire exposure and cooling, damaged concrete cover was removed and a 20 mm UHPC jacket was applied to selected specimens. The study evaluates the effectiveness of UHPC strengthening in terms of ultimate load capacity, load–displacement response, stiffness degradation, crack development, failure modes, and post-fire ductility. Unlike many previous studies that considered either unloaded fire exposure or conventional rehabilitation schemes, the present work examines the combined effects of sustained eccentric pre-loading, fire exposure severity, and thin UHPC jacketing within one experimental framework. The outcomes provide quantitative insight into the interaction between thermal damage severity and UHPC confinement efficiency for pre-loaded columns, thereby offering practical recommendations for rehabilitation of fire-damaged RC members under service loads.

2. Materials and Mix Proportions

2.1. Material

The materials used in this study are described below under the following headings.

2.1.1. Normal Strength Concrete (NSC)

All column specimens were cast using Normal Strength Concrete (NSC) designed to achieve a target 28-day compressive strength of 32 MPa. Ordinary Portland Cement (Type I) conforming to ASTM specifications was used as the binder. The mix proportions were 390 kg/m³ of cement, 685 kg/m³ of natural sand, and 1075 kg/m³ of crushed gravel with a maximum aggregate size of 14 mm. The water-to-cement ratio (w/c) was 0.47. The compressive strength was determined in accordance with ASTM C39 using standard cylindrical specimens with dimensions of 150 × 300 mm. All NSC specimens were cured in water for 28 days prior to testing.

2.1.1.1. Steel

Three types of steel reinforcing bars were used in the experimental program. Deformed Ø10 mm bars (Al-Mass production) served as longitudinal reinforcement in all column specimens. Deformed Ø12 mm bars (Al-Mass production) were used to reinforce all corbels, while Ø6 mm deformed bars (Turkish production) were employed as transverse ties in both the corbels and the column specimens.

2.1.2 Ultra-High-Performance Concrete (UHPC)

- Cement: Type V Portland cement was selected based on trial mixtures to achieve an optimal balance between compressive strength and workability.
- Fine Sand: Locally sourced natural sand, sieved through a 0.6 mm mesh, was used as the fine aggregate for UHPC in accordance with prior recommendations.
- Silica Fume (SF): Silica fume, a by-product of silicon and ferrosilicon alloy production, consists of particles much finer than cement. It reacts with calcium hydroxide to form C-S-H and fills micro-voids, thereby reducing porosity and improving the strength and durability of the concrete.
- Superplasticizer (SP): A carboxylic ether-based high-range water-reducing admixture, MasterGlenium 54 (BASF), was used to produce UHPC and obtain the desired workability. A dosage of 3.5% by weight of cementitious materials was adopted based on trial mixes.
- Steel Fibers: Copper-coated straight micro steel fibers (13 mm length, 0.2 mm diameter, aspect ratio 65, tensile strength 3005 MPa) were added to the UHPC at a volume fraction of 2%. The fibers comply with ASTM A820 requirements.

2.1.2.1 Concrete Mix Design

The mixing procedure and material proportions of the UHPC were developed and optimized after modifications to previous trial mixes. A total of eight trial mixes were conducted to identify the most suitable mixture achieving the required fresh and hardened properties.

Table 1. Mix proportions of UHPC

Constituents	The proportion by weight (kg/m ³)
Cement	950
Fine sand	1050
Silica fume	190 ^a
Water	195 ^b
Superplasticizer	40 ^c
Micro steel fiber	157 ^d

^a silica fume=20% of cement weight, ^b w/b =17.1%, ^c sp/ (cement + silica fume) = 3.5% and ^d micro steel fiber=2% of total volume

Steel fibers were then incorporated and mixed for 3–5 minutes to ensure uniform distribution. The flowability of the selected mix was evaluated in accordance with ASTM C230 and ASTM C1437,

resulting in a spread of 230 to 240 mm, which complies with the requirements of ASTM C1856. The optimized UHPC mix achieved a measured 28-day compressive strength of 135.7 MPa, determined in accordance with ASTM C109 using 50 mm cube specimens. The constituent materials and their corresponding weight proportions are presented in Table 1.

3. Specimens Details

All test specimens were fabricated with identical external geometry and configuration. The columns had a constant square cross section with dimensions of 150 × 150 mm and an overall height of 1400 mm. Two corbels were provided on each column, with a clear distance of 800 mm between their central axes. Each corbel measured 150 × 250 × 300 mm, and their primary function was to facilitate the application of an eccentric load to the column. Each column was constructed with a uniform concrete cover thickness of 20 mm and reinforced longitudinally with four deformed steel bars of 10 mm diameter, corresponding to a reinforcement ratio of $\rho = 0.0140$. Transverse reinforcement was provided in the form of steel ties with a diameter of 6 mm, spaced at 100 mm intervals along the column height. All columns were designed and detailed in accordance with the provisions of (ACI Code 318-25). The details of the column geometry and corbel reinforcement arrangement are illustrated in (Fig. 1). The experimental program comprised a total of nine column specimens cast using normal-strength concrete. Among these, one specimen (C₁) served as the control column and was neither preloaded nor exposed to fire. The remaining eight columns were subjected to fire exposure while carrying a pre-applied axial load equal to 50% of the ultimate load capacity (P_u). In the present experimental investigation, the specimens were categorized into two main groups, which are described in the following sections.

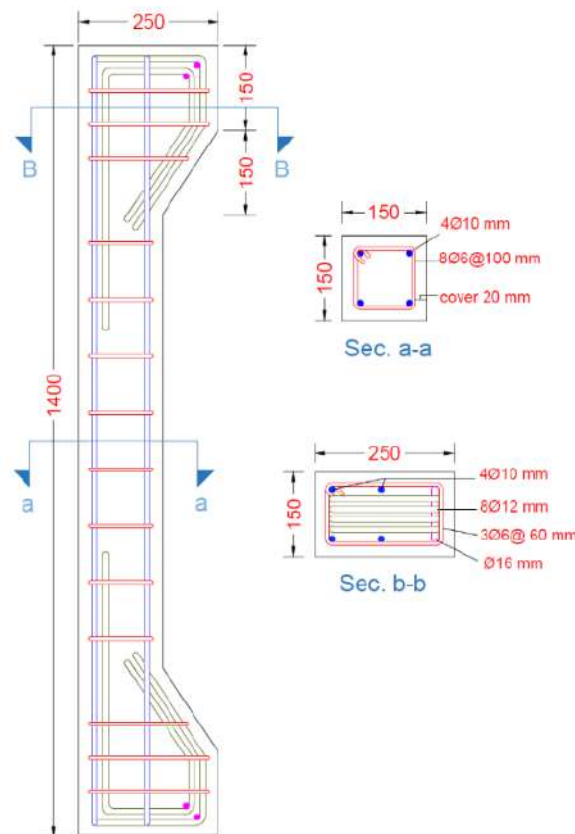


Fig. 1. Layout of reinforcement in the tested columns

The first group consists of four columns (C₂ to C₅). Columns C₂ and C₃ were exposed to fire at a temperature of 500 °C for 60 minutes. Column C₃ was subsequently rehabilitated after removing the damaged concrete cover and replacing it with Ultra-High-Performance Concrete (UHPC). Columns C₄ and C₅ were exposed to fire at a temperature of 500 °C for 120 minutes, and similarly, column C₅ was repaired in the same manner as column C₃. The second group also consists of four columns (C₆ to C₉), and its testing program was identical to that of the first group in all parameters,

except that the fire exposure temperature was 700 °C. It is worth noting that all eight columns from both groups were subjected to a pre-loading level equal to 50% (81.5 kN) of the ultimate load (P_u) during the fire exposure, with a constant load eccentricity of ($e = 90$ mm; $e/h = 0.6$) for all specimens. Table 2 presents all the details of the column groups.

Table 2. Details of the examined specimens

Group No.	Specimen Symbol (Ci)	Fire exposure (Ti) (°C)	Fire duration (Di) (min)	Repair of fire damaged specimen (R)
Control	C ₁	-	-	-
Group One	C ₂ T ₅₀₀ D ₆₀	500	60	-
	C ₃ T ₅₀₀ D ₆₀ R	500	60	R
	C ₄ T ₅₀₀ D ₁₂₀	500	120	-
	C ₅ T ₅₀₀ D ₁₂₀ R	500	120	R
Group Two	C ₆ T ₇₀₀ D ₆₀	700	60	-
	C ₇ T ₇₀₀ D ₆₀ R	700	60	R
	C ₈ T ₇₀₀ D ₁₂₀	700	120	-
	C ₉ T ₇₀₀ D ₁₂₀ R	700	120	R

4. Casting Procedures

A central batching mixer with a capacity of 10 m³, supplied by Al-Mustaqbal Ready-Mix Concrete Company, was used for concrete mixing in this project. To prevent adhesion of hardened concrete, the internal surfaces of the cube and cylinder molds were thoroughly cleaned and properly lubricated prior to casting. Subsequently, each steel reinforcement cage designated for the column specimens was placed horizontally inside the wooden formwork and securely fixed in position. Concrete was poured into each mold in a single layer, followed by compaction and vibration using an internal vibrator. After casting, the specimens were water-cured for a period of 28 days. The casting and curing procedures are illustrated in (Fig. 2).



Fig. 2. Casting process phases

5. Fire Test

The columns were exposed to fire several months after casting. A brick furnace with internal dimensions of 1400 × 1400 × 1150 mm was employed for the fire exposure tests, as shown in (Fig. 3). The furnace was programmed to follow the ASTM E119-88 standard temperature–time curve throughout the fire exposure period. Two exposure durations were considered, namely 60 and 120 minutes. According to the ASTM E119-88 curve, these exposure times correspond to furnace temperatures of approximately 500 °C and 700 °C, respectively, as illustrated in (Fig. 4). The

objective was to evaluate the residual structural performance of pre-loaded reinforced concrete columns at specific stages along the standard fire curve, representing moderate and severe fire conditions.

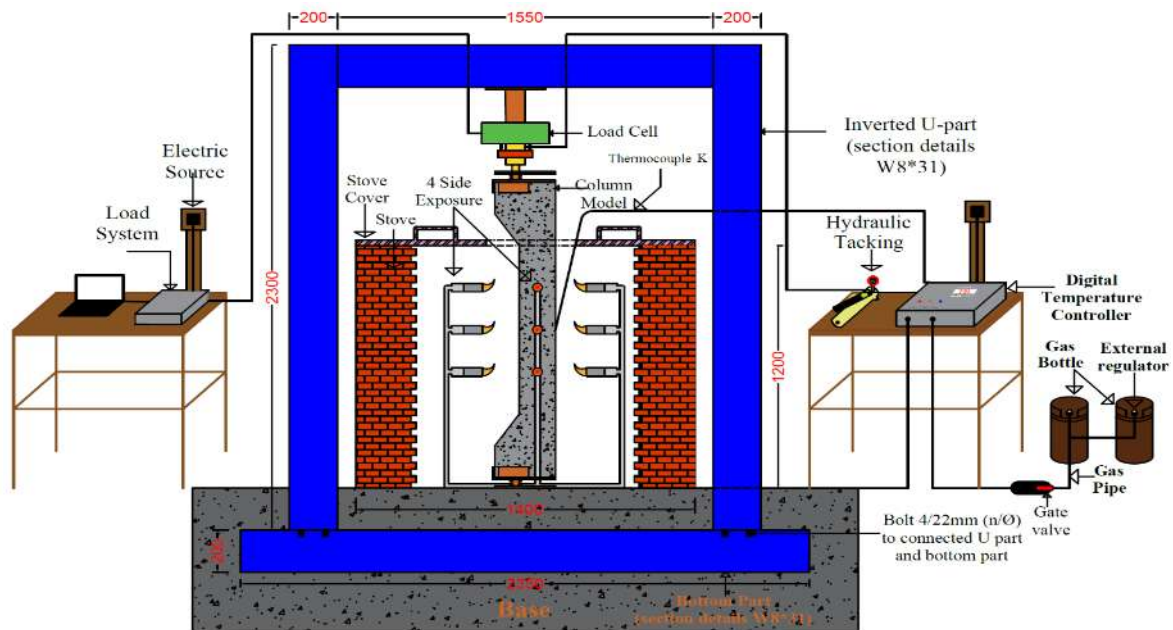


Fig. 3. Description of the loading frame

The furnace temperature was controlled using a digital controller connected to a gas regulator, which automatically regulated fuel supply based on real-time sensor feedback. Temperature measurements were recorded using 4 mm diameter K-type thermocouples installed on the external surface of each column at mid-height within the heated region. The thermocouples were connected to a data acquisition system to ensure continuous temperature monitoring during fire exposure. The recorded temperature–time history confirmed that the furnace followed the ASTM E119-88 curve within the operational tolerance of the testing system. The fire-exposed length of each column was approximately 800 mm, as illustrated in (Fig. 5). After completion of the designated exposure duration, the furnace cover was removed, and the specimens were allowed to cool naturally under ambient laboratory conditions.

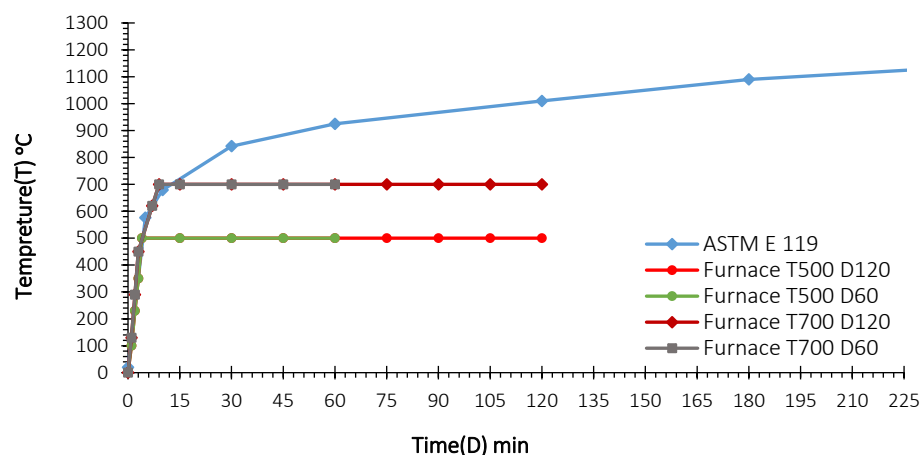


Fig. 4. Standard fire exposure according to ASTM

Column C_1 served as the unexposed control specimen. Columns C_2 to C_9 were subjected to a sustained eccentric axial load of 81.5 kN at an eccentricity of 90 mm prior to and during fire exposure, corresponding to 50% of the ultimate capacity determined from preliminary testing of the control column. The load was applied using a calibrated hydraulic jack and maintained

throughout the fire exposure period to simulate sustained service loading conditions during a fire event. The applied load was continuously monitored during fire exposure to ensure stability of the load magnitude. The loading frame was designed to preserve the predefined eccentricity throughout the fire exposure phase, minimizing potential load redistribution due to fire-induced material degradation and deformation.



Fig. 5. Specimens subjected to fire exposure under pre-loading curve

6. Repair of Fire-Damaged Column Specimens

The rehabilitation of columns exposed to fire (C₃, C₅, C₇, and C₉) involved the following key steps:

- **Removal of Damaged Concrete:** The deteriorated outer concrete cover was carefully removed manually until the longitudinal reinforcement was fully exposed. The average removal depth was approximately 20 mm, corresponding to the original concrete cover thickness. Mechanical methods were avoided to prevent dynamic vibrations that could weaken the elements. The removal depth was kept consistent for all repaired specimens to ensure uniform rehabilitation conditions.
- **Installation of Shear Connectors:** 4 mm-diameter shear connectors were horizontally fixed on all four sides of the stirrups to ensure a reliable bond between the existing concrete core and the new UHPC outer layer.





Fig. 6. Sequential steps of RC column repair

- **Epoxy Preparation and Application:** The epoxy resin (Sikadur ®-32 LP) was prepared by separately stirring the base and hardener, then mixing them thoroughly with a slow-speed drill for 2 minutes until a uniform color was achieved. The cleaned surface of the existing concrete core was coated with epoxy to enhance bonding between the old concrete and the UHPC layer.
- **Casting of UHPC Layer:** The removed concrete cover was replaced using Ultra-High-Performance Concrete (UHPC) with a uniform thickness of 20 mm, restoring the original cross-sectional dimensions of the columns (150 × 150 mm). No enlargement of the column cross-section was introduced during rehabilitation. All steps are illustrated in (Fig. 6).

7. Test Setup and Procedure

The specimens were tested using a hydraulic universal testing machine with a maximum capacity of 2500 kN under monotonic eccentric axial loading. The applied load was recorded through a calibrated load cell connected to a data acquisition system. Two Linear Variable Differential Transformers (LVDTs) were used to measure the column deformations. One LVDTs was installed at the mid-height of the column to measure lateral deflection due to eccentric loading. The second LVDTs was positioned at the base of the column to measure axial shortening. The load-displacement response was continuously recorded until failure. Crack development, failure mode, and ultimate load were carefully observed and documented, as illustrated in (Fig. 7).

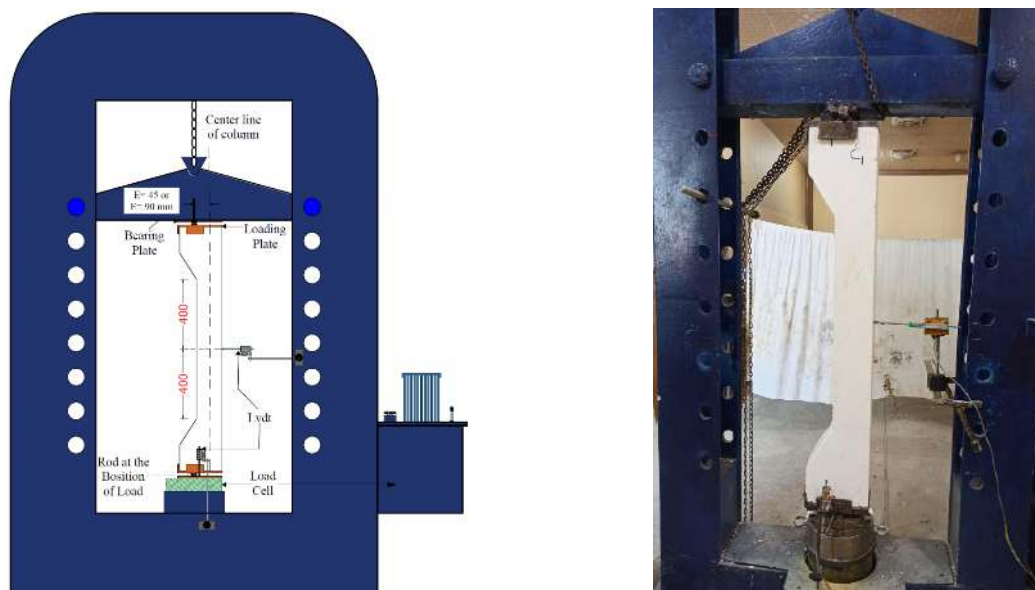


Fig. 7. Test setup with instrumentation details

8. Results and Discussion

The test results obtained for each column were compared with those of the other specimens to evaluate the influence of fire duration and intensity while maintaining constant load stability and eccentricity. The examined parameters included failure mode, ultimate load capacity, first cracking load, and axial deformation of the column specimens, as summarized in Table 3.

Table 3. Results of laboratory testing on column specimens

Group No.	Specimen Symbol (Ci)	Ultimate load capacity kN	Percentage Decreasing Load Carrying Capacity (%)	Ultimate axial deformation (mm)	Ultimate mid-height lateral deflection(mm)
Control	C ₁	163	0	9.3	13.85
Group One	C ₂ T ₅₀₀ D ₆₀	144	-11.65	8.27	14.67
	C ₃ T ₅₀₀ D ₆₀ R	295	+80.98	10.25	19.36
	C ₄ T ₅₀₀ D ₁₂₀	140	-14.11	8.67	15.14
	C ₅ T ₅₀₀ D ₁₂₀ R	281	+72.39	10.56	15.95
Group Two	C ₆ T ₇₀₀ D ₆₀	129	-20.85	10.34	16.42
	C ₇ T ₇₀₀ D ₆₀ R	284	+74.23	10.32	14.78
	C ₈ T ₇₀₀ D ₁₂₀	103	-36.8	11.42	18.21
	C ₉ T ₇₀₀ D ₁₂₀ R	265	+62.57	10.5	15.04

8.1. Load Carrying Capacity

The ultimate load-carrying capacity of each column specimen was taken as the maximum applied load recorded during testing. The unexposed control specimen (C₁) was used as the reference for assessing fire-induced capacity loss and for distinguishing post-fire strength recovery from capacity enhancement after rehabilitation.

Fire exposure caused a clear reduction in the load-carrying capacity of the tested columns. Relative to the control specimen (C₁), specimens C₂T₅₀₀D₆₀ and C₄T₅₀₀D₁₂₀, exposed to 500 °C for 60 and 120 min, exhibited reductions of (11.65% and 14.11%), respectively. Greater reductions were observed at 700 °C, where specimens C₆T₇₀₀D₆₀ and C₈T₇₀₀D₁₂₀ showed decreases of (20.85% and 36.8%), respectively. These results indicate that increasing fire exposure severity, in terms of both temperature and duration, led to progressive deterioration in the residual axial capacity of the pre-loaded columns.

After removal of the fire-damaged concrete cover and application of the 20 mm UHPC jacket, the rehabilitated specimens exhibited substantial strength recovery relative to their corresponding fire-damaged counterparts. The load-carrying capacities of specimens C₃T₅₀₀D₆₀R and C₅T₅₀₀D₁₂₀R increased by (104.86% and 100.71%), respectively, compared with specimens C₂T₅₀₀D₆₀ and C₄T₅₀₀D₁₂₀. Likewise, specimens C₇T₇₀₀D₆₀R and C₉T₇₀₀D₁₂₀R showed increases of (120.16% and 157.28%), respectively, relative to specimens C₆T₇₀₀D₆₀ and C₈T₇₀₀D₁₂₀, as shown in Fig. 8.

When compared with the unexposed control specimen (C₁), the rehabilitated columns demonstrated capacity enhancement rather than only recovery. The load-carrying capacities of specimens C₃T₅₀₀D₆₀R and C₅T₅₀₀D₁₂₀R exceeded that of C₁ by (80.98% and 72.39%), respectively, while specimens C₇T₇₀₀D₆₀R and C₉T₇₀₀D₁₂₀R exceeded the control capacity by (74.23% and 62.57%), respectively, as illustrated in Fig. 8. This distinction is important because the rehabilitated specimens did not merely regain the lost post-fire capacity, but developed higher ultimate resistance than the original unexposed column.

The strengthening effect of UHPC was more pronounced in the severely fire-damaged specimens, particularly specimen C₉T₇₀₀D₁₂₀R, which exhibited the highest improvement ratio. This behavior may be attributed to the greater degradation of the original concrete after severe fire exposure, which reduced the contribution of the residual core and increased the structural role of the UHPC

jacket in carrying load and providing confinement. Accordingly, the beneficial effect of UHPC became more evident as the level of fire-induced damage increased.

The increase in load-carrying capacity was accompanied by a clear change in the governing failure mode. Whereas the control and fire-damaged specimens failed mainly by concrete crushing and cover spalling, the UHPC-strengthened specimens showed a transition toward shear-dominated failure in the corbel region. Under eccentric loading ($e = 90$ mm), the increase in axial capacity after rehabilitation produced a corresponding increase in bending moment and associated local shear demand. Because the internal reinforcement detailing in the corbel region remained unchanged after rehabilitation, the shear resistance of that region did not increase in proportion to the enhanced compressive resistance provided by the UHPC jacket. As a result, once global compressive failure was delayed, localized shear in the corbel region became the governing limit state. Overall, the results demonstrate that UHPC jacketing is highly effective in restoring and enhancing the load-carrying capacity of pre-loaded fire-damaged RC columns, while also altering the structural response and shifting the failure hierarchy.

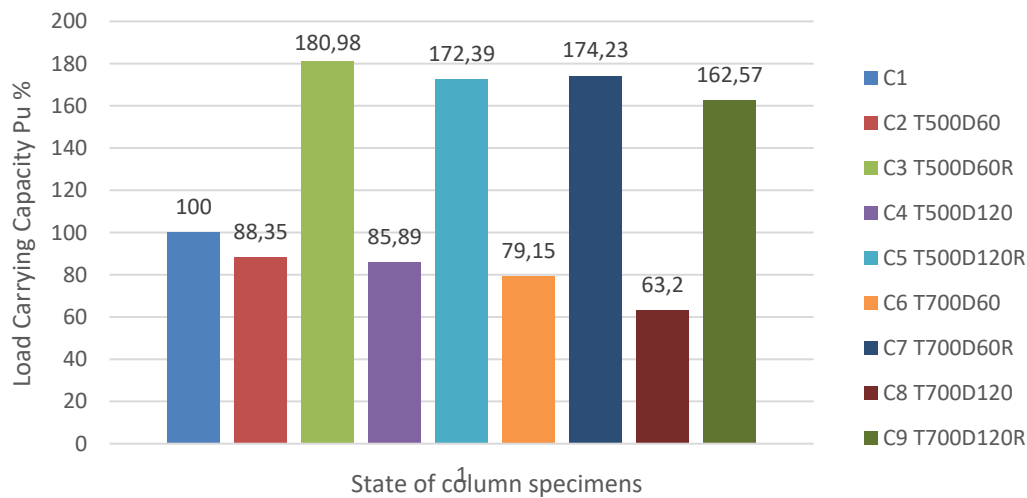
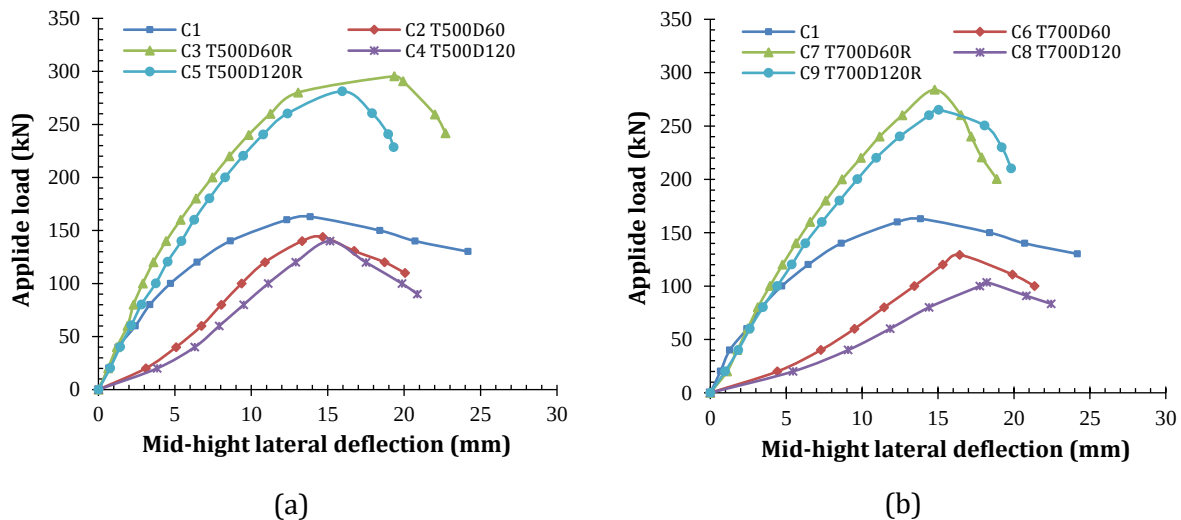


Fig. 8. Percentage of load carrying capacity for the tested column specimens

8.2. Load-Displacements Relationship of Reinforced Concrete Columns

As shown in Figs. 9a–f, the mid-height lateral deflection of the columns was noticeably influenced by the fire exposure duration. The comparison also indicates that the load-displacement curves of the second group were more sensitive to elevated temperature than those of the first group. This behavior can be attributed to the reduction in the effective cross-sectional area caused by cracking, as well as to the decrease in column stiffness resulting from the degradation of the concrete elastic modulus after fire exposure.



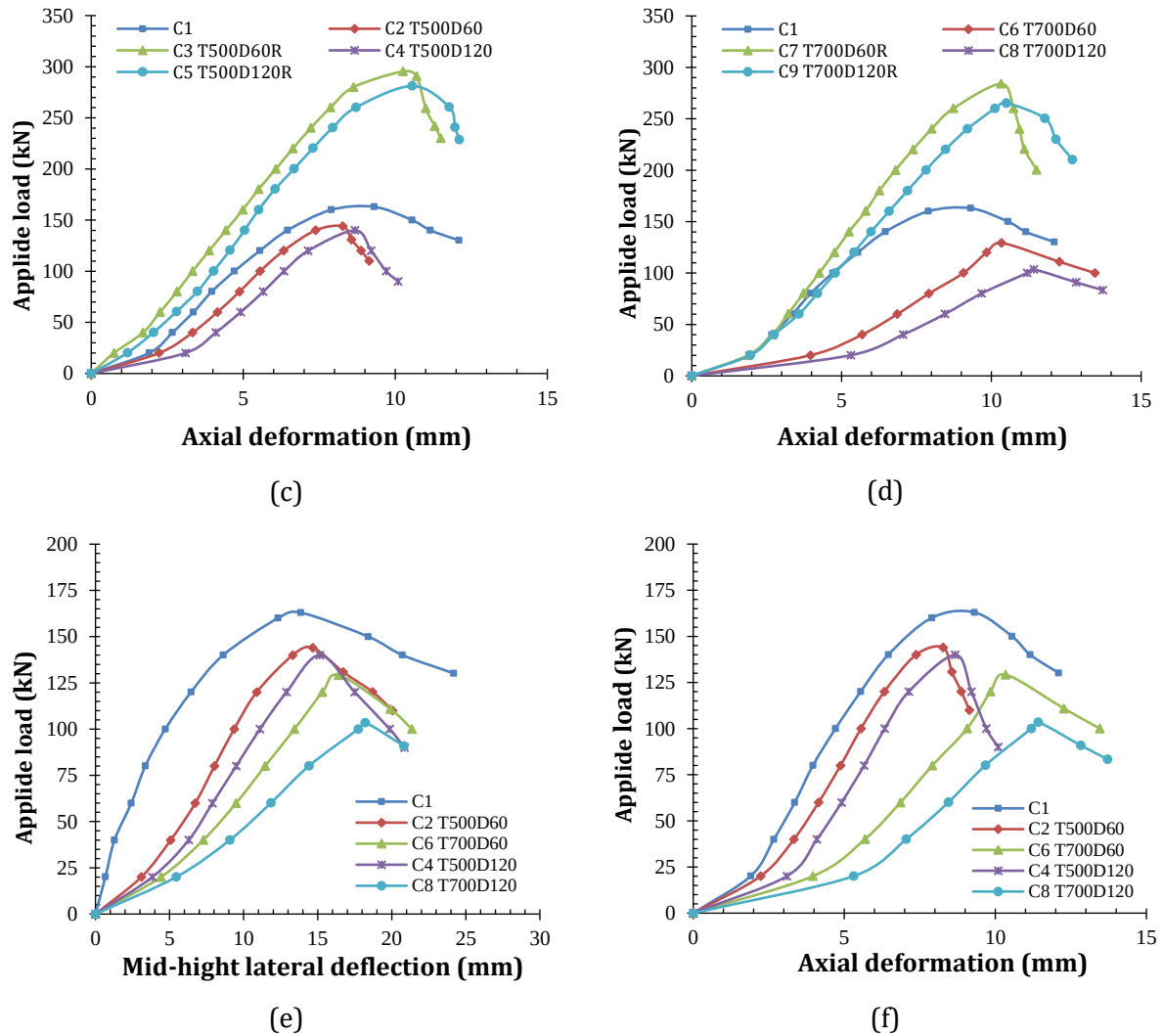


Fig. 9. Load-displacement of Columns for each group

In addition, rehabilitation with Ultra-High-Performance Concrete (UHPC) improved the overall deformation response of the fire-damaged columns under loading. This improvement can be mainly attributed to the high compressive strength and elastic modulus of UHPC, which increased the stiffness of the repaired columns, while the presence of steel fibers contributed to restricting crack propagation and improving deformation control.

8.3. First Crack Load

The control reinforced concrete columns exhibited no visible cracking during the initial stages of loading. Crack widths were measured using a crack gauge during mechanical testing. In eccentrically loaded specimens, transverse flexural cracks typically initiated in the tensile zone and propagated toward the compression zone, whereas longitudinal cracks generally developed as shear cracks near the corbel region. In the fire-exposed specimens, very fine hairline cracks were already present prior to mechanical testing as a result of sustained eccentric pre-loading combined with fire exposure. This indicates that damage had developed during the fire stage. Therefore, a clearly defined first-crack load during post-fire mechanical testing was not applicable for these specimens.

Table 4 presents the maximum crack width recorded for each individual specimen after fire exposure and at the maximum applied load prior to failure. In addition to the specimen-level values, the mean maximum crack width was calculated for each group to evaluate the influence of fire exposure and UHPC rehabilitation.

The control specimen exhibited a crack width of 0.40 mm at the peak load stage. For the fire-exposed specimens, the mean maximum crack width reached 0.44 mm after fire exposure and 0.69 mm at the maximum applied load prior to failure. In contrast, the rehabilitated specimens showed reduced mean maximum crack widths of 0.49 mm after fire exposure and 0.38 mm at the peak load stage, reflecting the crack-bridging and confinement effect provided by the UHPC jacket.

Table 4. Crack width of the columns

Group No.	Specimen Symbol (Ci)	Maximum Crack width After exposure to fire (mm)	Crack width at service load (mm)	Location of crack
Control	C ₁	-	0.4	In the last quarter of the column
Group One	C ₂ T ₅₀₀ D ₆₀	0.16	0.62	In the middle of the column
	C ₃ T ₅₀₀ D ₆₀ R	0.18	0.32	In the middle of the column
	C ₄ T ₅₀₀ D ₁₂₀	0.4	0.66	In the last quarter of the column
	C ₅ T ₅₀₀ D ₁₂₀ R	0.34	0.36	In the middle of the column
Group Two	C ₆ T ₇₀₀ D ₆₀	0.48	0.68	In the first quarter of the column
	C ₇ T ₇₀₀ D ₆₀ R	0.56	0.4	In the middle of the column
	C ₈ T ₇₀₀ D ₁₂₀	0.74	0.8	In the middle of the column
	C ₉ T ₇₀₀ D ₁₂₀ R	0.9	0.44	In the last quarter of the column

8.4. Failure Mode

All column specimens were tested under an eccentric axial load with an eccentricity of 90 mm. Under this loading condition, the control specimen (C₁) exhibited a typical compression-controlled failure mode. Cracks initiated in the tension zone and gradually propagated toward the compression side. Failure occurred through progressive crushing and spalling of the concrete cover in the compression region, followed by local instability of the longitudinal reinforcement, as shown in Fig. 10.

For the fire-exposed unstrengthened specimens (C₂T₅₀₀D₆₀, C₄T₅₀₀D₁₂₀, C₆T₇₀₀D₆₀, and C₈T₇₀₀D₁₂₀), pre-existing cracks induced by sustained eccentric loading during fire exposure were clearly visible before the mechanical test. Crack propagation started at lower load levels than in the control specimen. As the fire exposure severity increased, particularly with higher temperature and longer duration, cracking became more extensive and cover spalling occurred earlier. Nevertheless, failure remained primarily governed by compressive crushing and detachment of the concrete cover in the compression zone, as shown in Fig. 10. In contrast, the UHPC-strengthened specimens exhibited a clear transition in failure mode. The 20 mm UHPC jacket improved confinement and delayed crushing in the compression zone, thereby suppressing the global compression-controlled failure observed in the control and fire-damaged specimens. As a result, failure shifted toward localized shear in the corbel region, as illustrated in Fig. 11.

Although the corbel region was not directly exposed to fire, it was affected by heat transfer from the exposed portion of the column. Owing to the thermal retention characteristics of concrete, the temperature increase in this region may have caused localized material deterioration and a reduction in shear resistance. Under sustained eccentric loading ($e = 90$ mm), the increase in axial load capacity after UHPC strengthening also produced higher bending moments and associated local shear demand in the corbel region. Since the internal reinforcement detailing of the corbel was not modified during rehabilitation, its shear resistance did not increase in proportion to the enhanced compressive resistance provided by the UHPC jacket. Consequently, shear failure in the corbel region became the governing limit state.

An exception was observed in specimen (C₃T₅₀₀D₆₀R), which was exposed to the lower temperature and shorter fire duration. This specimen failed by compression spalling in the middle third of the column, similar to the behavior of the unstrengthened specimens, as shown in Fig. 11.

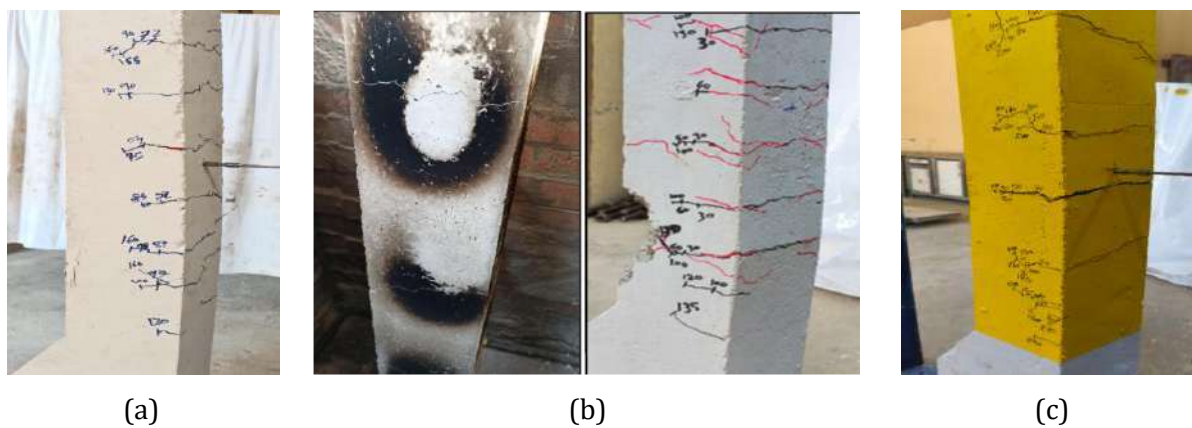
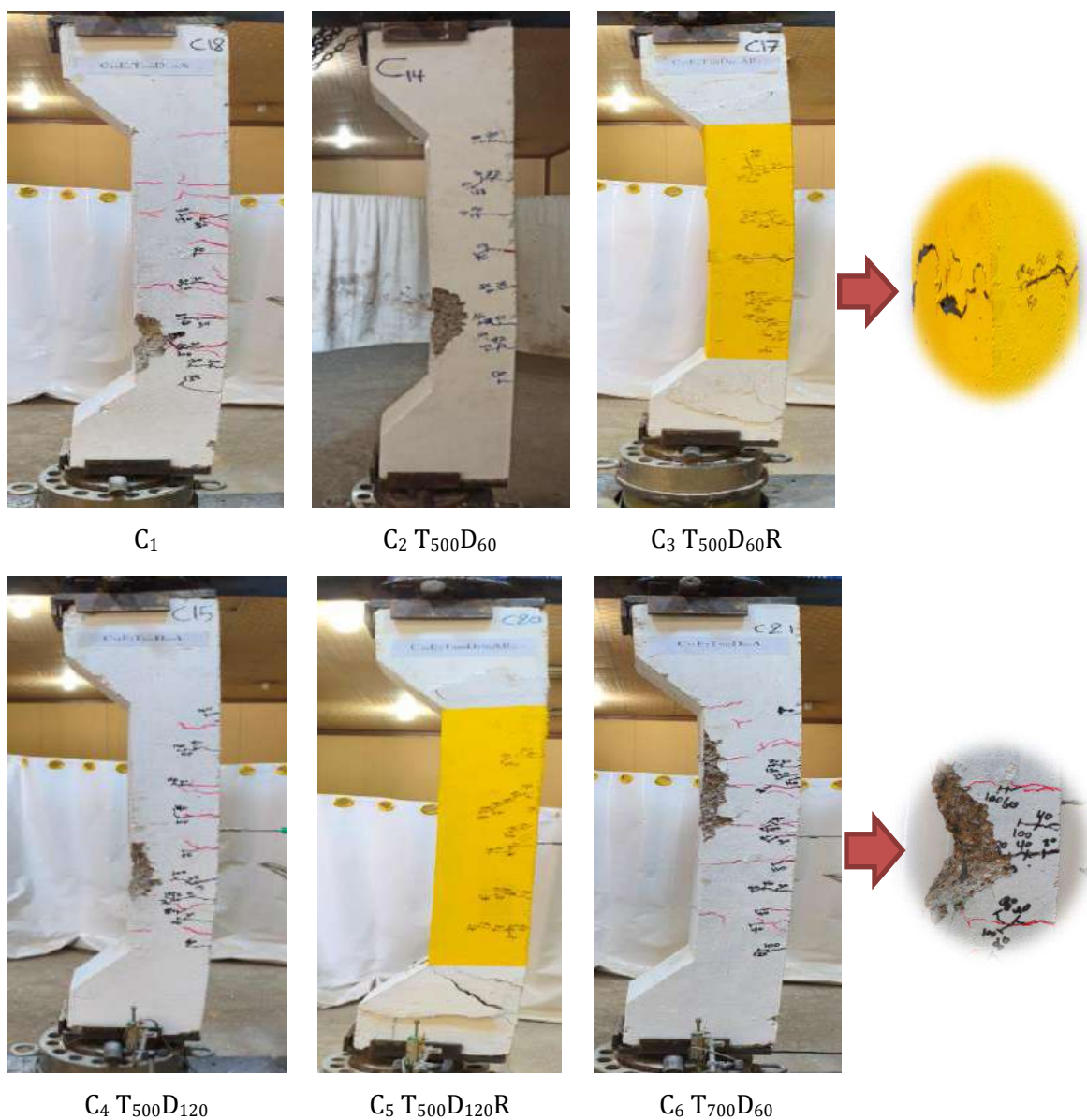


Fig. 10. Spread of crack in specimens, (a) Column specimens without fire, (b) Column specimens with fire exposure and (c) Repaired column specimen



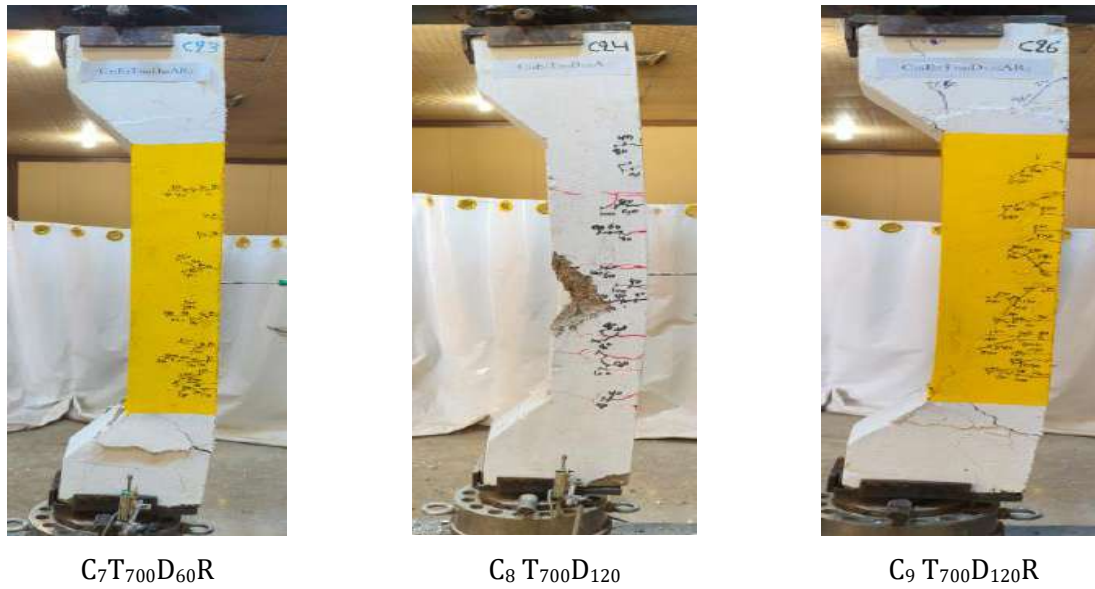


Fig. 11. Failure modes of columns after testing

8.5. Ductility

To evaluate the ductility of the investigated columns, this study adopted the Energy Absorption Capacity method proposed by Kumar et al. [20]. The energy absorption capacity of a concrete column is defined as the area enclosed beneath the load–displacement curve up to the maximum load. This method is considered an appropriate indicator for assessing the post-fire ductility of concrete columns, as it accounts for the reduction in load-carrying capacity resulting from exposure to elevated temperatures. (Fig. 12) illustrates the calculated areas under the load–displacement curves, which represent the corresponding ductility values.

In comparison with the control specimen, the results clearly indicate that the ductility of fire-exposed columns was significantly enhanced when strengthened with Ultra-High-Performance Concrete (UHPC). Specifically, UHPC strengthening increased the ductility of columns exposed to a temperature of 500 °C for durations of 60 and 120 minutes by 160.22% and 81.65%, respectively. Similarly, columns exposed to 700 °C for 60 and 120 minutes exhibited ductility improvements of 55.83% and 48.23%, respectively. These findings unequivocally demonstrate the effectiveness of UHPC in enhancing the post-fire ductile behavior of reinforced concrete columns by increasing their energy absorption capacity and improving their resistance to sudden failure.

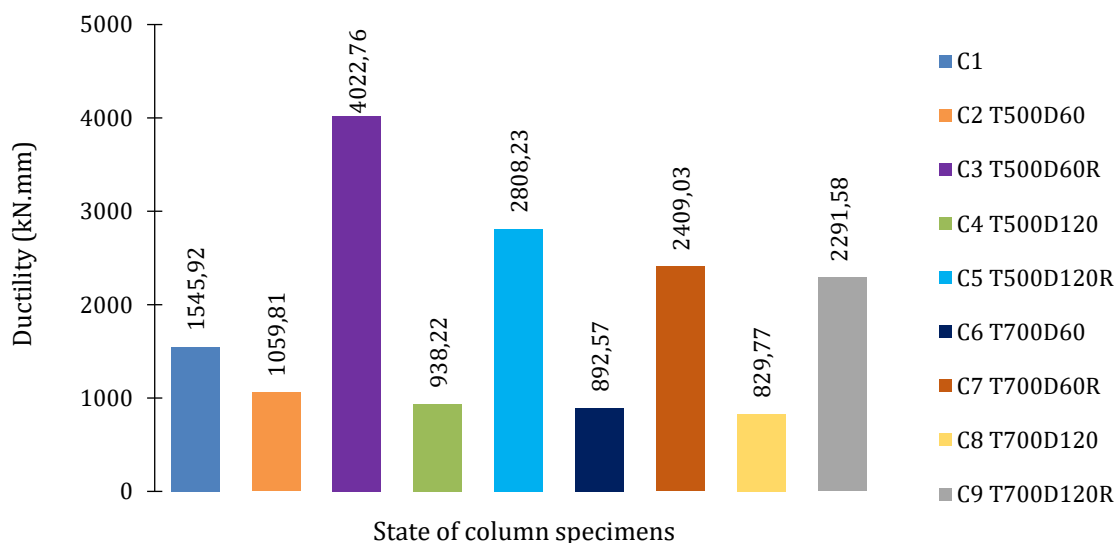


Fig. 12. The ductility of tested specimens

8.6. Stiffness Parameter

Stiffness is defined as the amount of load required to produce a unit deformation in a structural element. One common method to determine stiffness is by using the secant slope drawn on the load–displacement curve at a point corresponding to 75% of the ultimate load, as referenced in [21]. As illustrated in (Fig. 13), the stiffness of each column was calculated and compared with that of the reference column.

The results indicate that stiffness significantly decreases with increasing temperature and exposure duration following fire. At exposure temperatures of 500°C and 700°C, reductions in stiffness of 35.12% and 54.42% were recorded, respectively, accompanied by a decrease in load-carrying capacity. Conversely, the application of Ultra-High-Performance Concrete (UHPC) resulted in a notable increase in load-bearing capacity and a substantial improvement in column stiffness, with increases of 34.33% and 17.6% at 500°C and 700°C, respectively.

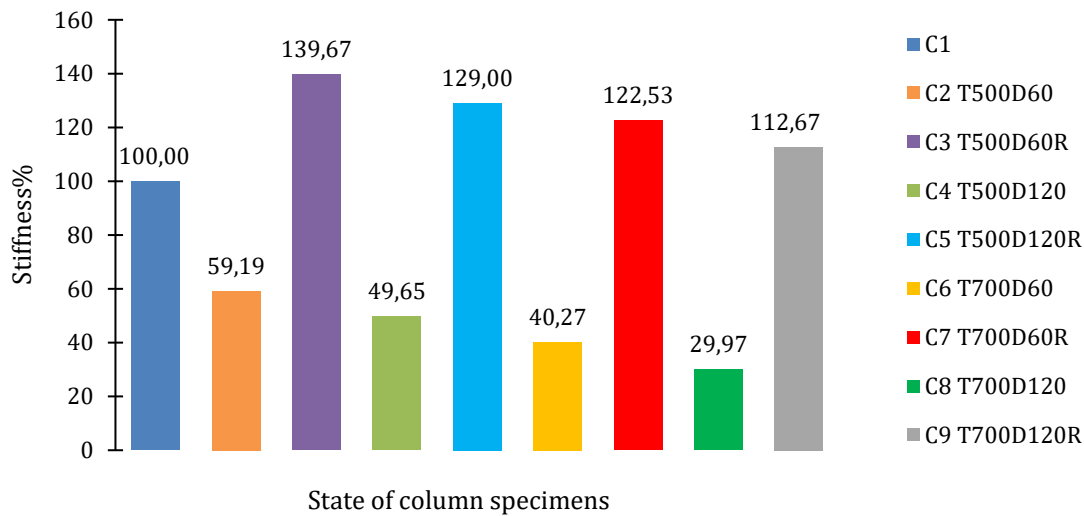


Fig. 13. Stiffness of tested specimens

9. Conclusions

The following conclusions are drawn from the experimental investigation of pre-loaded reinforced concrete columns subjected to controlled fire exposure and subsequently rehabilitated using Ultra-High-Performance Concrete (UHPC) jacketing:

- Fire exposure caused a clear reduction in the load-carrying capacity of the tested columns. Relative to the unexposed reference specimen, the ultimate load decreased by 11.65% and 14.11% after exposure to 500 °C for 60 and 120 min, respectively, whereas larger reductions of 20.85% and 36.8% were recorded at 700 °C for the same durations. These results confirm that both fire temperature and exposure duration had a significant influence on the residual capacity of the pre-loaded columns under sustained eccentric loading.
- Sustained eccentric pre-loading during fire exposure accelerated crack development and stiffness degradation. Increasing fire severity led to earlier crack initiation, more extensive crack propagation, and lower stiffness compared with the control specimen, indicating the combined adverse effect of mechanical loading and thermal damage.
- After removal of the damaged concrete cover, the application of a 20 mm UHPC jacket markedly improved the structural capacity of the fire-damaged columns. Compared with the corresponding fire-damaged specimens, the rehabilitated columns showed increases in ultimate load of 104.86% and 100.71% for specimens exposed to 500 °C for 60 and 120 min, respectively, and 120.16% and 157.28% for specimens exposed to 700 °C for 60 and 120 min, respectively.
- Compared with the unexposed reference specimen, the rehabilitated columns exceeded the original load-carrying capacity, indicating that UHPC jacketing provided not only recovery of

the fire-induced strength loss but also additional capacity enhancement through improved confinement and stress redistribution within the section.

- UHPC strengthening also improved the post-fire stiffness and ductility of the tested columns. The stiffness recovery reached up to 34.33% at 500 °C and 17.6% at 700 °C, while the ductility improvement reached up to 160.22% and 55.83% at 500 °C and 700 °C, respectively. These improvements reflect the beneficial role of the dense UHPC matrix and steel fibers in enhancing deformation control and crack resistance.
- Fire exposure altered the governing failure behavior from compression-controlled crushing in the reference specimen to more brittle damage characterized by concrete cover spalling and reinforcement instability in the fire-damaged columns. After UHPC rehabilitation, the failure mode shifted toward localized shear in the corbel region, indicating that the UHPC jacket delayed compression failure and changed the internal force transfer mechanism of the rehabilitated columns.

References

- [1] Kodur VKR, Banerji S. Comparative Fire Behavior of Reinforced Concrete Beams Made of Different Concrete Strengths. *Fire Technol* 2024;60:3117–46. <https://doi.org/10.1007/s10694-023-01375-x>.
- [2] Ali Ahmad A, Abdul-Rahman Alkhazraji F, Mohammed Omar S. Experimental study of Circular Short Columns made from Reactive Powder Concrete. *Eng Technol J* 2016;34:1860–72. <https://doi.org/10.30684/etj.34.9A.12>.
- [3] Sarsam K, Khalel R, Hadi M. Influence of Spirals on the Behavior of Short RC Columns Strengthened by External CFRP. *Eng Technol J* 2018;36:223–33. <https://doi.org/10.30684/etj.36.2A.15>.
- [4] Mohammed Z, Muhammad B, Resheq A. Slenderness effect on the behavior of composite RC columns under eccentric loads. *Eng Technol J* 2024;24 (11) (2:1–10. <https://doi.org/10.30684/etj.2024.147078.1702>.
- [5] Chen Q, Jiang Y. Performance Evaluation of Reinforced Concrete Columns under Simultaneously Combined Fire and Cyclic Loads. *Buildings* 2022;12:1062. <https://doi.org/10.3390/buildings12071062>.
- [6] Toska K, Hofer L, Faleschini F, Zanini MA, Pellegrino C. Seismic behavior of damaged RC columns repaired with FRCM composites. *Eng Struct* 2022;262:114339. <https://doi.org/https://doi.org/10.1016/j.engstruct.2022.114339>.
- [7] Jawad SA, Ali AY. Structural response of post-fire exposed reinforced concrete column with pre-load. *IOP Conf. Ser. Mater. Sci. Eng.*, vol. 928, IOP Publishing; 2020, p. 22151.
- [8] Shakir N, Ali A. Effect of Duration and Intensity of Burning on Columns Exposure to Cyclic Fire. *Anbar J Eng Sci* 2022;13:91–8. <https://doi.org/10.37649/aengs.2022.176362>.
- [9] Nadir W, Ali AY, Jawdhari A, Kadhim MMA. Static Behavior of UHPC Corner Beam–Column Joint Under Constant Axial and Increasing Bi-Directional Bending Loads. *Int J Civ Eng* 2024;22:1291–308.
- [10] Shah AH, Sharma UK. Fire resistance and spalling performance of confined concrete columns. *Constr Build Mater* 2017;156:161–74. <https://doi.org/10.1016/j.conbuildmat.2017.08.167>.
- [11] Mansour Kadhum Alkafaji M. Structural Performance of Short Square Self Compacting Concrete Columns in Fire. *Eng Technol J* 2015;33:237–56. <https://doi.org/10.30684/etj.33.1A.18>.
- [12] Balaji A, Luquman K M, Nagarajan P, Madhavan Pillai TM. Studies on the behavior of Reinforced Concrete Short Column subjected to fire. *Alexandria Eng J* 2016;55:475–86. <https://doi.org/10.1016/j.aej.2015.12.022>.
- [13] Yaqub M, Bailey CG. Repair of fire damaged circular reinforced concrete columns with FRP composites. *Constr Build Mater* 2011;25:359–70. <https://doi.org/10.1016/j.conbuildmat.2010.06.017>.
- [14] Darwin D, Dolan CW, Nilson AH. Design of concrete structures. vol. 2. McGraw-Hill Education New York, NY, USA; 2016.
- [15] Abbas S, Nehdi ML, Saleem MA. Ultra-High Performance Concrete: Mechanical Performance, Durability, Sustainability and Implementation Challenges. *Int J Concr Struct Mater* 2016;10:271–95. <https://doi.org/10.1007/s40069-016-0157-4>.
- [16] Raut N, Kodur V. Response of Reinforced Concrete Columns under Fire-Induced Biaxial Bending. *ACI Struct J* 2011;108.
- [17] Alasmari HA. Rehabilitation of overload-damaged reinforced concrete columns using ultra-high-performance fiber-reinforced concrete. *Open Eng* 2023;13. <https://doi.org/10.1515/eng-2022-0437>.
- [18] Toska K, Beaucour A-L, Faleschini F, Pellegrino C, Noumowe A. Thermally Damaged Concrete: Repair through Textile-Reinforced Mortar Jackets. *J Compos Constr* 2025;29:04025002. <https://doi.org/10.1061/JCCOF2.CCENG-5310>.

- [19] Kodur VKR, Raut NK, Mao XY, Khaliq W. Simplified approach for evaluating residual strength of fire-exposed reinforced concrete columns. *Mater Struct* 2013;46:2059–75. <https://doi.org/10.1617/s11527-013-0036-2>.
- [20] Kumar V, Iqbal MA, Mittal AK. Energy absorption capacity of prestressed and reinforced concrete slabs subjected to multiple impacts. *Procedia Struct Integr* 2017;6:11–8. <https://doi.org/10.1016/j.prostr.2017.11.003>.
- [21] Muthuswamy KR, Thirugnanam GS. Structural behaviour of hybrid fibre reinforced concrete exterior Beam-Column joint subjected to cyclic loading. *Int J Civ Struct Eng* 2014;4:262–73.

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Effect of epoxy/water-repellent on the mechanical behavior of concrete in aggressive environment

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Abstract

Many studies have shown that concrete structures are subject to varying degrees of deterioration due to chemical attacks, often associated with high soil salinity. As concrete generally exhibits low resistance to chemical aggression, it is sometimes necessary to implement protective measures on surfaces exposed to such harsh environments. This study investigates the effect of water-induced chemical aggressiveness on the degradation of concrete structures. It aims to evaluate the significance and effectiveness of protective measures applied to concrete exposed to aggressive environments, using an external epoxy coating and an internal water-repellent additive. Concrete specimens were exposed to both a normal environment and an aggressive one (Sebkha water from the Ouargla region in Algeria). The mixtures incorporated a water-repellent additive at a dosage of 2% by cement mass, using both of Portland cement resistant to sulfates (CEM I 42.5 N-SR 3) "CRS" and a Portland cement (CEMII /BL 42.5N) "CPJ", with a constant water-to-cement (W/C) ratio of 0.61 for all mixes. The engineering properties evaluated include compressive strength, water absorption (by immersion and capillarity), and density for both protected and unprotected concrete in fresh and hardened states. In addition, the protective coating layers were characterized using scanning electron microscopy (SEM). Cubic and cylindrical specimens were tested at 28 and 90 days. The results show that concrete formulated with CPJ cement and the water-repellent additive exhibited superior compressive strength performance compared to the other specimens.

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1. Introduction

Concrete is the most widespread building material in the world due to its mechanical performance, versatility, and comparatively low cost [1,2,5]. However, it is very sensitive to its microstructural integrity over the long term, particularly in the presence of environmental aggressors such as chlorides, sulfates, carbon dioxide, and freeze-thaw [6-8]. These chemicals infiltrate the porous concrete structure [6,9] and initiate internal degradation processes such as corrosion of steel, sulfate attack, and alkali-silica reaction, all of which are detrimental to the service life of concrete structures. [1,3,6]

The composition and type of cement is an important variable that affects the performance of concrete [10,11] when subjected to harsh chemical exposures. CPJ cement, commonly used for general construction [1,2,12], has a relatively high volume of calcium aluminates (C_3A) [13,14] and

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therefore can be more subject to sulfate or chloride attacks under certain conditions. CRS cement that is manufactured with ground granulated blast furnace slag typically has much greater chemical resistance due to its latent hydraulic activity, porosity, and permeability characteristics. All of these factors make pre-stressed concrete a naturally more resilient option in difficult conditions. However, it is important to understand that CPJ-based concrete can be expected to perform satisfactorily in aggressive conditions if protection techniques are properly used [4,15]. Hydrophobic agents, surface treatments such as epoxy resin (Fig. 1), or improved curing can help greatly in preventing the infiltration of harmful agents by reducing permeability and improving integrity of the matrix. While CRS cements have an innate advantage of better resistance, a CPJ system can still be cost effective and durable where protection systems are considered in many different exposure situations. It is really a matter of understanding the nature of the cement technology, the environment and the protection systems.

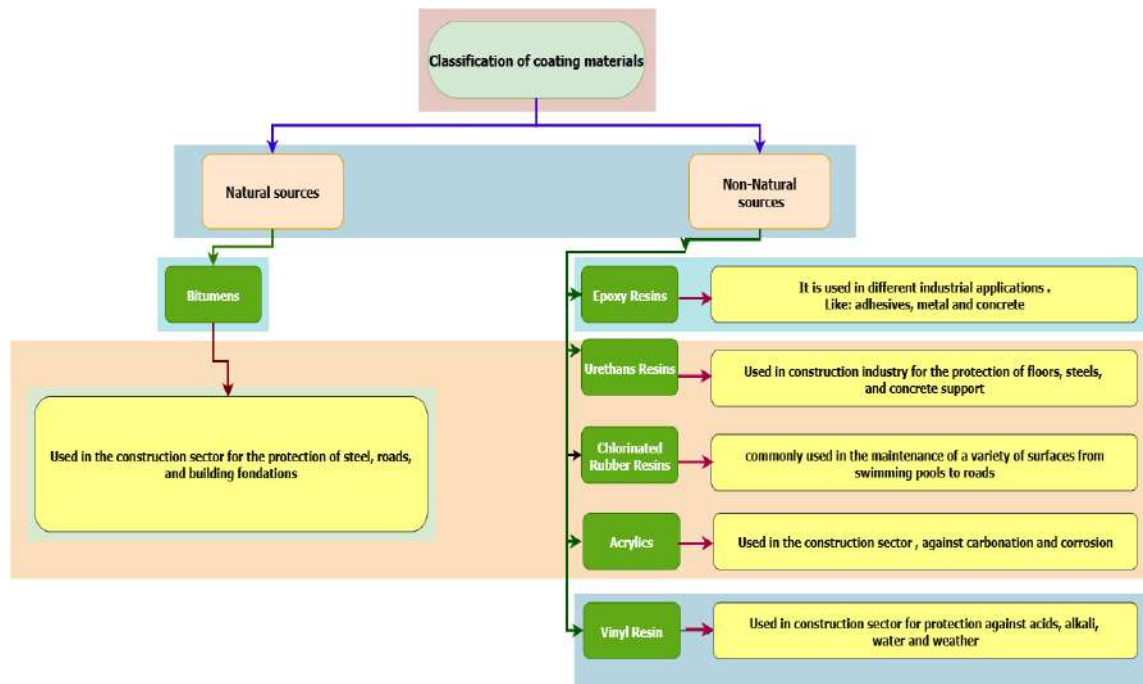


Fig. 1. Classification of coating materials [14-16]

To reduce the ingress of harmful agents, surface treatments or admixtures like hydrophobic (water-repellent agent) products have received a lot of attention. These products, which increase the energy of the surface of the pore walls, limit capillary absorption while allowing the pore to remain open and maintain vapor permeability. Epoxy-based coatings are also often applied to concrete surfaces in severely aggressive conditions because of their durability barrier [14, 17, 18], elongation, chemical inertness, and particularly their mechanical strength. With accurate combinations of cement formulations and protective treatments, concrete's durability performance can be dramatically improved. [19-21]

Our research is positioned within an experimental framework to assess the mechanical performance of concrete protected by surface-applied epoxy resin and an internally incorporated water-repellent admixture. Two cements were used for the concrete preparation: sulfate-resistant cement (CRS) and Portland composite cement (CPJ). The concrete specimens were subjected to two curing conditions: a standard environment (tap water) and an aggressive environment (Sebkha water from Hai Ennasr, Ouargla/ southeast of Algeria) [22,23]. Compression strength was measured on the specimens to evaluate both internal (water-repellent admixture) and external (sealed with an epoxy coating) means of protection under the various curing conditions. The results provide a platform to present comparative aspects of mechanical performance for concrete without protection and with protection applied internally and externally, while indicating the implications of protective layers for durability and resistance to external aggressors.

It should be noted that the testing period adopted in this study was relatively short, limited to 90 days, during which the specimens were exposed to a highly aggressive environment under continuous and constant conditions rather than a cyclic exposure regime. Furthermore, the scope of the investigation was restricted to the behavior of the cementitious matrix, and the effects of the aggressive environment on steel reinforcement, including potential corrosion-related phenomena, were not considered. Therefore, this study has opened new research perspectives, particularly regarding the influence of concrete age at the time of epoxy resin application. In the present work, the epoxy resin was applied at an early age; however, applying the coating at later ages, beyond 28 days, may affect its adhesion, penetration, and overall durability. This aspect is currently under investigation and will be addressed in future studies

This work will contribute to the overall optimization of protection mechanisms for concrete structures, especially of sections that will be subjected to highly aggressive environmental conditions. This work aims to contribute to a better understanding of the effectiveness of internal and external protections applied to concrete, and to provide practical recommendations to improve the durability of structures in regions with an aggressive environment, such as southern Algeria.

2. Study Materials

Before selecting of concrete materials, tests were conducted on various types of sand and gravels. This approach was intended to ensure that any potential chemical aggressions would originate from the external environment rather than from the internal constituents of the concrete, thereby allowing for more reliable and accurate study results. Therefore, the materials used in this study are described below.

2.1. Gravel

The gravel (3/8 and 8/15) used in these tests was sourced from the Ben Brahim quarry, located 40 km from the city of Ouargla/ Algeria, as illustrated in Fig. 2. The results of the various tests are presented in Table 1.



Fig. 2. Gravel used

Table 1. Characteristics of the gravel used

Tests	Standards	Results	
		Gravel 3/8	Gravel 8/15
Flakiness Index %	NF P 18-561	19.43	7.78
Absolute Density (g/cm ³)	NF P 18-554	2.62	2.62
Los Angeles Abrasion Resistance %	NF EN 1097 – 2		19.02
Micro-Deval Resistance %	NF P 18-572		11.8
Water Absorption %	NF P 18-554	1.91	1.68
Cleanliness %	NF P 18-591	0.96	0.7
Chemical Analysis	Insoluble		15,3
	Sulfate SO ₃	NF P 18-461, BS 1377,	0.31
	Carbonate CaCO ₃	NF P 94 048	85
	Chloride (Cl ⁻)		0.014

Both gravel fractions are whitish limestone aggregates, generally suitable for use in high-quality concrete. According to their chemical composition, assessed in accordance with the NF P18-461 standard and presented in Table 1, these aggregates are considered to be of good quality and chemically appropriate for concrete production.

2.2. Sand

The selected sand for this research is from the Djamaa quarry in El-Oued (South-East of Algeria); the choice is based on chemical analysis (low in sulfate) and a series of tests that demonstrated the chosen sand exhibits good characteristics. Table 2 summarizes the properties of the sands and the tests conducted.



Fig. 3. Sand used

Table 2. Characteristics of the sand

Test		Standard	Results
Fineness modulus		NF P 18-560	2.22
Absolute Density (g/cm ³)		NF P 18-554	2.62
Sand equivalent (SE)		NF EN 933 – 8	74.66
Water Absorption %		NF P 18-554	0.46
Methylene Blue Value (MBV).		NF P 94-068	0.48
Chemical Analysis	Insoluble		96
	Sulfate SO ₃	NF P 15-461, BS 1377, NF P 94 048	0.58
	Carbonate CaCO ₃		2
	Chloride Cl ⁻		0.026

2.3. Cement

In this research, two types of cement were used:

- Portland cement CEM II/B-L 42.5N (cement CPJ EL Mâtine);
- Portland cement resistant to sulfates (CEM I 42.5 N-SR 3) (cement CRS AL Mokaouam).

Table 3 presents the chemical properties of both cements utilized in this research.

Table 3. Chemical characteristics of cement used (%) [13,14,24].

CPJ cement	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	K ₂ O	LOI
	18.88	4.36	3.10	62.23	1.43	2.7	0.58	6.50
CRS cement	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	K ₂ O	LOI
	21.45	5	5	64	1.25	2	0.4	3.5

2.4. Water

The mixing water must be clean and free from harmful impurities (organic matter, alkalis). Potable water is always suitable [22, 25]. Analyses of potable water (used for concrete mixing and standard curing) and sebkha water (used for Aggressive conservation) as illustrated in Figs. 4. The results of the chemical water analysis are summarized in the table 4, below;



Fig. 4. The location of the Sebkha – Ouargla / Algeria

Table 4. Results of analyzed water (potable water and Sebkha)

Test	potable water	Sebkhas water	Specification NF EN 1008/July 2003
pH	7.51	7.77	PH $\geq$ 4
Sulfates SO_4^{2-} (mg/l)	753	11060	Max. 1 000 mg/l
Chloride Cl^- (mg/l)	883.13	82481.4	Max. 2 000 mg/l

3. Protective Products

For the protection method, we used two different products:

3.1 Internal Protection with Water Repellent

A liquid water-repellent admixture from Sika was used in the concrete during mixing. This additive acts internally by reducing the material's capillary absorption, thereby limiting water penetration through the formation of a water-repellent barrier within the concrete matrix. The detailed characteristics of the product are provided in the technical data sheets [26]

3.2 External Protection with Epoxy

We used a chemical protection product based on epoxy resin: Master Protect 1812, marketed under the name BASF 1812. This product is a two-component epoxy emulsion paint, composed of pigmented resins (component A) and cured resins (component B). It is specifically designed to protect both concrete and steel, and once applied, it forms an aesthetically pleasing and highly chemical-resistant coating.

Master-Protect 1812 is supplied in the correct proportions of resin (component A) and hardener (component B). Pour the hardener (component B) into the container of resin (component A) and ensure that container B is completely emptied. To achieve a homogeneous mixture, thoroughly mix the two components at approximately 300 rpm (27). Ensure the stirrer reaches the bottom and sides of the mixing container. Stir for at least 3 minutes or until the mixture is homogeneous (27).

4. Experimental Program

4.1 Formulation Method

The concrete mix design used for specimen preparation follows the Dreux method, resulting in a cement dosage of 350 kg/m³ (CPJ/CRS) (see Table 5), a gravel-to-sand ratio (G/S) of 1.91, a water-to-cement ratio (W/C) of 0.61 and HF dosage 2% by cement mass. According to the European standard NF EN 12390-1, cubic molds measuring (15 x 15 x 15) cm³ and cylindrical (7,5 x 15) cm² were used for specimen fabrication. The number of samples that were mixed and prepared in this study was estimated at 480 specimens.

Table 5. The percentages of formulations

Materials	Percentage	Mass for 1m ³ (Kg)
Concrete CPJ / CRS	-	350
Sand	38	625.07
Gravel 3/8	10	185.26
Gravel 8/15	52	1037.45
water	-	215.95

There are four (04) compositions to be studied (fig.6):

- Concrete based on CPJ cement + water repellent (B+HF -CPJ) and control concrete based on CRS cement + water repellent (B+HF -CRS): These are concrete (mixing with water repellent) formulations using different types of cement (CPJ or CRS) and stored in two conservation environments without any protection.
- Concrete based on CPJ or CRS cement protected with epoxy coating (B+HF+EP -CPJ and B+HF+EP -CRS) and mixing with water repellent: These concrete samples are coated with an epoxy-based protective product and stored in both standard and aggressive conservation environments.

4.2 Conservation and Protection of Specimens

In this study, two environments were selected for the conservation of concrete specimens:

- Aggressive conservation environment: The specimens were immersed in water from Sebkhia El-Khafdji-Ouargla under controlled conditions at 20°C ± 2°C.
- Standardized conservation environment: The specimens were stored in potable water at a temperature of 20°C ± 2°C.

After 14 days of standardized conservation for all specimens, apply two coats of 150 µm each; the second coat is applied after the first coat has dried for 4-6 hours at +40°C or 16 hours at +20°C. Two layers of protective coating were applied with a roller (See fig. 5). Before applying epoxy, it is must important to ensure that the surface of concrete has been completely dusted and that all impurities and damaged parts have been removed (27).



Fig. 5. Application of epoxy

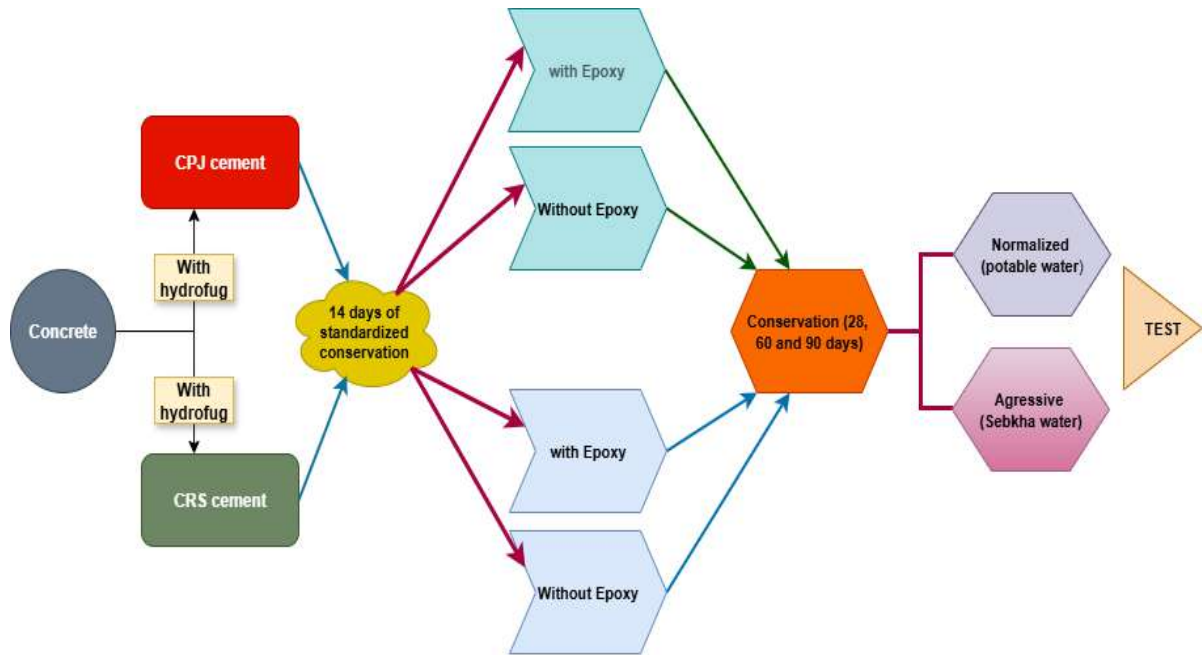


Fig. 6. Experimental protocol

4.3 Presentation of Experimental Tests

Four tests were applied to both unprotected (control) and protected concrete samples. The number of specimens tested for each result is 4 specimens (All results shown in the figures are the average result of 4 samples).

4.3.1 Compression Test

Compressive strength is defined as the ability of a material to withstand direct pressure from an applied compressive force [1,2]. In this study, compression tests were conducted following the European standard NF EN 12390-3, applying a load to a cubic specimen (15 × 15 × 15 cm) at a speed of 0.6 MPa/s, in a direction perpendicular to the casting axis, until failure. The samples were tested after air-drying.

4.3.2 Determination of Water Absorption by Immersion

This test measures the amount of water absorbed by a hardened concrete specimen (7,5x15) cm² (the specimens of this test, taken by drilling from a molded specimen). It indicates the concrete's porosity and durability. The test is typically performed in accordance with standards such as NF P18-459 and ASTM C642. The operations are performed in the following order:

- Immersion in a water bath at 20 ± 2 °C for a minimum of 48 hours and until a constant wet mass is reached; before weighing (M_h), the test tube is wiped with a damp chamois cloth to remove surface water;
- Drying for a minimum of 72 hours and until constant dry mass in a ventilated oven whose temperature is maintained at 105 ± 3 °C.
- The mass is considered constant when two successive weighing 24 hours apart do not show a difference greater than 0.1%.

Water absorption (Abs) is expressed as a percentage of dry mass (M_s) and is calculated using the following relationship (Eq.1):

$$Abs = \frac{M_h - M_s}{M_s} \cdot 100 \quad (1)$$

4.3.3 Determination of Water Absorption Rates by Capillarity

The determination of water absorption by capillarity is a standardized test method used to evaluate the rate at which water is absorbed by specimen or mortar specimens (5x10 cm) through

capillary action. This test was conducted following the ASTM C1585. It has provided essential information about the material's porosity, permeability, and resistance to water ingress, which are critical indicators of durability, especially in exposure to aggressive environments.

The data collected allows us to calculate the initial (Si) and secondary (Ss) sorpivity, expressed in $\text{mm/s}^{0.5}$, we can also calculate the index I , that it represents the *cumulative water absorption per unit area*, expressed as an equivalent height of water, hence the unit millimeters (mm).

$$I = \frac{\Delta M}{\rho \cdot A} \quad (\text{mm}) \quad (2)$$

Where; ΔM = increase in mass of the specimen due to water absorption (*kg or g*), ρ = density of water ($\approx 1000 \text{ kg/m}^3$), A = area of the surface in contact with water (m^2)

4.3.4 Analysis of Concrete with A Scanning Electron Microscope

The scanning electron microscope (SEM) is a valuable tool for the study of concrete. It allows the surface of concrete to be observed at high magnifications, revealing microscopic details such as cracks, pores and inclusions, which helps in understanding its microstructure and mechanical behavior. The magnification used in the test is 100, with a scale bars $10\mu\text{m}$, accelerating voltage $\text{ETH}=20 \text{ KV}$. The spectra supporting mineral identification will be presented in the next part. The SEM also makes it possible to carry out elemental analyses with an EDX spectrometer (Energy Dispersive X-ray spectrometry), which detects the X-radiation emitted by the sample and allows identifying the chemical composition of the elements. [28].

5. Results and Discussion

To facilitate comparison between the different types of concrete, all are manufactured with consistent workability, assessed by the Abrams cone slump of the consistency class for plastic concrete. Table 6 below provides information on the quantities of water actually added to the mixes, the workability, and the density of the concretes produced.

Table 6. The fresh properties of concerts

		Effective water used (<i>l</i>)	E/C	Workability (cm)	Density (g/cm^3)
B HF	CPJ	215.95	0.617	7.3	2.31
	CRS	215.95	0.617	7.3	2.35

5.1. The Effect of Adding Water-Repellent and Epoxy Coating on Mechanical Strength

Figure 7 shows the mechanical performance of concrete under various conditions was evaluated through compressive strength tests at 28 and 90 days, in both normal and aggressive environments. The comparison focuses on concrete mixes incorporating a water-repellent admixture (HF) with two types of cement: Portland cement "CPJ" and Portland cement resistant to sulfates "CRS", and with or without surface protection using an epoxy coating.

At 28 days and under normalized curing, the compressive strength of CRS-based concretes was higher than that of CPJ-based concretes. For instance, the compressive strength of the B + HF + CRS reached 41.89 MPa while that of the B + HF + CPJ was 37.52 MPa. Further, at 90 days, the strengths reached 45.00 MPa and 44.22 MPa respectively. This performance is attributed to the superior sulfate resistance and lower C_3A content in CRS cement, which enhances long-term durability [29,30]. The addition of a hydrophobic admixture water-repellent (HF), perhaps leads to an enhancement of the compressive strength development by reducing internal moisture movement and decreasing pore water saturation, as also indicated in [26,31,32].

For aggressive environments (Sebkha water rich in salt and sulfates), they led to a decrease of strength especially for CPJ-based concrete. At 90 days, while CPJ + HF concrete decreased from 44.22 MPa (standard) to 37.46 MPa (aggressive), the degree of reduction was recorded at $\sim 15\%$. In contrast, the CRS + HF concrete marginally increased while exposed to a harsh condition (45.00

to 46.90 MPa), which means the CRS-based concrete resistance to sulfate attack is high as supported by findings in [12,33-35].

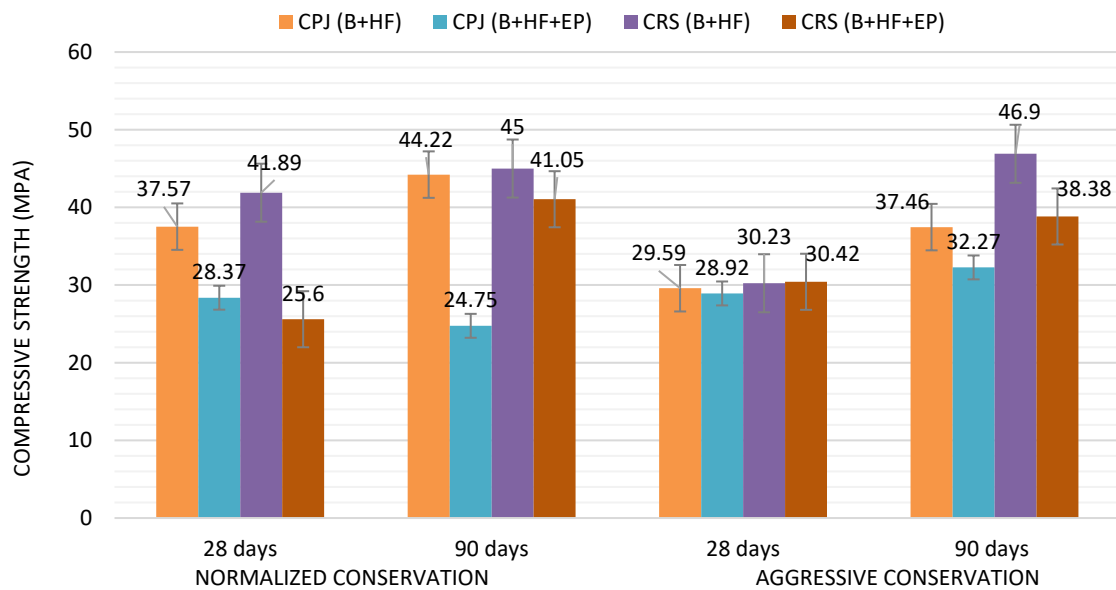


Fig. 7. Compressive strength of different concrete

Adding epoxy coating protection with the water-repellent additive produced mixed results. Under normal curing conditions, the compressive strength of CPJ + HF + Epoxy concrete decreased to 24.75 MPa at 90 days, compared to 44.22 MPa without epoxy. This indicates that the epoxy coating may have hindered cement hydration by restricting moisture exchange [3,32]. However, under aggressive exposure, the epoxy coating helped maintain strength. At 90 days, CPJ + HF + Epoxy reached 32.27 MPa, showing less loss compared to the uncoated version. This demonstrates that epoxy coatings effectively block external aggressive agents, especially when curing is properly controlled [32,36,37]

The decrease in the compressive strength of concrete with water-repellent and epoxy-protected materials in the standard medium is due to insufficient water, limited by the epoxy, which hinders the complete progression of cement hydration reactions and prevents the attainment of optimal strength. These findings align with [3] and [32], which demonstrated that epoxy coatings significantly reduce the ingress of chloride and sulfate ions. However, as noted by [36,38], early application of impermeable coatings can trap unreacted water, delaying hydration and weakening the microstructure. Medeiros et al. [32] highlighted the synergy between internal and surface water-repellent treatments in improving durability without sacrificing strength, as long as the curing process is properly managed.

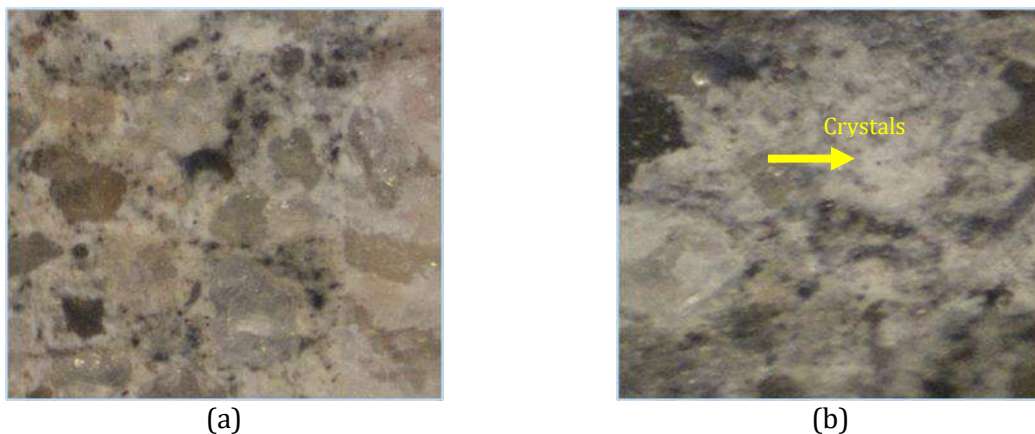


Fig. 8. The formation of crystalline deposits (a) Normalized environment and (b) Aggressive environment

The decrease in the compressive strength of concrete with water repellent and protected by epoxy in the aggressive environment can be explained by the formation of crystalline deposits, as supported by findings in [29,39], inside the sample indeed of Sebkha water, which is trapped between the concrete, promotes over time the precipitation of crystals as indicated in the Figure 8. Epoxy-protected concrete with water repellent did not retain moisture (fig. 9-a), like epoxy-protected concrete without water repellent (fig. 9-b), which positively influenced the compressive strength.



Fig. 9. Form of rupture of specimens without and with water-repellent

Fig. 9 (a) shows the presence of residual moisture at the time of testing likely influenced the failure behavior. High internal humidity can result in:

- Reduced strength due to incomplete hydration or internal pore pressure, especially if the specimen was poorly cured [29].
- More brittle or irregular fracture surfaces, as internal water disrupts the cohesive strength between the paste and aggregates.
- An early onset of cracking along weaker zones.

The specimen demonstrates in fig. 9(b):

- Uniform failure (barrel shape) typical of a well-compacted and well-cured concrete.
- Higher compressive strength and better aggregate–matrix bond, suggesting good material homogeneity and less internal moisture disruption.
- Likely results from adequate drying or curing prior to testing, enhancing performance as shown in previous studies [40,41].

These results are in agreement with findings from Neville [29], and Ambad V . and al. [42], who highlighted the role of moisture content at the time of testing in influencing compressive strength measurements. Overly moist concrete can give misleadingly lower strength values, while properly cured, drier specimens typically yield more accurate and higher readings.

5.2 The Effect of Adding Water-Repellent and Epoxy Coating on Water Absorption Rates

5.2.1 Water Absorption Rates by Immersion

The results of the water absorption by immersion test indicate a significant difference in porosity between the tested concrete formulations. The control specimen made with CPJ cement (without any protective additive) showed the highest water absorption value at 5.36%, exceeding the commonly accepted threshold of 5% for durable concrete in aggressive environments, as specified in standards such as ASTM C642 or NF P18-459.

In contrast, the concretes incorporating a water-repellent admixture (HF) demonstrated improved performance. The CPJ + HF formulation showed an absorption of 4.25%, while the CRS

+ HF formulation presented the lowest value, at 4.07%. These results confirm the effectiveness of the water-repellent additive in reducing the capillary absorption of concrete, thus enhancing its resistance to water ingress.

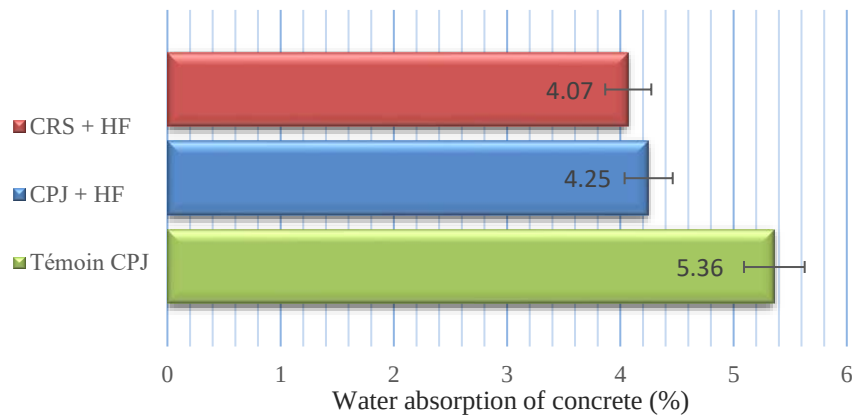


Fig. 10. Coefficient of water absorption by immersion

These results confirm the effectiveness of hydrophobic agents in reducing water ingress. The CRS-based mixture showed the lowest absorption, suggesting that the combination of CRS cement and water-repellent additive produces a denser, more water-resistant concrete. This observation aligns with findings by Djerbi et al. and Antoni et al. [43,44], who demonstrated that hydrophobic additives could significantly reduce the permeability of concrete by creating internal barriers to capillary suction. Moreover, the reduction in absorption observed with CRS-based mixes may be attributed to their lower C_3A content and higher sulfate resistance, as supported by [44-46]. These results are crucial for structures exposed to aggressive environments (e.g., marine or sulfate-rich soils), where limiting water ingress is key to improving durability and reducing chloride penetration.

5.1.2 Water Absorption Rates by Capillarity

The results of the capillary water absorption test, illustrated in figure 11 (A and B), highlight the influence of both water-repellent additives and cement type on the rate and extent of water ingress in concrete.

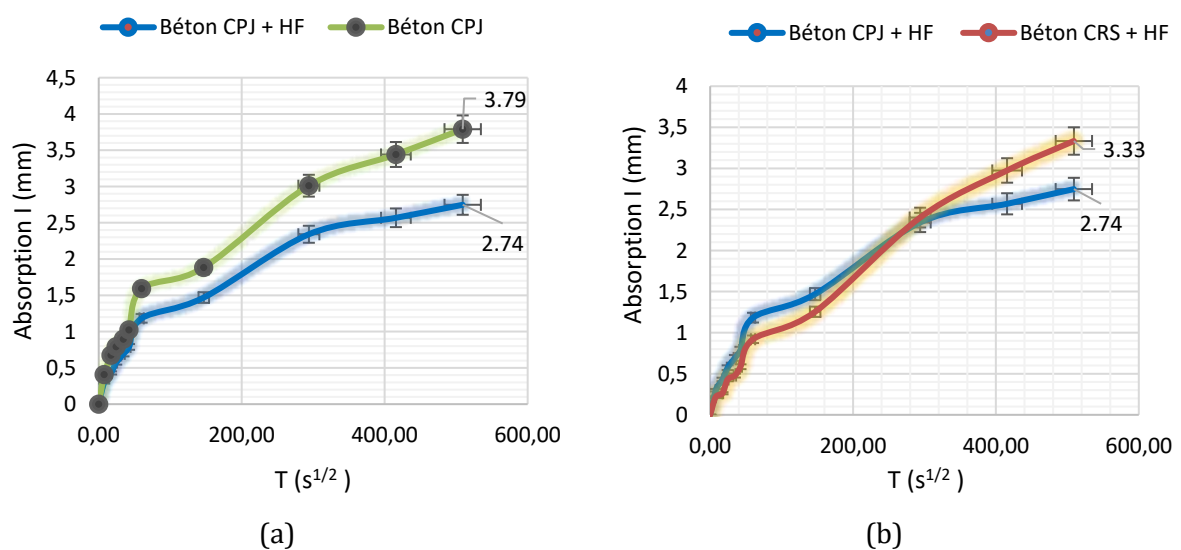


Fig. 11. Evolution of the water absorption rate of concrete by capillarity

Figure 11 (a), CPJ concrete with HF absorbs less water than control CPJ concrete at every time step. After $500\sqrt{s}$, CPJ + HF absorbs about 2.8 mm, while control CPJ absorbs approximately 3.8 mm. Figure 11 (b), Both CPJ + HF and CRS + HF show similar trends, but CRS + HF exhibits slightly higher absorption than CPJ + HF over time. This behavior demonstrates again the effectiveness of the HF additive, which acts internally to reduce capillary water transport. It is important to note that although CRS + HF shows slightly more absorption than CPJ + HF in capillarity (Figure B), it performs better in immersion (Figure above), which may reflect different pore structures and surface interactions of the concrete. These findings support the work of Dias and al. and Snoeck et al. [47,48], who demonstrated that water-repellent admixtures significantly reduce both initial and long-term capillary water uptake, enhancing resistance to freeze-thaw cycles and chloride penetration.

Wang and al. [49] show that the capillary absorption coefficient of the CRS+HF mixture is greater compared to CPJ+HF, CPJ+HF displays greater water absorption in immersion conditions. This difference in performance suggests that there are fundamental differences in their pore structures. The greater sorptivity of CRS+HF suggests that there is a well-connected capillary pore structure that facilitates rapid water absorption. Conversely, CPJ+HF displays lower capillary absorption as a result of a refined pore system with smaller pore entry diameters. During prolonged immersion conditions, CPJ+HF displays greater water absorption as a result of more accessible pores in its pore system, which suggests that capillary absorption is mainly affected by pore connectivity, whereas immersion absorption is affected by total open porosity. [49, 50, 51]

5.2 Analysis of Concrete with A Scanning Electron Microscope

The following figures (from 12 to 16) present SEM micrographs for different concrete compositions exposed to both normalized and aggressive environments. The analysis focuses on the influence of cement type (CPJ and CRS), and the use of hydrophobic agents (water repellent) on the microstructural development of the concrete matrix.

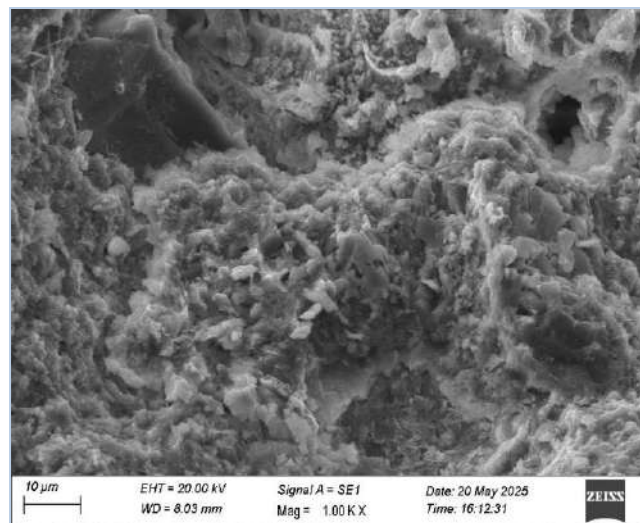


Fig. 12. CPJ Cement's concrete - normalized environment

Fig. 12 indicates the microstructure appears relatively dense with the presence of microcracks and large capillary pores, also, it reveals a granular texture, typical of partially hydrated cementitious compounds. The visible porosity indicates that the hydration is not complete, and there is a potential for permeability to water and ions.

In Fig. 13, the SEM micrograph of the samples reveals the addition of hydrophobic agent leads to significant microstructural changes. It reveals well-defined layered or crystalline hydration products, with a relatively compact structure, and shows a smoother overall surface and reduced visible porosity. Nevertheless, the surface texture appears highly compact with fewer visible pores. The morphology indicates a denser cement matrix, with abundant C-S-H gel-like formations seen on Fig. 14.

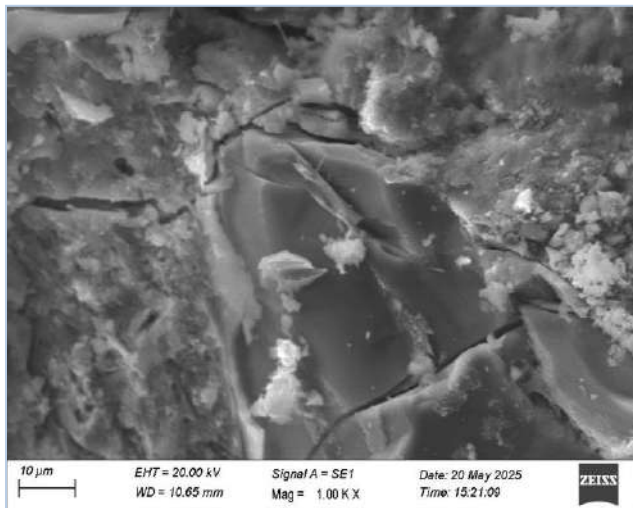


Fig. 13. CPJ Concrete with water repellent - normalized environment

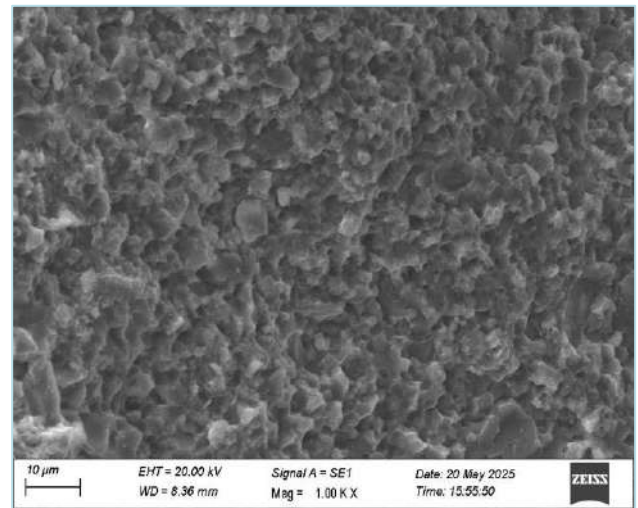


Fig. 14. CRS Concrete with water repellent - normalized environment

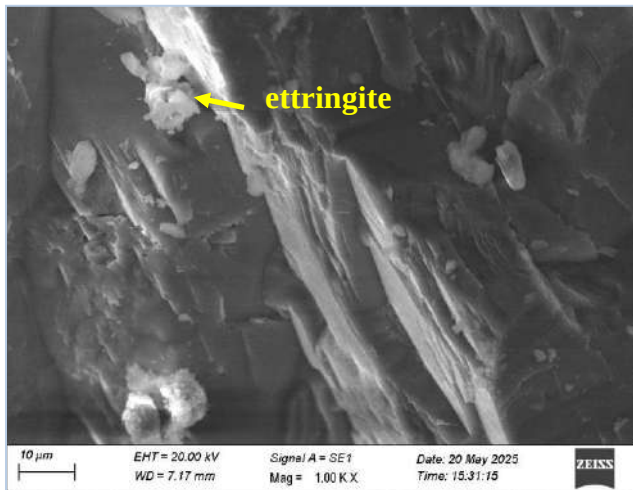


Fig. 15. CPJ Concrete with water repellent - aggressive environment

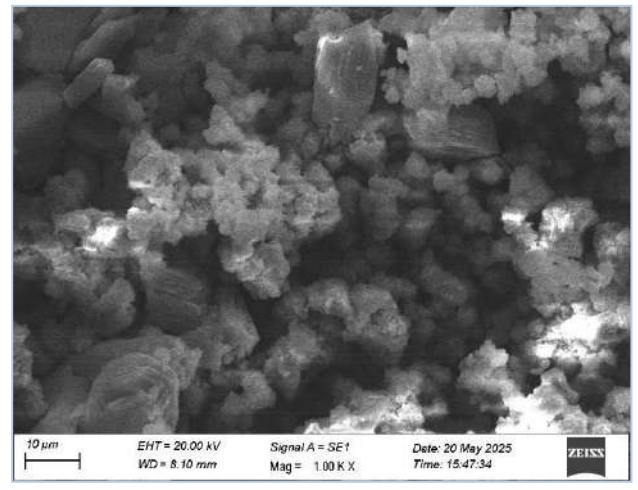


Fig. 16. CRS Concrete with water repellent - aggressive environment

Fig. 15 shows a long, needle-like or plate-like crystalline structures dominate the micrograph, indicating the presence of secondary ettringite or other sulfate-related expansive phases. Cracks are visible, suggesting damage due to chemical attack. However, Fig. 16 shows the microstructure retains a globular, dense morphology with minimal evidence of crack formation or expansive crystal growth. Hydration products are compact and interlocked. Concrete formulated with CPJ cement under normalized conditions exhibited a moderate level of compactness. Nevertheless, the presence of visible pore networks and microcracks suggests a relatively open microstructure, which could facilitate the ingress of water and aggressive ions. This porous morphology likely stems from incomplete matrix densification, potentially due to insufficient curing duration or the limited pozzolanic reactivity inherent to the CPJ formulation.

The incorporation of hydrophobic additives into CPJ-based concrete significantly modifies the pore system. The treated matrix displayed improved cohesion and reduced visible porosity, indicating that the additives contribute to filling and sealing capillary pores. This densification effect enhances water resistance and may also improve long-term durability. These observations are consistent with previous studies highlighting the ability of hydrophobic agents to enhance the microstructural integrity of cementitious systems [32,52,53].

A more pronounced improvement was observed in the concrete based on CRS (slag-based) cement with hydrophobic treatment. Under normalized conditions, this system exhibited superior microstructural refinement, characterized by a dense, gel-rich matrix with fewer interconnected voids. The enhanced compactness is attributed to the latent hydraulic activity of slag, which fosters

extended hydration, as well as the synergistic action of the hydrophobic admixture in reducing pore continuity. These factors collectively contribute to a substantial decrease in permeability, as supported by literature findings [35,54].

In aggressive environments, however, the CPJ-based concrete—even when treated with hydrophobic agents—demonstrated signs of deterioration. The formation of expansive crystalline products, likely ettringite or gypsum, resulted in microcracking and internal stresses. These degradation phenomena suggest that the hydrophobic treatment alone is insufficient to mitigate chemical attack in systems with high C_3A content, emphasizing the need for chemically stable cement compositions in such conditions [6,55]. Conversely, CRS-based concrete treated with hydrophobic agents maintained a stable and compact microstructure under the same aggressive exposure. The absence of expansive products and visible degradation can be attributed to the low C_3A content of slag cement, which inherently offers improved sulfate resistance. The combined effect of the chemical stability of CRS and the physical sealing action of the hydrophobic agent provides a robust defense against both physical and chemical deterioration. These results reinforce prior evidence regarding the superior durability of slag-based cementitious systems under aggressive conditions [3,7,54,56].

EDS analysis (figures from 17 to 21) revealed Ca-Si-Al-O-rich compositions, characteristic of hydrated cementitious matrices, with minor S and Cl detected in samples subjected to aggressive environments. The Ca-Si-O signals are indicative of C-S-H-type phases, whereas the simultaneous presence of Al and S suggests the formation of aluminate-sulfate phases, consistent with the identification of ettringite observed in the SEM test.

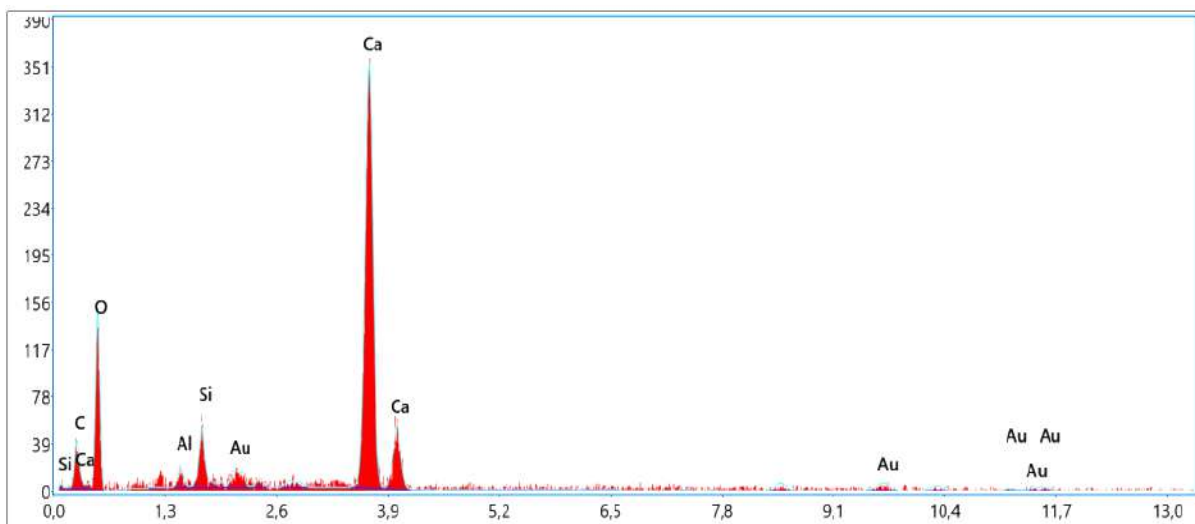


Fig. 17. CPJ Concrete without water repellent - normalized environment

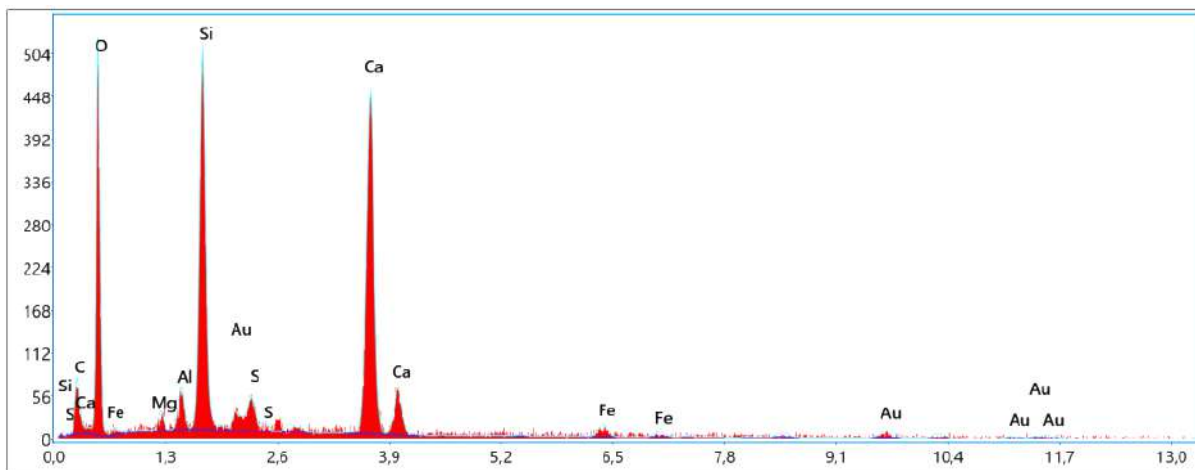


Fig. 18. CPJ Concrete with water repellent - normalized environment

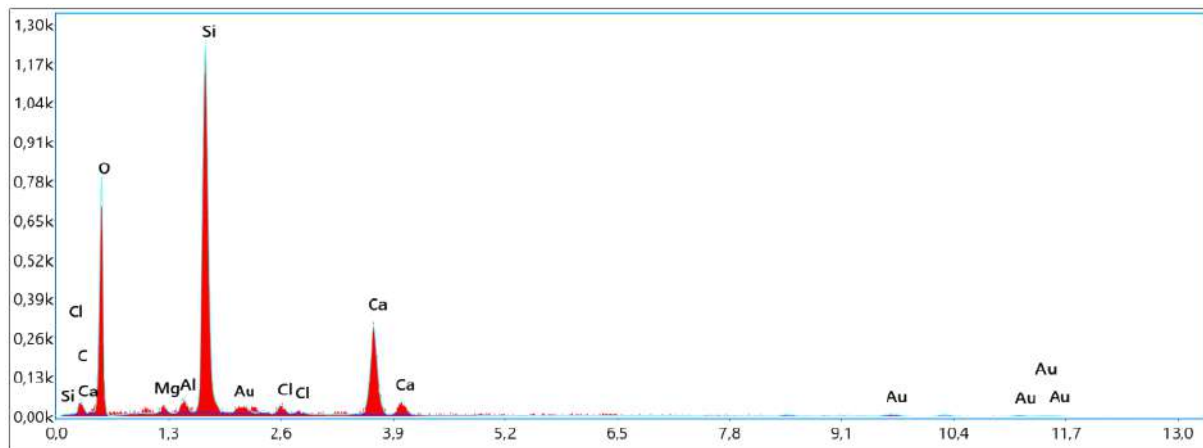


Fig. 19. CPJ concrete with water repellent - aggressive environment

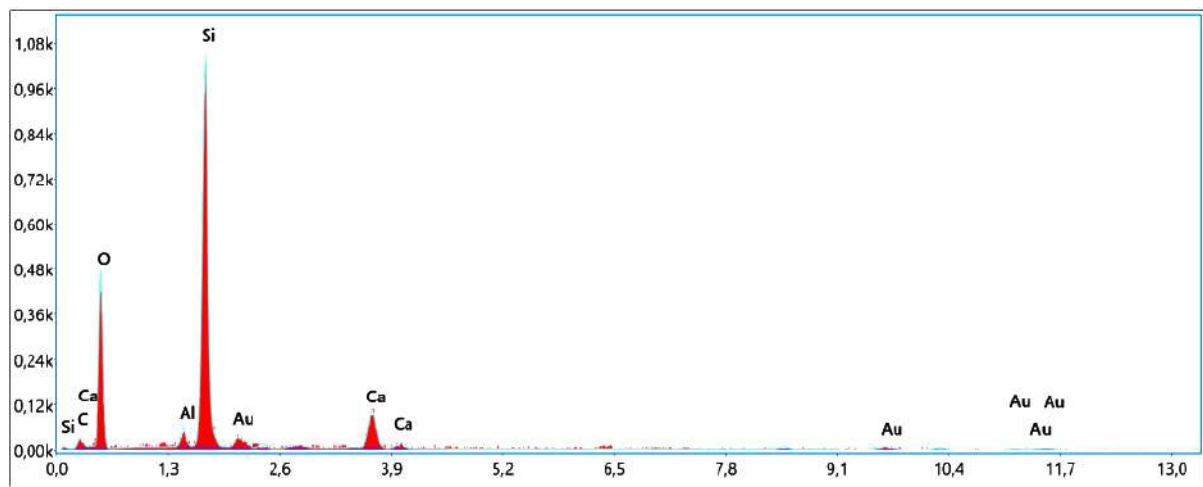


Fig. 20. CRS concrete with water repellent - normalized environment

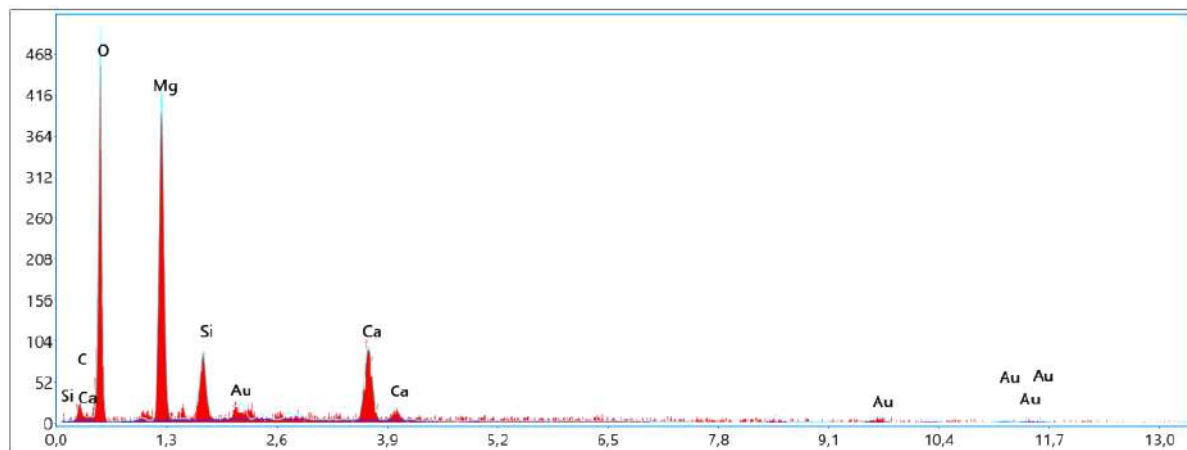


Fig. 21. CRS concrete with water repellent - aggressive environment

6. Conclusions

The objective of this research is to assess and evaluate the significance and effectiveness of various concrete protection methods in high-salinity environments. Identifying effective and durable solutions in this field contributes to enhancing infrastructure quality and extending their lifespan, benefiting societies and economies at large. The use of water-repellent and epoxy-based products as protective coatings for concrete made with CPJ (Portland Composite Cement) and CRS (Sulfate-

Resistant Cement) has demonstrated very satisfactory results in terms of improving the mechanical strength of concrete submerged in aggressive environments.

Water repellent admixture significantly decreases capillary water absorption capacity of concrete and greatly reduces its porosity. Its effectiveness is further enhanced when combined with CPJ cement. As the CPJ + HF system demonstrated, better in terms of coefficient of absorption than CRS + HF concrete. This could be due to a more favorable condition of the water repellent and CPJ concrete matrix interaction. Thus, yielding better long-term durability.

Epoxy protection should be reserved for aggressive exposure and not applied too early in normal curing conditions. However, epoxy coatings offer external protection, their application should be carefully timed to avoid compromising strength development, particularly in CPJ-based concretes.

The results for concrete based on CPJ cement, when protected with epoxy and mixed with water-repellent, yield acceptable values, suggesting the suitability of this product and cement type even in the most unfavorable conditions. Moreover, using CPJ cement protected by an epoxy-based and water repellent product in concrete offers numerous advantages; this cement can be more cost-effective than CRS cement, thereby reducing the overall financial burden in production. Additionally, it can enhance the durability of concrete structures in aggressive environments. In this way, CPJ cement can be considered as making "friendship with the environmental", not just for its moderate clinker reduction but also for enabling sustainable building design when used in a well-designed and well protected system. Its existing availability, affordability, and adaptability to green designs provide a further building block for transition to green concrete.

This study also opens new research perspectives regarding the effect of the concrete age at the time of application of epoxy resin. Moreover, other practical aspects regarding the durability of epoxy resin in environments with saline and sulfate solutions, the risk of blistering in immersion conditions, and the application of epoxy resin under real field conditions should be studied.

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References

- [1] Gagne R. Durabilité et réparations du béton. Cours en ligne de la durabilité de Béton, GCI-714. Université de Sherbrooke; 2011 Aug 29.
- [2] LCPC. Aide à la gestion des ouvrages atteints de réactions de gonflement interne. Technique et méthodes des LPC. Paris: LCPC; 2003.
- [3] Basheer PAM, Kropp J, Cleland DJ. Assessment of the durability of concrete from its permeation properties: A review. *Constr Build Mater*. 2001;15(2-3):93-103. [https://doi.org/10.1016/S0950-0618\(00\)00058-1](https://doi.org/10.1016/S0950-0618(00)00058-1)
- [4] Filip A, Covatariu D. The Influences Of The Durability Factors Over A Special Reinforced Concrete Structure's In-Time Behaviour. *Bul Inst Polit Iași*. 2019;65(69)(2).
- [5] Patil M, Dubey S, Patil H. Optimized properties of concrete at various exposure conditions. *Res Eng Struct Mater*. 2023;9(2):579-596. <https://doi.org/10.17515/resm2022.577ma1107>
- [6] Venkata Rambabu V, Amit B Mahindrakar. Impact of aggressive environment on concrete - a review. *Int J Civ Eng Technol (IJCIET)*. 2017;8(9):777-788.
- [7] Du Q, Li Y, Qiu B, Ren L, He X. Exploration of the Composition and Microstructure of the Historic Concrete Material in Shanghai. *Int J Archit Herit*. 2024. <https://doi.org/10.2139/ssrn.4327592>
- [8] Selejda J, Khmil R, Blikharsky Z. The influence of simultaneous action of the aggressive environment and loading on strength of RC beams. *MATEC Web Conf*. 2018;183:02002. <https://doi.org/10.1051/mateconf/201818302002>
- [9] Pepenar I. Comportement à la corrosion des constructions en béton armé dans des milieux agressifs acides: études de cas, tests de laboratoire et mécanismes de corrosion. *MATEC Web Conf*. 2018;149:01018. <https://doi.org/10.1051/mateconf/201814901018>
- [10] Ismail M, et al. Inhibiting Sulphate Attack on Concrete by Hydrophobic Green Plant Extract. *Adv Mater Res*. 2011;250-253:3837-3843. <https://doi.org/10.4028/www.scientific.net/AMR.250-253.3837>

- [11] Kurdowski W. Cement and Concrete Chemistry. Dordrecht: Springer Science + Business Media B.V.; 2014. <https://doi.org/10.1007/978-94-007-7945-7>
- [12] Taylor HFW. Cement Chemistry. 2nd ed. London: Thomas Telford; 1997.
- [13] Parande AK, et al. Environmental effects on concrete using ordinary and Portland pozzolana Portland cement. *Constr Build Mater*. 2011;25:288-297. <https://doi.org/10.1016/j.conbuildmat.2010.06.027>
- [14] CICA. Les constituants des bétons et des mortiers. Fiches techniques, Tome 1. Collection Technique CIMBETON. Centre d'Information sur le Ciment et ses Applications; 2005.
- [15] Fiche Technique. MATINE ciment béton exigeant CEMII/B-L42.5N. NA5042.
- [16] Kumar V. Protection of steel reinforcement for concrete - a review. 1998;16(4). <https://doi.org/10.1515/CORRREV.1998.16.4.317>
- [17] Millán Ramírez GP, Bylinski H, Niedostatkiewicz M. Deterioration and Protection of Concrete Elements Embedded in Contaminated Soil: A Review. *Materials*. 2021;14:3253. <https://doi.org/10.3390/ma14123253>
- [18] Erden S, Ertekin M, Ertekin G, Marmaralı A. Comparison of the compression characteristics of 2D braided and circular knitted carbon/epoxy composite tubes. *Res Eng Struct Mater*. 2019;5(3):203-212. <https://doi.org/10.17515/resm2019.80me1206>
- [19] Unal HY, Çok SS, Koç F, Gizli N, Pekbey Y. Investigating the effect of silica aerogel content on the mechanical properties of epoxy resin system. *Res Eng Struct Mater*. 2020;6(1):85-95. <http://dx.doi.org/10.17515/resm2019.111me0201>
- [20] Boulebnane A, Djeghader D, Tioua T. Weibull Analysis of Charpy Impact Test in Short Date Palm Fiber Reinforced Epoxy Composite. *Period Polytech Civ Eng*. 2023 Sep. <https://doi.org/10.3311/PPci.22500>
- [21] Dębska B, Wójcik K. Evaluation of the influence of aggregate type on selected properties of epoxy mortars. *E3S Web Conf*. 2018;49:00018. <https://doi.org/10.1051/e3sconf/20184900018>
- [22] Dębska B, Lichołai L. Resin Composites with High Chemical Resistance for Application in Civil Engineering. *Period Polytech Civ Eng*. 2016;60(2):281-287. <https://doi.org/10.3311/PPci.7744>
- [23] ANRH. Inventaire des forages et enquête sur les débits extraits de la Wilaya de Ouargla. Ouargla, Algérie; 2005 Jan.
- [24] B.E.T.S. BG. Études D'assainissement Des Eaux Résiduares, Pluviales Et D'irrigation. Mesures Complémentaires De Lutte Contre La Remontée De La Nappe Phréatique de Ouargla. Bureau d'études B.G. Suisse; 2002 Jul.
- [25] MOKAOUEM. Ciment pour béton en milieu agressifs. Fiche technique, Usine De LAFARGE en Algérie; 2012.
- [26] INSID. Les sols salins en Algerie. Algérie; 2008 Jul.
- [27] Medeiros MHF, Silva DA, Helene P. Durability performance of concrete surface treated with silane-based water repellent agent. *Mater Struct*. 2012;45(3):419-430.
- [28] Fiche technique. Master Protect 1812, BASF Construction Chemicals Algérie SARL.
- [29] Nanoscience Instruments. Scanning Electron Microscopy [Internet]. [cited 2025 May 17]. <https://www.nanoscience.com/techniques/scanning-electron-microscopy>
- [30] Neville AM. Properties of Concrete. 5th ed. Pearson Education; 2011.
- [31] Abimouloud Y. Contribution à l'étude de l'effet de la remontée des eaux de la nappe phréatique de Ouargla sur le béton à base de malzemeleri locaux [dissertation]. Université d'Ourgla UKMO; 2017.
- [32] Medeiros MHF, Helene P. Surface treatment of reinforced concrete in marine environment: Influence on chloride diffusion coefficient and capillary water absorption. *Constr Build Mater*. 2009;23(3):1476-1484. <https://doi.org/10.1016/j.conbuildmat.2008.06.013>
- [33] Topcu BT, Uzunomeroglu A. Experimental investigation of utilizing chemical additives and new generation corrosion inhibitors on reinforced concrete. *Res Eng Struct Mater*. 2021;7(1):13-34. <http://dx.doi.org/10.17515/resm2020.214st0929>
- [34] Aguiar JB, Camões A, Moreira PM. Performance of Concrete in Aggressive Environment. *Int J Concr Struct Mater*. 2008;2(1):21-25. <https://doi.org/10.4334/IJCSM.2008.2.1.021>
- [35] Blikharskyy Y, Selejdak J, Kapiika N, Vashkevych R. Study of Concrete under Combined Action of Aggressive Environment and Long-Term Loading. *Materials*. 2021;14:6612. <https://doi.org/10.3390/ma14216612>
- [36] Mohammed TU, Mahmood MH, Nihal A. Effect of epoxy and cementitious coating on chloride resistance of concrete exposed to marine environment. *Mater Today Proc*. 2018;5(9):19294-19300.
- [37] Demircan O, Çolak P, Kadioğlu K, Gunaydın E. Flexural properties of glass fiber/epoxy/MWCNT composites. *Res Eng Struct Mater*. 2019;5(2):91-98. <http://dx.doi.org/10.17515/resm2018.63is0828>
- [38] Balcioglu HE, Sakin R, Gun H. The design of multi-sample flexural fatigue device and fatigue behavior of glass/epoxy laminated composites. *Res Eng Struct Mater*. 2018;4(4):279-296. <http://dx.doi.org/10.17515/resm2018.67me0917>
- [39] Collepardi M. Damage by Delayed Ettringite Formation - A Holistic Approach and New Hypothesis. *Concrete International*. 1999;21(1):69-74.

- [40] Mehta PK, Monteiro PJM. Concrete: Microstructure, Properties, and Materials. 4th ed. McGraw-Hill Education; 2014.
- [41] K A, P SR. Effect of elevated temperature on strength and durability properties of concrete using nano-silica and alccofine. *Res Eng Struct Mater.* 2022;8(1):101-115. <http://dx.doi.org/10.17515/resm2021.281st0419>
- [42] Ambad V, Narwade R, Nagarajan K. Impact of controlled permeable formwork liner against chloride penetration on the concrete structures. *Res Eng Struct Mater.* 2022;8(1):117-126. <http://dx.doi.org/10.17515/resm2021.306st0605>
- [43] Djerbi A, Lafhaj R, Dheilly M, Quéneudec M. Influence of aggregates on the mechanical and transport properties of hydrated cement-treated clay. *Cem Concr Res.* 2007;37(7):1033-1040.
- [44] Antoni M, Rossignol J, Clastres J. Hydrophobic treatments and water absorption of cement-based materials: influence of aging and porosity. *Constr Build Mater.* 2016;124:219-226.
- [45] Marchand J, Samson E, Maltais R. Influence of curing on chloride diffusion in cement-based materials. *Cem Concr Res.* 1999;29(10):1429-1433. [https://doi.org/10.1016/S0008-8846\(99\)00101-5](https://doi.org/10.1016/S0008-8846(99)00101-5)
- [46] European Committee for Standardization. EN 13057: Products and systems for the protection and repair of concrete structures - Test methods - Determination of resistance of capillary absorption. 2002.
- [47] Dias WL, Lima FL. Influence of hydrophobic admixtures on water absorption and drying of concrete. *J Mater Civ Eng.* 2012;24(1):76-83.
- [48] Snoeck D, Steuperaert N, De Belie N. Effect of hydrophobicity on water ingress and durability of cracked self-healing concrete. *Cem Concr Res.* 2018;112:67-76.
- [49] Wang Y, Li L, An M, Sun Y, Yu Z, Huang H. Factors Influencing the Capillary Water Absorption Characteristics of Concrete and Their Relationship to Pore Structure. *Appl Sci.* 2022;12(4):2211. <https://doi.org/10.3390/app12042211>
- [50] Yang L, Gao D, Zhang Y, Tang J, Li Y. Relationship between sorptivity and capillary coefficient for water absorption of cement-based materials: theory analysis and experiment. *R Soc Open Sci.* 2019;6(6):190112. <https://doi.org/10.1098/rsos.190112>
- [51] Ding X, Liang X, Zhang Y, Fang Y, Zhou J, Kang T. Capillary Water Absorption and Micro Pore Connectivity of Concrete with Fractal Analysis. *Crystals.* 2020;10(10):892. <https://doi.org/10.3390/cryst10100892>
- [52] Arunkumar K, Sureskumar A, Ganesh AC, Parthiban L, Premkumar V. Optimization of hybrid fibre reinforced geopolymer concrete using hardened and durability properties. *Res Eng Struct Mater.* 2023;9(1):113-130. <http://dx.doi.org/10.17515/resm2022.450ma0620>
- [53] Zhang X, Wei J, Shi C. Effect of hydrophobic admixtures on pore structure and durability of concrete. *Cem Concr Compos.* 2020;105:103426. <https://doi.org/10.1016/j.cemconcomp.2019.103426>
- [54] Pacheco-Torgal F, Jalali S. Sulfate resistance of plain and blended cement mortars in terms of their composition. *Constr Build Mater.* 2010;24(3):372-378.
- [55] Bernal SA, et al. Performance of alkali-activated slag mortars exposed to different aggressive environments. *Cem Concr Res.* 2014;40(4):1358-1365.
- [56] Skalny J, Marchand J, Odler I. Sulfate Attack on Concrete. Spon Press; 2002. <https://doi.org/10.4324/9780203301623>

Frequency-dependent effects of CFRP retrofitting on the equivalent viscous damping ratio of RC frames

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Abstract

Conventional seismic assessment of reinforced concrete (RC) frames retrofitted with externally bonded carbon fiber-reinforced polymer (CFRP) typically relies on quasi-static force-deformation metrics, overlooking the frequency-dependent evolution of energy dissipation. This study investigates the effects of excitation frequency and CFRP detailing system on the dynamic response of retrofitted frames using an energy-based framework and the equivalent viscous damping ratio (EVDR). A three-story RC frame was subjected to harmonic base displacement over a range of frequencies and evaluated using two CFRP retrofit configurations at beam plastic hinges: an extended flange-bonded CFRP sheet configuration and a configuration designed to promote plastic hinge relocation. Results show that, at resonance, the extended flange configuration increased elastic energy by 67% and reduced hysteretic damping by 40%, leading to excessive force amplification that compromised cyclic endurance. In contrast, the hinge relocation configuration limited elastic energy growth to 30%, maintained higher hysteretic damping, and improved cyclic endurance. Moreover, CFRP retrofitting was most effective below resonance and became progressively less effective at higher excitation frequencies. These findings underscore that effective CFRP retrofit design must jointly consider the targeted excitation frequency and CFRP detailing system to optimize energy dissipation and structural resilience.

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1. Introduction

Fiber-reinforced polymer composites, when externally bonded to RC members, have become a widely used technique for seismic strengthening due to their high strength-to-weight ratio, corrosion resistance, and ease of installation [1]. Over the past two decades, experimental and analytical investigations have shown that properly detailed FRP retrofitting can significantly enhance the strength and deformation capacity of RC beam-column joints [2–4]. These advances have motivated the development of design guidelines and supported broader implementation of FRP retrofit systems to upgrade aging, seismically vulnerable RC structures [1,5,6].

A key mechanism by which FRP retrofitting improves the behavior of RC frames is the strategic relocation of plastic hinges away from column faces and joint cores toward the beam span. This relocation reduces inelastic demand on joint regions and promotes a ductile response characterized by a strong-column/weak-beam mechanism, thereby shifting the global failure pattern toward a controlled beam-hinging mode.

The concept of plastic hinge relocation has been extensively explored in structural retrofitting research, with two mains externally bonded FRP retrofit schemes reported in the literature: web-bonded and flange-bonded configurations. Both techniques have demonstrated the ability to enhance joint performance and shift inelastic deformations into the beam span [2,4,7,9]. However,

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the web-bonded approach imposes significant practical limitations in three-dimensional framing systems. Slabs and transverse beams obstruct access to the beam webs and joint faces, rendering this method largely infeasible for widespread field implementation [2]. In contrast, flange-bonded techniques address these access constraints by reinforcing the top and bottom beam flange surfaces, which remain accessible even in the presence of floor systems. A comparative analysis by Zarandi et al. [2] further showed that flange-bonded configurations can achieve hinge relocation and ductility enhancement comparable to those of web-bonded systems, while requiring less FRP material, thereby reducing retrofit costs and increasing feasibility for field implementation.

Regardless of whether a web-bonded or flange-bonded configuration is adopted, effective plastic hinge relocation using externally bonded FRP is governed primarily by two design variables: CFRP ply count (thickness) and the CFRP development length L_f , defined as the distance from the column face along the beam to the laminate termination (cut-off). Experimental and numerical studies have consistently shown that these parameters control the ability to relocate plastic hinging into the beam span. Mahini and Ronagh [7], through a systematic investigation of web-bonded layouts, demonstrated that adequate ply count and development length are indispensable for successful hinge relocation and the restoration of joint integrity. They further reported that insufficient CFRP thickness can concentrate damage at the joint interface and may not yield meaningful improvements in joint ductility.

For flange-bonded systems, Arowojolu et al. [4] conducted monotonic pushover and cyclic tests on deficient exterior beam–column joints, systematically varying the CFRP development length from approximately two-thirds to one time the beam depth ($L_f \approx 0.67h_b - 1h_b$, where h_b is the beam depth) and adjusting ply thickness to assess hinge relocation. They reported that increasing L_f from $0.67h_b$ to $1h_b$ produced no appreciable gain in load capacity but resulted in slightly increased joint cracking. They suggested an optimal L_f of approximately two-thirds the beam depth for successful hinge relocation, shorter than the conventional $1h_b$ length recommended by Paulay and Priestley [8].

Complementing this work, Hejazi et al. [9] conducted parametric pushover analyses of interior joints with moderate ductility, adopting the development length proposed by Paulay and Priestley [8] and varying the number of flange-bonded plies. They concluded that a five-layer configuration was the most cost-effective option for successful plastic hinge relocation, as applying further layers provided no additional benefit to the relocation mechanism. Furthermore, they recommended selecting the ply count based on the shear and moment demands imposed on the intersecting beam and column members at the joint.

Another critical requirement for reliable FRP retrofitting is robust anchorage of externally bonded sheets to ensure effective engagement of the FRP reinforcement under seismic loading. Without special anchorage, premature debonding may occur before the FRP approaches its tensile capacity, thereby limiting the effectiveness of the retrofit. Accordingly, prior studies have proposed and validated several anchorage systems, such as FRP wraps and steel fasteners, to delay or prevent debonding [3,13,21]. In cyclic tests on shear-critical RC beam–column joints investigated by Antonopoulos and Triantafillou [3], specimens retrofitted with externally bonded FRP and provided with anchorage systems achieved significantly higher shear strength, energy dissipation capacity, and stiffness than their unanchored counterparts. Similarly, Sakr et al. [13] reported premature debonding in several CFRP-retrofitted exterior beam–column joints tested under cyclic loading. To overcome this limitation, they further investigated a mechanically fastened alternative using steel plates and anchors. Their results indicated that such fasteners substantially enhanced the ultimate joint capacity while promoting a ductile beam-hinging response.

Despite extensive component-level validation, comparatively few studies have examined the efficacy of FRP retrofit at the whole-frame system scale, where interactions among retrofitted members and damage redistribution govern the overall response. Wang et al. [14] demonstrated through pseudo-static cyclic testing of previously damaged RC frames retrofitted with externally bonded CFRP that retrofitting not only restores lateral strength, stiffness, and ductility but can also yield performance metrics that exceed those of the original, undamaged frames. Comparable system-level enhancements have been observed in additional full-scale frame studies [2,12,15-20].

Another important metric is the seismic performance level achieved after retrofitting, as defined by performance-based design guidelines [10,11,21]. In this context, Zarandi et al. [2] evaluated and compared the system-level seismic response of an original RC frame and its retrofitted counterparts employing flange-bonded and web-bonded FRP schemes. Capacity curves obtained from nonlinear pushover analysis were integrated into a capacity–demand procedure [11] to determine the performance point and the associated performance level for each frame. Their results showed that, under an earthquake demand corresponding to a design base acceleration of 0.25g, the original frame failed to satisfy the Life Safety (LS) requirement, whereas both FRP-retrofitted frames met this criterion. However, a direct comparison of the capacity curves showed that the flange-bonded scheme significantly outperformed the web-bonded counterpart, improving not only the frame's global strength and ductility but also its overall seismic performance. This superiority was further highlighted at a higher seismic demand of 0.30g, where the flange-bonded configuration continued to satisfy the LS requirement, whereas the web-bonded scheme no longer met this criterion.

However, the vast majority of prior studies have relied on quasi-static cyclic or monotonic loading to assess retrofit effectiveness, focusing primarily on force–deformation metrics. A key research gap remains in understanding the frequency-dependent dynamic behavior of CFRP-retrofitted RC frames, specifically how energy dissipation and damping characteristics evolve under harmonic or broadband frequency excitations that include the structure's fundamental frequency. The effect of excitation frequency is critical, as earthquake ground motions can induce pronounced response amplification when their dominant spectral components approach a structure's natural frequencies, leading to resonance. In such scenarios, the structural response is governed by the combined influence of excitation amplitude and resonance effect, which in turn can cause excessive stiffness degradation, energy dissipation, and cumulative damage. Moreover, prior research has shown that CFRP retrofitting can modify the structure's hysteretic behavior and stiffness evolution [14–16]. This, in turn, can change the partitioning of input energy among recoverable strain energy, damping-related dissipation, and damage-associated mechanisms. Therefore, explicit consideration of excitation-frequency effects is essential for reliable performance-based seismic assessment of CFRP-retrofitted RC frames.

The primary objective of this study is to assess how excitation frequency and CFRP retrofit detailing system influence the energy dissipation and damping characteristics of retrofitted RC frames. By systematically varying the excitation frequency relative to the structure's fundamental frequency and comparing different retrofit configurations, the study establishes a unified framework that links excitation frequency, retrofit detailing, and dynamic energy dissipation, thereby addressing a critical gap in the system-level assessment of CFRP-retrofitted RC frames.

2. Numerical Investigation

2.1. Description of Test Structures: Primary and CFRP-Retrofitted Frames

To evaluate the effect of CFRP retrofitting on the seismic performance of reinforced concrete frames, a numerical simulation was carried out using Abaqus/CAE [22] on a three-story, single-bay, moment-resisting RC frame. The model, characterized by uniform cross-sectional dimensions for all beams and columns across all stories, was developed without CFRP retrofitting and is referred to as the primary frame. It served as the baseline for assessing the dynamic response and energy-dissipation characteristics under sinusoidal base excitation across a range of frequencies.

For comparative analysis, two additional models, exact replicas of the primary frame, were developed with externally bonded CFRP sheets applied at the anticipated plastic hinge regions. These models employed two distinct retrofit configurations, designated as Configuration A and Configuration B, allowing for a systematic comparison of energy dissipation behavior between the primary and retrofitted frames under identical base displacement histories (Fig. 1). Both configurations utilized CFRP composites but differed in retrofit strategy, ply thickness, and development length, as summarized in Table 1.

Table 1. CFRP retrofit configurations: Mechanical and geometric properties

Configuration	Location	CFRP System	Fiber Orientation	No. of Layers	E_f (GPa)	ε_{fu} %	Total thickness (mm)	L_f
A	Column wrapping	UD fabric	$[45^\circ/-45^\circ]_2$	4	133.86	1.36	1	$2h_c$
	Beam flanges	HM laminate	0°	3	165	1.87	3	$2h_b$
B	Column wrapping	UD fabric	$[45^\circ/-45^\circ]_2$	4	133.86	1.36	1	$2h_c$
	Joint core/beam webs	HM laminate (U-shaped)	0°	1	165	1.87	1	$0.67h_b$ (from beam-column interface)
	Beam flanges	HM laminate	0°	3	165	1.87	3	$0.67h_b$

UD = unidirectional; HM = high modulus; E_f = elastic modulus; ε_{fu} = ultimate tensile strain

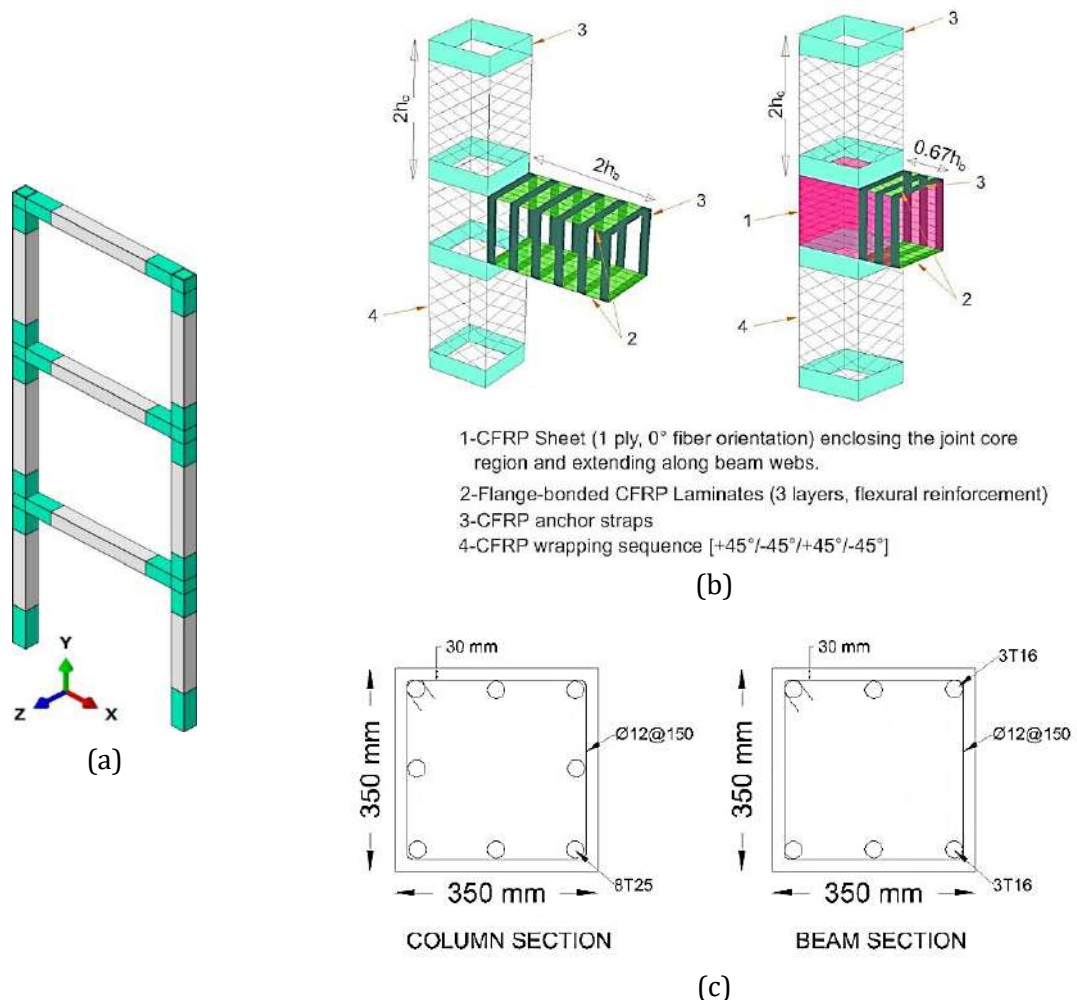


Fig. 1. Integration of CFRP retrofitting into the moment-resisting RC frame: (a) 3D model of the three-story RC frame; (b) details of CFRP application at beam and column joints for Configurations A and B; (c) cross-sectional details of beam and column members

Configuration A employed a two-stage CFRP retrofit scheme. In the first stage, the column plastic hinge regions were wrapped with four plies of unidirectional CFRP sheets arranged in an

alternating stacking sequence, extending a distance of $2h_c$ (where h_c is the column depth) above and below the beam–column interface to enhance shear resistance. In the second stage, flexural retrofit was provided by bonding high-modulus CFRP laminates to both beam flanges, with fibers oriented longitudinally. The laminate development length, $L_f = 2h_b$ (where h_b is the beam depth), measured from the column face, was selected in accordance with ACI 440.2R-17 [1].

Configuration B employed a three-stage CFRP retrofit scheme specifically designed to relocate the plastic hinge away from the beam-column interface. In the first stage, column plastic hinge zones were wrapped with four plies of unidirectional CFRP in an alternating $[45^\circ/-45^\circ/45^\circ/-45^\circ]$ lay-up (relative to the longitudinal axis), similar to Configuration A. In the second stage, high-modulus CFRP laminates were bonded to the beam flanges (top and bottom faces), with fibers oriented longitudinally. A reduced development length of $L_f = 0.67h_b$, measured from the column face, was adopted in accordance with Arowojolu et al. [4], who demonstrated that plastic hinge relocation can be effectively achieved at both $0.67h_b$ and $1h_b$, with the shorter extension minimizing joint-core damage while preserving comparable flexural capacity.

In the third stage, a single-ply U-shaped CFRP laminate (nominal thickness ≈ 1.0 mm) was applied over the joint core and adjacent beam web regions. The fibers were aligned with the beam axis (0° orientation), and the wrap terminated at the same development length as the flange-bonded laminates. This web-bonded detailing has been shown to effectively shift plastic hinge formation away from the beam–column interface toward the CFRP termination region [7]. All CFRP configurations were anchored using strap systems placed at strategic locations to prevent end-peeling failures and ensure effective transfer of tensile forces, thereby enabling full development of the CFRP strengthening capacity, as illustrated in Fig. 1(b).

2.2. Numerical Simulation Investigation

Modal analysis was initially performed on the primary frame and the two CFRP-retrofitted frame configurations, A and B, to extract their fundamental vibration modes and associated natural frequencies. Based on these frequencies, the Rayleigh damping coefficients, α (mass-proportional) and β (stiffness-proportional), were then determined for a prescribed material damping ratio of 5%. Subsequently, the calculated damping parameters were incorporated into each model prior to conducting the nonlinear dynamic simulations.

Each finite element model was then subjected to a one-second static gravity preload to incorporate self-weight effects into the global mass matrix. Following this, Incremental Dynamic Analysis (IDA) was initiated. The primary frame was subjected to a sinusoidal base excitation $u(t) = A \sin(\omega t)$ at the corresponding resonance frequency, where A denotes the displacement amplitude (mm), ω is the circular frequency (rad/s) associated with the fundamental natural mode of the structure, and t is the time variable (s). The amplitude A was calibrated so that the primary frame reached its threshold limit after 14 consecutive loading cycles. The IDA simulation was terminated once any cross-section reached its ultimate curvature ϕ_u , defined as the earliest occurrence of one of two local failure modes outlined by Valles [23]:

- Crushing of concrete at the extreme compression fiber
- Reaching the ultimate tensile strain of longitudinal steel reinforcement

Following amplitude calibration of the primary frame, the same base excitation was applied to both CFRP-retrofitted configurations to enable direct comparison of energy dissipation and structural response. The evaluation criteria and limit-state definitions established for the primary frame were consistently maintained for the retrofitted configurations. Throughout the analysis, CFRP strain was continuously monitored, with failure defined as the point at which the composite material reached its ultimate tensile strain, as specified in Table 1.

It is worth noting that throughout all conducted dynamic analyses, the governing limit state corresponded to the attainment of the ultimate curvature in the beam plastic hinge region, consistently occurring in the beam segment rather than in the column elements across all frames. No premature column yielding, global instability, or CFRP rupture was observed before reaching this curvature threshold, indicating that the beam flexural capacity controlled the structural response under the applied cyclic loading protocol. Steel reinforcement failure was assumed to

occur at a tensile strain of 5%, in accordance with the maximum usable strain limit for longitudinal bars as specified in FEMA 356 [10], beyond which rebar fracture is expected.

Three primary energy components were extracted and quantified from Abaqus history output across loading cycles: cumulative viscous dissipated energy (E_v), cumulative plastic (hysteretic) dissipated energy (E_p), and maximum recoverable elastic strain energy per cycle (E_{se}). These quantities were used to compute equivalent viscous damping ratios, decomposed into viscous ($\xi_{viscous}$) and hysteretic (ξ_{hyst}) components to enable systematic comparison between the primary frame and the CFRP-retrofitted frames. The investigation was complemented by off-resonance analyses at $\pm 25\%$ of the fundamental frequency, during which energetic response and EVDR trends were evaluated and compared with the baseline resonant behavior.

3. Theoretical Considerations: Energy Balance Principles and Strain Energy Method

3.1. Energy Balance Under Harmonic Excitation

As outlined in Chopra's work on structural dynamics [24], the energy response terms for a single-degree-of-freedom (SDOF) system subjected to earthquake ground motion can be derived from the equation of motion, expressed as follows:

$$E_k + E_v + E_I = E_{input} \quad (1)$$

where E_k represents the kinetic energy, and E_I denotes the internal energy, which comprises the recoverable elastic strain energy and the cumulative hysteretic energy (E_{hyst}) resulting from inelastic deformation. The latter includes both E_p and damage-related energy dissipation (E_D). E_{input} corresponds to the total input energy imparted to the system by the ground motion.

In the present study, hysteretic losses arise almost entirely from plastic deformation; damage-related dissipation contributes only a negligible share. Consequently, E_{hyst} will be treated as equivalent to the plastic dissipated energy for all subsequent analyses.

3.2. Evaluation of the EVDR Using the Strain Energy Method

The strain energy method calculates the EVDR as the ratio of dissipated energy to the elastic strain energy at the maximum force and displacement per hysteretic loop (Fig. 2). Initially developed for linear-elastic systems, Jacobsen extended it to accommodate nonlinear behavior [25-27]. In nonlinear systems, it is crucial to distinguish between the equivalent viscous damping component ($\xi_{viscous}$), which is frequency-dependent (Eq. (2)), and the equivalent hysteretic damping component (ξ_{hyst}), which arises from frequency-independent plastic dissipation (Eq. (3)) [28]. Both components are evaluated on a cycle-by-cycle basis.

$$\xi_{viscous} = \frac{E_{v,cyc}}{2\pi E_{se}} \left(\frac{\omega_n}{\omega} \right) \quad (2)$$

$$\xi_{hyst} = \frac{E_{hyst,cyc}}{2\pi E_{se}} \quad (3)$$

$$\xi_{eq,total} = \xi_{viscous} + \xi_{hyst} \quad (4)$$

where ω_n and ω are the natural and excitation angular frequencies, respectively; $E_{v,cyc}$, and $E_{hyst,cyc}$ are the viscous and hysteretic dissipated energy per cycle. In this study, Abaqus directly extracts these energy components through history output requests, as previously mentioned, eliminating the need to compute Jacobsen's areas from hysteretic loops.

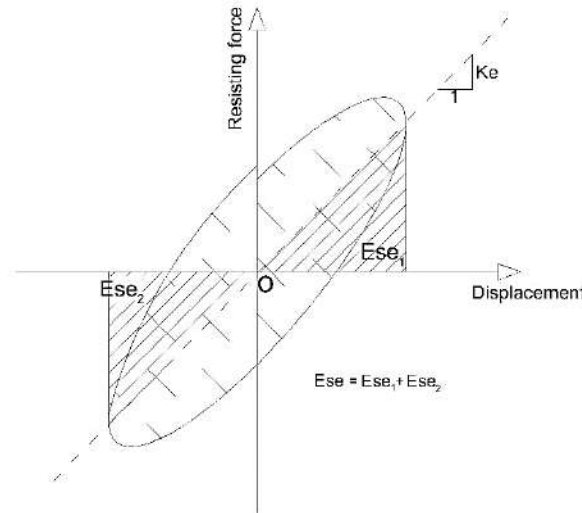


Fig. 2. Hysteresis loop illustrating energy dissipation and absorption

3.3. Takeda Degrading Model

A principal methodological contribution of this study is the explicit integration of stiffness degradation into the cycle-by-cycle evaluation of the EVDR. In conventional equivalent viscous damping formulations, $\xi_{viscous}$ is typically assumed constant (often 5%), implicitly treating it as independent of stiffness degradation during inelastic cyclic loading [29-31]. Likewise, nonlinear time-history analyses frequently model viscous dissipation using the initial stiffness. However, because the damping force depends on both the adopted stiffness and the response velocity, retaining the initial stiffness after structural yielding can generate spurious damping forces, as demonstrated by Priestley et al. [31]. Accordingly, using the tangent (degraded) stiffness as the basis for $\xi_{viscous}$ calculations provides a more physically transparent and consistent representation of damping in degrading RC frames [29,30].

The present study incorporates a Takeda degrading hysteresis model directly into the equivalent damping evaluation by updating the degraded stiffness and, consequently, the corresponding ω_n at each loading cycle. The viscous component of the EVDR is then evaluated for each cycle using Eq. (2), enabling $\xi_{viscous}$ and, consequently, $\xi_{eq,total}$, to evolve consistently with damage accumulation and structural softening. By embedding this cycle-dependent stiffness degradation into an energy-based damping evaluation, the proposed procedure overcomes a central limitation of traditional EVDR methods and provides a more physically transparent description of damping in degrading RC frames. The degraded stiffness over cycles is calculated as:

$$K_i = K_0 (\Delta_y / \Delta_m)^\alpha \quad (5)$$

where K_i is the unloading stiffness in cycle i , K_0 is the initial elastic stiffness, Δ_y is the yield displacement, Δ_m is the maximum displacement experienced per cycle, and α is the unloading stiffness coefficient, taken as 0.5 in this study.

The yield displacement was determined from the nonlinear dynamic analysis by monitoring the evolution of longitudinal reinforcement stress throughout the time-history response. The yield criterion was defined as the instant when the average longitudinal steel stress in critical regions first reached the specified yield stress (f_y), following the approach adopted by Peng and Guner [32] for direct displacement-based seismic assessment of concrete frames.

4. Material Characteristics and Finite Element Discretization

4.1. Materials

4.1.1 Concrete Stress-Strain Relation

The nonlinear behavior of concrete is modeled using the Concrete Damage Plasticity Model (CDPM), incorporating a constitutive law for compression based on Hafezolzhorani et al. [33].

$$\sigma_c = \sigma_{cu} \left[2 \left(\frac{\varepsilon_c}{\varepsilon'_c} \right) - \left(\frac{\varepsilon_c}{\varepsilon'_c} \right)^2 \right] \quad (6)$$

where σ_c and ε_c represent the compressive stress and strain, respectively, and σ_{cu} and ε'_c denote the ultimate compressive strength and strain of unconfined concrete.

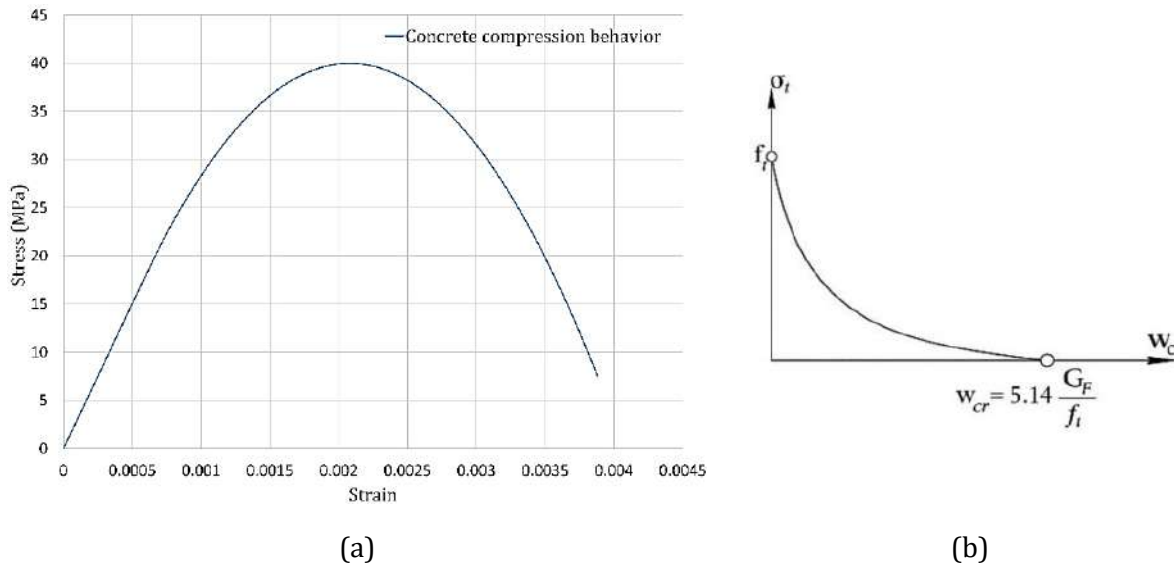


Fig. 3. (a) Concrete uniaxial compressive stress-strain diagram; (b) stress–crack opening diagram for uniaxial concrete in tension [34, 35]

The tensile behavior of concrete is modeled using an exponential softening law to capture the progressive degradation of tensile stress as a function of crack opening displacement. This formulation is based on the crack-opening model proposed by Cornelissen et al. [34], which defines the relationship between the normal tensile stress σ_t and the crack opening displacement, w_t , as follows:

$$\frac{\sigma_t}{f_{ct}} = \left[1 + \left(c_1 \frac{w_t}{w_{cr}} \right)^3 \right] e^{(-c_2 \frac{w_t}{w_{cr}})} - \frac{w_t}{w_{cr}} (1 + c_1^3) e^{(-c_2)} \quad (7)$$

In this expression, f_{ct} denotes the peak tensile strength, and w_{cr} is the critical crack-opening displacement corresponding to complete tensile stress release. The dimensionless softening parameters are taken as $c_1 = 3.00$ and $c_2 = 6.93$, as recommended in the original model [34].

The associated fracture energy G_f , representing the energy dissipated per unit of crack surface during complete tensile failure, is evaluated in accordance with the CEB-FIP Model Code 1990 [35]:

$$G_f = G_{f0} \left(\frac{f_{cm}}{f_{cm0}} \right)^{0.7} \quad (8)$$

where $f_{cm} = f_{ck} + 8$ MPa denotes the mean compressive strength of concrete, and $f_{cm0} = 10$ MPa is the reference strength.

Moreover, the CDP model enables progressive damage analysis of concrete by introducing a scalar damage variable, " d ", ranging from 0 (undamaged) to 1 (fully damaged or fractured). The model

distinguishes two damage parameters: d_c for compression-induced damage and d_t for tension-induced damage, each quantifying the reduction in elastic modulus associated with crushing and cracking, respectively.

Under cyclic loading, concrete exhibits complex degradation behavior characterized by repeated opening and closing of microcracks. The CDPM in Abaqus/CAE captures the asymmetric stiffness recovery of concrete under reversible loading by using independent recovery parameters, w_t for tension and w_c for compression, to control the degree of stiffness regained upon load reversal. The effective elastic modulus is governed by $E = (1 - d_t) E_0$ in the compression branch following tensile damage and by $E = (1 - d_t)(1 - d_c) E_0$ in the tension branch after compressive crushing. The default Abaqus/CAE stiffness recovery parameters ($w_t = 0$, $w_c = 1$) specify complete compressive stiffness restoration upon crack closure while preventing tensile stiffness recovery following compressive crushing. Figure 4 illustrates the stress-strain response under a complete tension-compression-tension loading cycle.

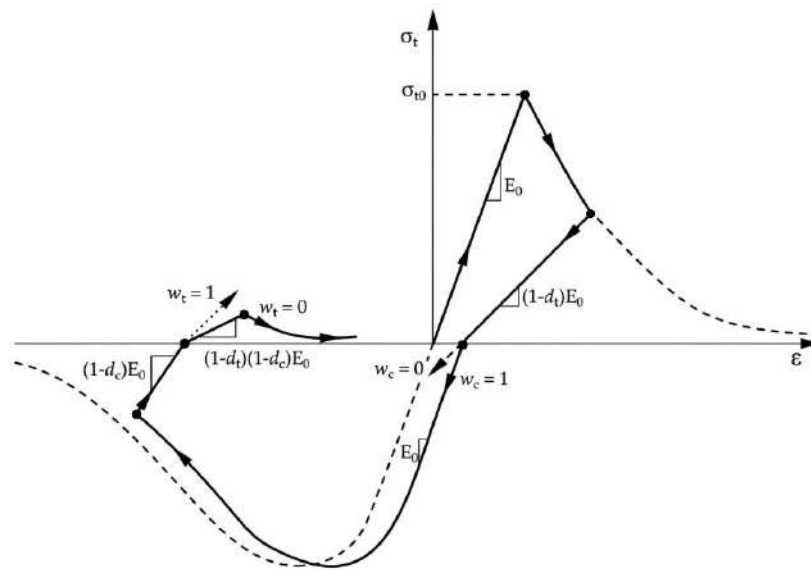


Fig. 4. Uniaxial cyclic behavior [22]

Sinha et al. [36] showed that concrete cylinders under cyclic loading exhibit significant stiffness degradation at a critical compression damage value of $d_c = 0.30$. This threshold aligns with findings from multiple CDP model studies that associate this value with severe crushing and is therefore adopted as the concrete crushing criterion in this study [37].

4.1.2 Properties of Steel Reinforcement and Steel-Concrete Interaction

HRB400 steel reinforcement (nominal yield strength $f_y = 400$ MPa) was used for all structural members. Steel behavior was modeled using a Chaboche-type combined isotropic-kinematic hardening plasticity law based on the von Mises yield criterion. This formulation captures key nonlinear cyclic phenomena, including the Bauschinger effect and cyclic hardening or softening through concurrent translation and expansion of the yield surface. All hardening parameters were adopted from Lu et al. [38]. A perfect bond was assumed at the concrete-steel interface.

4.2. Finite Element Meshing and Discretization Strategy

Concrete was discretized using eight-node linear brick solid elements with reduced integration (C3D8R) to mitigate shear locking and accurately capture flexural deformations in RC joint regions [39]. Steel reinforcement was modeled using two-node truss elements (T3D2), whereas CFRP layers were modeled using four-node reduced-integration shell elements (S4R) with orthotropic material properties defined for each ply.

5. Modal Analysis

A modal analysis using the Lanczos method in Abaqus/CAE [22] was performed to assess the dynamic characteristics of both the primary and the CFRP-retrofitted structures, assuming linear elastic material behavior for all frames. The first four eigenfrequencies were extracted and evaluated, as shown in Table 2. Rayleigh damping coefficients were calculated using the first and third vibrational modes. This mode selection ensured that the cumulative mass participation in the X-direction (Fig. 1), which aligns with the direction of the imposed base excitation, exceeded 95%.

Table 2. Comparison of modal characteristics: Primary vs. CFRP-retrofitted frames

Mode	Primary frame		CFRP retrofitted frame (Configuration A)		CFRP retrofitted frame (Configuration B)	
	Eigen frequencies (Cycle/time)	Mass participating ratio in X direction (%)	Eigen frequencies (Cycle/time)	Mass participating ratio in X direction (%)	Eigen frequencies (Cycle/time)	Mass participating ratio in X direction (%)
1	2.89	82.43	2.91	82.42	2.94	82.54
2	9.62	12.21	9.68	12.21	9.76	12.16
3	17.52	5.36	17.58	5.37	17.68	5.30
4	26.01	0.0	26.04	0	26.10	0

A comparison of the modal characteristics of the primary and CFRP-retrofitted frames showed only minor variations in natural frequencies and modal mass participation ratios across all extracted modes. This suggests that CFRP retrofitting did not significantly alter the global linear elastic stiffness of the frames. Consequently, the differences observed in the subsequent nonlinear IDA can be attributed primarily to changes in post-elastic behavior induced by the retrofit.

Moreover, the near equivalence in natural frequency between the primary and retrofitted frames enables a fair comparison of their energetic behavior under identical sinusoidal base displacements. This similarity ensures that any differences in structural response observed during IDA can be confidently attributed to the CFRP retrofit rather than to resonance effects arising from a mismatch between the imposed excitation frequency and the natural frequency of each frame.

6. Nonlinear Model Validation Procedure

An energy-based validation procedure was developed to assess the accuracy of the nonlinear primary frame model by comparing its response to that of an equivalent linear system. The comparison was based on two critical parameters, the effective stiffness K_{eff} and the EVDR, following the methodology outlined in [40].

The nonlinear primary frame was subjected to harmonic base excitation at its fundamental frequency, defined as $u(t) = A \sin(\omega t)$ with $\omega = \omega_n$. The displacement amplitude, $A = 6$ mm, was calibrated so that the model reached the specified threshold after 14 consecutive loading cycles. A 5% damping ratio was assigned using Rayleigh damping coefficients. Figure 5 shows the time history of the primary frame's top displacement response.

Since the validation procedure aims to verify the nonlinear frame behavior through comparison with its linear substitute model, the accuracy of the linear model must first be established. To this end, the linear substitute model was subjected to the same base displacement excitation as the nonlinear frame, and the EVDR was evaluated using the strain energy method once a stable cyclic response had been achieved. The analysis yielded an EVDR of approximately 4.8%, closely matching the prescribed 5% damping ratio. This confirms that the linear model accurately captures the prescribed damping characteristics and can therefore be considered fully validated for use as a reliable reference in the subsequent nonlinear model evaluation.

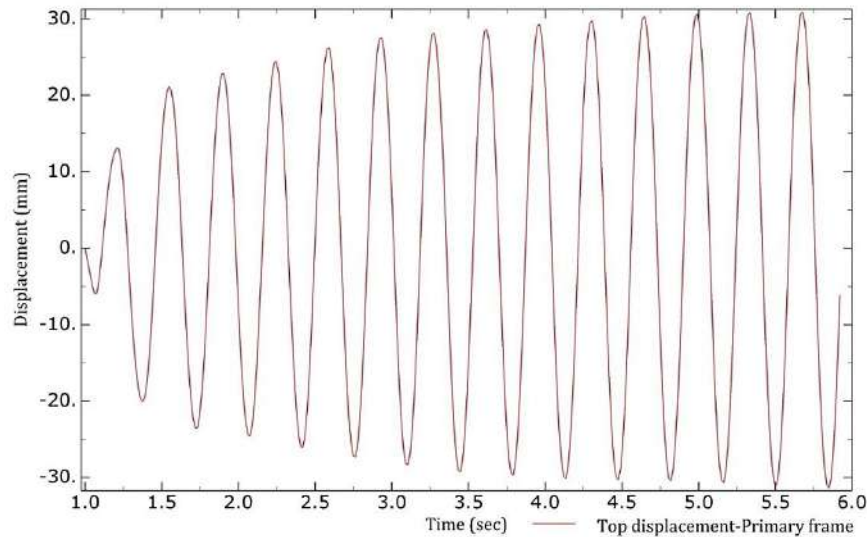


Fig. 5. Top displacement response of the primary frame subjected to sinusoidal base displacement at its resonance frequency

The validation procedure for the nonlinear frame consisted of four sequential steps:

- Ductility assessment: The displacement ductility ratio $\mu = \Delta_m / \Delta_y$ was evaluated based on the final three cycles of the nonlinear response, during which the frame exhibited stable inelastic behavior with peak displacement amplitudes approximately equal to Δ_m .
- Damping assignment: The average EVDR computed from the last three cycles of the nonlinear response was assigned to the linear substitute model, which was then subjected to the same base displacement input as the nonlinear frame.
- Stiffness reduction: The linear substitute model was initially assigned the elastic stiffness K_0 and subjected to the same base excitation as the nonlinear frame. To account for inelastic stiffness degradation, the elastic stiffness of the linear model was reduced using the Takeda degradation ratio, computed from the mean unloading stiffness degradation observed over the last three stabilized cycles of the nonlinear response. This stiffness adjustment was applied together with the EVDR specified in step 2, ensuring a consistent representation of both stiffness and energy dissipation characteristics in the linear model. Once the linear model reached a steady-state response, the peak roof displacement was extracted and systematically compared with the mean value of Δ_m obtained from the last three cycles of the nonlinear analysis. Close agreement between the linear and nonlinear responses, specifically matching peak displacement within an acceptable tolerance, confirmed that the equivalent linear model adequately captured the essential inelastic behavior.
- Energetic verification: The energy dissipated per cycle during the steady-state response (corresponding to the attainment of Δ_m) was computed for the linear elastic model and compared with the average per-cycle energy dissipation observed over the final three cycles of the nonlinear analysis. The corresponding energy dissipation values at the associated peak roof displacement are summarized in Table 3.

Table 3. Comparison of energy dissipation between the nonlinear primary frame model and the linear substitute model

Average $\xi_{eq,total}$ over the last three cycles (%)	Maximum top displacement Δ_m reached in the nonlinear model (mm)	Reduced frequency in the equivalent linear substitute model (Hz)	Top displacement in the equivalent linear substitute model (mm)	Average dissipated energy in the nonlinear model over the last three cycles (J)	Dissipated energy in the linear substitute model per cycle at steady-state response (J)
9.72	31.4	2.63	31	1.3×10^3	1.36×10^3

The results showed that the nonlinear primary frame accurately captured the expected energy dissipation behavior when compared with the validated linear reference model. This consistency indicates that the nonlinear model provides a reliable representation of the inelastic response under cyclic loading, confirming its adequacy for subsequent dynamic analysis.

7. Results and Discussion

7.1. Energy-Based Performance Evaluation of Configuration A (Flange-Bonded CFRP) Under Resonant Loading

7.1.1 Energy Dissipation Characteristics

A comparative analysis of energy dissipation mechanisms was conducted for the primary frame and its CFRP-retrofitted counterpart employing Configuration A under resonant excitation conditions. The IDA revealed an unexpected outcome: the retrofitted frame reached its ultimate member curvature after only 10 cycles, compared to 14 cycles sustained by the primary frame before failure.

Cycle-by-cycle energy analysis during the common operational period (cycles 1–10) showed that the retrofit significantly altered the system's energy dissipation mechanisms (Fig. 6). The retrofitted frame exhibited higher E_{se} and E_v than the primary frame, with increases of approximately 67% and 48%, respectively, by cycle 10. However, E_p in the retrofitted system increased at a slower rate, resulting in a reduction of up to 20% relative to the primary frame by the tenth cycle, indicative of ongoing stiffness degradation. According to Akbas et al. [41], both E_k and E_{se} constitute relatively small portions of E_{input} at any given time during the vibrational response. Consequently, E_v and E_{hyst} emerge as the dominant contributors to input energy dissipation.

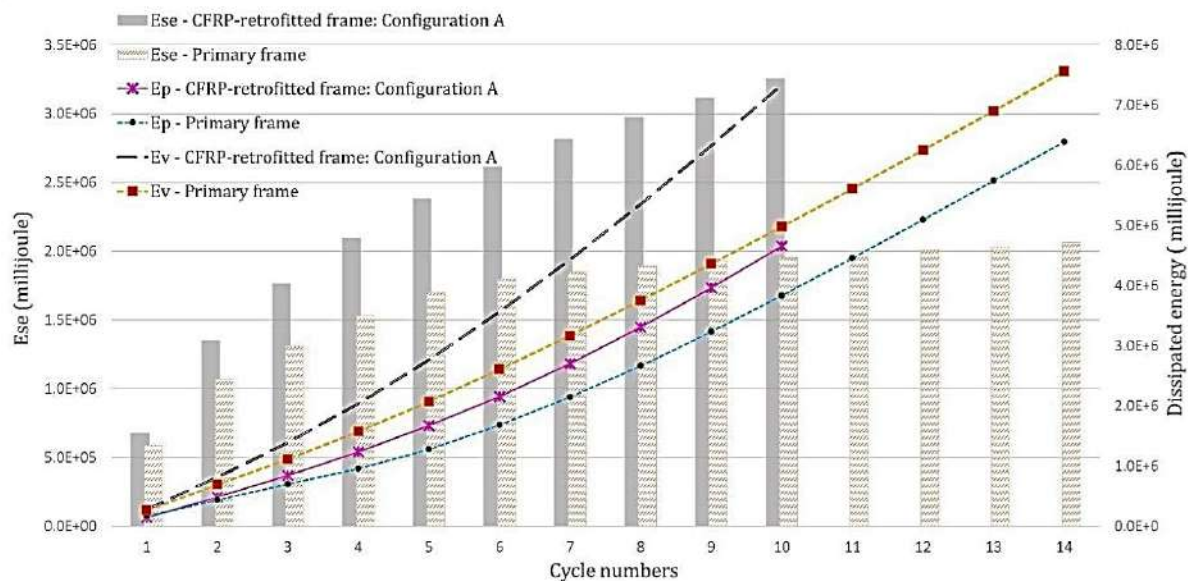


Fig. 6. Maximum elastic strain energy per cycle and cumulative viscous and plastic energy dissipation at resonance: primary vs. Configuration A CFRP-retrofitted frame

Over the full loading history, both E_v and E_p were consistently higher in the CFRP-retrofitted frame than in the primary frame, indicating that the retrofit enabled the structure to attract more E_{input} across cycles. The associated increase in input energy demand was accommodated primarily through greater viscous energy dissipation rather than additional inelastic hysteretic dissipation, as evidenced by the significantly higher accumulation rate of E_v in the retrofitted configuration. These findings indicate that CFRP retrofitting alters the energy dissipation mechanism, with a larger portion of the input energy temporarily stored as recoverable elastic strain energy and subsequently dissipated through viscous damping, thereby reducing reliance on inelastic hysteretic mechanisms.

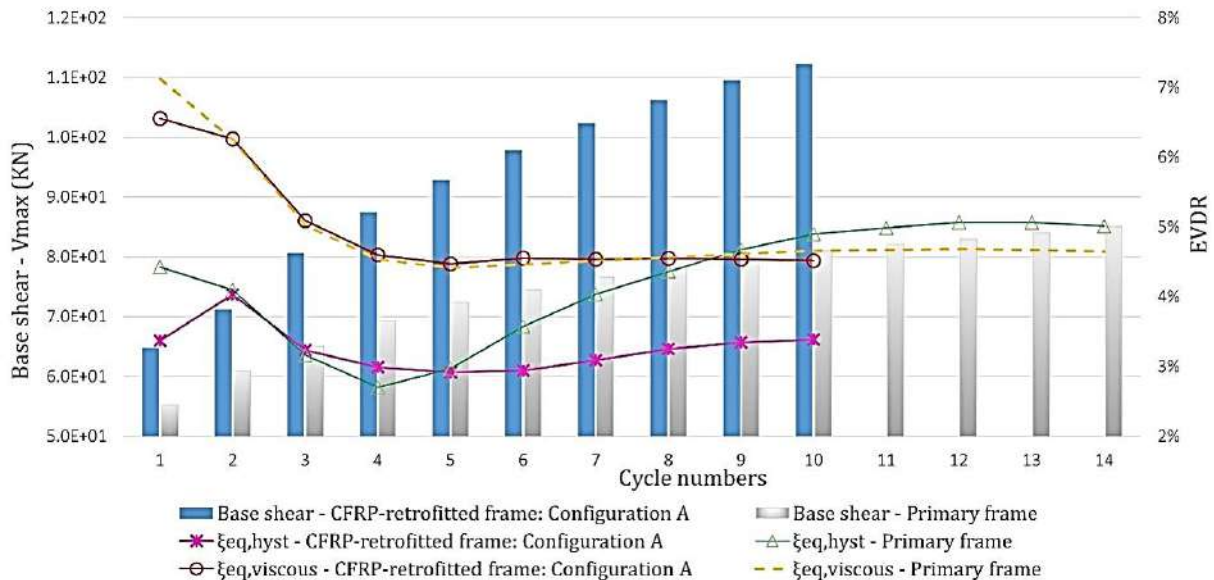
7.1.2 Equivalent Viscous Damping Ratio Evolution

Analysis of the EVDR components revealed distinct damping characteristics between the CFRP-retrofitted and the primary frames (Fig. 7(a)). Both systems exhibited a transient decrease in $\xi_{viscous}$ and ξ_{hyst} during the initial loading cycles, reflecting structural adaptation to cyclic excitation. Following this phase, $\xi_{viscous}$ converged to approximately 5% in both frames, consistent with the assigned inherent damping ratio, whereas ξ_{hyst} evolved differently.

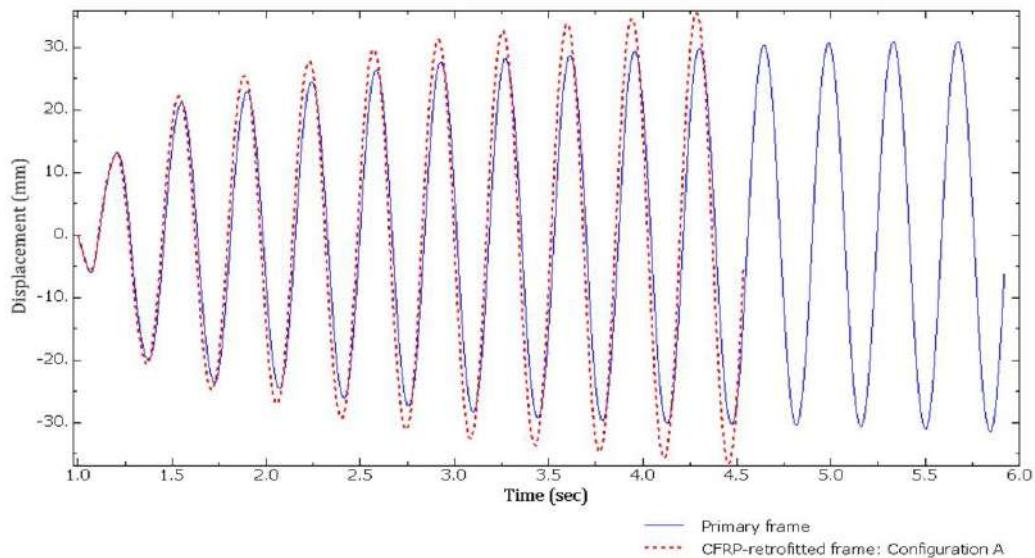
The primary frame showed progressive growth in ξ_{hyst} from 2.7% at cycle 4 to 5% at cycle 11, maintaining this plateau through cycle 14. In contrast, the CFRP-retrofitted frame achieved early stabilization of ξ_{hyst} within the narrow range of 2.8–3.2% after only four consecutive cycles, representing an approximately 40% reduction relative to the peak ξ_{hyst} observed in the primary frame. As a result, the CFRP retrofit reduced the overall EVDR, driven primarily by lowering ξ_{hyst} . This reduction stems from the higher E_{se} of the retrofitted frame, which results in a smaller fraction of E_{input} being dissipated through hysteretic mechanisms and a larger fraction being stored and released elastically, consistent with the energy-based observations previously established in Fig. 6.

Time-history analysis of roof displacements revealed that the retrofitted frame exhibited greater displacement responses than the primary frame (Fig. 7(b)). Given the lower EVDR induced by the retrofit, this behavior is consistent with the dynamic response of systems operating under resonance conditions, where damping plays a critical role in controlling displacement amplitudes [42]. In fact, a lower EVDR indicates the structure is more elastic (storing more E_{se}) but less damped, and therefore dissipates vibrational energy less effectively per cycle, resulting in larger oscillatory displacements.

A cycle-by-cycle evaluation of the maximum base shear values (V_{max}) showed that the retrofitted frame consistently exhibited higher V_{max} in each cycle than the primary frame (Fig. 7(a)). The concurrent increase in roof displacement and V_{max} resulted in substantially greater E_{se} per cycle in the retrofitted system, thereby increasing member force demands, particularly in the CFRP-retrofitted members. This behavior highlights the need for appropriate CFRP detailing and local strengthening measures to accommodate the additional force demands that develop under resonant excitation. Nonetheless, the observed premature failure of the retrofitted frame relative to its primary counterpart suggests potential limitations of the adopted retrofit configuration.



(a)



(b)

Fig. 7. Primary vs. Configuration A CFRP-retrofitted frame under resonance-frequency excitation: (a) evolution of EVDR and V_{max} at resonance; (b) comparative top displacement responses

Finite element analyses showed that elevated member forces triggered crack-induced delamination at the beam-column interface, substantially compromising stress transfer between the flange-bonded CFRP laminates and the reinforced concrete substrate. In combination with amplified dynamic forces, this mechanism precipitated premature plastic hinge formation in beam segments adjacent to the joint (Fig. 8). Consequently, the retrofitted frame reached its ultimate curvature limit earlier than the primary frame, resulting in localized member failure at the beam-column interface. This counterintuitive result indicates that, under resonant excitation, CFRP retrofitting may reduce structural endurance when amplified dynamic forces cause premature interface delamination and localized member failure before the retrofitted system's intended capacity is fully mobilized.

To address these limitations, engineers should consider alternative strengthening strategies that employ selective retrofit configurations. These configurations should be designed to limit excessive deformation at critical beam-column interfaces by promoting plastic hinge redistribution and shifting inelastic strains away from vulnerable joint regions. Configuration B, analyzed later in this study, exemplifies this approach by demonstrating how targeted retrofitting can redistribute plastic deformation away from vulnerable beam-column interfaces while controlling the magnitude of the force amplification that led to premature failure in Configuration A.

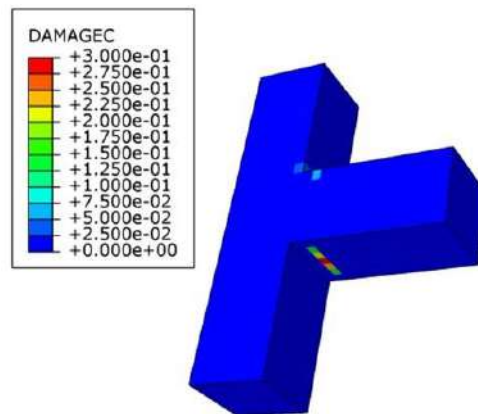


Fig. 8. Ultimate curvature at the beam-column interface following Configuration A after local CFRP delamination

7.2. Comparative Analysis of CFRP Flange-Web Bonded Configuration: Enhanced Performance Through Plastic Hinge Relocation

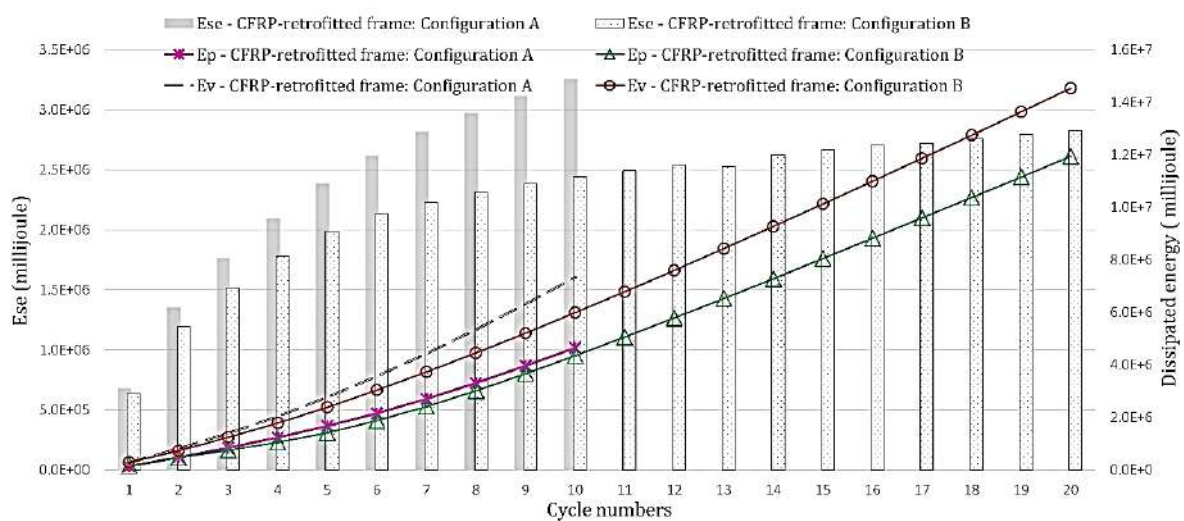
7.2.1 Energy Dissipation Characteristics

The energy dissipation characteristics of Configuration B, featuring reduced CFRP length ($L_f = 0.67h_b$) and combined flange-web bonding (Fig. 1(b)), were evaluated under identical sinusoidal base displacement $u(t) = 6 \sin(\omega t)$ to enable direct comparison with Configuration A. In all remaining energy-related plots in this study, the response histories were truncated after 20 loading cycles, as this range was sufficient to capture the essential energy dissipation trends, rendering additional cycles less informative for comparative analysis.

Comparative energy analysis over the shared cycle range (cycles 1–10; Fig. 9(a)) showed that Configuration A mobilized higher E_{se} and greater cumulative E_v than Configuration B, with maximum increases of approximately 33% and 22%, respectively, by cycle 10. During this interval, both configurations exhibited comparable E_p levels. Beyond cycle 10, E_{se} , E_v , and E_p components in Configuration B continued to increase steadily throughout the remaining cycles.

The key performance distinction between the two CFRP-retrofitted configurations lies in their cyclic durability. Configuration B demonstrated significantly greater endurance, withstanding 22 consecutive loading cycles before reaching the ultimate curvature limit, compared with only 10 cycles for Configuration A. This enhanced cyclic durability is attributed to the successful relocation of the plastic hinge achieved through the modified CFRP layout. By strategically limiting the flange-bonded length and incorporating web-bonded reinforcement, Configuration B redistributed plastic strain accumulation away from the vulnerable beam-column interface toward the CFRP termination region. Longitudinal beam reinforcement strain measurements confirmed this redistribution: steel rebars at the CFRP termination point exhibited significant plastic strains, whereas those within the CFRP-covered zone showed minimal inelastic deformation (Fig. 10).

Moreover, energy-based response evaluation of the primary frame and the CFRP-retrofitted Configuration B (Fig. 9(b)) revealed dynamic trends consistent with those observed in Configuration A, with progressive increases in both recoverable and dissipative energy components across loading cycles. By cycle 14, Configuration B exhibited approximately 30% higher maximum per-cycle E_{se} , 23% higher E_v , and 14% higher E_p than the primary frame. Although these increases were less pronounced than those observed in Configuration A, they nonetheless represent substantial improvements in both E_{se} and E_v relative to the primary frame, with only a moderate increase in E_p across the 14-cycle range.



(a)

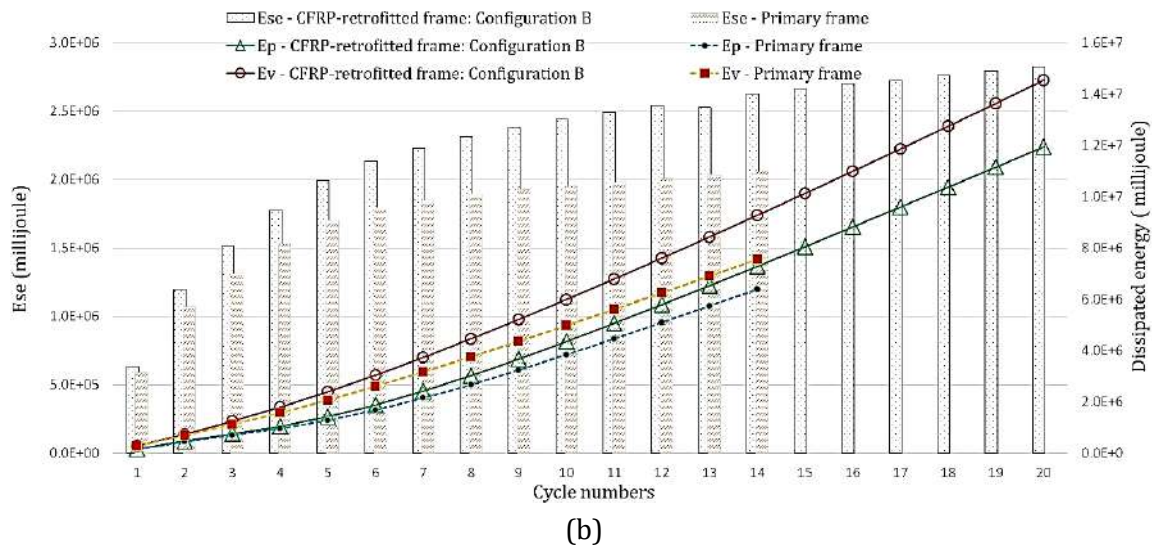


Fig. 9. Maximum elastic strain energy per cycle and cumulative viscous and plastic energy dissipation at resonance: (a) Configuration A vs. Configuration B CFRP-retrofitted frames; (b) primary vs. Configuration B CFRP-retrofitted frame

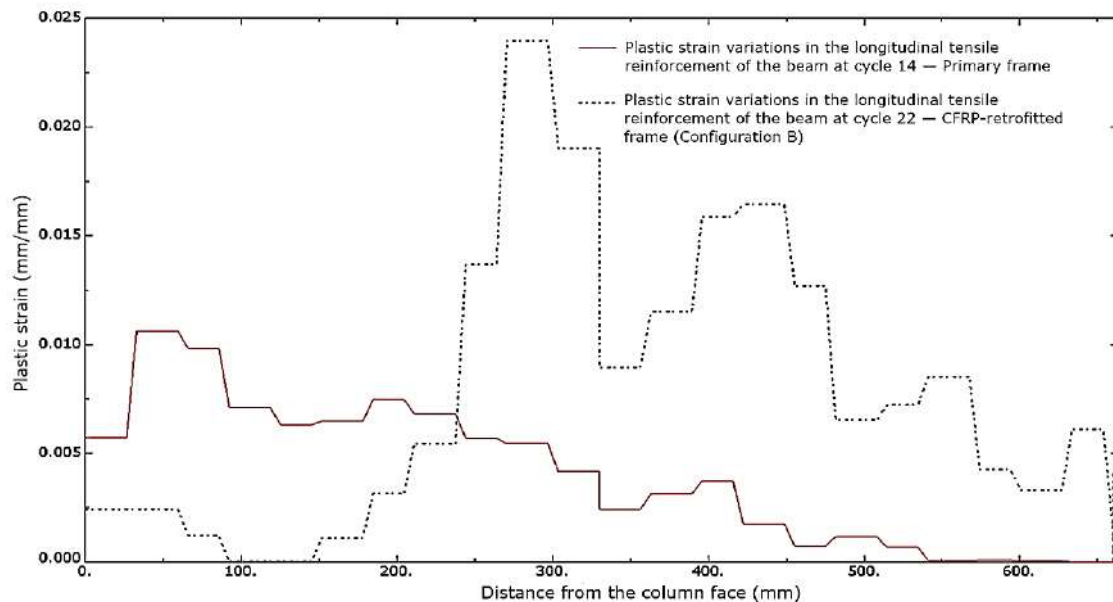


Fig. 10. Strain variation in longitudinal beam reinforcement: primary vs. Configuration B CFRP-retrofitted frame

7.2.2 Equivalent Viscous Damping Ratio Evolution

Comparison between Configurations A and B reveals distinct EVDR characteristics (Fig. 11). In both cases, $\xi_{viscous}$ converged to the assigned 5% inherent damping ratio following the initial transient cycles. However, the evolution of the hysteretic component differed significantly. Configuration B exhibited a gradual increase in ξ_{hyst} , rising from 2.7% at cycle 4 to approximately 4.6% by cycle 10 before stabilizing in the remaining cycles. In contrast, Configuration A consistently showed lower ξ_{hyst} values, plateauing between cycles 8 and 10 at 2.8% to 3.2%, as previously noted. This indicates a markedly reduced reliance on hysteretic energy dissipation in Configuration A relative to Configuration B. Moreover, the cycle-by-cycle increase in V_{max} was less pronounced in Configuration B than in Configuration A, indicating lower force demands on the RC members. This reduction in force demand, coupled with the strategic relocation of plastic hinges away from critical joint regions, mitigated early member failure and enhanced the frame's capacity to sustain cyclic deformation. In summary, under resonant sinusoidal excitation, CFRP retrofitting significantly altered the frame's energy dissipation behavior. Both configurations increased E_{se} and shifted energy dissipation toward viscous mechanisms. However, these improvements were accompanied

by a reduction in overall EVDR and increased internal force and base shear demands, with the severity of these effects strongly dependent on the adopted retrofit configuration.

Configuration A, which used continuous flange bonding with an extended development length ($L_f = 2h_b$), achieved the highest E_{se} but also caused delamination at the CFRP-concrete interface, leading to premature member failure relative to the primary frame. In contrast, Configuration B mitigated these adverse effects by limiting the flange-bonded length and adding web-bonded reinforcement, thereby promoting plastic hinge relocation and redistributing inelastic demands to less critical beam regions. Although it generated lower E_{se} than Configuration A, Configuration B substantially enhanced the frame's cyclic durability by delaying member failure. These findings highlight the importance of balancing increased elastic energy with optimized energy dissipation, manageable force demands, and controlled damage mechanisms to enhance overall seismic resilience.

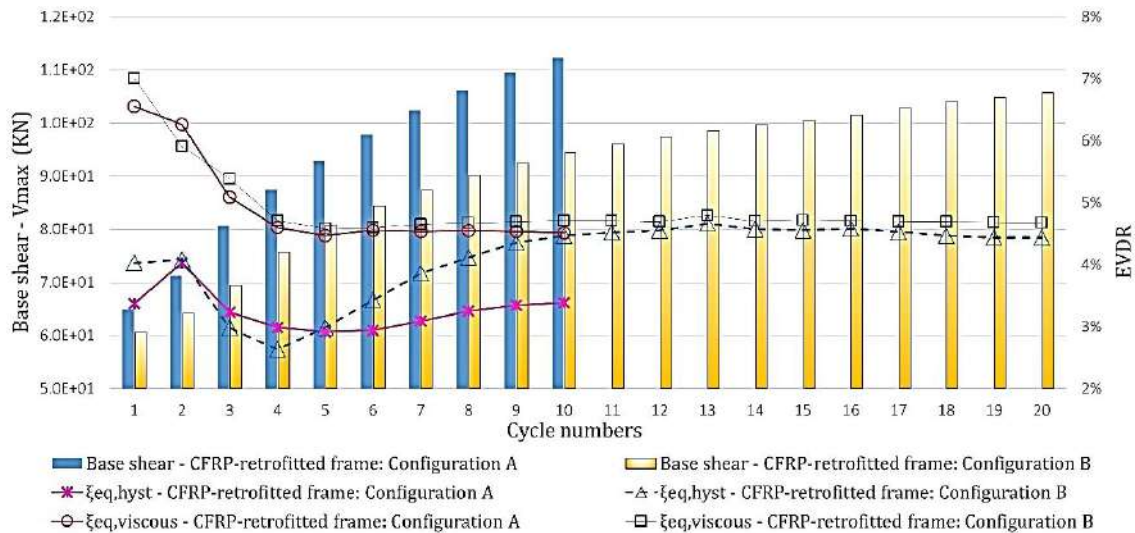
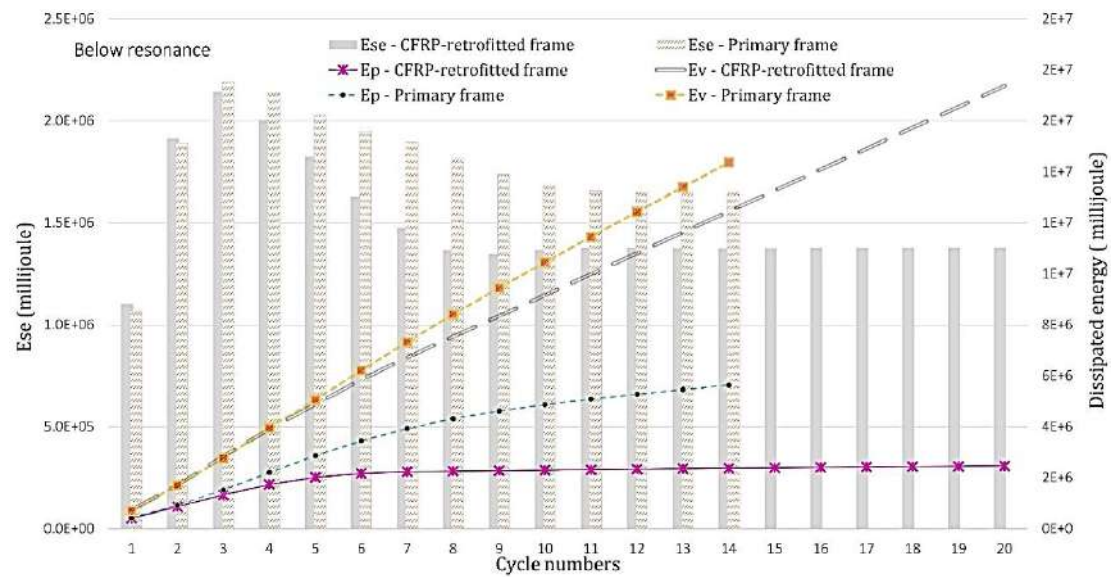


Fig. 11. Cycle-by-cycle evolution of EVDR and V_{max} in CFRP-retrofitted frames under resonance-frequency excitation: Configuration A vs. Configuration B

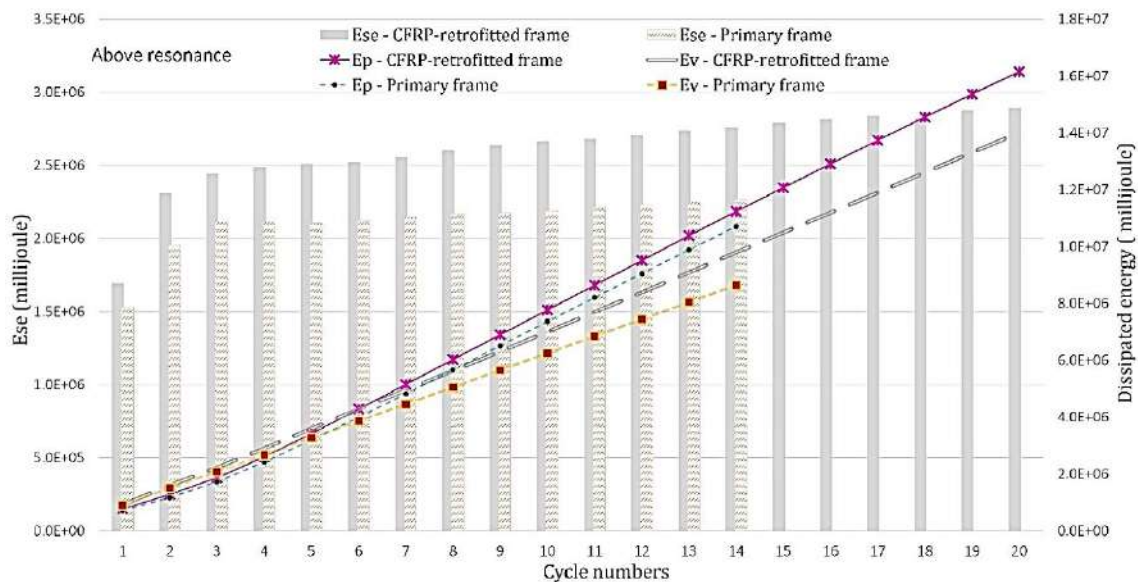
7.3. Assessment of EVDR Consistency Under Off-Resonance Excitation

To assess whether the EVDR trends observed at resonance also held under off-resonance conditions, a complementary set of analyses was performed. Parametric off-resonance simulations were conducted at frequency ratios of $0.75\omega_n$ (below resonance, B-RES) and $1.25\omega_n$ (above resonance, A-RES). A performance-based calibration procedure, consistent with that used for the resonance case, was implemented to ensure comparable inelastic demand in the primary frame across all excitation frequencies. For each frequency ratio, the sinusoidal base displacement amplitude was iteratively adjusted until the primary frame reached its ultimate curvature limit state after 14 loading cycles. The same calibrated displacement history was then applied to the CFRP-retrofitted configuration, enabling a direct comparison of energy partitioning under identical base excitations. The resulting calibrated base displacement amplitudes were $A_{B-RES} = 11.8$ mm and $A_{A-RES} = 12.5$ mm for the below-resonance and above-resonance conditions, respectively.

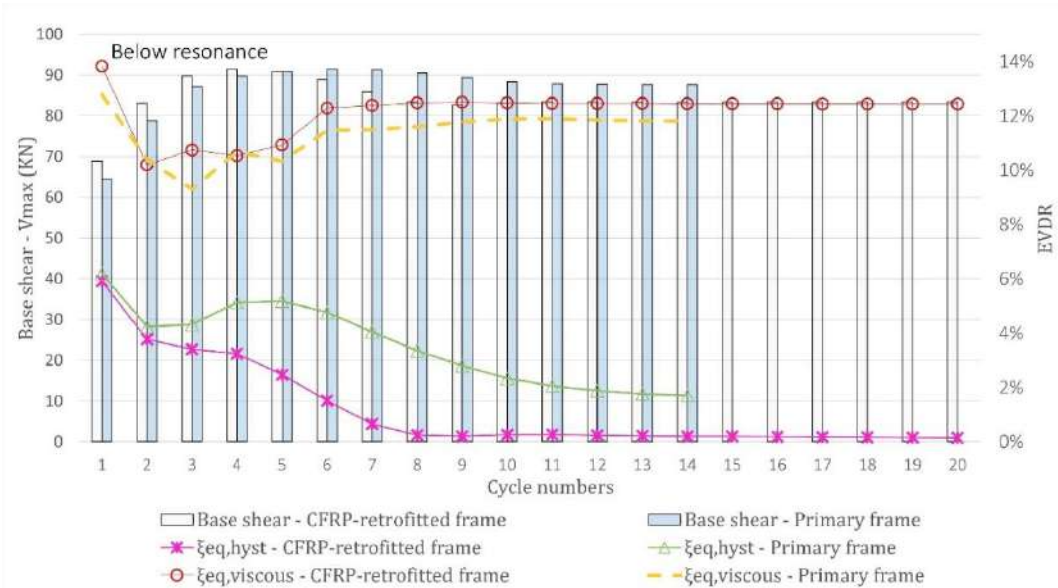
This methodology ensures that observed variations in EVDR are primarily attributable to frequency effects rather than to differences in structural deformation demand. Four simulation cases were analyzed: B-RES-primary, B-RES-CFRP, A-RES-primary, and A-RES-CFRP. Following the comparative assessment of energy dissipation behavior previously performed at resonance, Configuration B was selected as the CFRP retrofit configuration for off-resonance analyses. Its superior cyclic durability makes it a more representative and practical solution for seismic retrofit applications, justifying its use for subsequent off-resonance investigations. Figure 12 shows the evolution of the principal energy components (E_{se} , E_v , and E_p), along with the corresponding EVDR values, for both the below-resonance and above-resonance cases.



(a)



(b)



(c)

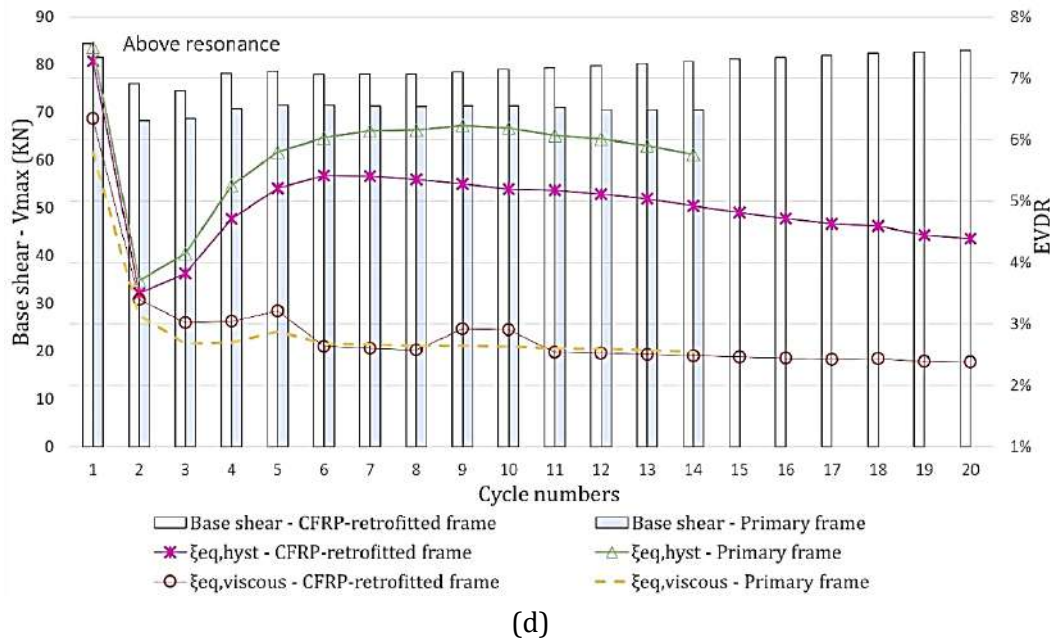


Fig. 12. Off-resonance response for the primary and CFRP-retrofitted frames: (a, b) maximum elastic strain energy per cycle and cumulative viscous and plastic dissipation; (c, d) cycle-by-cycle evolution of EVDR and V_{max}

7.3.1 Energy Dissipation Characteristics

Under below-resonance excitation, fundamental differences in energy dissipation mechanisms emerged between the primary and CFRP-retrofitted frames. The retrofitted frame exhibited superior cyclic durability, withstanding more than 20 loading cycles without reaching its ultimate curvature limit (Fig. 12(a)). Cycle-by-cycle comparison revealed that the retrofitted frame exhibited a rapid increase in E_p , followed by a clear plateau at approximately 2.4×10^6 mJ from around cycle 6 onward. This plateau indicates the cessation of plastic dissipation, consistent with elastic shakedown behavior, where further stiffness degradation is not expected. In contrast, the primary frame showed continuous growth of E_p , reaching approximately 5.6×10^6 mJ by cycle 14, indicative of progressive structural deterioration. Following the onset of elastic shakedown, the retrofitted frame transitioned to a purely viscous-dominated energy dissipation regime, reflecting the retrofit's ability to suppress plastic mechanisms and promote a predominantly elastic, non-damaging response under these loading conditions.

Throughout the loading protocol, the CFRP-retrofitted frame consistently exhibited lower values of both E_v and E_{se} than the primary frame. By cycle 14, the retrofitted system had accumulated approximately 1.24×10^7 mJ of E_v , about 16% less than the primary frame, and continued to increase linearly thereafter. From around cycle 8 onward, both systems reached a stable peak E_{se} for the remaining cycles, with the retrofitted frame sustaining a value near 1.38×10^6 mJ, approximately 20% lower than the peak value observed in the primary frame. Neglecting the minor contributions of both E_{se} and E_k relative to E_{input} [41], the CFRP-retrofitted frame dissipated less total energy than the primary frame over the shared cycle interval, indicating that less E_{input} was imparted to the system.

Comparative analysis of the response histories indicates that the retrofitted frame exhibited markedly lower peak roof displacements than the primary frame (Fig. 13), while maintaining comparable V_{max} values (Fig. 12(c)). This combination of reduced displacement demand and similar base shear response reflects an enhancement in the global secant stiffness, defined here as the ratio of V_{max} to the corresponding roof displacement, resulting from the CFRP retrofit. The response behavior observed in the retrofitted frame is consistent with a stiffness-controlled regime, which typically occurs when the excitation frequency lies well below the structure's fundamental frequency. In such regimes, displacement amplitudes are primarily governed by the system's restoring stiffness rather than inertial effects [42]. The increased secant stiffness introduced by the

retrofit significantly constrained lateral displacements, thereby limiting both the onset and progression of plastic deformation. As a result, the structure rapidly transitioned into an elastic shakedown state, characterized by stable, recoverable elastic behavior under repeated loading. This elastic stabilization effectively curtailed further E_{input} from the ground motion, leading to a corresponding reduction in both E_v and E_{se} components throughout the response duration.

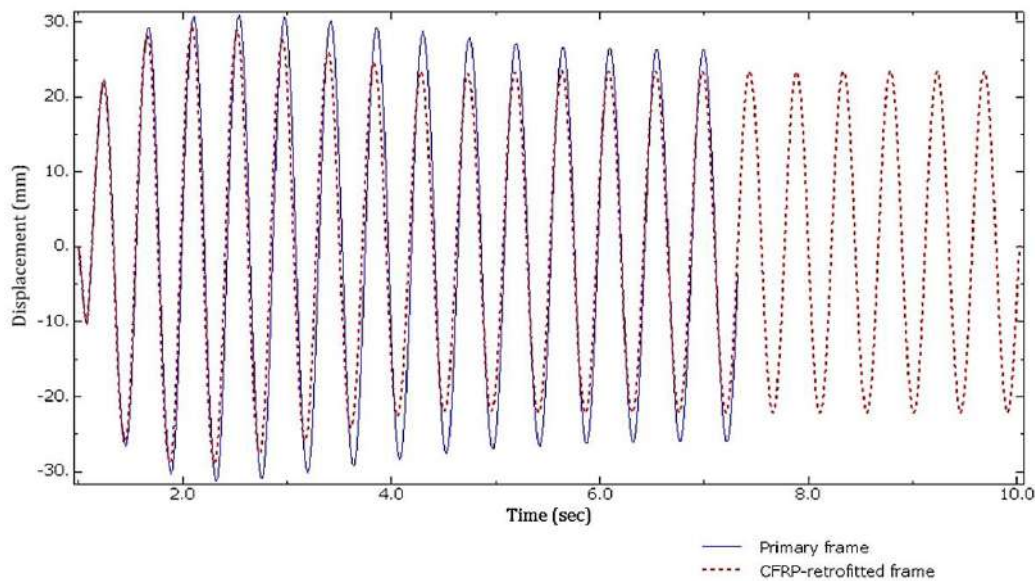


Fig. 13. Top displacement responses of the primary and CFRP-retrofitted frames under below-resonance sinusoidal base excitation

7.3.2 EVDR Response Below Resonance

In the CFRP-retrofitted frame, ξ_{hyst} exhibited pronounced decay after the initial transient cycles. It dropped sharply after the fifth cycle and continued to decrease until it effectively vanished, indicating the cessation of plastic dissipation (Fig. 12(c)). In the primary frame, ξ_{hyst} also began to decrease from approximately the fifth cycle onward; however, its rate of reduction per cycle was lower than in the retrofitted frame and eventually stabilized at a plateau of about 2% before reaching the deformation limit state.

In both frames, $\xi_{viscous}$ converged to relatively high, nearly constant values (approximately 12%), indicating that energy dissipation is predominantly governed by velocity-dependent mechanisms. These results show that, under the stiffness-controlled excitation levels considered, CFRP retrofitting can induce a pronounced shift toward elastic-dominated behavior. The retrofit effectively suppresses inelastic deformation, causing the system to dissipate most of its input energy through viscous mechanisms, whereas the primary frame retains a non-negligible hysteretic contribution.

7.3.3 Energy Partition Trends Above Resonance

At $1.25\omega_n$, the CFRP-retrofitted frame sustained loading up to cycle 28 before reaching its ultimate curvature limit state. Figure 12b shows that E_p remained the dominant energy dissipation mechanism in both frames over the shared 14-cycle interval, in contrast to the behavior observed in the below-resonance regime. However, the retrofitted frame exhibited a slightly higher rate of E_p accumulation, resulting in an overall increase of approximately 5% by cycle 14 relative to the primary frame.

From cycle 8 onward, the retrofitted frame exhibited a steeper rate of increase in E_v , surpassing that of the primary frame by roughly 15% at cycle 14. This progressive cycle-by-cycle increase in total energy dissipation indicates that the retrofit enabled the structure to absorb more E_{input} across cycles [41], consistent with observations under resonance-frequency excitation. Most of this additional energy was dissipated through viscous mechanisms, as also observed in the resonant case.

Regarding E_{se} , the retrofitted system reached a per-cycle peak of approximately 2.76×10^6 mJ by cycle 14. In contrast, the primary frame reached a lower peak of approximately 2.26×10^6 mJ by cycle 4, after which E_{se} plateaued. Beyond the shared cycle range, all energy components of the retrofitted frame continued to increase progressively.

A comparison of the roof displacement histories (Fig. 14) indicates that the two compared frames reached similar peak displacements. In contrast, the cycle-by-cycle base shear records (Fig. 12(d)) show consistently higher base shear forces in the retrofitted frame. This combination of similar displacement levels coupled with higher base shear reflects the enhanced secant stiffness provided by the retrofit. The observed response is indicative of an inertia-controlled regime, which arises when the excitation frequency greatly exceeds the structure's fundamental frequency. In this regime, displacement amplitudes are governed primarily by inertial forces and the frequency content of the ground motion, rather than by the structure's restoring stiffness [42]. Under such conditions, increasing stiffness does not necessarily reduce displacement demands. Instead, the enhanced secant stiffness of the retrofitted frame increased its dynamic coupling with the high-frequency components of the excitation, resulting in amplified acceleration responses and elevated base shear forces. Consequently, greater internal forces and higher E_{se} were generated during each vibration cycle. Additionally, the higher acceleration demands experienced by the retrofitted frame led to an increased velocity response, which, in turn, amplified E_v due to its velocity-dependent nature. However, under these amplified inertial forces, CFRP retrofitting was unable to fully suppress plastic deformation, which continued to develop once inertia-driven response became dominant.

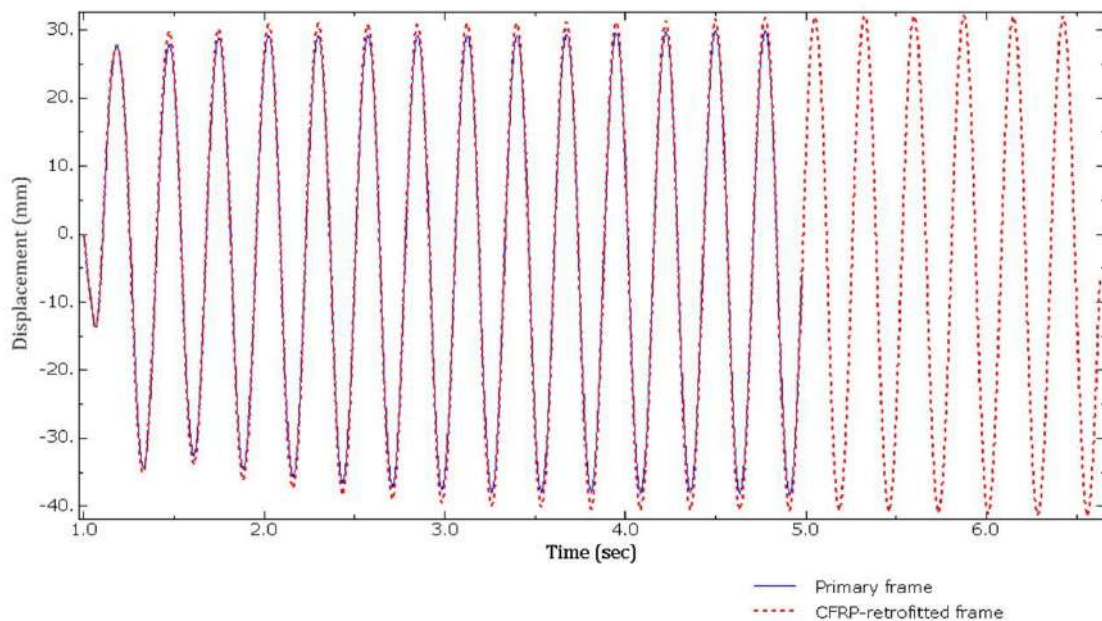


Fig. 14. Top displacement responses of the primary and CFRP-retrofitted frames under above-resonance sinusoidal base excitation

7.3.4 EVDR Response Above Resonance

Excluding the initial transient cycles, ξ_{hyst} in the CFRP-retrofitted frame increased rapidly between cycles 2 and 6, peaking at approximately 4.5% around cycles 6 to 8, before gradually declining in the subsequent cycles (Fig. 12(d)). In contrast, the primary frame exhibited a steeper initial rise in ξ_{hyst} , followed by a relatively stable plateau between 6% and 6.5% up to cycle 14. This consistently lower ξ_{hyst} reflects the enhanced recoverable elastic storage capacity provided by the CFRP retrofit. As a result, a greater proportion of E_{input} is accommodated through recoverable elastic mechanisms rather than irreversible plastic dissipation, as confirmed by the energy response plots (Fig. 12(b)).

Moreover, $\xi_{viscous}$ converged to approximately 2.5% in both frames shortly after the initial cycles, indicating that ξ_{hyst} remained the dominant contributor to the total EVDR.

Although the CFRP retrofit reduced the hysteretic contribution to $\xi_{eq,total}$ compared to the primary frame, this reduction was less significant than in the resonant case and considerably less pronounced than in the below-resonance regime. Consequently, in this regime, the retrofit was comparatively less effective at shifting the primary energy dissipation mechanism away from hysteretic behavior.

8. Conclusions

This study investigated the influence of CFRP retrofitting on the dynamic response of reinforced concrete frames subjected to harmonic base displacement excitation. An energy-based analytical framework was employed to evaluate the influence of various CFRP retrofit configurations on the global structural response, compared with a geometrically and materially identical unretrofitted primary frame under the same loading conditions. The comparative performance of the frames was primarily evaluated through the EVDR, which served as the principal performance indicator. In addition, the study examined the frequency-dependent behavior of CFRP-retrofitted frames, assessing how variations in excitation frequency affect their dynamic response. The key conclusions are:

- Effect of CFRP retrofitting on EVDR: CFRP retrofitting consistently reduced the global EVDR across the range of base excitation frequencies and retrofit configurations examined. However, the magnitude of this reduction was strongly influenced by both the adopted CFRP configuration and the imposed excitation frequency.
- Implications of CFRP configuration on cyclic durability: Results demonstrated the critical role of CFRP retrofit configuration in governing the cyclic durability of reinforced concrete frames. Under resonant excitation, Configuration A, characterized by extended flange-bonded CFRP sheets (e.g., $2h_b$ extensions consistent with ACI 440 guidelines [1]), reduced structural resilience under cyclic loading. This configuration significantly increased elastic energy storage and, in turn, decreased EVDR, thereby amplifying global base shear demands and internal force resultants. The resulting dynamic response promoted premature CFRP delamination and early member failure, effectively negating the intended benefits of the retrofit. In contrast, configurations that intentionally promote controlled plastic hinge relocation away from vulnerable joint zones (achieved by shortening flange-bonded lengths and adding web-bonded reinforcement, as in Configuration B) were shown to mitigate these adverse effects by providing a more favorable balance between moderate gains in elastic energy storage, optimized energy dissipation capacity, and acceptable force and damage demands, thereby enhancing cyclic durability and seismic resilience.
- Frequency-dependent retrofit effectiveness: CFRP retrofitting was found to modify the distribution of energy dissipation across the entire range of excitation frequencies considered. In all cases, a larger portion of the input energy was shifted from inelastic to viscous dissipation mechanisms, reducing the hysteretic contribution to the total EVDR. However, the magnitude of this reduction was highly dependent on the base excitation frequency. CFRP retrofitting was most effective in the stiffness-controlled (below-resonance) range, where it markedly reduced inelastic energy dissipation and promoted an essentially elastic response, thereby suppressing ξ_{hyst} . As the excitation frequency approached and exceeded resonance, this benefit progressively diminished. In the inertia-controlled (above-resonance) regime, ξ_{hyst} regained importance in the retrofitted frame, indicating that a larger share of the E_{input} was still dissipated through inelastic mechanisms despite the retrofit intervention. This frequency-dependent behavior highlights the critical importance of assessing the relationship between a structure's natural frequencies and the anticipated excitation frequency content when designing CFRP-retrofitted systems for seismic applications.
- No impact on linear-elastic properties: CFRP retrofitting preserved the structure's linear-elastic modal characteristics, indicating that it does not significantly alter the global linear-elastic stiffness of the retrofitted frames. Consequently, the performance gains observed in

nonlinear dynamic analyses are primarily attributable to modifications in the post-elastic behavior induced by CFRP retrofitting.

Based on the findings of this study, a practical design recommendation is proposed for CFRP-retrofitted RC frames subjected to dynamic loading: to maximize seismic resilience, retrofit strategies should prioritize shorter flange development lengths L_f , which intentionally promote controlled plastic hinge relocation away from the beam–column interface. This detailing can mitigate the pronounced EVDR reduction associated with extended flange-bonded configurations (e.g., $L_f = 2h_b$), which, in the present study, compromised cyclic endurance under resonant excitation. Furthermore, given the high sensitivity of retrofit performance to excitation frequency, an effective retrofit design approach must jointly consider both the CFRP detailing system and the targeted excitation frequency to ensure optimized energy dissipation, controlled force demand, and improved cyclic durability.

It is important to acknowledge that the present analysis was limited to the fundamental vibration mode, a single structural configuration, and specific CFRP retrofit layouts, with all observed failure modes governed by ductile beam mechanisms. Future investigations should therefore extend this framework by incorporating higher-mode contributions and assessing the influence of ground motion records with broad frequency content on the EVDR of FRP-retrofitted systems. In addition, studies exploring a broader range of structural typologies, alternative CFRP layouts, and diverse failure modes (such as column-dominated mechanisms and joint shear degradation) would contribute to a more comprehensive and generalized seismic performance assessment. Such investigations would clarify how energy dissipation patterns evolve across distinct damage states, ultimately supporting seismic design approaches that more effectively integrate strength enhancement with energy-dissipation objectives in CFRP retrofit applications.

References

- [1] ACI Committee 440.2R-17. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, American Concrete Institute, Farmington Hills, Michigan, USA, 2017.
- [2] Zarandi S, Maheri MR. Seismic performance of RC frames retrofitted by FRP at joints using a flange-bonded scheme. *Iranian Journal of Science and Technology, Transactions of Civil Engineering*. 2015;39(C1):103-123.
- [3] Antonopoulos CP, Triantafillou TC. Experimental investigation of FRP-strengthened RC beam-column joints. *Journal of Composites for Construction*. 2003;7(1):39-49. [https://doi.org/10.1061/\(asce\)1090-0268\(2003\)7:1\(39\)](https://doi.org/10.1061/(asce)1090-0268(2003)7:1(39))
- [4] Arowojolu O, Ibrahim A, Rahman MK, Al-Osta M, Al-Gadhib AH. Plastic hinge relocation in reinforced concrete beam–column joint using carbon fiber–reinforced polymer. *Advances in Structural Engineering*. 2019;22(14), 2951–2965. <https://doi.org/10.1177/1369433219855901>
- [5] International Federation for Structural Concrete (fib). Externally applied FRP reinforcement for concrete structures. fib Bulletin no.90, Lausanne, Switzerland, 2019. <https://doi.org/10.35789/fib.bull.0090>
- [6] CNR-Advisory Committee on Technical Recommendations for Construction. Guide for the design and construction of externally bonded FRP systems for strengthening existing structures, Rome, Italy, 2004.
- [7] Mahini SS, Ronagh HR. Web-bonded FRPs for relocation of plastic hinges away from the column face in exterior RC joints. *Composite Structures*. 2011;93(10):2460-2472. <https://doi.org/10.1016/j.compstruct.2011.04.002>
- [8] Paulay T, Priestley MJN. Seismic design of reinforced concrete and masonry buildings, John Wiley & Sons, New York, USA, 1992.
- [9] Hejazi F, Azarm R, Firoozi AA. Efficiency of flange-bonded CFRP sheets in relocation of plastic hinge in RC beam–column joints. *Applied Sciences*. 2023;13(21):11870. <https://doi.org/10.3390/app132111870>
- [10] FEMA 356. Prestandard and commentary for the seismic rehabilitation of buildings, Federal Emergency Management Agency, Washington, D.C., USA, 2000.
- [11] ATC. Seismic evaluation and retrofit of concrete buildings, Rep. No. ATC-40, Applied Technology Council, Redwood City, California, USA, 1996.
- [12] Shaikh ZK, Sohoni VS. Efficacy of carbon fibre reinforced polymer (CFRP) laminates in retrofitting the damaged moment resisting RC frames. *Discover Civil Engineering*. 2025;2(1):166. <https://doi.org/10.1007/s44290-025-00326-5>
- [13] Sakr MA, Seleemah AA, Khalifa TM, El-Samak G. Studying behavior of exterior RC beam–column joints strengthened using CFRP for achieving "strong column–weak beam" in RC frames. *East African Scholars Journal of Engineering and Computer Sciences*. 2019;2(2):53–68.

- [14] Wang L, Xuan W, Zhang Y, Cong S, Liu F, Gao Q, Chen H. Experimental and numerical research on seismic performance of earthquake-damaged RC frame strengthened with CFRP sheets. *Advances in Materials Science and Engineering*. 2016;6716329. <https://doi.org/10.1155/2016/6716329>
- [15] Wang Y, Chen W, Li D, Xu H, Zhang F, Guo X. Experimental and numerical investigations of the seismic performance of reinforced concrete frames strengthened with CFRP sheets. *Buildings*. 2023;13(9):2195. <https://doi.org/10.3390/buildings13092195>
- [16] Zhou Y, Liu X, Zhang X, Guo X. Investigation on seismic performance of reinforced concrete frame retrofitted by carbon fiber-reinforced polymer. *Buildings*. 2024;14(6):1604. <https://doi.org/10.3390/buildings14061604>
- [17] Pohoryles DA, Minas S, Melo J, Bournas DA, Varum H, Rossetto T. A novel FRP retrofit solution for improved local and global seismic performance of RC buildings: Development of fragility curves and comparative cost-benefit analyses. *Journal of Earthquake Engineering*. 2024;28(7):1914-1932. <https://doi.org/10.1080/13632469.2023.2256891>
- [18] Eslami A, Dalalbashi A, Ronagh HR. On the effect of plastic hinge relocation in RC buildings using CFRP. *Composites Part B: Engineering*. 2013;52:350-361. <https://doi.org/10.1016/j.compositesb.2013.04.025>
- [19] Ronagh HR, Eslami A. Flexural retrofitting of RC buildings using GFRP/CFRP—A comparative study. *Composites Part B: Engineering*. 2013;46:188-196. <https://doi.org/10.1016/j.compositesb.2012.09.072>
- [20] Balsamo A, Colombo A, Manfredi G, Negro P, Protà A. Seismic behavior of a full-scale RC frame repaired using CFRP laminates. *Engineering Structures*. 2005;27(5):769-780. <https://doi.org/10.1016/j.engstruct.2005.01.002>
- [21] Le-Trung K, Lee K, Shin M, Lee J. Analytical assessment and modeling of RC beam-column connections strengthened with CFRP composites. *Composites Part B: Engineering*. 2011;42(7):1786-1798. <https://doi.org/10.1016/j.compositesb.2011.07.002>
- [22] Abaqus Analysis User's Manual. Abaqus, 2017.
- [23] Valles RE, Reinhorn AM, Kunnath SK, Li C, Madan A. IDARC2D, Version 4.0: A computer program for the inelastic damage analysis of buildings, National Center for Earthquake Engineering Research (NCEER), Technical Report NCEER-96-0010, New York, USA, 1996.
- [24] Chopra AK. *Dynamics of structures: theory and applications to earthquake engineering*, 4th edition, Pearson, Boston, Massachusetts, USA, 2014.
- [25] Carlos IA, Manuel SHJ, Ivan MD, Higinio GPJ. Energy balance analysis in non linear dynamic equivalent systems. *MATEC Web of Conferences*, 2018;148:03002. <https://doi.org/10.1051/mateconf/201814803002>
- [26] Heitz T, Giry C, Richard B, Ragueneau F. Identification of an equivalent viscous damping function depending on engineering demand parameters. *Engineering Structures*. 2019;188:637-649. <https://doi.org/10.1016/j.engstruct.2019.03.058>
- [27] Jacobsen LS. Damping in composite structures. *Proceedings of the 2nd World Conference on Earthquake Engineering*, Tokyo, Japan, 1029-1044, 1960.
- [28] Rodrigues H, Varum H, Arêde A, Costa A. A comparative analysis of energy dissipation and equivalent viscous damping of RC columns subjected to uniaxial and biaxial loading. *Engineering Structures*. 2012;35:149-164. <https://doi.org/10.1016/j.engstruct.2011.11.014>
- [29] Priestley MJN, Grant DN. Viscous damping in seismic design and analysis. *Journal of Earthquake Engineering*. 2005;9(sup2):229-255. <https://doi.org/10.1142/s1363246905002365>
- [30] Priestley MJN, Grant DN, Blandon CA. Direct displacement-based seismic design. New Zealand Society for Earthquake Engineering (NZSEE) conference, 2005.
- [31] Priestley MJN, Calvi GM, Kowalsky MJ. Direct displacement-based seismic design of structures. New Zealand Society for Earthquake Engineering (NZSEE) conference, 2007.
- [32] Peng C, Guner S. Direct displacement-based seismic assessment of concrete frames. *Computers and Concrete*. 2018;21(4):355-365. <https://doi.org/10.12989/cac.2018.21.4.355>
- [33] Hafezolghorani M, Hejazi F, Vaghei R, Jaafar MS Bin, Karimzade K. Simplified damage plasticity model for concrete. *Structural Engineering International*. 2017;27(1):68-78. <https://doi.org/10.2749/101686616x1081>
- [34] Cornelissen HAW, Hordijk DA, Reinhardt HW. Experimental determination of crack softening characteristics of normal-weight and lightweight concrete. *Heron Journal*. 1986;31(2):45-56.
- [35] MC90. CEB-FIP Model Code 1990, Bulletin d'Information No. 213/214, Comité Euro-International Du Béton, Thomas Telford, Lausanne, Switzerland, 1993.
- [36] Sinha BP, Gerstle KH, Tulin LG. Stress-strain relations for concrete under cyclic loading. *ACI Journal Proceedings*. 1964;61(2):195-212. <https://doi.org/10.14359/7775>
- [37] Nguyen NH, Whittaker AS. Numerical modelling of steel-plate concrete composite shear walls. *Engineering Structures*. 2017;150:1-11. <https://doi.org/10.1016/j.engstruct.2017.06.030>
- [38] Lu J, Mou T, Wang C, Huang H, Han W. Research on hysteretic behavior of FRP-confined concrete core-encased rebar. *Polymers*. 2023;15(12):2728. <https://doi.org/10.3390/polym15122728>

- [39] Shi K, Gao Z. Experimental and numerical study on flexural behavior of steel fiber reinforced high-strength concrete (SFRHC) beams. *Scientific Reports*. 2025;15:18338.
<https://doi.org/10.1038/s41598-025-02220-7>
- [40] Gulkan P, Sozen MA. Inelastic responses of reinforced concrete structure to earthquake motions. *ACI Journal Proceedings*. 1974;71(12):604-610. <https://doi.org/10.14359/7110>
- [41] Akbaş B, Akşar B, Doran B, Alacalı S. Hysteretic energy to energy input ratio spectrum in nonlinear systems. *Deu Muhendislik Fakultesi Fen ve Muhendislik*. 2016;18(53):239-254.
<https://doi.org/10.21205/deufmd.20165318384>
- [42] White C. Resonance revealed: Understanding what really happens at resonance. *Vibrations*. 2018;35(1).

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Controlled-combustion emission benchmarking of cow-manure-rice-husk-sawdust briquettes as firewood substitutes for traditional salt production

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Abstract

Traditional salt production in coastal Indonesia still relies heavily on firewood, contributing to deforestation pressure, unstable fuel supply, and local air-pollution exposure. This study evaluates multi-feedstock biomass briquettes made from cow manure, rice husk, and sawdust as potential firewood substitutes by assessing their proximate properties and controlled-combustion emissions. Briquettes were prepared using a mixture of 74% cow manure, 5% rice husk, 9% sawdust, 1% tapioca starch, and 11% water, then dried to approximately 12% moisture. Proximate analysis (moisture, ash, volatile matter, and fixed carbon by difference) was conducted following standardized wood/biomass fuel test methods. Flue-gas emissions (SO₂, NO₂, CO, opacity, and O₂) were measured in a controlled combustion chamber (garden incinerator) by an accredited laboratory with triplicate measurements, and results were benchmarked against commonly used local firewood (petai, sentang, rambutan) and published pellet data. The briquettes exhibited 12% moisture (as received) and, on a dry basis, 29% ash, 60% volatile matter, and 11% fixed carbon. Emissions showed very low SO₂ (1.2 mg/Nm³) and moderate NO₂ (7.4 mg/Nm³), but comparatively high CO (1,857.6 mg/Nm³), indicating incomplete combustion under the tested airflow and mixing conditions. Overall, the briquettes offer a clear sulfur-emission advantage over firewood, while combustion-system optimization (air supply and fuel-air mixing) is required to reduce CO and improve combustion completeness for salt-processing applications.

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1. Introduction

The increasing demand for thermal energy in rural and small-scale industries continues to place pressure on traditional biomass resources, particularly firewood. In Indonesia's coastal salt-producing regions, firewood remains the dominant fuel used for seawater evaporation [1], contributing to high operational costs, inconsistent heat supply, and increased carbon emissions. The reliance on firewood also accelerates deforestation and threatens the long-term sustainability of salt production systems that depend heavily on steady heat availability [2-3].

In response to these concerns, there has been an increasing emphasis on the adoption of renewable energy alternatives, such as biomass, which play a vital role in mitigating environmental degradation [4-7]. Biomass, in particular, holds substantial promise due to its renewable nature

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and wide availability from sources such as agricultural residues, livestock waste, and other organic materials [8-12].

The utilization of biomass offers multiple benefits, including the potential for conversion into various alternative fuels [13-15]. Among these, biomass briquettes represent a particularly effective form, produced by compressing organic waste into dense, energy-efficient solid fuels. This conversion improves storage, transportation, and combustion efficiency compared to unprocessed biomass [16-21]. Especially in rural and off-grid communities, biomass briquettes serve as a cost-effective and accessible energy option for everyday activities such as cooking, heating, and small-scale industrial use [22, 23]. Meanwhile, Indonesia generates significant volumes of agricultural and livestock waste—including cattle manure, rice husks, and sawdust—that are often discarded or used inefficiently. These residues hold considerable potential as renewable bioenergy resources when converted into densified bio-briquettes. Biomass briquettes from agricultural have shown promising fuel and combustion properties [24-26]. However, briquette quality and environmental performance vary substantially depending on the mixture, proportion, and physicochemical characteristics of the raw materials.

In our previous study, a briquette product of cow manure, rice husk and wood dust were produced [27]. Figure 1 shows the dimensions of the briquette product, which is 20 cm x 10 cm x 12.5 cm. From the combination of dimensional measurements and mass weighing of briquettes, the average density of dry briquettes is 0.337 gr/cm³.



Fig. 1. Geometry and appearance of the produced briquette: (a) top view (left) showing surface perforation holes; (b) side view (right). Nominal dimensions are 20 cm × 10 cm × 12.5 cm (length × width × height)

The utilization of briquettes produced from a mixture of cow manure, rice husk, and wood dust has demonstrated strong performance across several quality assessments. The financial evaluation in our previous study also indicated strong feasibility, with a Net Present Value (NPV) of IDR 144,074,566, an Internal Rate of Return (IRR) of 72.154%, a Net Benefit–Cost ratio (Net B/C) of 4.37, and a payback period of 3.22 years [27]. These indicators collectively show that briquette production is not only technically viable but also financially attractive, while simultaneously offering the potential to reduce dependence on firewood—a major driver of deforestation and environmental degradation in rural communities.

Despite the growing attention to biomass-based solid fuels, research integrating cattle manure, rice husk, and sawdust into a single multi-feedstock briquette remains extremely limited, particularly in the context of coastal salt farming. Existing studies tend to focus on either briquette from a single waste source, household-scale cooking applications, or generalized combustion parameters without examining sector-specific needs. Only limited evidence exists on studies that systematically compare both proximate properties and controlled-combustion emissions of multi-feedstock cow-manure–rice-husk–sawdust briquettes against the firewood types used in traditional salt production systems. This represents a critical gap, given that salt farmers require fuel with stable combustion, low moisture, and reduced pollutants. A representative firewood stockpile commonly used by salt farmers is shown in Figure 2.



Fig. 2. Representative stockpiled firewood commonly used as fuel in traditional coastal salt production (field photograph)

Despite extensive research on agricultural-residue briquettes, evidence remains limited for (i) multi-feedstock briquettes that jointly combine cow manure, rice husk, and sawdust in a single formulation, and (ii) emission benchmarking against the specific firewood types used in traditional salt-processing furnaces. Most prior studies focus on single-feedstock briquettes, household cooking stoves, or general fuel characterization without linking performance to salt-production operational needs (stable heat release, manageable ash handling, and reduced pollutant exposure). Therefore, the scientific contribution of this work is to provide controlled-combustion emission data (SO_2 , NO_2 , CO , opacity, and O_2) and proximate properties for a locally feasible multi-feedstock briquette formulation, and to directly compare its emission profile with representative tropical hardwood firewood used by salt farmers. The objectives of this study are: (1) to determine moisture, ash, volatile matter, and fixed carbon of the briquettes using standardized proximate-analysis methods; (2) to quantify flue-gas emissions under controlled combustion conditions; and (3) to benchmark briquette emissions against commonly used firewood and published pellet references to assess feasibility as a firewood substitute for traditional salt production.

1.1 Research Questions and Scope

This study addresses three practical research questions: (RQ1) What are the proximate characteristics (moisture, ash, volatile matter, fixed carbon) of the proposed cow-manure-rice-husk-sawdust briquettes under standardized testing? (RQ2) Under controlled combustion conditions, how do the flue-gas emissions (SO_2 , NO_2 , CO , opacity, and O_2) of the briquettes compare with representative tropical hardwood firewood used by salt farmers? (RQ3) Based on energy-content estimation and emission benchmarking, what is the practical implication for using the briquettes as a partial or full substitute for firewood in traditional salt production?

The scope of the study is limited to controlled combustion tests in a laboratory-type combustion chamber and literature-based benchmarking; field-scale stove trials and long-duration performance tests in operational salt furnaces are recommended as follow-up work.

1.2 Literature Review

Biomass densification into briquettes and pellets has been widely reported as an effective strategy to increase bulk density, improve handling and storage, and stabilize combustion relative to loose residues. Multiple studies show that densified biomass fuels can improve practical usability in rural and small-scale industrial contexts by reducing transport volume and enabling more consistent feeding rates during combustion [16–21,24–26]. However, densification alone does not guarantee cleaner emissions; emission outcomes remain strongly dependent on feedstock composition (e.g., ash-forming minerals and fuel-bound nitrogen) and on combustion conditions (air supply, mixing, and residence time) [14,24,25,35].

Manure- and faecal-based solid fuels have gained attention due to their availability in livestock-dominant regions and their potential to convert sanitation or agricultural residues into usable energy carriers [6,11,12]. Prior studies demonstrate that dung-based briquettes/pellets can be feasible fuels, but they commonly exhibit high ash fractions due to mineral-rich composition, which can increase ash handling requirements and influence combustion stability and residue management [28–30]. In particular, the ash contribution becomes more pronounced when manure is blended with silica-rich residues such as rice husk, potentially affecting both combustion dynamics and post-combustion maintenance [15,18].

Rice husk is frequently highlighted for its availability and its porous structure, which can influence ignition behavior; nevertheless, its high silica content typically elevates ash levels and can pose challenges such as clinker formation or airflow obstruction in certain combustion systems if not managed properly [15,18]. Sawdust and other woody residues, by contrast, often contribute more favorable lignocellulosic combustion characteristics and have been associated with improved briquette strength and stable flaming under appropriate conditions [13,24,25]. Therefore, blending manure, rice husk, and sawdust is technically attractive but requires explicit evaluation of both fuel properties and emission performance due to opposing effects (improved cohesion and availability versus elevated ash and potential incomplete combustion).

Emission performance is a critical criterion for adopting alternative fuels in semi-enclosed work environments such as artisanal processing sites. Studies on pellets and briquettes show that sulfur dioxide (SO₂) can vary widely depending on fuel sulfur content and contamination, while carbon monoxide (CO) is strongly indicative of combustion completeness and is highly sensitive to airflow and mixing conditions [29,35]. Consequently, comparative emission benchmarking against real fuels used in the target application (here, tropical hardwood firewood for salt production) is necessary to assess whether a proposed briquette can realistically reduce pollutant exposure and environmental burden while maintaining sufficient thermal performance.

Although briquette research has grown substantially, evidence remains limited for multi-feedstock formulations that combine cow manure, rice husk, and sawdust in a single blend and, simultaneously, report controlled-combustion emission data benchmarked against the specific firewood types used by traditional salt producers. This limitation motivates the present study, which links proximate properties and controlled emission measurements to the salt-production fuel context.

2. Materials and Methods

2.1 Basic Materials

The basic materials for this research were dried cow dung mixed with rice husks and sawdust. The formulation of Sample was designed to optimize the physical integrity and combustion characteristics of the biomass briquette by utilizing readily available and low-cost organic waste materials. As shown in Table 1, cow dung constitutes the majority of the composition at 74%. This component serves as both a binder and a combustible material, contributing significantly to the fixed carbon content while improving the mechanical cohesion of the briquette due to its fibrous and organic matter content. Cow dung is widely accessible in rural agricultural communities and has been previously recognized for its potential in low-emission biomass fuel production.

Table 1. Raw materials composition

Raw Material	Composition Ratio (%)
Cow Dung	74
Rice Husk	5
Sawdust	9
Tapioca Starch	1
Water	11

2.1.1 Formulation Rationale

The composition in Table 1 was adopted from a prior development and feasibility study of the same briquette product [27] and refined through preliminary mixing trials aimed at achieving (i) sufficient mechanical integrity during molding and drying, and (ii) stable ignition and flame sustainability. Cow manure was set as the dominant fraction because it provides natural binding capacity and structural cohesion, while also contributing combustible organic matter. Rice husk was limited to a low proportion to avoid excessive ash accumulation associated with its high silica content, whereas sawdust was included to increase the carbon-rich fraction and support more stable flaming combustion. Tapioca starch was added as a supplementary binder to improve inter-particle bonding, and water content was adjusted to obtain a workable paste for molding and to activate binder functionality prior to drying.

2.2 Proximate Analysis

Proximate analysis was conducted to determine moisture content, ash, volatile matter, and fixed carbon as key indicators of fuel quality. Moisture content was determined using a standard oven-drying approach for particulate biomass/wood fuels (ASTM E871). Ash content was determined by dry oxidation of the oven-dried sample (ASTM E1755). Volatile matter was determined under standardized high-temperature conditions (ASTM E872). Fixed carbon was calculated by difference as commonly defined in proximate analysis standards (ASTM E870 / ISO 17225-1). Moisture content (as received).

$$MC(\%) = \frac{m_2 - m_3}{m_2 - m_1} \times 100 \quad (1)$$

where m_1 is the mass of the empty crucible (g), m_2 is the mass of crucible + wet sample before oven drying (g), and m_3 is the mass of crucible + sample after oven drying at $103 \pm 2^\circ\text{C}$ to constant mass (g). Ash content (dry basis).

$$Ash_{db}(\%) = \frac{m_4 - m_1}{m_3 - m_1} \times 100 \quad (2)$$

where m_1 is the mass of the empty crucible (g), m_3 is the mass of crucible + oven-dried sample used for ashing (g), and m_4 is the mass of crucible + ash after oxidation in a muffle furnace (g). Volatile matter (dry basis).

$$VM_{db}(\%) = \frac{m_3 - m_5}{m_3 - m_1} \times 100 \quad (3)$$

where m_5 is the mass of crucible + residue after volatile-matter determination (g). Fixed carbon (dry basis, by difference).

$$FC_{db}(\%) = 100 - (Ash_{db} + VM_{db}) \quad (4)$$

All reported proximate fractions (Ash_{db} , VM_{db} , FC_{db}) are on a dry basis, while moisture content is reported as received.

2.3 Comparative Analysis

To evaluate the performance of the briquettes, the proximate analysis results were compared with briquettes produced from different biomass feedstocks reported in previous studies and commonly used firewood types sourced from the literature. This comparative approach was used to assess whether the briquettes meet or exceed standard fuel characteristics associated with efficient and clean combustion.

2.4 Emission Analysis

The second stage of the study focused on evaluating the combustion emissions of the multi-feedstock briquettes. The pollutants measured included sulfur dioxide (SO_2), carbon monoxide

(CO), and nitrogen dioxide (NO₂). Emission testing was performed using a Garden Incinerator. A generic emission analyzer was employed to measure gas emission concentrations. The procedures carried out are as follows:

2.4.1 Emission Measurements

During combustion, gaseous emissions from the briquettes and the comparative wood fuels—petai (*Parkia speciosa*), sentang (*Azadirachta excelsa*), and rambutan (*Nephelium lappaceum*)—were quantified using a generic emission analyzer. The measurement procedures adhered to established national testing protocols, with sulfur dioxide (SO₂) assessed following Method Ik.5.04.02.01, carbon monoxide (CO) using Method Ik.5.04.02.03, and nitrogen dioxide (NO₂) using Method Ik.5.04.02.02. Opacity measurements were performed in accordance with SNI 19.7117.11.2005. All analytical methods were implemented in compliance with the emission monitoring requirements outlined in Appendix 1 of the Ministry of Environment and Forestry Regulation No. 11/2021. The testing was carried out by the Industrial Standardization and Service Agency of Banda Aceh (Balai Standardisasi dan Pelayanan Jasa Industri Banda Aceh), ensuring conformity with accredited national standards.

The applied procedures and reporting format follow the emission monitoring framework and definitions in the Ministry of Environment and Forestry Regulation No. 11/2021 and the Indonesian National Standard for opacity testing (SNI 19.7117.11.2005).

2.4.2 Accuracy, Calibration, and Repeatability

All emission measurements in this study were conducted by the Industrial Standardization and Service Agency of Banda Aceh (BSPJI/BARISTAND), following the official Indonesian test methods for stationary combustion sources, namely IK.5.04.02.01 (SO₂), IK.5.04.02.02 (NO₂), and IK.5.04.02.03 (CO). These methods comply with the Ministry of Environment and Forestry Regulation Appendix 1 No. 11 of 2021 on Emission Standards for Combustion. The procedures include mandatory calibration and accuracy verification steps prior to each test. According to IK.5.04.02.03, the emission analyzer undergoes free-air calibration and instrument self-check for approximately 180 seconds to ensure measurement stability and accuracy. Each pollutant measurement is performed in three consecutive repetitions, as required by Section 6.2.3 of the method, with a measurement duration of ten minutes per repetition. This triplicate measurement protocol ensures data repeatability and reduces variability in the reported concentrations. The accuracy, resolution, and detection limits of the analyzer follow the specifications validated by BSPJI through routine certified reference gas calibration, ensuring that the emission data generated in this study meet nationally standardized metrological requirements.

2.4.3 Combustion Testing of Briquettes

Combustion trials were conducted for the briquettes formulated from cow manure, rice husks, and wood dust, alongside comparative tests using petai wood (*Parkia speciosa*), sentang wood (*Azadirachta excelsa*), and rambutan wood (*Nephelium lappaceum*). Each fuel type was burned individually in a Garden Incinerator under controlled conditions.

Controlled combustion tests were conducted using a commercially available garden incinerator operated as a batch, natural-draft combustion chamber. The incinerator body is equipped with multiple air holes (ventilation openings) that function as primary air inlets to supply combustion air to the fuel bed. Flue gas exits through an upper chimney/funnel that promotes draft and directs exhaust upward. Flue-gas sampling was conducted at the chimney/funnel outlet, where the sampling probe was positioned in the exhaust stream to ensure consistent and comparable measurements across fuels. The same sampling location and test configuration were applied for all fuel types.

The tests were performed at an ambient air temperature of $33 \pm 1^\circ\text{C}$, with the measured oxygen concentration in the flue gas approximately $19 \pm 1\%$ to ensure consistent combustion environments across all samples. Throughout the combustion process, key gaseous and optical emission parameters—including sulfur dioxide (SO₂), carbon monoxide (CO), nitrogen dioxide

(NO₂), oxygen (O₂), and flue-gas opacity—were continuously monitored to characterize the emission profile of each fuel.

2.4.4 Data Analysis

The emission data recorded by the generic emission analyzer for each gas during combustion were analyzed to evaluate the emission levels of the briquettes made from cow manure, rice husks, and wood dust compared to the emissions produced by firewood commonly used by salt farmers under identical testing conditions, and emission data reported in previous studies for other types of briquettes and firewood. This comparative framework enabled a broader assessment of whether the briquettes offer environmental advantages over conventional fuels and other biomass-based alternatives documented in the literature.

2.5 Research Flow Chart

A flow chart is used to visually represent the process or sequence of steps undertaken in this research. It illustrates the procedures, starting from the preparation of equipment and materials to the data analysis phase. The flow chart in this study is as shown in Figure 3.

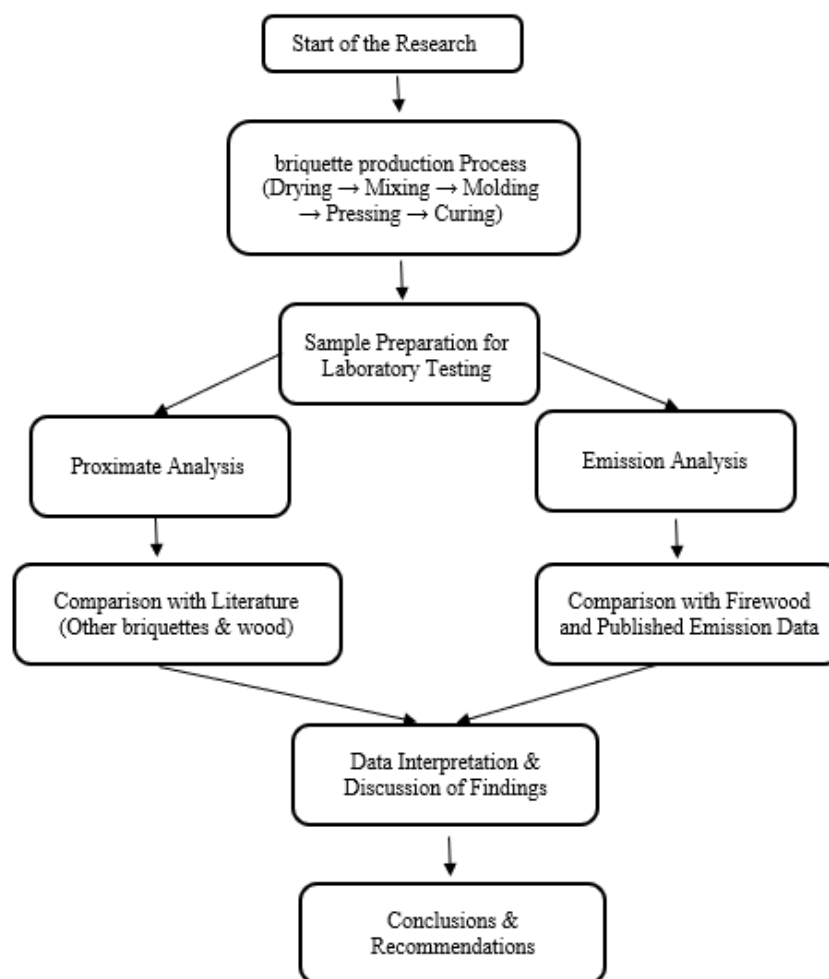


Fig. 3. Research Flow chart

2.6 Conceptual Framework

The analytical framework of this study links feedstock formulation to end-use feasibility through two measurable domains: (i) proximate properties as indicators of ignition behavior, ash handling, and char-burning potential; and (ii) controlled-combustion emissions as indicators of environmental and occupational exposure risks. The briquette formulation (cow manure–rice husk–sawdust) is treated as the primary design variable, proximate fractions (MC, ash, VM, FC) as intermediate fuel-quality descriptors, and emission outputs (SO₂, NO₂, CO, opacity, O₂) as

performance indicators. The framework enables benchmarking against firewood used in salt production and against pellet references from the literature to identify advantages (e.g., low SO₂) and constraints (e.g., high CO under limited mixing), thereby informing practical recommendations for combustion-system optimization.

3. Results and Discussion

3.1. Results

3.1.1 Proximate Characteristics of the Produced Briquettes

The proximate analysis revealed distinct combustion-related properties of the briquettes formulated from cow dung (74%), rice husk (5%), sawdust (9%), tapioca starch (1%), and water (11%). The results indicate that the produced briquettes contain 12% moisture, 29% ash, 60% volatile matter, and 11% fixed carbon, positioning them within the general performance profile of non-carbonized biomass briquettes.

Table 2. Comparison of proximate characteristics of cow-dung-based briquettes with previous studies

Parameter (%)	This Study	Cow-dung Briquette [28]	Cow-dung Pellets [29]	Composite Briquette with Cow Dung Binder [30]
Moisture	12.00	6.94	~18	3.16
Ash	29.00	25.47	10–20	3.60
Volatile Matter	60.00	18.58	50–70	80.20
Fixed Carbon	11.00	49.02	~18.61	13.04

When compared with previous studies, the moisture content of the briquettes is higher than the value reported by Rahmaisyanti but lower than the typical moisture levels found in cow-dung pellets reviewed by Szymajda. This moderate moisture level suggests that the briquettes remain suitable for combustion without requiring excessive drying time.

The ash content (29%) is relatively high and closely aligns with the ash level of cow-dung-dominant briquettes (25.47%) in Rahmaisyanti. The elevated ash is consistent with the mineral-rich composition of cow dung and the additional silica contribution from rice husk. Compared with composite briquettes that use cow dung only as a binder, the ash content in this study is substantially higher, reflecting the greater proportion of inorganic residues inherent in the feedstock blend.

Volatile matter of the produced briquettes reached 60%, which is significantly higher than the value reported for cow-dung-only briquettes in Rahmaisyanti but still within the 50–70% range documented by Szymajda. This high volatile fraction indicates that the briquettes ignite easily and burn quickly, which is advantageous for fuel applications requiring fast combustion, such as traditional salt processing.

The fixed carbon content (11%) is lower than reported in the cow-dung-only briquettes of Rahmaisyanti, where fixed carbon reached 49.02%. This discrepancy reflects differences in formulation, as the cow-dung briquettes in prior studies often use carbonized or partially carbonized feedstocks. In contrast, the non-carbonized formulation used in this study—combined with low-fixed-carbon biomass such as rice husk and sawdust—naturally results in lower fixed-carbon content. Nevertheless, the fixed carbon value remains comparable to composite briquettes using cow dung as a binder [30], which reported 13.04%.

3.1.2 Comparison Between Briquette Characteristics and Conventional Firewood

The proximate characteristics of the cow-dung-based briquettes produced in this study were compared with benchmark values for conventional firewood reported in the literature. The briquettes exhibited a moisture content of 12%, which falls within the typical range of 8–15% observed in several fuelwood species such as *Eucalyptus pellita*, *Acacia* spp., mangrove species, and

multiple Indonesian hardwoods. This indicates that the drying process of the briquettes is comparable to the standard practices for seasoned firewood, and moisture is unlikely to hinder ignition performance.

Table 3. Typical proximate characteristics of common firewood species (literature benchmarks)

Wood / Source (type)	Moisture (%)	Ash (%)	Volatile matter (%)	Fixed carbon (%)	Source
<i>Eucalyptus pellita</i> (wood) — Indonesia study	10–14	0.8–2.5	70–78	18–25	[31]
<i>Acacia</i> spp. (fuelwood / charcoal feedstock) — tropical study	8–12	0.5–4.0	65–80	15–25	[32]
Ulin / <i>Eusideroxylon</i> (several Indonesian woods — proximate reported)	6–12	0.5–3.0	65–80	15–28	[33]
Mangrove species (wood, multiple species) — charcoal/proximate report	8–15	0.6–3.5	60–78	17–28	[34]

However, a substantial contrast emerges in the ash content. The briquettes contained 29% ash, which is considerably higher than the 0.5–5% range commonly reported for firewood. Studies on *Eucalyptus pellita*, *Acacia* spp., mangrove wood, and Indonesian species such as *Eusideroxylon* consistently show low mineral residue after combustion. In comparison, the briquettes' ash content is an order of magnitude higher. This difference reflects the intrinsic mineral composition of cow dung and the contribution of rice husk silica, which together elevate inorganic content. Consequently, the briquettes are expected to produce more post-combustion residue than firewood, a factor that may affect stove maintenance and ash disposal requirements.

In terms of volatile matter, the briquettes recorded 60%, which aligns with the reported values for firewood, typically ranging between 60–80%. This suggests that the briquettes, like natural biomass, contain a substantial proportion of easily combustible components that support rapid ignition and flame formation in the early combustion stage. Meanwhile, the fixed carbon content of the briquettes (11%) is lower than the 15–28% typical of hardwood fuel. Because fixed carbon governs the duration of the glowing-phase combustion, firewood is expected to burn longer and retain embers more effectively compared to the briquettes. These differences collectively indicate that the briquettes may ignite readily but may not sustain long-lasting heat without frequent fueling.

31.3. Emission Performance

The emission characteristics of the cow manure–rice husk–wood dust briquette was compared with three local wood fuels (Petai, Sentang, Rambutan) and two reference fuels from the literature (cow dung pellets and woody pellets). The results are summarized in Table 4.

Table 4. Comparison of research results with other briquettes with cow manure base material

No.	Test Parameters	Cow manure, rice husk, and wood dust	Petai wood	Sentang wood	Rambutan Wood	Cow Dung Pellets [29]	Woody Pellets [35]
1	SO ₂ (mg/Nm ³)	1.2	75.4	7.6	94.4	244.38	2.68
2	NO ₂ (mg/Nm ³)	7.4	3.6	12.8	4.6	-	-
3	CO (mg/Nm ³)	1,857.6	1,643.8	1,502.4	1,568.2	854.77	329.87
4	Opacity (%)	20	20	20	20	-	-
5	Oxygen (O ₂ %)	18.9	19.53	18.72	19.39	11.59	9.14

3.1.4 Sulfur Dioxide (SO_2)

The briquette produced a very low SO_2 concentration of 1.2 mg/Nm^3 , substantially lower than Petai (75.4 mg/Nm^3) and Rambutan (94.4 mg/Nm^3), and also lower than the high- SO_2 cow dung pellets reported in the literature (244.38 mg/Nm^3). Its SO_2 value was comparable to woody pellets (2.68 mg/Nm^3), indicating that the sulfur content of the briquette's raw materials is inherently low. This suggests that the mixture of cow manure, rice husk, and sawdust does not introduce significant sulfur-bearing compounds and thereby minimizes the potential for acidifying emissions.

3.1.5 Nitrogen Dioxide (NO_2)

The briquette emitted 7.4 mg/Nm^3 of NO_2 , a value within the range observed for the three wood species ($3.6\text{--}12.8 \text{ mg/Nm}^3$). Although reference values for NO_2 from cow dung pellets and woody pellets were not available, the data indicate that the fuel blend produces nitrogen oxides at levels comparable to conventional wood combustion. This suggests that fuel nitrogen content in the briquette is moderate and does not significantly elevate NO_x formation during combustion.

3.1.6 Carbon Monoxide (CO)

Measured CO concentration for the cow manure–rice husk–wood dust briquette was $1,857.6 \text{ mg/Nm}^3$, which reference fuels: Petai ($1,643.8 \text{ mg/Nm}^3$), Sentang ($1,502.4 \text{ mg/Nm}^3$), Rambutan ($1,568.2 \text{ mg/Nm}^3$), cow-dung pellets from literature (854.77 mg/Nm^3), and woody pellets (329.87 mg/Nm^3). The markedly elevated CO emissions indicate a high degree of incomplete combustion for the briquette under the applied test conditions.

Overall, Table 4 indicates a clear trade-off profile: the briquettes provide a strong SO_2 reduction benefit relative to the tested firewood types, while CO levels suggest incomplete oxidation under the present airflow/mixing conditions. This trade-off is consistent with the expectation that dense, heterogeneous fuels with high volatile release require adequate secondary air and mixing to complete volatile oxidation, especially in natural-draft chambers.

3.2 Discussion

The proximate analysis (Table 2) provides the fuel-quality context required to interpret the emission trends (Table 4) and to assess practical feasibility for traditional salt boiling. Overall, the produced briquettes represent a non-carbonized dung–agro-residue fuel characterized by moderate moisture (12% as received) and a volatile-rich composition ($VM = 60\%$ on a dry basis), accompanied by a high ash burden ($Ash = 29\% \text{ db}$) and relatively low fixed carbon ($FC = 11\% \text{ db}$). These attributes are broadly consistent with reported variability of cow-dung-based solid fuels, where moisture and proximate fractions are strongly influenced by feedstock composition, drying practice, and densification conditions [28–29].

Moisture plays a key role in ignition behavior and effective heat release because part of the fuel energy is consumed to heat and evaporate water before stable combustion is achieved. The 12% moisture content measured here is higher than the 6.94% reported by Rahmaisanti et al. [28], yet lower than the $\sim 18\%$ moisture level reported for cow-dung pellets in Szymajda et al. [29]. This difference is plausibly associated with differences in pre-drying intensity and with the present formulation process, where 11% water was added to activate binder functionality and improve molding consistency prior to drying. Importantly, 12% remains within the typical operational range reported for non-carbonized biomass briquettes and should not prevent practical handling and ignition under appropriate firing conditions [25,29].

Ash content and operational implication. A major distinguishing characteristic of the studied briquettes is the high ash fraction (29% db). This value is comparable to the 25.47% ash content reported for cow-feces/coffee-ground briquettes by Rahmaisanti et al. [28], supporting the interpretation that elevated ash is a feedstock-driven feature of dung-dominant formulations rather than a production defect. For end-use applications, high ash implies increased residue management and a higher likelihood of airflow obstruction if ash accumulates in the primary-air zone. Therefore, ash handling and grate/airflow design become more critical when deploying these briquettes in salt-production furnaces than when using low-ash hardwood firewood (Table 3).

Volatile matter and fixed carbon balance. The briquettes exhibit VM = 60% db, which falls within the broad range commonly reported for non-carbonized biomass briquettes and is consistent with the “volatile-rich” character of many dung-based fuels reviewed by Szymajda et al. [29]. In contrast, Rahmaisyanti et al. reported substantially lower VM (18.58%) and higher FC (49.02%) for briquettes where processing choices and material composition favored a more char-like behavior [28]. The present briquettes show FC = 11% db, closer to the lower fixed-carbon regime expected for non-carbonized blends and comparable to other residue-briquette reports (e.g., FC ~13% in Geberehiet et al.) [30]. Practically, this VM–FC balance suggests rapid initial flaming (good for quick heat establishment) but a shorter ember/char phase than fuels with higher fixed carbon, potentially requiring more frequent refueling during long salt-boiling cycles.

Taken together, the proximate profile confirms that the developed briquettes fall within expected ranges for non-carbonized dung–residue fuels [28–30] and provides a mechanistic basis for interpreting the emission outcomes in Table 4, particularly the trade-off between very low SO₂ and elevated CO under the tested natural-draft combustion configuration.

3.2.1 Suitability of the Briquettes for Thermal Applications

Based on the proximate profile (Table 2), the briquettes exhibit characteristics compatible with practical is the highest value among all fuels tested in this study and substantially exceeds the CO reported for the thermal uses that require reliable ignition and sustained heating, including traditional salt boiling. The moderate as-received moisture (12%) together with a volatile-rich dry-basis composition (VM = 60%) indicates that the fuel can ignite readily and establish a stable flame when adequate air supply is available, consistent with general briquette combustion behavior reported for agro-residue and dung-derived solid fuels [25,29]. At the same time, the high ash fraction (Ash = 29%)—likely influenced by mineral-rich manure and silica-bearing rice husk—implies that residue management and maintaining airflow through the fuel bed will be more critical than for low-ash hardwood firewood [15,25]. The relatively low fixed carbon (FC = 11%) also suggests a shorter ember/char phase than many hardwood fuels, so long salt-boiling cycles may require more frequent feeding or operational adjustments to maintain consistent heat output [25,29].

3.2.2 Implications of Differences in Fuel Properties for End-Use Performance

Compared with the firewood benchmarks (Table 3), the briquettes are expected to offer comparable ignition convenience under similar moisture conditions, but with distinct operational requirements. The substantially higher ash content implies more frequent ash removal and a higher risk of airflow restriction if ash accumulates in the primary-air zone; therefore, end-use feasibility for salt boiling depends not only on fuel availability but also on the ability of the furnace/stove configuration to maintain airflow despite ash generation (e.g., through periodic cleaning and appropriate ash handling) [25]. In addition, the lower fixed carbon fraction indicates shorter burn endurance per unit mass than many hardwood fuels, which can be accommodated through feeding schedules already typical in artisanal boiling processes, but should be recognized as a key difference affecting heat maintenance behavior [25,29]. These property-driven implications provide the practical context for interpreting the emission trade-offs discussed in the next subsection (Table 4).

3.2.3 Mechanistic Interpretation and Implications of Emission Performance

The contrasting emission behaviors observed among the tested fuels highlight the complex interplay between feedstock composition, physicochemical properties, and combustion dynamics. The extremely low SO₂ emissions from the cow-manure–rice-husk–wood-dust briquette can be attributed to its inherently low sulfur content. Rice husk contributes a high proportion of silica and negligible sulfur, while cow manure contains sulfur predominantly in organic forms that decompose without substantial SO₂ release. This composition-driven advantage positions the briquette as a cleaner alternative to sulfur-bearing hardwoods, which consistently exhibited higher SO₂ concentrations. From an environmental standpoint, the exceptionally low sulfur footprint is critical for mitigating acid gas formation and aligning with stringent emission thresholds adopted in clean-energy policies.

The moderate NO₂ emission level reflects both the nitrogen composition of cow manure and the combustion temperature profile. Manure-based fuels tend to generate some fuel-NO_x due to nitrogenous biochemical residues. However, the low to moderate flame temperature typical of manure–biomass blends inhibit thermal-NO_x formation, which explains why NO₂ concentrations remained far below those commonly associated with herbaceous biomass. This reduced NO_x formation enhances the suitability of the briquette for small-scale thermal applications where ventilation is limited and NO₂ exposure becomes a functional health concern. Compared to woody biomass that tends to burn at higher temperatures, the briquette’s-controlled combustion temperature indirectly benefits nitrogen emission control.

The most striking behavior is the elevated CO emissions relative to all comparison fuels, including pelletized cow dung and woody pellets. Mechanistically, this is likely influenced by a combination of structural and combustion factors: (i) higher briquette density limiting internal air diffusion, (ii) heterogeneity of its organic–inorganic matrix affecting heat transfer, and (iii) partial devolatilization of manure components generating combustible gases that are not fully oxidized. Pelletized fuels, by contrast, are mechanically densified under precisely controlled parameters, ensuring uniform ignition, stable temperature rise, and enhanced mixing of oxygen with volatile gases—conditions that facilitate more complete oxidation and thereby reduce CO significantly. The findings highlight the need for improvements in briquette formulation, such as optimizing particle size, pressing force, and binder ratios to enhance porosity and combustion stability.

In the broader context of thermal energy development, these results demonstrate that while the briquette delivers significant environmental benefits in terms of SO₂ and NO₂ suppression, optimization is essential to meet desirable CO emission standards. Nonetheless, the emission patterns observed indicate strong potential for deployment in rural or small-industry applications, provided that burning technologies are adapted to improve combustion completeness. The briquette’s low-sulfur and low-NO_x characteristics may contribute to cleaner air and reduced exposure risk for traditional salt producers and households relying on biomass fuels.

3.2.4 Study Limitations and Implications for Interpretation

The combustion tests were conducted in a garden-incinerator chamber as a controlled and repeatable setup; however, this configuration does not fully replicate the airflow pattern, heat transfer, and fuel-feeding practice of traditional salt-production furnaces. Consequently, the absolute CO concentrations reported here should be interpreted as chamber-specific indicators of combustion completeness rather than direct field-equivalent values. Future work should include (i) field trials in representative salt furnaces, (ii) tests under controlled excess-air ratios and secondary-air staging, and (iii) reporting of oxygen-corrected concentrations to a defined reference O₂ level where required by the target standard.

3.3 Energy Requirement Implication for Salt Boiling (Illustrative Estimation)

In addition to emission benchmarking, an energy-demand context is useful for judging feasibility of the proposed briquettes for traditional salt boiling. Because direct calorific value testing (e.g., bomb calorimetry) was not performed in this study, the briquette higher heating value (HHV) was estimated from proximate analysis using the correlation by Parikh et al. [36]. The correlation expresses HHV on a dry basis as:

$$HHV_{db} \text{ (MJ/kg)} = 0.3536 \cdot FC + 0.1559 \cdot VM - 0.0078 \cdot Ash \quad (5)$$

where FC, VM, and Ash are wt% on a dry basis. Using the measured proximate fractions of the briquettes (Table 2: FC = 11%, VM = 60%, Ash = 29%), the estimated HHV is ≈ 13.02 MJ/kg (dry basis). With 12% moisture as received, a first-order as-received estimate is ≈ 11.46 MJ/kg briquette (13.02×0.88).

For salt boiling, the dominant theoretical energy requirement is water evaporation from brine. As a simple illustration, evaporating 1 kg of water typically requires on the order of ~ 2.5 MJ (heating from ambient to boiling plus latent heat). If brine entering the boiling step is concentrated to 20% salt, producing 1 kg salt requires evaporating approximately ~ 4 kg water, corresponding to ~ 10

MJ/kg salt of theoretical heat demand. In practice, actual fuel consumption will be higher due to heat losses and combustion completeness (consistent with the elevated CO observed in Table 4); therefore, improving airflow management and mixing (secondary air) is expected to increase useful heat release and reduce the required briquette mass during salt boiling.

The increasing utilization of waste biomass (including rice husk and sawdust) as an engineered feedstock has been driven by both waste-management needs and concerns over deforestation, supporting the rationale for converting locally available residues into solid fuels [37]. Beyond technical performance, adoption among salt farmers is strongly influenced by institutional and socio-economic factors. A recent logistic regression study among salt farmers in rural Aceh indicates that policy support, perceived economic feasibility, and environmental awareness are the strongest predictors of willingness to adopt cattle-manure briquettes [38]. Therefore, technical improvements (e.g., CO reduction through better air supply and mixing) should be complemented by implementation strategies that address these adoption drivers.

4. Conclusions

This study assessed multi-feedstock biomass briquettes produced from cow manure, rice husk, and wood dust as potential substitutes for tropical hardwood firewood used in traditional salt production. The work combined proximate analysis with controlled-combustion emission testing and benchmarking against commonly used local firewood (petai, sentang, rambutan) and pellet references from the literature. The proximate analysis indicates that the briquettes represent a non-carbonized biomass fuel with 12% moisture (as received) and a dry-basis composition of 29% ash, 60% volatile matter, and 11% fixed carbon. These characteristics imply easy ignition and rapid volatile-driven flaming but a comparatively limited char-burning phase relative to many hardwood fuels. The high ash fraction is an inherent consequence of mineral-rich manure and silica-bearing rice husk, and it signals a practical requirement for more frequent ash removal and furnace/grate designs that avoid airflow blockage during long boiling operations.

Emission measurements show a distinctive environmental profile. The briquettes produced exceptionally low SO₂ concentrations (1.2 mg/Nm³), substantially lower than all tested firewood types and lower than the cow-dung pellet reference, indicating a strong sulfur-emission advantage for reducing acid gas formation and potential corrosion/irritation risks in work environments. NO₂ emissions (7.4 mg/Nm³) were within the range observed for the tested woods, suggesting that the fuel blend does not elevate NO_x to atypical levels under the applied conditions. In contrast, CO emissions were the highest among the compared fuels (1,857.6 mg/Nm³), indicating incomplete combustion in the present chamber configuration. This outcome is consistent with the need for improved oxygen delivery and mixing (e.g., staged/secondary air) when burning dense heterogeneous briquettes with high volatile release under natural-draft conditions.

From a feasibility standpoint, the briquettes provide a clear emission benefit with respect to sulfur, while their practical deployment for salt production should be accompanied by operational adaptation and combustion-system optimization. Recommendations include: (i) improving furnace or stove airflow management (primary and secondary air pathways), (ii) adjusting briquette geometry/porosity or densification parameters to enhance internal diffusion and stable burning, and (iii) implementing ash management practices suitable for high-ash fuels. The present study is limited to controlled combustion in a garden-incinerator chamber; therefore, field trials in representative salt furnaces, reporting of oxygen-corrected emissions where required, and direct calorific value measurement (bomb calorimetry) are recommended to refine energy-demand estimates and strengthen deployment guidance. Overall, the briquettes show strong potential as a locally sourced, renewable fuel alternative that can reduce firewood dependency, provided that combustion design is improved to lower CO and increase thermal efficiency.

From a broader sustainability perspective, aligning material and energy choices with climate and environmental targets requires a lifecycle-conscious approach and policy-aware decision making, reinforcing the motivation to evaluate alternative fuels beyond purely technical performance [39].

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References

- [1] Athaillah T, Rahmi S. The effectiveness of tunnel salt construction in Pidie District, Aceh. *Russ J Agric Socio-Econ Sci.* 2020;10(118):198-205. <https://doi.org/10.18551/rjoas.2020-10.22>
- [2] Amru K, Anjani R, Aryantie MH, Ihsan IM, Oktivia R. Calculation of carbon emissions and sequestration to support the net zero emission target of Central Java Province (case study: Cilacap and Brebes Regencies). *J Nat Resour Environ Manage.* 2024;14:525-33. <https://doi.org/10.29244/jpsl.14.3.525>
- [3] Mawardi I, Winanti WS, Sudinda TW, Alimin A. Analysis of achievements of net zero carbon emissions as Indonesia's reference in fulfillment of climate change mitigation commitments. *J Sain dan Teknol Mitig Bencana.* 2022;16:71-89.
- [4] Agustiar, Bantacut T, Romli M, Pramudya B, Aulia MR. The potential of bio-try briquettes for biomass power plant in Aceh Province - case study in South-West Aceh, Indonesia. *J Ecol Eng.* 2023;24:115-24. <https://doi.org/10.12911/22998993/170079>
- [5] Cao L, Yu IK-M, Xiong X, Tsang DC-W, Zhang S, Clark JH, et al. Biorenewable hydrogen production through biomass gasification: A review and future prospects. *Environ Res.* 2020;186:109547. <https://doi.org/10.1016/j.envres.2020.109547>
- [6] Sanka PM, Germain O, Khalifa L, Komakech H, Magambo H. Production of low-emission briquettes from carbonized faecal sludge as an alternative source of cooking energy. *Energy Sustain Soc.* 2024;14:1-15. <https://doi.org/10.1186/s13705-024-00449-0>
- [7] Waheed MA, Akogun OA, Enweremadu CC. Dataset on the performance characteristics of briquettes from selected agricultural wastes using a piston-type briquetting machine. *Data Brief.* 2023;50:109476. <https://doi.org/10.1016/j.dib.2023.109476>
- [8] Akhator PE, Bazuaye L, Ewere A, Oshiokhai O. Production and characterisation of solid waste-derived fuel briquettes from mixed wood wastes and waste PET bottles. *Heliyon.* 2023;9:e21432. <https://doi.org/10.1016/j.heliyon.2023.e21432>
- [9] Siregar AR, Harahap LA, Panggabean S. Pemanfaatan sekam padi dan limbah teh sebagai bahan briket arang dengan perekat tetes tebu. *J Rekayasa Pangan Pertanian.* 2015;3:396-402.
- [10] Wang S, Luo K, Hu C, Sun L, Fan J. Impact of operating parameters on biomass gasification in a fluidized bed reactor: An Eulerian-Lagrangian approach. *Powder Technol.* 2018;333:304-16. <https://doi.org/10.1016/j.powtec.2018.04.027>
- [11] Sulmiyati, Said NS. Pengolahan briket bio-arang berbahan dasar kotoran kambing ve cangkang kemiri di Desa Galung Lombok, Kecamatan Tinambung, Polewali Mandar. *J Pengabdian Masyarakat.* 2017;3:108-18. <https://doi.org/10.22146/jpkm.25529>
- [12] Radityaningrum AD, Harnawan BY. Karakteristik biobriket dari kotoran sapi dengan kulit durian. *J Teknol Lingkungan.* 2022;23:222-8. <https://doi.org/10.29122/jtl.v23i2.4730>
- [13] Antwi-Boasiako C, Acheampong BB. Strength properties and calorific values of sawdust-briquettes as wood-residue energy generation source from tropical hardwoods of different densities. *Biomass Bioenergy.* 2016;85:144-52. <https://doi.org/10.1016/j.biombioe.2015.12.006>
- [14] Sher F, Pans MA, Sun C, Snape C, Liu H. Oxy-fuel combustion study of biomass fuels in a 20 kW fluidized bed combustor. *Fuel.* 2018;215:778-86. <https://doi.org/10.1016/j.fuel.2017.11.039>
- [15] Yank A, Ngadi M, Kok R. Physical properties of rice husk and bran briquettes under low pressure densification for rural applications. *Biomass Bioenergy.* 2016;84:22-30. <https://doi.org/10.1016/j.biombioe.2015.09.015>
- [16] Adu-Poku KA, Appiah D, Asosega KA, Derkyi NSA, Uba F, Kumi EN, et al. Characterization of fuel and mechanical properties of charred agricultural wastes: Experimental and statistical studies. *Energy Rep.* 2022;8:4319-31. <https://doi.org/10.1016/j.egyr.2022.03.015>
- [17] Akolgo GA, Awafo E, Essandoh EO, Owusu PA, Uba F, Adu-poku KA. Assessment of the potential of charred briquettes of sawdust, rice and coconut husks: Using water boiling and user acceptability tests. *Sci Afr.* 2021;12:e00789. <https://doi.org/10.1016/j.sciaf.2021.e00789>
- [18] Akpenpuun TD, Salau RA, Adebayo AO, Adebayo OM, Salawu J, Durotoye M. Physical and combustible properties of briquettes produced from a combination of groundnut shell, rice husk, sawdust and wastepaper using starch as a binder. *J Appl Sci Environ Manag.* 2020;24:171. <https://doi.org/10.4314/jasem.v24i1.25>
- [19] Noah AS, Okon-Akan OA, Abiola JK. Energy characterization of briquettes produced from admixture of *Arundo donax* L. and coconut coir. *Am Sci Res J Eng Technol Sci.* 2019;56:136-51.
- [20] Ofori P, Akoto O. Production and characterization of briquettes from carbonized cocoa pod husk and sawdust. *OALib.* 2020;7:1-20. <https://doi.org/10.4236/oalib.1106029>

- [21] Vyas D, Sayyad F, Khardiwar M, Kumar S. Physicochemical properties of briquettes from different feed stocks. *Curr World Environ*. 2015;10:263-9. <https://doi.org/10.12944/CWE.10.1.32>
- [22] Kidmo DK, Deli K, Bogno B. Status of renewable energy in Cameroon. *Renew Energy Environ Sustain*. 2021;6:11. <https://doi.org/10.1051/rees/2021001>
- [23] Tamba JG. An analysis of Cameroon's energetic system, 2001-2010. *Energy Sources B*. 2017;12:216-22. <https://doi.org/10.1080/15567249.2015.1060547>
- [24] Deac T, Fechete-Tutunaru L, Gaspar F. Environmental impact of sawdust briquettes use - experimental approach. *Energy Procedia*. 2016;85:178-83. <https://doi.org/10.1016/j.egypro.2015.12.324>
- [25] Falemara BC, Joshua VI, Aina OO, Nuhu RD. Performance evaluation of the physical and combustion properties of briquettes produced from agro-wastes and wood residues. *Recycling*. 2018;3:37. <https://doi.org/10.3390/recycling3030037>
- [26] Sunnu AK, Adu-Poku KA, Ayetor GK. Production and characterization of charred briquettes from various agricultural wastes. *Combust Sci Technol*. 2023;195:1000-21. <https://doi.org/10.1080/00102202.2021.1977803>
- [27] Athaillah T, Masykur M, Husin H, Adib A, Aulia MR. Briquettes from a mixture of cow manure, rice husks and wood dust as alternative fuel. *J Ecol Eng*. 2024;25:290-9. <https://doi.org/10.12911/22998993/177194>
- [28] Rahmaisyanti A, Sofyan A, Sulistyati M, Hidayati YA. Physico-chemical and energy properties of biomass briquettes composed from dairy cow feces and coffee ground with tapioca flour addition. *J Ilmu Teknol Kayu Tropis*. 2024;24(2). <https://doi.org/10.24198/jit.v24i2.56662>
- [29] Szymajda A, Łaska G, Joka M. Assessment of cow dung pellets as a renewable solid fuel in direct combustion technologies. *Energies*. 2021;14:1192. <https://doi.org/10.3390/en14041192>
- [30] Geberehiet GA, Gebreegziabher TG, Mekonen AG, Hagos GK, Gebresilasie TN. Development of briquettes suitable for energy generation from residue of sorghum stalk and groundnut husk. *Mater Renew Sustain Energy*. 2025;14:35. <https://doi.org/10.1007/s40243-025-00309-7>
- [31] Wahyuni NS, Sulistyo J. Fuelwood characteristics of five species grown in Merauke Forest. *Wood Res J*. 2016;7(1):13-7. <https://doi.org/10.51850/wrj.2016.7.1.13-17>
- [32] García-Quezada J, Musule-Lagunes R, Wehenkel C, Prieto-Ruíz JA, Núñez-Retana V, Carrillo-Parra A. Effect of firewood moisture content on quality, yield and economic gain during charcoal production in a modified half-orange kiln. *Fuels*. 2024;5(1):1-16. <https://doi.org/10.3390/fuels5010001>
- [33] Fahrussiam F, Lestari D, Ningsih RV. Calorific value of several types of wood through proximate analysis and chemical components approach. *J Bioindustri*. 2023;23(1):22-31. <https://doi.org/10.29303/jbt.v23i1.4416>
- [34] Efiyanti L, Wati SA, Setiawan D, Saepulloh, Pari G. Sifat kimia dan kualitas arang lima jenis kayu asal Kalimantan Barat [Chemical properties and quality of charcoal from five types of wood from West Kalimantan]. *J Penel Has Hutan*. 2020;38(1):55-68. <https://doi.org/10.20886/jphh.2020.38.1.45-56>
- [35] Dołżyńska M, Obidziński S, Piekut J, Yildiz G. The utilization of plum stones for pellet production and investigation of post-combustion flue gas emissions. *Energies*. 2020;13(19):5107. <https://doi.org/10.3390/en13195107>
- [36] Parikh J, Channiwala SA, Ghosal GK. A correlation for calculating HHV from proximate analysis of solid fuels. *Fuel*. 2005;84(5):487-94. <https://doi.org/10.1016/j.fuel.2004.10.010>
- [37] Ikubanni PP, Adeleke AA, Adekanye TA, Aladegboye OJ, Agboola OO, Ogunsemi BT. Particleboard from biomass wastes: A review of production techniques, properties, and future trends. *Res Eng Struct Mater*. 2025;11(2):713-740. <https://doi.org/10.17515/resm2024.265ma0502rv>
- [38] Athaillah T, Masykur M, Muzammil A, Rizki A, Bagio B, Safrika S. Determinants of adoption of cattle manure briquettes: a logistic regression study among salt farmers in rural Aceh, Indonesia. *Front Sustain Food Syst*. 2026;9:1724877. <https://doi.org/10.3389/fsufs.2025.1724877>
- [39] Ozmen HB, Tanriverdi B. Concrete paradox: Economic importance, environmental impacts, and the sustainability of concrete material. *Res Des*. 2025;2(2):66-84. <https://doi.org/10.17515/rede2025-008en0801rs>

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Heat and fire resistance of geopolymer concrete

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Abstract

Geopolymer concrete, made from aluminosilicate materials such as fly ash and an alkaline solution, is a more environmentally friendly alternative to Portland cement. While its use is growing, more research is needed to explore its commercial applications, particularly for high-temperature resistance. The effects of high-temperature exposure on geopolymer concrete were evaluated and compared with those of normal concrete. Geopolymer concrete specimens were heat-cured at 115°C for 24 hours, then stored at room temperature until 28 days of age. In contrast, normal concrete specimens were cured in water at room temperature. Both concrete types were then heated in an iron furnace at 400°C-800°C for 2 hours, after which they were cooled and tested. A visual inspection assessed the concrete for spalling, cracking, and discoloration, while tests measured mass loss and residual compressive strength. Results indicated that both geopolymer and traditional concrete lost compressive strength after burning; however, at 800°C, geopolymer concrete maintained a 10.09% higher residual strength. Geopolymer concrete also showed less weight loss due to lower water content and exhibited no spalling. In summary, geopolymer concrete resists high temperatures better than normal concrete.

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1. Introduction

Concrete is the most applied construction material globally because of its favorable mechanical properties, which support the development of rigid, reliable, and durable infrastructure. In 2025, global concrete production reached 4.15 billion tons, and with a constant need to develop infrastructure to serve a growing population, it is projected to grow at around 4% annually [1].

The large-scale production and consumption of concrete pose another significant environmental issue, with concrete production accounting for 5–8% of global emissions [2-3]. Considering this, innovation in more environmentally friendly concrete varieties, such as geopolymer concrete, is very much needed [4]. This analysis aims to evaluate the heat resistance of geopolymer concrete and determine how it could be implemented across various types of infrastructure.

The term "geopolymer material" is no longer uncommon in scholars' research topics and experiments. Geopolymer concrete has been gaining significant attention in recent years due to its unique properties, benefits, and ability to reduce emissions in the construction industry. Research also indicates that an FA- or fly-ash-based geopolymer concrete beam has properties similar to those of a regular reinforced concrete beam used in construction [5]. This concrete has been a hot topic since 2016, even though it hasn't been internationally recognized for its cost-effectiveness and production efficiency [2].

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Geopolymer concrete is a concrete mixture composed of aluminosilicate materials such as fly ash (FA), metakaolin (MK), slag (SG), rice husk ash (RHA), etc. Geopolymer concrete works by polymerizing aluminosilicate materials with an alkaline base solution, which reacts rapidly with them to promote the reaction between silica (Si) and alumina (Al) under alkaline conditions [2]. This reaction forms strong, three-dimensional polymeric chains (Si-O-Al-O) that result from the polycondensation of silica, alumina, and high alkali content [2]. This polycondensation will then contribute to the concrete's compressive strength. Materials categorized as geopolymers are usually amorphous rather than crystalline, making them easier to polymerize than other zeolitic materials [6].

Depending on the types of aluminosilicate materials used, the concrete's result and strength can vary. While MK-based geopolymer boasts the most persistent properties, it requires a lot of water, leading to flow issues. Fly ash-based geopolymer offers superior durability, and slag-based geopolymer shines with its rapid initial strength and strong acid resistance [7]. In addition to the materials used, the activator ratio also affects the concrete. For example, an activator such as $\text{Na}_2\text{SiO}_3/\text{NaOH}$ at a 3:1 ratio can significantly improve the compressive strength of fly ash-based geopolymer concrete [8]. Since its initial discovery decades ago, numerous studies and advancements have been conducted to enhance the mechanical, physical, and chemical properties of geopolymer concrete. Laboratory findings suggest that geopolymer concrete samples can match or even surpass the properties of ordinary Portland cement concrete (OPC) [9-10].

Geopolymer concrete also offers sustainability advantages, with a significant portion of the admixture materials listed above, often being waste products used as partial replacements for cement in the mix design [10]. Hence, geopolymer concrete leads to lower CO_2 emissions than ordinary concrete, with OPC production accounting for 9% higher emissions. [11]. The superior sustainability of geopolymer concrete is well suited to the concept of green buildings and the principles of environmental responsibility and resource efficiency [12].

Fires are frequent disasters. The Director General of Regional Administration at the Ministry of Home Affairs revealed that there were 17,768 fire cases in Indonesia. The most common cause is a short circuit in the electrical system. Based on the ISO 834 curve in EN 1991-1.2, the temperature can reach 600°C during the first 5 minutes of fire. Therefore, structural materials with high-temperature resistance are important [13]. The inorganic structure of geopolymer concrete offers advantages such as non-toxicity, chemical resistance, and, notably, fire resistance [14]. Geopolymer concrete has been shown to withstand loading even after 2 hours of exposure to hydrocarbon fire and to exhibit better shrinkage and spalling performance than Portland cement concrete [15-17]. This suggests that geopolymer concrete has strong potential as a high-temperature-resistant construction material.

A study investigated the heat resistance of geopolymer concrete exposed to a hydrocarbon fire for 120 minutes, in accordance with Australian standards [15]. The binder for the geopolymer concrete was prepared by activating Gladstone fly ash with an alkaline solution containing Na_2SiO_3 and NaOH . The ratio of activator to fly ash was 0.4, while the sodium silicate to sodium hydroxide ratio in the alkaline solution was 2.5. Testing was conducted on two sizes of cylinders with diameters of 150 mm and 100 mm and corresponding heights of 300 mm and 200 mm, as well as panels measuring 1075 mm by 1075 mm by 200 mm. The cylindrical samples were subjected to fire from all directions, while the panels were exposed on one side. The geopolymer panels exhibited lower thermal diffusivity than normal concrete with aggregates no larger than 14 mm. Due to this lower diffusivity, geopolymer concrete panels offer superior heat resistance. The material's reduced permeability increases its heat storage capacity, making geopolymer concrete less prone to spalling during hydrocarbon fires.

Due to the abovementioned properties of geopolymer concrete, it has many potential applications across various infrastructure sectors. Firstly, geopolymer concrete has been used in the construction of multi-story buildings, with the first fully geopolymer-concrete-based residential building built in 1989 in Russia [10]. Geopolymer concrete is also advantageous for the production of precast concrete elements due to the controlled conditions required during the manufacturing process [10].

Table 1. Previous studies have examined how geopolymer concrete performs under high-temperature exposure

References	Temperature	Note
Heikal et al. [18]	Room temperature, 200, 400, 600, 800°C	- The specimens are composed of normal concrete with fly ash incorporated as a partial substitution. - After immersion in water for curing, the specimens were oven-dried at 105°C for 24 hours ahead of thermal treatment. - The heating rate was 3°C/min. At the target temperature, specimens were held for 3 hours. - At 400°C, the compressive strength increased by approximately 33%, whereas at 600–800°C, it decreased by approximately 9% (residual strength of about 91%).
Park et al. [19]	20, 200, 400, 600, 800°C	- The specimens underwent a 28-day curing period at room temperature. - Heating was applied at a rate of 10°C/min. Upon reaching the target temperature, the specimens were held at that level for 2 hours. - The compressive strength of the specimens increased with heating, only starting to decline at 800°C.
Luhar et al. [20]	Room temperature, 200, 400, 600, 800°C	- The concrete specimens underwent oven curing at 90°C for 48 hours. - Heating was applied at a rate of 4.4°C per minute, and once the target temperature was reached, the specimens were held at that level for 2 hours. - The compressive strength began to decrease as soon as heating commenced from the initial temperature.
Tayeh et al. [21]	Room temperature, 100, 200, 300, 400, 500, 600, 700, 800°C	- The specimens were oven-cured at 80°C for 24 hours and then held at the target temperature for 2 hours. - When heated, the compressive strength increased at 100°C and decreased at 200–800°C.
Razak et al. [22]	25, 500, and 1200°C.	- Normal concrete specimens were cured at ambient temperature for 28 days. - Geopolymer concrete specimens were oven-heated to 60°C for 24 hours, then held at the target temperature for 2 hours. - Heating reduced the compressive strength of normal concrete (OPC) specimens. - In contrast, geopolymer concrete specimens showed an increase in compressive strength at 500°C, but a decrease at 1200°C.

A review of research and evaluations on the heat resistance of geopolymer concrete indicates that it generally performs better at high temperatures than OPC, highlighting its strong potential for future applications [9]. However, upon closer examination, studies on this topic exhibit

considerable diversity in the variables considered and significant variation in admixture components across studies. Previous studies regarding how geopolymer concrete performs at elevated temperatures are shown in Table 1 and will be compared with the results of this study. This variance has a substantial impact on heat resistance, as previously discussed. This presents a challenge when comparing results across different studies, which is why more studies and analyses are needed on this topic, particularly those that build on previous findings.

Previous research has found that geopolymer concrete is a suitable material for use as a structural component in infrastructure. One experimental study investigated the mechanical properties of geopolymer concrete made from low-aluminum, iron-rich, calcium fly ash. The study assessed its compressive strength and moment capacity and compared the results to those of normal Portland cement concrete. The study found that geopolymer concrete can match or even outperform regular Portland cement-based concrete in some areas, such as ultimate moment capacity [23].

Accordingly, this study investigates how geopolymer concrete behaves under high-temperature exposure and compares its performance to that of normal concrete. The research specifically evaluates visual deterioration, residual compressive strength, and mass loss following exposure to temperatures ranging from 400°C to 800°C. Through this investigation, the study seeks to provide further insight into the potential application of geopolymer concrete in structures subjected to high-temperature conditions such as fire.

2. Research Significance

This analysis aims to investigate the effects of high temperature and fire exposure on the compressive strength of fly ash-based geopolymer concrete and Portland cement-based concrete (normal concrete). This analysis differs from previous analyses by Park et al. [19] and by Razak [22]. The curing method is conducted at a much higher temperature (115°C) than the more common approaches of curing at room temperature [19] or at a lower temperature (60°C) [22], as used for geopolymer concrete. Hence, the findings of this study will provide further insight and knowledge within this field of research, aiding future studies and the real-world application of geopolymer concrete.

3. Methodology

3.1. Limitations of the Study

This study has several limitations that should be considered when interpreting the results. The geopolymer concrete was produced using Class F fly ash obtained from a specific coal-fired power plant in Malang; therefore, the findings may not be directly generalizable to geopolymer systems with different source materials or chemical compositions. In addition, the specimens used in this study were limited to cylindrical samples with a diameter of 10 cm and a height of 20 cm, which may not fully represent the behavior of larger structural elements under fire exposure.

The curing method used in this study also reflects a specific condition, as geopolymer concrete specimens were heat-cured at 115°C to achieve higher compressive strength, whereas other studies commonly use ambient curing. This difference in curing regime may influence the material's microstructure and thermal response. Furthermore, the burning process was conducted using direct flame exposure rather than standard furnace heating curves, which may differ from real-world fire scenarios or standardized fire-testing conditions.

The heating protocol was limited to temperatures ranging from 400°C to 800°C in 100°C increments, with a constant heating rate of 15°C/min and a fixed exposure duration of 2 hours. While this approach allows for controlled comparison, it does not fully capture the variability of actual fire events. Finally, the evaluation of material performance was limited to compressive strength, mass loss, and microstructural observation using scanning electron microscopy (SEM), without considering other mechanical or durability properties. Therefore, further studies are recommended to explore different material sources, curing conditions, fire exposure scenarios, and additional performance parameters.

3.2. Procedure

The analysis methodology can be broken down into five stages.

- Stage 1 involves preparation of materials and equipment, ensuring everything necessary for the research is readily available.
- Stage 2, material suitability is assessed through tests like determining the specific gravity of aggregates with ASTM C127 [24].
- Stage 3 focuses on specimen casting. Here, 45 normal concrete and 45 geopolymer concrete specimens are cast in Ø100 mm × 200 mm cylindrical molds. The planned strength for normal concrete is 30 MPa, while the geopolymer concrete utilizes a 12 M NaOH alkaline solution. Curing then followed, with geopolymer concrete cured in an oven at 115°C for 24 hours and normal concrete cured by immersion for 7 days.
- Stage 4 deals with specimen testing. This includes a compressive strength test on day 7, followed by another on day 28 (without burning the sample). A burning test is then performed on a separate sample on day 28, and finally, a Scanning Electron Microscope (SEM) test is conducted.
- Stage 5 is dedicated to analyzing, discussing, and drawing conclusions from the gathered test results. This final stage ties together all the research findings.

3.3. Materials

The geopolymer concrete mix design comprises coarse aggregate, fine aggregate, water, and Portland Composite Cement (PCC), in accordance with the provisions of SNI 7657:2012 [25]. Here, the geopolymer concrete mix design uses coarse and fine aggregates, fly ash, and an alkaline solution. The aggregates used in both varieties are the same to ensure a consistent and accurate comparative analysis. The maximum sizes for coarse and fine aggregates were 25 mm and 0.6 mm, respectively, and both were used under Saturated Surface Dry (SSD) conditions. Semen Tiga Roda was used as the PCC cement in accordance with SNI 7064:2014 [26]. The fly ash was sourced from Malang and classified as F per ASTM C618 [27].

The alkaline solution used consists of NaOH and Na₂SiO₃ (water glass) mixed at a 1:3 ratio. The NaOH solution used has a molarity of 12 M [23]. The water glass solution was obtained from PT. Ajidharmamas Tritunggal Sakti. All materials in the mix design underwent preliminary testing, as highlighted in stage 2 of the diagram above.



Fig. 1. Alkaline solution consisting of (a) NaOH crystals and (b) NaOH solution

3.4. Equipment

The testing process involved various pieces of equipment at each stage. For material testing (Stage 2), scales, a 500mL measuring cup, and sieves (with a sieving machine) were used. Stage 3 involved formwork (or cylinder molds), oil for releasing the molds, a concrete mixer for large batches, and Abram's cone to test the aggregate. An oven was used for curing geopolymer specimens. Stage 4 relied on a concrete compression tester to measure strength and a shake table for additional

testing. Specialized equipment used for the burning test includes an iron furnace with a thermocouple for temperature control.

3.5. Concrete Specimens

This study analyses both normal and geopolymer concrete specimens to serve as a basis for comparison and to provide a control for evaluating the behavior of the geopolymer variety. The mix design for the normal concrete samples is shown in the table below, with a target compressive strength of 30 MPa and a slump of 75-100 mm. The mix design for the geopolymer concrete specimens is presented in the table below and was calculated based on the findings of an analysis by Prinya Chindaprasirt et al. [28].

Table 2. Mix design for normal concrete

Material	Percentage by Weight (%)
Coarse Aggregate	44.80
Fine Aggregate	23.68
Cement	23.32
Water	8.20

Table 3. Mix design for geopolymer concrete

Material	Percentage by Weight (%)
Coarse Aggregate	35
Fine Aggregate	30
Fly ash	23
NaOH	3
Na ₂ SiO ₃ (waterglass)	9

The steps of creation for both sets of concrete specimens include material and equipment preparation; mixing the admixture components of the mix design in a concrete mixer; casting the specimens; removing the specimens from cylinder molds; and finally, leaving the specimens to cure before the main testing stage. The curing conditions for each concrete specimen type differed. Normal concrete samples were cured by immersion in water at room temperature for 7 days, whereas geopolymer concrete samples were cured in an oven at 115°C for 24 hours. After the initial curing stage, all specimens were stored at room temperature until 28 days, then subjected to the burning test.



(a)



(b)

Fig. 2. Casting of (a) normal concrete and (b) geopolymer concrete specimens



Fig. 3. Unmolded (a) normal concrete and (b) geopolymer concrete specimens

A curing temperature of 115°C was used to ensure sufficient geopolymerization of the fly ash-based binder prior to high-temperature exposure. Elevated heat curing is commonly used in geopolymer concrete to accelerate the dissolution of aluminosilicate materials and promote the formation of a dense geopolymer matrix [29]. Previous studies have shown that curing temperatures above 90°C can further enhance the reaction rate and strength development of fly ash-based geopolymers [30-31]. Therefore, a curing temperature of 115°C was applied in this study to ensure that the geopolymer matrix was fully developed before the fire exposure tests.



Fig. 4. Curing process of specimens: (a) Submerged curing for normal concrete and (b) heated curing for geopolymer concrete specimens

3.6. Testing of Specimens

The burning test involves placing the concrete specimens in an iron furnace. The temperature inside the furnace will rise at a controlled rate of 15°C per minute. The heating rate was set to 15°C/min to ensure a gradual temperature increase and avoid thermal shock in the concrete specimens. A controlled heating rate helps reduce excessive thermal gradients between the specimen's surface and core, which may otherwise induce internal stresses and premature cracking. In addition, maintaining a constant heating rate ensures that all specimens experience the same thermal history, allowing the effect of the target temperature to be evaluated consistently.

The specimens were placed at the center of the closed furnace chamber to ensure uniform heat exposure. Adequate spacing was maintained between specimens to allow uniform heat circulation. The specimens remained stationary during the heating process and were not rotated or repositioned during exposure.

Once the target temperatures of 400, 500, 600, 700, and 800°C are reached, the specimens will be held at each temperature for approximately 2 hours. After the heating period, the specimens were allowed to cool naturally in the furnace until room temperature was reached, after which further

testing was performed. The distance between the specimen and the heat source was maintained at approximately 20 cm, following the experimental setup used by Razak et al. [22].

Temperature levels of 400°C to 800°C, in 100°C increments, were selected to evaluate the progressive degradation of concrete under elevated temperatures. Previous studies on conventional concrete commonly used similar temperature ranges, allowing comparison between geopolymer concrete and ordinary Portland cement concrete under high-temperature exposure [32,35-36]. The selected temperature range also represents critical stages of thermal damage in concrete, including dehydration of hydration products (100-300°C), development of microcracking (400-600°C), and significant deterioration of the internal matrix (600-800°C) [29-30,36]. The 100°C increment provides sufficient resolution to observe the degradation trend and identify potential critical temperatures where significant reductions in mechanical properties occur.

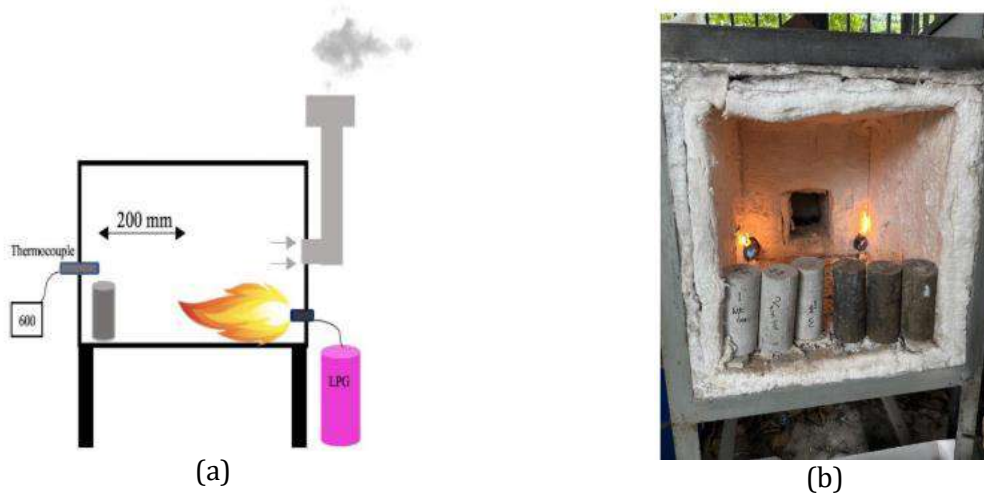


Fig. 5. Burning test: (a) experimental setup of the furnace used for high-temperature exposure of concrete specimens and (b) the burning test of the specimens



Fig. 6. Compressive strength test: (a) cured and heated concrete prior to testing and (b) specimens testing using a universal compression testing machine

The heating protocol used in this study differs from standard fire testing methods such as ASTM E119 [39], which follow predefined temperature–time curves. In this study, a constant heating rate with direct flame exposure was adopted, following approaches reported in previous experimental studies. This method was selected to simulate more severe, localized heating conditions, serving as a conservative scenario for evaluating concrete's high-temperature performance. Heat and fire resistance are evaluated based on compressive strength, with specimens subjected to heating and firing under controlled conditions tested in accordance with the standards specified in SNI 1974:2011 [40]. Specimens are first weighed and then heated. Molten sulfur is used to create a

smooth surface on the specimen, ensuring accurate results. The specimens are then placed into a Concrete Compression Tester to obtain results on the concrete's compressive strength. To analyze the effects of burning at the microscopic level, each specimen will be examined using Scanning Electron Microscopy (SEM). This technique provides high-resolution images of the concrete's microstructure by scanning the surface with focused electron beams [37], revealing how the burning process has altered it. This information is crucial for understanding the concrete's behavior under heat exposure, improving the accuracy of the overall results and analysis, and investigating the concrete's mineral composition.

4. Results and Discussion

Concrete and geopolymer concrete are subjected to each test, with three specimens of each type per test. The tests include a preliminary test (an early test to determine specimen suitability), a burning test, a compressive strength test, and an SEM test. The results of the various tests conducted on the specimens are presented in the following paragraphs.

4.1. Preliminary Test Results

The results of preliminary testing of the base materials of coarse aggregate, fine aggregate, and fly ash are shown in the following sections.

4.1.1. Specific Gravity and Absorption of Aggregate

Specific Gravity testing was done on both coarse (based on ASTM C127 [24]) and fine aggregates (based on ASTM C128 [33]) to calculate the mix design for the concrete samples. The specific gravity in these tests was measured at the Saturated Surface Dry (SSD) condition; therefore, absorption tests were also conducted to determine the amount of water to add to the respective aggregate materials to reach that condition. Below are the results of the specific gravity and absorption tests on both coarse and fine aggregates.

Table 4. Specific gravity and absorption test results for coarse aggregate

No	Parameter	Test				Unit
		I	II	III		
1	Container Weight	164.2	134.5	198.2	-	gr
2	Container + Dry Aggregate Weight	2173.5	2163.2	2208.3	-	gr
3	Dry Aggregate Weight	2009.3	2028.7	2010.1	-	gr
4	Container + (SSD) Aggregate Weight	2244.9	2217.6	2268.4	--	gr
5	(SSD) Aggregate Weight	2080.7	2083.1	2070.2	-	gr
6	(Submerged Aggregate Weight	1208.3	1208.3	1208.2	-	gr
					Avg	-
7	Bulk Specific Gravity	2.30	2.32	2.33	2.32	gr/ cm ³
8	Bulk Specific Gravity (SSD)	2.39	2.38	2.40	2.39	gr/ cm ³
9	Apparent Specific Gravity	2.51	2.47	2.51	2.50	gr/ cm ³
10	Absorption	3.55	2.68	2.99	3.07	%

The test results show that the coarse aggregate used has a bulk specific gravity of 2320 kg/m³, a bulk specific gravity in SSD condition of 2390 kg/m³ and a percentage absorption of 3.07% of the aggregate weight. The test results show that the fine aggregate used has a bulk specific gravity of 2,230 kg/m³, a bulk specific gravity in SSD condition of 2,320 kg/m³, and a percentage absorption of 3.84% by weight.

Table 5. Specific gravity and absorption test results for fine aggregate

No	Parameter	Test				Unit
		I	II	III		
1	Container Weight	82.8	72.3	97.4	-	gr
2	Container + Dry Aggregate Weight	564	555.8	577.3	-	gr
3	Dry Aggregate Weight	481.2	483.5	479.9	-	gr
4	(SSD) Aggregate Weight	500	500	500	--	gr
5	Measuring Cylinder Weight	905.8	894.2	903.5	-	gr
6	Measuring Cylinder + Aggregate + Water	1185.6	1182.3	1187.5	-	gr
					Avg	-
7	Bulk Specific Gravity	2.19	2.28	2.22	2.23	gr/ cm ³
8	Bulk Specific Gravity (SSD)	2.27	2.36	2.31	2.32	gr/ cm ³
9	Apparent Specific Gravity	2.39	2.47	2.45	2.44	gr/ cm ³
10	Absorption	3.91	3.41	4.19	3.84	%

4.1.2. Sieve Analysis for Coarse and Fine Aggregates

Sieve analyses were performed on both coarse and fine aggregates (in accordance with ASTM C136 [34]) to assess particle-size gradation relative to the respective base materials. The tables below show the results for the sieve analyses.

Table 6. Sieve analysis results for coarse aggregate

Sieve No.	Sieve Size (mm)	Average Weight Captured from 3 Repetitions (gr)	% Captured	% Cumulative	
				Captured	Passed
1 1/2"	37.5	0.000	0	0	100
1"	25	4.100	0.4	0.4	99.6
3/4"	19	549.933	54.29	54.7	45.30
1/2"	12.5	419.000	41.37	96.07	3.93
3/8"	9.5	39.300	3.88	99.95	0.05
4	4.75	0.000	0	99.95	0.05
8	2.36	0.000	0	99.95	0.05
Pan	-	0.533	0.05	100.0	0
Total		1012.9	100		

Table 7. Sieve analysis results for fine aggregate

Sieve No.	Sieve Size (mm)	Average Weight Captured from 3 Repetitions (gr)	% Captured	% Cumulative	
				Captured	Passed
3/8"	9.5	1.433	0.14	0.14	99.86
4	4.75	2.833	0.28	0.42	99.58
8	2.36	38.333	3.81	4.23	95.77
16	1.18	99.700	9.90	14.13	85.87
30	0.5	313.700	31.16	45.29	54.71
50	0.3	311.600	30.95	76.25	23.75
200	0.15	237.100	23.55	99.80	0.20
Pan	-	2.033	0.20	100.0	0
Total		1006.7	100		

4.1.3. Water Content Analysis for Fine Aggregate

The water content of the fine aggregate was evaluated to determine the additional water required to reach SSD. The measured water content 0.143%.

Table 8. Water content analysis results for fine aggregate

No	Parameter	Test			Avg.	Unit
		I	II	III		
1	Container Weight	67.5	72.0	59.4	-	gr
2	Container + Aggregate Weight	588.9	581.1	570.4	-	gr
3	Weight of Aggregate before oven drying	521.4	509.1	511.0	-	gr
4	Container + Dry Aggregate Weight	588.2	580.2	569.8	--	gr
5	Dry Aggregate Weight	520.7	508.2	510.4	-	gr
6	Water Content	0.134	0.177	0.117	0.143	%

4.1.4. Sludge Content Analysis for Fine Aggregate

The sludge content in this analysis is evaluated using volume and weight methods, ensuring it does not exceed 5%. The sludge content was found to be 1.985% by volume and 1.330% by weight.

Table 9. Sludge content analysis results (volume method)

No	Parameter	Test			Avg.	Unit
		I	II	III		
1	Aggregate + Sludge Volume	256	255	245	-	gr
2	Aggregate Volume	251	250	240	-	gr
3	Sludge Content	1.953	1.961	2.041	1.985	%

Table 10. Sludge Content Analysis Results (Weight Method)

No	Parameter	Test			Avg.	Unit
		I	II	III		
1	Container Weight	67.5	72	59.4	-	gr
2	Container + Aggregate + Sludge Weight	588.2	580.2	569.8	-	gr
3	Aggregate + Sludge Weight	520.7	508.2	510.4	-	gr
4	Container + Dry Aggregate Weight	579.9	575.1	562.7	--	gr
5	Dry Aggregate Weight	512.4	503.1	503.3	-	gr
6	Sludge Content	1.594	1.004	1.391	1.330	%

4.1.5. X-Ray Fluorescence Analysis of Fly Ash

X-Ray Fluorescence (XRF) analysis was conducted on the fly ash incorporated in the geopolymer mix design, resulting in the chemical composition breakdown shown in the table below. The results above show that fly ash can be classified as F per ASTM C618 [27].

Table 11. Chemical composition in percentage of fly ash

No	Compound	Percentage (%)
i.	SiO ₂	49.48
ii.	Al ₂ O ₃	16.46
iii.	CaO	13.13
iv.	Fe ₂ O ₃	9.22
v.	MgO	5.52
vi.	Na ₂ O	4.54
vii.	K ₂ O	1.71
viii.	TiO ₂	0.80
ix.	P ₂ O ₅	0.42

4.2 Burning Test Result

The burning test results are divided into two groups, each yielding a different type of data. Those two test results are the physical properties of the concrete, such as color and physical differences of the specimens. The second test result will be the different in weight of the concrete before and after burning.

4.2.1. Color and Physical Differences in Samples

The results in this section will be analyzed based on the physical differences between normal and geopolymer concrete before and after the burning process. The analysis data is based on scientific observations of the physical properties of each concrete after burning at 400, 500, 600, 700, and 800°C. Below are figures of the resulting specimen before and after burning.



Fig. 7. Physical condition of normal concrete specimens before and after exposure to 400°C: (a) before burning, bright gray in color; (b) after burning, showing no significant color change and fine surface cracks

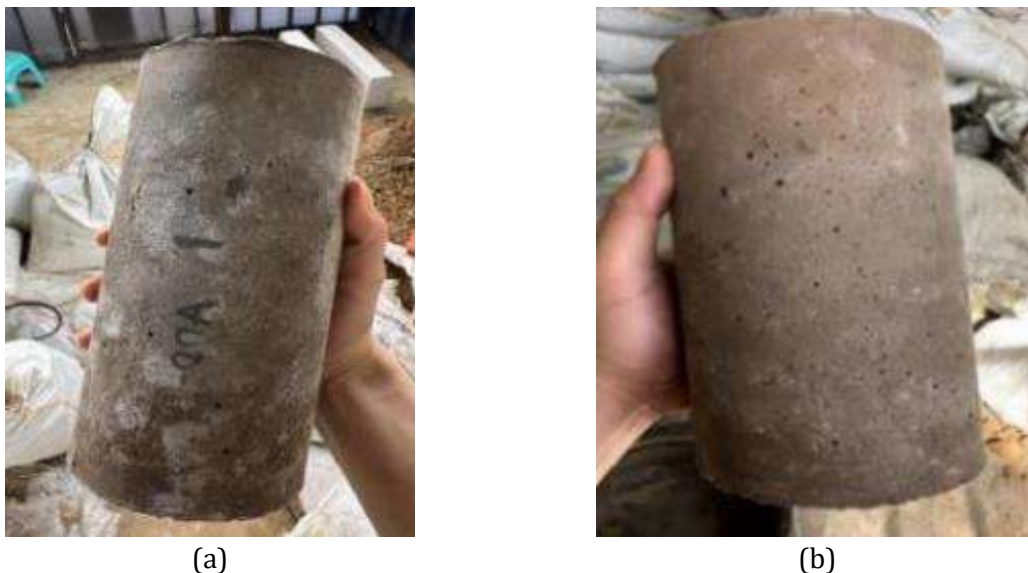


Fig. 8. Physical condition of geopolymer concrete specimens before and after exposure to 400°C: (a) before burning, dark gray in color; (b) after burning, showing no significant color change and fewer visible cracks



Fig. 9. Physical condition of normal concrete specimens before and after exposure to 500°C: (a) before burning, bright gray in color; (b) after burning, showing no significant color change, with localized yellowish spots, fine cracking, and spalling



Fig. 10. The figures show the physical condition of geopolymer concrete specimens prior to and after exposure to 500°C: (a) before burning, dark gray in color; (b) after burning, showing a slight orange hue, fine cracking, and no evidence of spalling

At 400°C, both normal concrete (Fig. 7) and geopolymer concrete (Fig. 8) exhibited no significant color change after burning. Fine surface cracking was observed in both materials; however, the geopolymer concrete exhibited less extensive cracking than normal concrete, indicating greater resistance to thermal exposure.

At 500°C, yellow spots were observed on some parts of the surface of the normal concrete (Fig. 9), while the geopolymer concrete (Fig. 10) changed colour to orange and grey. Both normal and geopolymer concrete developed fine cracks, but the geopolymer concrete had fewer cracks than the normal concrete. Only the normal concrete showed spalling.

At 600°C, some parts of the surface of normal concrete (Fig. 11) exhibited yellow-grey spots, whereas geopolymer concrete (Fig. 12) exhibited a colour change to orange, grey with additional red accents in some areas. Geopolymer concrete exhibited greater cracking with larger fibre sizes than normal concrete. Another phenomenon observed only in geopolymer concrete was expansion. However, geopolymer concrete did not undergo spalling as normal concrete did.

At 700°C, the color changes in both normal (Fig. 13) and geopolymer concrete (Fig. 14) were similar to those observed at 600°C. Normal concrete exhibited cracking and spalling at a higher fiber density than at 600°C. Geopolymer concrete also exhibited cracking of a similar size to that at 600°C, but with a higher fiber concentration. Additionally, geopolymer concrete experienced more severe cracking and exhibited swelling, while no spalling was observed.



Fig. 11. Physical condition of normal concrete specimens before and after exposure to 600°C: (a) before burning, bright gray in color; (b) after burning, showing no significant color change, with increased localized yellowish spots, longer cracks, and spalling



Fig. 12. The figures show the physical condition of geopolymer concrete specimens prior to and after exposure to 600°C: (a) before burning, dark gray in color; (b) after burning, showing a gray-orange color with reddish accents in some areas, more extensive cracking, no evidence of spalling, and noticeable expansion



Fig. 13. Physical condition of normal concrete specimens before and after exposure to 700°C: (a) before burning, bright gray in color; (b) after burning, showing no significant color change, with increased localized yellowish spots, longer cracks, and spalling

The result shows that normal concrete (Fig. 15) changed color to yellowish-white, while geopolymer concrete (Fig. 16) turned brownish at 800°C. Normal concrete experienced more cracking and spalling than at 700°C. In geopolymer concrete, numerous cracking fibers were observed, and the concrete swelled, but no spalling was observed.

The results indicated that normal concrete exhibited no significant color change at any tested temperature, with only isolated yellow spots observed on certain areas of the surface. Meanwhile, geopolymer concrete displayed a brown coloration. Normal concrete experienced cracking, with crack width increasing with temperature; however, the cracks in geopolymer concrete appeared wider and more numerous. Geopolymer concrete experienced severe cracking, the severity of which increased with increasing burning temperature.



Fig. 14. Physical condition of geopolymer concrete specimens before and after exposure to 700°C: (a) before burning, dark gray in color; (b) after burning, showing a brownish-gray color with reddish accents in some areas, more extensive cracking, no evidence of spalling, and noticeable expansion



Fig. 15. Physical condition of normal concrete specimens before and after exposure to 800°C: (a) before burning, bright gray in color; (b) after burning, showing a yellowish-gray color, longer cracks, and increased spalling



Fig. 16. Physical condition of geopolymer concrete specimens before and after exposure to 800°C: (a) before burning, dark gray in color; (b) after burning, showing a brownish-red color, more extensive cracking, no evidence of spalling, and noticeable expansion

It was observed that spalling was found in normal concrete starting at 500°C and increased with increasing burning temperature. In contrast, no spalling was found in geopolymer concrete up to 800°C. However, geopolymer concrete experienced both lateral and longitudinal deformation, or swelling. Spalling in normal concrete occurs because steam trapped within its pores builds up pressure. Conversely, geopolymer concrete has a more complex pore structure, including nanopores, which reduce internal pore pressure and prevent spalling—cracking in concrete—resulting from the differing thermal expansion coefficients of the aggregate and the binder, which weaken the bond and lead to deformation. The color change in geopolymer concrete is caused by oxidation of the fly ash component [38].

4.2.2. Weight Comparisons Before and After Burning

The result of this test will be a comparison of the weight of the concrete specimen before and after burning. The tables below will show the results.

Table 12. Comparison of specimen weight before and after high-temperature exposure for normal concrete at temperatures ranging from 400°C to 800°C

Normal concrete weight comparison			
Temperature	Average Weight (kg)		Reduction (%)
	Before	After	
400	3.4	3.01	11.66
500	3.44	3.02	12.22
600	3.46	3.04	12.13
700	3.66	3.21	12.39
800	3.64	3.16	13.28
Average Reduction			12.33

Table 13. Comparison of specimen weight before and after high-temperature exposure for geopolymer concrete at temperatures ranging from 400°C to 800°C

Geopolymer concrete weight comparison			
Temperature	Average Weight (kg)		Reduction (%)
	Before	After	
400	3.61	3.42	5.26
500	3.54	3.36	5.18
600	3.43	3.24	5.54
700	3.58	3.36	6.05
800	3.61	3.35	7.20
Average Reduction			5.85

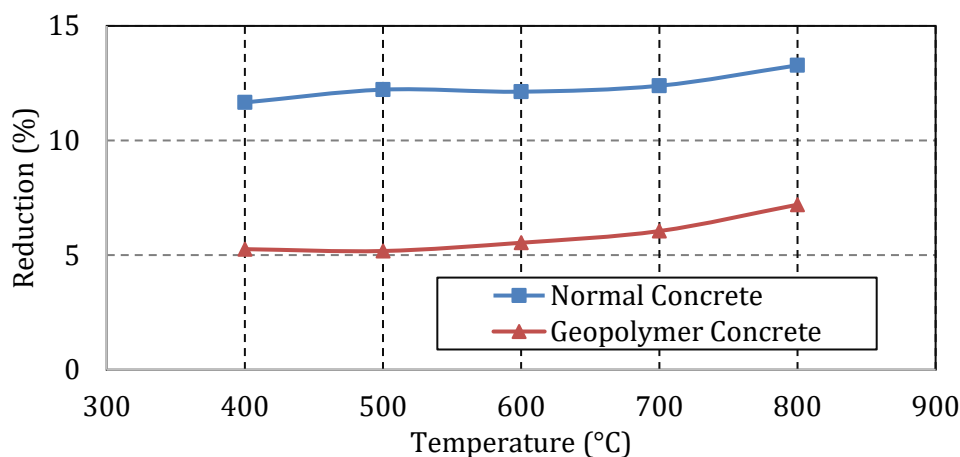


Fig. 17. Comparison of weight reduction between normal and geopolymer concrete specimens after exposure to temperatures ranging from 400°C to 800°C

Normal concrete experiences an average weight shrinkage of 12.33%, while geopolymer concrete experiences an average weight shrinkage of 5.85%. The increase in burning temperature does not significantly affect the weight shrinkage of concrete per unit increase in temperature. As can be seen in Figure 17, the percentage decrease is not very substantial. From the results of Table 12, Table 13 dan Fig. 17, we can conclude that after both concrete specimens are burned, normal concrete experiences significantly greater weight shrinkage than geopolymer concrete.

4.3. Compressive Strength Test Results

The figure below shows the compressive strength of normal and geopolymer concrete before and after the burning test. Error bars representing the standard deviation are included in the figures to indicate the variability of the test results. The variability is evaluated using the coefficient of variation (COV), which ranges from 4% to 19% in this study.

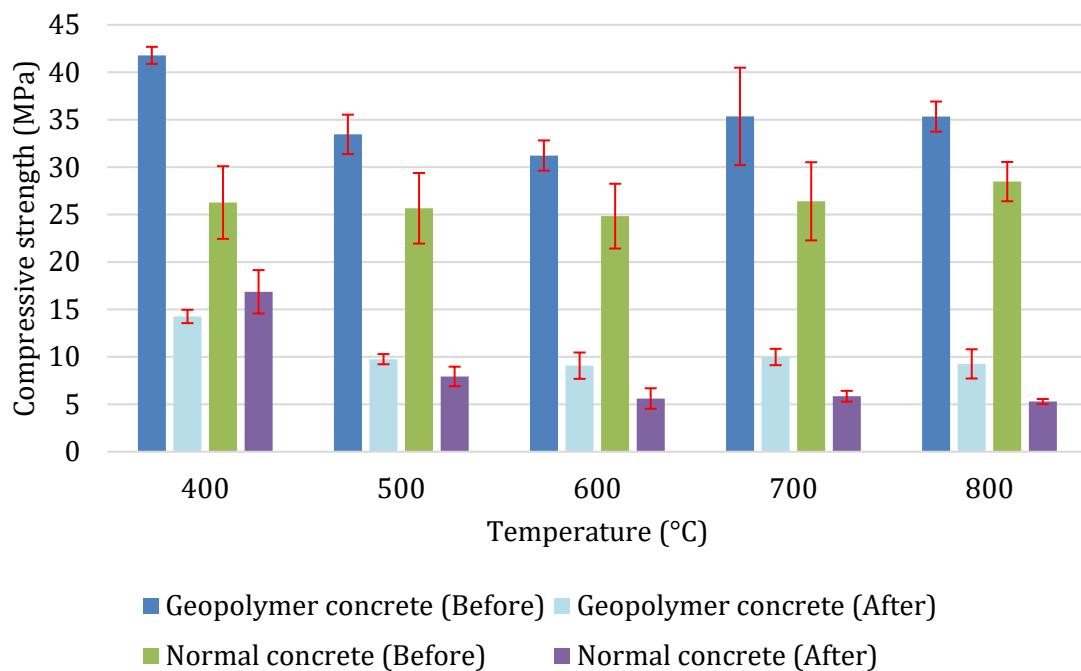


Fig. 18. Comparison of compressive strength in normal and geopolymer concrete prior to and after exposure to elevated temperatures ranging from 400°C to 800°C.

The tables below present the compressive strength test results for both normal and geopolymer concrete samples before and after varying degrees of fire and high-heat exposure. The data above shows that geopolymer concrete exhibited a greater reduction in compressive strength at 400°C than normal concrete. However, at higher temperatures (500-800°C), the reduction in compressive strength of geopolymer concrete was lower than that of normal concrete. The residual strength of normal concrete is 64.18%, 28.78%, 22.58%, 21.87%, and 18.58%, respectively. A similar trend was reported by Tayeh et al. [21], who observed residual strength values of 63.28%, 50.22%, 38.16%, 27.11%, and 18.08% at comparable temperatures [21].

Heikal et al. [18] reported different results. Their study used normal concrete incorporating fly ash. The results indicate that the compressive strength increased up to 400°C but decreased at higher temperatures. This behavior differs from the findings of the present study, in which normal concrete did not exhibit strength gain at 400°C, suggesting that the effect of fly ash on thermal resistance may depend on its proportion and composition [18].

The residual strengths of geopolymer concrete were 34.13%, 29.16%, 29.06%, 28.25%, and 28.67%, respectively. In contrast, Park et al. [19] and Razak et al. [22] observed an increase in compressive strength of geopolymer concrete following exposure to high temperatures [19, 22]. Specifically, Park et al. [19] reported compressive strength increases of 61%, 85%, and 45% at 200°C, 400°C, and 600°C, respectively, followed by a decrease at 800°C, resulting in a residual

strength of 76%. These findings differ from the present study, which did not observe strength gains at comparable temperatures. The increase in strength reported by Park et al. [19] is commonly attributed to continued geopolymerization and densification of the geopolymer matrix at moderate temperatures. In their study, specimens were cured at room temperature for 28 days. In contrast, the specimens in the present study were cured at 115°C, which likely resulted in more complete geopolymerization prior to heating. As a result, further strength gain during subsequent heating was limited, and the specimens primarily experienced thermal degradation.

Table 14. Compressive strength of geopolymer concrete specimens prior to and after being subjected to elevated temperatures ranging from 400°C to 800°C

Temp. (°C)	Average Compressive Strength for 3 Samples (Before)		Average Compressive Strength for 3 Samples (After)		Residual%	Reduction%
	MPa	kN	MPa	kN		
400	41.79	328.01	14.26	111.94	34.13	65.87
500	33.46	262.63	9.76	76.59	29.16	70.84
600	31.22	245.10	9.07	71.23	29.06	70.94
700	35.35	277.50	9.99	78.40	28.25	71.75
800	35.33	277.34	10.13	79.52	28.67	71.33

Table 15. Compressive strength of normal concrete specimens prior to and after being subjected to elevated temperatures of 400°C to 800°C

Temp. (°C)	Average Compressive Strength for 3 Samples (Before)		Average Compressive Strength for 3 Samples (After)		Residual%	Reduction%
	MPa	kN	MPa	kN		
400	26.27	206.22	16.86	132.35	64.18	35.82
500	25.66	201.46	7.39	57.97	28.78	71.22
600	24.83	194.94	5.61	44.01	22.58	77.42
700	26.76	210.03	5.85	45.92	21.87	78.13
800	28.48	223.53	5.29	41.53	18.58	81.42

Comparable trends were reported in studies where geopolymer specimens were cured at elevated temperatures. For instance, Luhar et al. [20] cured specimens at 90°C and observed a decrease in compressive strength upon exposure to high temperatures. The residual compressive strengths reported were 72.62%, 62.96%, 50.87%, and 54.78% at 200°C, 400°C, 600°C, and 800°C, respectively [20]. Similarly, Tayeh et al. [21] found that compressive strength increased at 100°C but decreased at temperatures from 200°C to 800°C [21]. These results align with the present study, which demonstrated a continuous reduction in strength of geopolymer concrete after exposure to elevated temperatures.

Overall, these findings indicate that geopolymer concrete exhibits superior performance at elevated temperatures compared to normal concrete. This enhanced performance is generally attributed to the thermal stability of the aluminosilicate geopolymer binder, which decomposes less at high temperatures than the calcium-based hydration products in Portland cement concrete.

The curing conditions may also influence this difference in behavior. Comparing different curing methods for geopolymer concrete reveals that elevated-temperature curing (high-strength geopolymer) results in greater susceptibility to weight loss, volume instability, dehydration, and cracking upon fire exposure, primarily due to its compact internal structure [36,41]. In contrast, room-temperature curing (low-strength geopolymer) is more effective in mitigating these effects, mainly because the specimens undergo further polymerization and sintering when exposed to high temperatures. Figure 19 illustrates the difference in internal structure between geopolymer concrete cured at elevated temperature and that cured at room temperature.

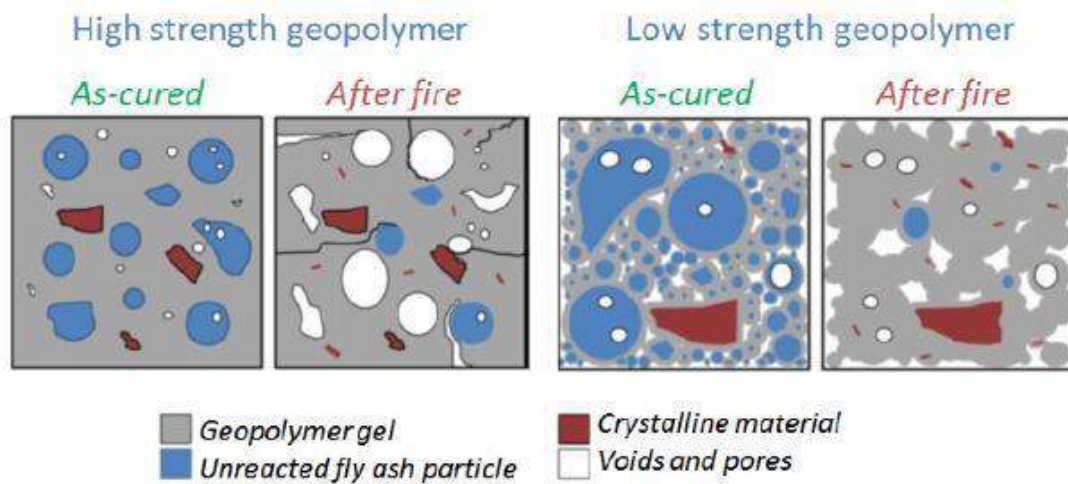


Fig. 19. Comparison of the internal structure of Geopolymer concrete with different curing methods

Due to the results of the study of Tayeh et al. [21], further analysis was conducted to verify the theory of the effect of curing on the reduction in compressive strength after firing. The results of this analysis showed a 34.27% discrepancy between the decrease in oven-cured/elevated high-temperature curing and room-temperature curing. Hence, it is concluded that elevated-temperature curing may increase the susceptibility of geopolymer concrete to strength degradation after fire exposure.

4.4. Scanning Electron Microscope (SEM) Test Results

The results of this scan are used to compare the geopolymer concrete before and after burning at a structural level. Four geopolymer concrete samples are included in this test, each cured using a different method. The first sample will be cured at room temperature. The second sample will be cured using an elevated temperature of 115°C. The third and fourth samples will also be cured at elevated temperatures of 115°C, but will then be burned at 600°C and 800°C, respectively.

The SEM image of geopolymer concrete with room-temperature curing shows spherical particles, which represent either fly ash that has not yet fully reacted or fly ash that has not reacted at all. As explained in the theoretical basis, heat is crucial for activating fly ash and continuing the polymerization reaction [29]. The cracks in the geopolymer concrete are caused by stress during the compressive strength test.

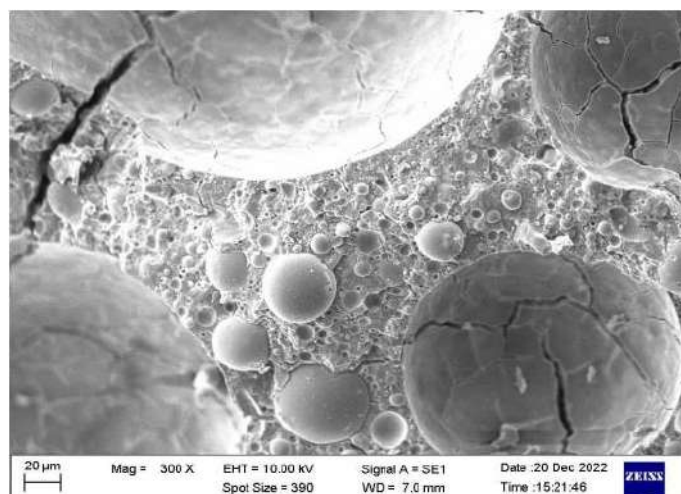


Fig. 20. SEM result of geopolymer concrete at room temperature curing

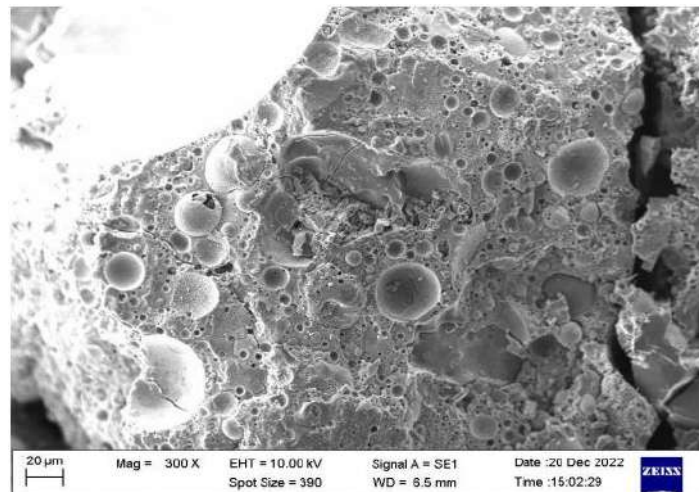


Fig. 21. SEM result of geopolymer concrete at elevated temperature curing of 115°C, showing less unreacted fly ash than room temperature curing specimens

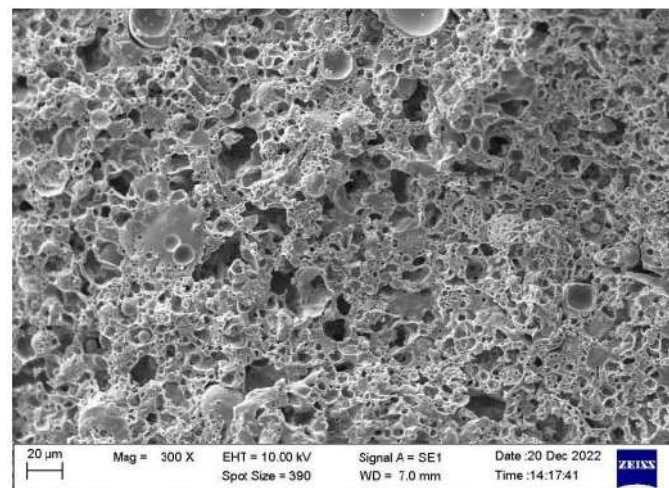


Fig. 22. SEM result of geopolymer concrete with elevated curing and burned at 600°C, showing increased porosity and microstructural changes after high-temperature exposure

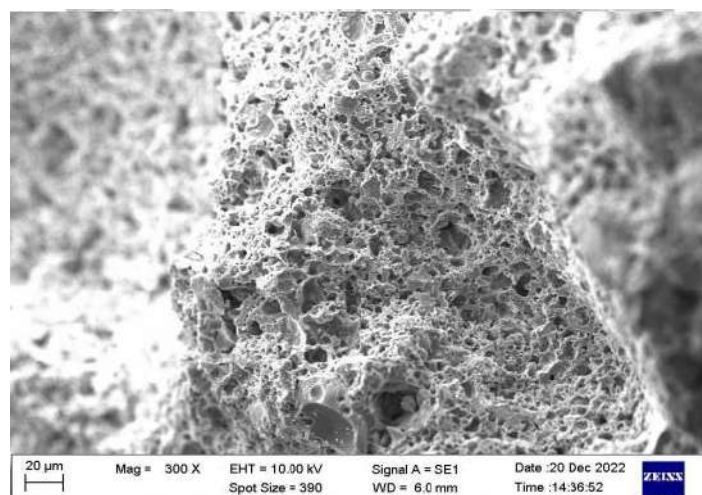


Fig. 23. SEM result of geopolymer concrete with elevated curing and burned at 800°C

Figure 21 shows SEM images of geopolymer concrete cured at 115°C for 24 hours. The structural condition in Figure 21 appears denser and smoother compared to Figure 20. Additionally, the number of unreacted fly ash particles, represented by spherical particles, is significantly reduced

compared to Figure 20. This observation suggests that an appropriate curing temperature, namely 115°C, is sufficient to induce further polymerization reactions in fly ash [31].

Figure 22 shows the SEM of geopolymer concrete after being burned at 600°C with elevated-temperature curing at 115°C. After burning, there was an increase in porosity, pore size, and volume. This increase in porosity may be associated with structural changes occurring during high-temperature exposure, such as crystallization of geopolymer gel [41].

Figure 23 shows the SEM image of geopolymer concrete after burning at 800°C with elevated-temperature curing at 115°C. Compared to the results at 600°C, the pore structure at 800°C is denser [16]. This behavior may be associated with sintering and densification processes occurring in geopolymer materials at elevated temperatures [38]. The permeable nano-pore structure of geopolymer concrete allows free water and water vapor to easily escape, preventing pore pressure buildup that can lead to spalling, unlike in normal concrete.

In conclusion, the curing method's polymerization likely causes fly ash to react, forming a stronger, denser structure that increases compressive strength. Concrete burned at 800°C exhibits a denser structure than that burned at 600°C. This is due to sintering and densification processes occurring at higher temperatures.

5. Conclusions

The analysis demonstrates that fire exposure significantly reduces the performance of geopolymer concrete as a structural material. Specifically, compressive strength decreased by 65.87%, 70.84%, 70.94%, 71.75%, and 71.33% at 400°C, 500°C, 600°C, 700°C, and 800°C, respectively. Fire exposure also alters the physical and mechanical properties of the concrete, resulting in an average weight loss of 5.85%, cracking at 600°C to 700°C, colour changes from dark grey to brown, and observable deformities. SEM results further indicate increased porosity in the concrete following fire exposure.

The curing method also significantly affects the reduction in compressive strength. Elevated-temperature curing influenced the microstructure and thermal behavior of geopolymer concrete during fire exposure, making the samples more susceptible to adverse fire effects.

Compared with normal concrete, geopolymer concrete exhibits superior performance at elevated temperatures. Normal concrete samples experienced an 81.42% reduction in compressive strength at 800°C, whereas geopolymer concrete samples exhibited a 71.33% reduction. Regarding physical and mechanical properties, normal concrete demonstrated a 12.33% weight loss, while geopolymer concrete showed only a 5.85% reduction. Additionally, normal concrete samples displayed more pronounced spalling and cracking. These findings indicate that geopolymer concrete is better suited to applications involving high temperatures and fire exposure than conventional Portland cement concrete.

Temperature increases seemed to have no significant impact on the weight reduction for both varieties. At 400°C and 800°C, the geopolymer variety experiences reductions of 5.26% and 7.2%, respectively, while the normal variety experiences reductions of 11.66% and 13.28%, respectively. On the contrary, temperature increases have different impacts on the reduction of compressive strength amongst the two varieties. The geopolymer concrete does not experience a significant rise in reduction (65.87% at 400°C and 71.33% at 800°C); however, the normal concrete does (35.82% at 400°C and 81.42% at 800°C)

Through this analysis, several further considerations and suggestions can be made for future studies on this topic. Firstly, more studies into curing methods would greatly benefit our understanding of the relationship between the polymerization reaction and the strength of concrete. Aside from that, further analysis is needed to investigate the relationship between the Si/Al ratio and the strength of geopolymer concrete at high temperatures. Additional research is needed on the admixture for geopolymer concrete to determine an optimum mix that enhances heat and fire resistance.

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References

- [1] Volume Concrete. (2025, December). Worldwide cement consumption: An in-depth survey. <https://volumeconcrete.com/worldwide-cement-consumption/>
- [2] Ma CK, Awang AZ, Omar W. Structural and material performance of geopolymer concrete: A review. *Construction and Building Materials*. 2018;186:90-102. <https://doi.org/10.1016/j.conbuildmat.2018.07.111>
- [3] Ozmen HB, Tanriverdi B. Concrete paradox: Economic importance, environmental impacts, and the sustainability of concrete material. *Research & Design* 2025;2(2):66-84. <http://dx.doi.org/10.17515/rede2025-008en0801rs>
- [4] Ezuldin NY, Raoof SM, Alattar AA, Hamada H. Factors affecting durability properties of GPC: A review. *Res. Eng. Struct. Mater.*, 2025; 11(1): 21-43. <http://dx.doi.org/10.17515/resm2024.145ma0109rv>
- [5] Sumajouw MDJ, Rangan BV. Research report GC 3. Low-calcium fly ash-based geopolymer concrete: Reinforced beams and columns, Faculty of Engineering, Curtin University of Technology, Australia, 2006, 11-13.
- [6] Palomo A, Grutzeck MW, Blanco MT. Alkali-activated fly ashes: A cement for the future. *Cement and Concrete Research*. 1999; 29(8):1323-1329. [https://doi.org/10.1016/S0008-8846\(98\)00243-9](https://doi.org/10.1016/S0008-8846(98)00243-9)
- [7] Duxson P, Fernández-Jiménez A, Provis JL, Lukey GC, Palomo A, van Deventer JSJ. Geopolymer technology: the current state of the art. *J Mater Sci*. 2007;42(9):2917-2933. <https://doi.org/10.1007/s10853-006-0637-z>
- [8] Hosan A, Haque S, Shaikh F. Compressive behaviour of sodium and potassium activators synthesised fly ash geopolymer at elevated temperatures: A comparative study. *J Build Eng*. 2016;8:123-130. <https://doi.org/10.1016/j.jobe.2016.10.005>
- [9] Luhar S, Nicolaidis D, Luhar I. Fire resistance behaviour of geopolymer concrete: An overview. *Buildings*. 2021;11(3):82. <https://doi.org/10.3390/buildings11030082>
- [10] Almutairi AL, Tayeh BA, Adesina A, Isleem HF, Zeyad AM. Potential applications of geopolymer concrete in construction: A review. *Case Stud Constr Mater*. 2021;15:e00733. <https://doi.org/10.1016/j.cscm.2021.e00733>
- [11] Lingyu T, Dongpo H, Jianing Z, Hongguang W. Durability of geopolymers and geopolymer concretes: A review. *Rev Adv Mater Sci*. 2021;60(1):1-14. <https://doi.org/10.1515/rams-2021-0002>
- [12] Komnitsas KA. Potential of geopolymer technology towards green buildings and sustainable cities. *Procedia Eng*. 2011;21:1023-1032. <https://doi.org/10.1016/j.proeng.2011.11.2108>
- [13] Hang Y, Zhu M, Fang C. Study on fire resistance of concrete structures. *E3S Web Conf*. 2021;237:01017. <https://doi.org/10.1051/e3sconf/202123701017>
- [14] Amran M, et al. Long-term durability properties of geopolymer concrete: An in-depth review. *Case Stud Constr Mater*. 2021;15:e00661. <https://doi.org/10.1016/j.cscm.2021.e00661>
- [15] Mohd Ali AZ, Sanjayan J, Guerrieri M. Performance of geopolymer high strength concrete wall panels and cylinders when exposed to a hydrocarbon fire. *Constr Build Mater*. 2017;137:195-207. <https://doi.org/10.1016/j.conbuildmat.2017.01.099>
- [16] Sarker PK, Kelly S, Yao Z. Effect of fire exposure on cracking, spalling, and residual strength of fly ash geopolymer concrete. *Mater Des*. 2014;63:584-592. <https://doi.org/10.1016/j.matdes.2014.06.059>
- [17] Zhao R, Sanjayan JG. Geopolymer and Portland cement concretes in simulated fire. *Mag Concr Res*. 2011;63(3):163-173. <https://doi.org/10.1680/mac.9.00110>
- [18] Heikal M, El-Didamony H, Sokkary TM, Ahmed IA. Behavior of composite cement pastes containing microsilica and fly ash at elevated temperature. *Construction and Building Materials*. 2013;38:1180-1190. <http://dx.doi.org/10.1016/j.conbuildmat.2012.09.069>
- [19] Park SM, Jang JG, Lee NK, Lee HK. Physicochemical properties of binder gel in alkali-activated fly ash/slag exposed to high temperatures. *Cement and Concrete Research*. 2016;89:72-79. <http://dx.doi.org/10.1016/j.cemconres.2016.08.004>
- [20] Luhar S, Chaudhary S, Luhar I. Thermal resistance of fly ash based rubberized geopolymer concrete. *Journal of Building Engineering*. 2018;19:420-428. <https://doi.org/10.1016/j.jobe.2018.05.025>
- [21] Tayeh BA, Zeyad AM, Agwa IS, Amin M. Effect of elevated temperatures on mechanical properties of lightweight geopolymer concrete. *Case Studies in Construction Materials*. 2021;15:1-21. <https://doi.org/10.1016/j.cscm.2021.e00673>

- [22] Abd Razak SN, Shafiq N, Hasan Nikbakht E, Mohammed BS, Guillaumat L, Farhan SA. Fire performance of fly-ash-based geopolymer concrete: Effect of burning temperature on mechanical and microstructural properties. *Mater Today Proc.* 2022;66:2665-2669. <https://doi.org/10.1016/j.matpr.2022.06.491>
- [23] Widjajakusuma J, Bali I, Ng GP, Wibowo KA. An experimental study on the mechanical properties of low-aluminum and rich-iron-calcium fly ash-based geopolymer concrete. *Adv Technol Innov.* 2022;7(4):295-302. <https://doi.org/10.46604/aiti.2022.10525>
- [24] ASTM International. Standard test method for density, relative density (specific gravity), and absorption of coarse aggregate (ASTM C127). West Conshohocken (PA): ASTM International; 2001.
- [25] Standar Nasional Indonesia (Indonesian National Standards). (Standard Practice for Selecting Proportions for Normal, Haevyweight, and Mass Concrete) (SNI 7657:2012). Jakarta: Badan Standardisasi Nasional (National Standardization Agency); 2012.
- [26] Standar Nasional Indonesia (Indonesian National Standards). Semen Portland Komposit (Composite Portland Cement) (SNI 7064:2014). Jakarta: Badan Standardisasi Nasional (National Standardization Agency); 2014.
- [27] ASTM International. Standard specification for coal fly ash and raw or calcined natural pozzolan for use in concrete (ASTM C618). West Conshohocken (PA): ASTM International; 2022. <https://doi.org/10.1520/C0618-22>
- [28] Chindaprasirt P, Chareerat T, Sirivivatnanon V. Workability and strength of coarse high calcium fly ash geopolymer. *Cem Concr Compos.* 2007;29(3):224-229. <https://doi.org/10.1016/j.cemconcomp.2006.11.002>
- [29] Hardjito D, Rangan BV. Development and properties of low-calcium fly ash-based geopolymer concrete. Perth: Curtin University of Technology; 2005.
- [30] Al Bakria AMM, Kamarudin H, BinHussain M, Nizar IK, Zarina Y, Rafiza AR. The effect of curing temperature on physical and chemical properties of geopolymers. *Phys Procedia.* 2011;22:286-291. <https://doi.org/10.1016/j.phpro.2011.11.045>
- [31] Atiş CD, Görür EB, Karahan O, Bilim C, Ilkentapar S, Luga E. Very high strength (120 MPa) class F fly ash geopolymer mortar activated at different NaOH amount, heat curing temperature and heat curing duration. *Constr Build Mater.* 2015;96:673-678. <https://doi.org/10.1016/j.conbuildmat.2015.08.089>
- [32] Tanash AO, Abu Bakar BH, Muthusamy K, Al Biajawi MI. Effect of elevated temperature on mechanical properties of normal strength concrete: An overview. *Mater Today Proc.* 2024;107:152-157. <https://doi.org/10.1016/j.matpr.2023.09.053>
- [33] ASTM International. Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate (ASTM C128-25). West Conshohocken (PA): ASTM International; 2025. DOI: 10.1520/C0128-22.
- [34] ASTM International. Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates (ASTM C136M/C136M-19). West Conshohocken (PA): ASTM International; 2019. DOI: 10.1520/C0136_C0136M-19
- [35] He F, Biolzi L, Carvelli V, Li G. Cracking stress thresholds of thermally damaged hybrid fiber reinforced concrete under compression. *Constr Build Mater.* 2025;494:143352. <https://doi.org/10.1016/j.conbuildmat.2025.143352>
- [36] Khoury GA. Effect of fire on concrete and concrete structures. *Prog Struct Eng Mater.* 2001;2(4):429-447. <https://doi.org/10.1002/pse.51>
- [37] Al-Jaberi L, Al-Saraj ASKW, Al-Serai SJ. Scanning electron microscopy of metakaolin based geopolymer concrete. *J Phys Conf Ser.* 2021;2114:012061. <https://doi.org/10.1088/1742-6596/2114/1/012061>
- [38] Hager I, Sitarz M, Mróz K. Fly-ash based geopolymer mortar for high-temperature application – effect of slag addition. *J Clean Prod.* 2021;316:128168. <https://doi.org/10.1016/j.jclepro.2021.128168>
- [39] ASTM International. Standard Test Methods for Fire Tests of Building Construction and Materials (ASTM E119-26). West Conshohocken (PA): ASTM International; 2026. DOI: 10.1520/E0119-26
- [40] Standar Nasional Indonesia (Indonesian National Standards). Cara Uji Kuat Tekan Beton dengan Benda Uji Silinder (Standard Test Methods for Compressive Strength of Cylindrical Concrete) (SNI 1974:2011). Jakarta: Badan Standardisasi Nasional (National Standardization Agency); 2011.
- [41] Bakharev T. Thermal behaviour of geopolymers prepared using class F fly ash and elevated temperature curing. *Cem Concr Res.* 2006;36(6):1134-1147. <https://doi.org/10.1016/j.cemconres.2006.03.022>

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Impact of preprocessing in UAV images on road surface damage detection

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Abstract

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The global deterioration of transportation infrastructure necessitates automated, high-precision pavement monitoring systems. While Unmanned Aerial Vehicles (UAVs) provide unparalleled spatial flexibility for infrastructure inspection, aerial photogrammetry suffers from severe environmental degradations, including motion blur, specular glare, and deep structural shadows. This research investigates the quantitative impact of spatial and photogrammetric preprocessing techniques on the detection accuracy of deep learning architectures. Evaluations conducted on global benchmarks, notably the RDD2022 and UAV-PDD2023 datasets, reveal that algorithmic input optimization directly dictates neural network performance. Specifically, applying Contrast Limited Adaptive Histogram Equalization (CLAHE) enhanced fine crack detection accuracy from 88% to 95% by improving local feature visibility without amplifying global noise. High-frequency amplification via Unsharp Masking drove a profound 12.77% improvement in Mean Average Precision (mAP) for Faster R-CNN models, effectively preventing the loss of hairline features during down-sampling. Furthermore, illumination-invariant Retinex transformations increased information entropy by 63% in low-light environments, enabling a 97.5% overall recognition accuracy. Finally, the shift toward implicit preprocessing mechanisms in single-stage YOLO variants, achieved through integrated attention modules, reduced computational parameters by 88% while maintaining a robust mAP@0.5 of 63.2%. This study proves that the systematic integration of mathematical image filters with convolutional feature extraction is a fundamental requirement for achieving reliable, autonomous aerial infrastructure assessment. By bridging the gap between raw optical data and high-fidelity feature extraction, this research establishes a robust framework for real-time, weather-invariant roadway condition assessment in dynamic operational environments.

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1. Introduction

Transportation networks form the backbone of national economic stability and public mobility. Pavement distresses—ranging from superficial linear cracking to severe structural potholes and surface raveling—directly compromise traffic safety and accelerate the structural degradation of the infrastructure matrix if left unaddressed [1, 2]. Historically, road condition assessments and performance-verification tasks have relied almost entirely on manual visual inspections or specialized multi-function ground vehicles equipped with surface sensors. However, these traditional paradigms carry substantial operational costs, require intensive manual labor, and introduce human subjectivity into condition reporting, alongside exposing inspectors to active

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traffic hazards. Moreover, ground-based vehicles often struggle in congested, complex, or mountainous topographies, heavily limiting their spatial scalability. Consequently, the intelligent transportation systems face a critical need for scalable, highly accurate, and cost-efficient automated pavement assessment methodologies [3].

1.1. Transition from Ground Vehicles to Unmanned Aerial Vehicles

To address the inherent limitations of ground-based and manual surveys, the integration of Unmanned Aerial Vehicles (UAVs) has emerged as a transformative modality for infrastructure health monitoring. UAV platforms equipped with high-resolution optical sensors offer significant advantages, including exceptional spatial flexibility, rapid deployment capabilities, and the capacity to capture wide-area topographical data without interrupting active traffic flows [4]. Aerial photogrammetry enables the rapid generation of detailed two-dimensional orthomosaics and three-dimensional surface models, facilitating a more comprehensive spatial evaluation of the pavement network. Concurrently, the rapid evolution of deep learning architectures—specifically Convolutional Neural Networks (CNNs), the You Only Look Once (YOLO) series, and emerging Vision Transformers—has provided a robust computational framework for automated object detection and semantic segmentation. Extensive research demonstrates that coupling UAV-acquired imagery with deep learning algorithms substantially outperforms traditional ground-based approaches in both operational efficiency and detection robustness, establishing aerial inspection as a highly viable alternative for modern infrastructure management [2, 5].

1.2. Current Literature Gaps and Trends

Despite promising advancements in UAV-based road damage detection, the shift from controlled ground imaging to dynamic aerial acquisition introduces distinct optical and computational challenges. UAV-captured imagery is particularly vulnerable to severe environmental degradation; factors such as motion blur from high-frequency rotor vibration, extreme illumination variance from dynamic solar angles (glare), and deep structural shadows cast by urban architecture or overhanging foliage severely distort visual data [6]. These optical artifacts compress the dynamic range of image tensors, obscure critical high-frequency features like hairline cracks, and elevate the false positive rates of downstream algorithms by producing background noise that visually mimics genuine structural defects.

A notable gap in the current literature is the absence of systematic, quantitative analyses that isolate the impact of explicit image preprocessing techniques on the detection metrics of deep learning architectures. While contemporary studies heavily emphasize the architectural "lightweighting" of models to satisfy the strict edge-computing constraints of UAVs [3, 6], the foundational role of photogrammetric and spatial preprocessing—such as Contrast Limited Adaptive Histogram Equalization (CLAHE), Bilateral Filtering, Retinex illumination correction, and Unsharp Masking—remains underexplored in comparative ablation studies. To address this critical gap, the present research conducts a systematic, quantitative investigation into the efficacy of photogrammetric and spatial preprocessing methodologies. This study explicitly evaluates how tailored algorithms mathematically optimize the input data space to mitigate complex environmental degradations, thereby driving tangible and empirically verified improvements in the precision and recall metrics of UAV-deployed road damage detection networks [4].

2. Deep Learning Architectures for Road Damage Detection

The translation of raw, high-resolution Unmanned Aerial Vehicle (UAV) optical data into actionable infrastructure metrics relies fundamentally on the underlying computer vision architecture. Modern automated road inspection systems have largely abandoned classical digital image processing methods—such as standalone thresholding and manual edge detection—in favor of deep learning networks capable of hierarchical and autonomous feature extraction [2]. Currently, the architectural landscape for pavement monitoring is dominated by three primary paradigms: single-stage detectors, two-stage region proposal networks, and emerging hybrid models incorporating Vision Transformers (ViTs). The choice of architecture dictates the critical trade-off

between mean Average Precision (mAP), parameter count, and the feasibility of real-time inference on edge-computing devices onboard the UAV [3].

2.1. Single Stage Detectors and the YOLO Series

The You Only Look Once (YOLO) family of single-stage detectors (ranging from YOLOv5 to the recent YOLOv8) constitutes the foundational architecture for the majority of real-time UAV pavement monitoring systems [7]. YOLO models frame object detection as a single regression problem, simultaneously predicting bounding box coordinates and class probabilities directly from full images. This unified approach facilitates exceptional inference speeds—reportedly exceeding 60 to 100 frames per second (FPS) under optimal conditions—making them highly suitable for the constrained computational payloads and battery limitations of UAVs [2].

Recent advancements in the YOLO series focus heavily on structural "lightweighting" and attention-driven feature extraction to compensate for the complex backgrounds of aerial images. As illustrated in Figure 1, the emerging implicit lightweighting paradigm effectively integrates spatial and photogrammetric filters (such as Ghost Convolutions and Generalized Feature Pyramid Networks) directly into the convolutional architecture. This integration allows the network to bypass computationally heavy external preprocessing on edge devices while maintaining robust feature extraction.

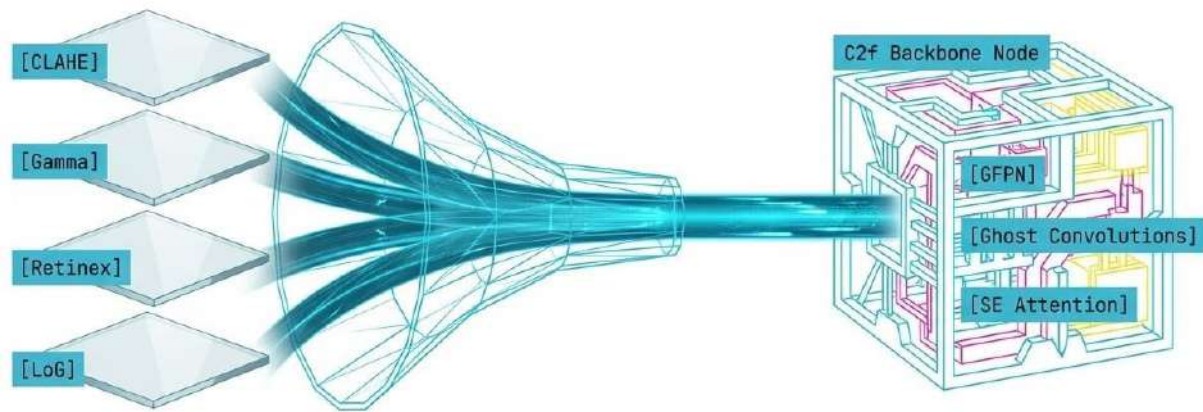


Fig. 1. The implicit lightweighting paradigm in single-stage detectors

For instance, Chen, Wu [6] proposed the ALC-Net, a highly optimized derivative that integrates Ghost convolutions and Squeeze-and-Excitation (SE) attention mechanisms to drastically reduce the parameter count while preserving vital channel-wise feature dependencies. Furthermore, ALC-Net utilizes a dedicated "Focus module" for down-sampling and a Coordinate Attention (CA) mechanism to aggregate horizontal and vertical spatial data. This aggregation is mathematically vital for detecting narrow, directional defects such as longitudinal cracks in high-resolution UAV imagery without being distracted by linear road markings [6].

Similarly, the CSGEH-YOLO architecture, built upon the YOLOv8s baseline, incorporates a Generalized Feature Pyramid Network (GFPN) for cross-scale feature fusion and introduces the Star operation from StarNet to map complex, nonlinear feature spaces [3]. This optimization demonstrated an approximate 3.1% improvement in mAP@0.5 on UAV remote sensing datasets while reducing the computational burden to roughly 78% of the baseline model [3]. Other notable variants, such as YOLO-ERCD, deliberately embed Convolutional Block Attention Modules (CBAM) and dynamic gamma correction directly into the network pipeline to natively suppress complex background noise and illumination variance, suggesting that single-stage detectors can achieve high precision without sacrificing edge-deployment speeds [7].

2.2. Region Proposal Networks and Faster RCNN

While single-stage models prioritize speed, two-stage detectors—most notably Faster R-CNN and its Mask R-CNN derivatives—are leveraged when absolute localization precision and pixel-level instance segmentation are the paramount objectives [7]. Faster R-CNN operates via a dual-phase

mechanism: a Region Proposal Network (RPN) first hypothesizes potential object bounding boxes based on dense "objectness" scores, followed by a separate convolutional network that extracts high-level features from these specific spatial regions to classify the damage and refine the bounding box coordinates.

In comparative evaluations utilizing UAV-acquired crack datasets, Faster R-CNN frequently observes higher absolute accuracy, precision, and recall metrics than standard YOLO variants. This makes it highly appropriate for offline, high-fidelity pavement distress evaluation where data is processed post-flight at ground workstations [7]. However, the intrinsic mathematical complexity of the RPN generates a substantial computational bottleneck, rendering standard Faster R-CNN architectures largely incompatible with real-time, onboard UAV processing.

To bridge this operational gap, recent literature has explored aggressive lightweight modifications to two-stage networks. For example, Magdy, Moustafa [8] proposed a novel bi-stage compression approach to create a lightweight Faster R-CNN specifically tailored for optical remote sensing imagery. Rather than relying solely on architectural structural changes, their methodology leverages mixed-precision (FP16) aware training combined with post-training unstructured weight pruning and dynamic quantization. This optimization reportedly yielded an approximate 56.6% reduction in total parameters and a 25.6% decrease in model size, successfully mitigating computational loads while rigorously preserving the model's baseline mean Average Precision (mAP) [8].

Furthermore, when standard two-stage architectures are coupled with explicit image preprocessing steps—such as unsharp masking or Laplacian of Gaussian (LoG) filters that artificially steepen the edge gradients of ultra-fine cracks—the anchor generation efficiency of the RPN layer is significantly enhanced. As illustrated in Figure 2, foundational analyses indicate that this targeted spatial amplification isolates high-frequency structural signals from optical background noise, providing the RPN with the necessary gradient slopes for precise bounding box regression.

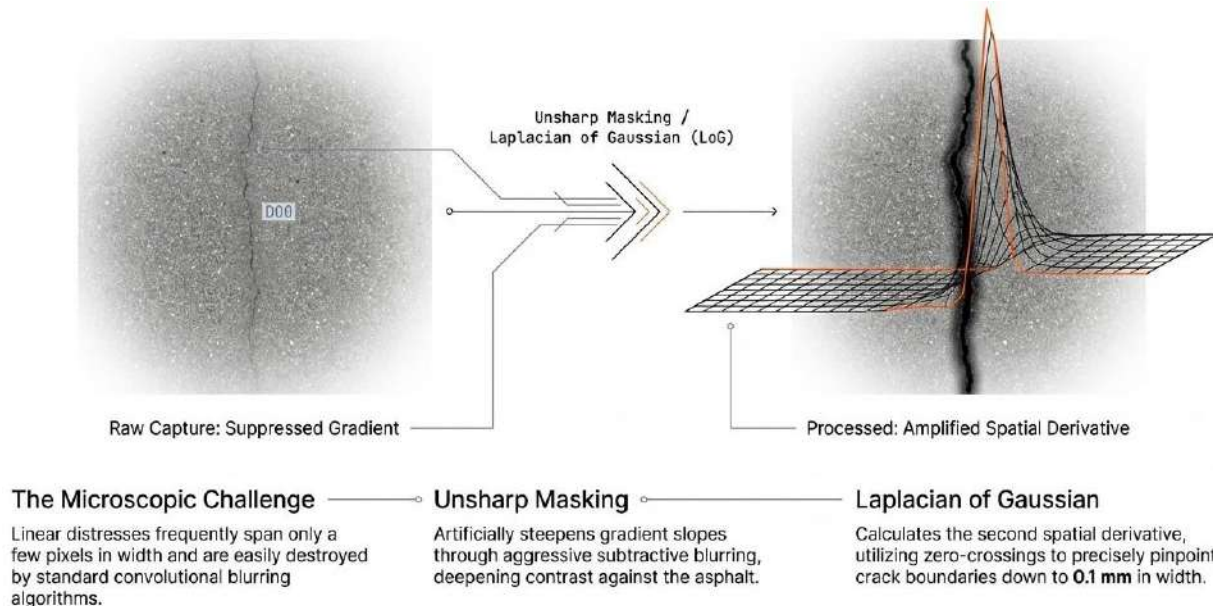


Fig. 2. Spatial amplification via Laplacian of Gaussian (LoG) and Unsharp Masking

Collectively, such findings demonstrate that two-stage detectors can be mathematically compressed and optically optimized to improve edge-computing efficiency without discarding their inherent precision advantages.

2.3 Vision Transformers and Hybrid Global Context Models

The most recent paradigm shift in UAV road damage detection is the integration of Vision Transformers (ViTs) and hybrid attention networks. Traditional CNNs are fundamentally constrained by the localized receptive field of their convolutional kernels, which restricts their

ability to model long-range, global dependencies across the entire pavement matrix [9]. Vision Transformers bypass this limitation by employing self-attention mechanisms that calculate the mathematical relationships between all pixel patches globally, making them exceptionally adept at understanding macroscopic distress patterns like expansive alligator cracking and large-scale structural deformations.

Because pure Transformer models (such as the standard Swin Transformer) often require massive parameter counts that strictly prohibit UAV edge deployment, recent studies have explored alternative optimization paradigms. For instance, Peng [10] demonstrated that joint training frameworks utilizing image generators (e.g., GANs) can computationally synthesize extreme lighting anomalies and structural noise during training. This generative augmentation allows the detector to outperform massive Transformer models while utilizing significantly fewer parameters [10]. Concurrently, to bypass the parameter bottleneck of pure ViTs without relying solely on data synthesis, researchers have rapidly pivoted toward hybrid CNN-Transformer architectures. As conceptually illustrated in Figure 3, the future of infrastructure monitoring relies on this necessary synthesis: harmonizing generative deburring, dynamic retinex correction, and ultra-lightweight attention-driven architectures. These hybrid topologies allow the network to utilize CNNs for efficient, localized gradient extraction while leveraging the Transformer block to mathematically grasp the global structural context of the road surface.

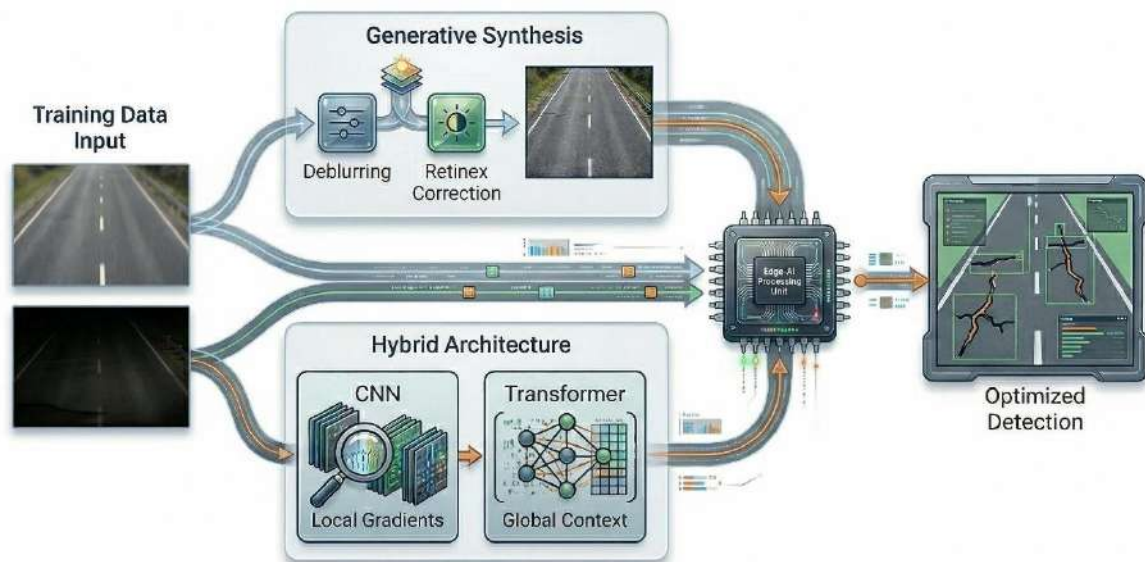


Fig. 3. The architectural synthesis of hybrid models for weather-invariant inspection

A prime example is the OBC-YOLOv8 model, which appends a Bottleneck Transformer (BoTNet) to the terminus of the YOLOv8 backbone [9]. This specific structural fusion is crucial for identifying fine crack edges locally while understanding the road surface globally, resulting in an observed 1.8% increase in mAP@0.5 and a 1.6% increase in the F1-score on standard multi-national road damage datasets [9].

Similarly, advanced architectures that fuse the high-precision instance segmentation capabilities of Mask R-CNN with the global contextual awareness of Vision Transformer models have achieved state-of-the-art segmentation metrics on complex, noisy crack datasets [11]. Furthermore, embedding Transformer Prediction Heads directly into models like YOLOv7 has been shown to elevate mAP@0.5 scores beyond 73%, indicating that hybridizing local convolutions with global self-attention is a highly effective strategy for managing the complex, noisy visual environments encountered during aerial UAV inspections [4].

3. Quantitative Impact of Image Preprocessing Techniques

The performance of deep learning object detectors is bound by the input optical quality. Advanced architectures like YOLO and Faster R-CNN require high-resolution and noise-free tensors. However, aerial images often suffer from sensor noise and varying illumination. Consequently, these factors frequently degrade the feature extraction capabilities of neural networks [2]. Preprocessing algorithms are utilized to optimize the feature space before convolution occurs. In this context, advanced optimization approaches, such as the Weighted Differential Evolution Algorithm, have shown significant potential in automatically enhancing UAV aerial captures to mitigate these environmental degradations [12]. The following subsections detail the quantitative impact of spatial and photogrammetric filters. These filters influence critical detection metrics such as mean Average Precision (mAP) and recall. Table 1 provides a systematic comparison of these preprocessing strategies. It outlines their operational advantages and specific limitations based on recent literature.

Table 1. Comparative analysis of verified preprocessing filters

Filter Type	Primary Mechanism	Optimal Target	Literature Benchmark
CLAHE	Localized histogram redistribution.	Low-contrast cracks	Freitas et al. (2025) [14]
Bilateral Filter	Edge-preserving spatial smoothing.	3D Denoising	Zhang et al. (2019) [15]
Adaptive Gamma	Non-linear luminance scaling.	Sun glare/Shadows	X. Chen et al. (2024) [7]
Multi-Scale Retinex	Reflectance-based normalization.	Low-light tunnels	Sun et al. (2025) [18]
Unsharp Masking	High-frequency edge amplification.	Hairline micro-cracks	Magdy et al. (2025) [8]

3.1 Contrast-Limited Adaptive Histogram Equalization

Contrast Limited Adaptive Histogram Equalization (CLAHE) is a highly effective photogrammetric transformation. It is designed to improve localized image contrast without artificially amplifying high-frequency background noise. Global histogram equalization redistributes pixel intensities across the entire image. Consequently, this global approach often washes out subtle crack gradients. To resolve this, CLAHE partitions the image into smaller contextual tiles. It applies equalization locally and enforces a strict clipping limit on the histogram bins. Any excess intensity is uniformly redistributed to prevent noise over-amplification [13].

The integration of CLAHE has demonstrated profound quantitative impacts on overall detection accuracy. Recent ablation studies have utilized UAV aerial image datasets to evaluate this methodology. Applying CLAHE alongside HSV color space transformations allowed YOLOv8s algorithms to suppress aggregate noise. Simultaneously, this specific configuration successfully highlighted critical structural features. This targeted preprocessing pipeline reportedly improved object detection accuracy from 88% to approximately 95% [13]. Despite these robust peak performances, the efficacy of CLAHE depends heavily on the target distress. Comparative analyses evaluated CLAHE against other low-light enhancement techniques, such as the LIME method. In these studies, standard CLAHE sometimes produced the lowest True Positive rates for complex macroscopic damages like potholes. This indicates that localized contrast limits require meticulous algorithmic tuning. Specifically, these parameters must mathematically match the spatial frequency of the targeted road defect [14].

3.2 Edge Preserving Smoothing via Bilateral Filtering

Asphalt and concrete pavements possess inherently coarse, granular micro-textures. To high-resolution UAV optical sensors, these textures manifest as intense spatial noise. Consequently, this noise severely disrupts the edge-detection capabilities of early convolutional layers. Traditional Gaussian smoothing uniformly blurs all high-frequency data. Unfortunately, this process destroys

the fine boundaries of linear cracks. Bilateral filtering resolves this by acting as a non-linear, edge-preserving smoother [15]. It utilizes two distinct Gaussian kernels for pixel weighting. The spatial kernel weights pixels based on physical proximity. The range kernel weights pixels based on photogrammetric similarity. When traversing a sharp structural boundary, the intensity kernel drastically reduces the smoothing weight. This preserves the hard mathematical gradient while aggressively blurring the surrounding pavement texture. Integrating bilateral filtering prior to feature extraction effectively regularizes the road matrix. Multi-stage detection systems frequently deploy these filters after initial convolutional assessments. This smooths isolated regions and suppresses false positive predictions triggered by pavement aggregates [16]. Providing a mathematically cleaner tensor benefits downstream bounding box regression algorithms. As a result, the network anchors onto authentic distress geometries, directly enhancing localization metrics.

3.3 Channel-Wise Adaptive Gamma Correction

UAVs operating in dynamic outdoor environments inevitably encounter severe solar irradiance fluctuations. This results in captures suffering from specular overexposure or severe underexposure. Gamma correction addresses this by applying a non-linear transformation to pixel luminance. This mathematically expands shadowed regions or compresses saturated regions. Recent literature has advanced this concept into Channel-wise Adaptive Gamma Correction (CAGC). The YOLO-ERCD framework was developed to improve detection robustness under complex lighting. It embeds a CAGC module that introduces stochastic deviations during the training phase [17]. Rather than applying a static coefficient, CAGC perturbs base values for different color channels independently. This forces the neural network to adapt to real-world ambient light fluctuations. Furthermore, it effectively models the non-linear response of human visual perception [17]. Quantitative assessments demonstrate that dynamic gamma modulation substantially reduces false negative rates. Consequently, it provides a highly resilient feature extraction process across varying diurnal cycles.

3.4 Illumination Normalization using Multi-Scale Retinex

Retinex theory is derived from the human visual system's ability to maintain color constancy. It postulates that an image is the product of ambient illumination and structural reflectance. UAVs frequently photograph roads obscured by tree canopies or urban structures. In these scenarios, separating temporary shadows from actual asphalt damage is critical. The Multi-Scale Retinex (MSR) algorithm utilizes Gaussian center-surround functions to estimate the illumination map. Dividing out this map yields an illumination-invariant representation of the road surface [18]. The quantitative benefits of MSR for low-light crack segmentation are exceptional. The CrackNex framework utilized Retinex-based extraction for robust segmentation on low-light datasets. This pushed the mean Intersection over Union (mIOU) to approximately 68.82 under specific learning configurations [19]. In UAV applications targeting profoundly dark environments, MSR is integrated with advanced segmentation algorithms. This combination reportedly elevated the information entropy of processed images by 63%. Consequently, the overall crack recognition accuracy reached up to 97.5% [18]. By mathematically neutralizing shadows early, MSR prevents networks from misclassifying shadow boundaries as longitudinal distresses.

3.5 High Frequency Amplification with Unsharp Masking

Detecting hairline cracks from high-altitude UAV imagery is severely constrained by low pixel density. Unsharp Masking artificially steepens the gradient transitions of these micro-cracks. The algorithm first applies a low-pass blur to the raw image. It subtracts this blurred version to isolate high-frequency edge signals. Finally, it scales and adds these isolated edges back to the original tensor. Applying Unsharp Masking has yielded notable mAP improvements in recent architectural evaluations. Explicit integration of unsharp masking enhances edge definition prior to region proposal networks. In targeted lightweight architectures, this spatial amplification drove reported surges of up to 12.77% in mAP [8]. Similarly, hybrid frameworks demonstrate that combining high-frequency amplification with brightness redistribution optimizes lightweight models. Evaluations on complex aerial datasets yielded improvements of approximately 5.6% in mAP@50 [20]. These

findings confirm a critical principle for autonomous infrastructure monitoring. For ultra-fine linear distresses, artificial edge amplification is mathematically superior to raw captures.

4. Enhancing Model Robustness Under Environmental Degradation

The transition to UAVs introduces highly volatile environmental variables into data acquisition. Deep learning models learn strict spatial and pixel-intensity representations during training. Consequently, these networks are notoriously brittle when deployed in environments deviating from original distributions [14]. Aerial photogrammetry is consistently subjected to severe optical degradations. These include dynamic solar angles, deep structural shadows, and vibration-induced motion blur. Furthermore, physical meteorological occlusions, such as snow and ice cover, drastically alter the spectral signature of the target surface. While traditional multi-spectral index methods are highly effective for general environmental mapping [21], recent literature explicitly confirms that robust deep learning frameworks, specifically CNNs, are definitively required to extract complex features amidst such severe topographical noise [22]. Preprocessing serves as a critical mathematical bridge in these challenging scenarios. It maps erratic UAV captures back into a normalized statistical distribution for reliable neural processing. Figure 4 demonstrates this physical distortion, illustrating how deep shadows, specular glare, and flight motion blur severely compromise the raw optical input.

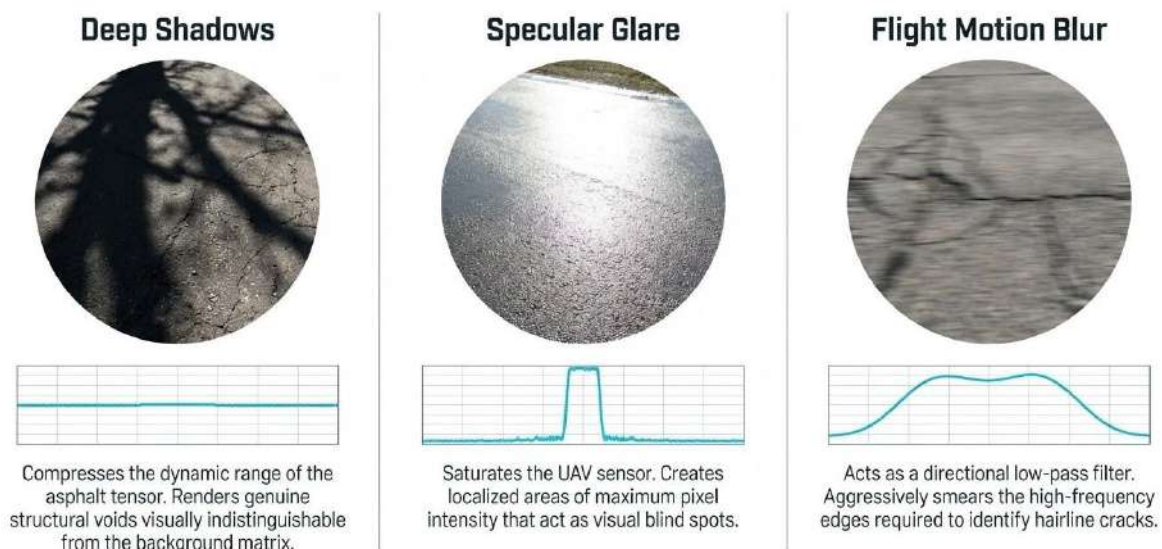


Fig. 4. The profound impact of severe environmental degradations on UAV optical captures

4.1 Strategies for Deep Shadows and Low Light Conditions

In urban and forested environments, road surfaces are punctuated by sharp, high-contrast shadows. These shadows are frequently cast by buildings, overpasses, and dense vegetation. For CNN, dark linear shadows can mimic severe longitudinal cracks. Conversely, deep shadows severely compress the dynamic range of the underlying asphalt. This renders genuine structural voids visually indistinguishable from the surrounding background matrix [23]. Consequently, this dynamic range compression suppresses the activation maps of early convolutional layers. This suppression leads to common false negatives and misclassifications during instance segmentation. Models must integrate illumination-invariant preprocessing frameworks to ensure robust detection. This prevents the need for exhaustive re-training on artificially shadowed datasets. The Low-Light Image Enhancement (LIME) algorithm demonstrates exceptional efficacy in these specific scenarios. Recent ablation studies compared LIME against unprocessed images and standard histogram equalizations. LIME-processed datasets reportedly achieved an approximate Mean Average Precision (mAP) of 78.8% [14]. Furthermore, this method achieved a precision metric of 91.1% by reclaiming hidden structural contrast. Additionally, specialized spatial strategies such as image tiling have proven highly effective. This technique subdivides high-resolution captures into smaller contextual grids before applying localized enhancements. Image

tiling prevents global illumination algorithms from being mathematically skewed by massive dark regions. Consequently, both illuminated and shadowed distress features are preserved simultaneously during extraction [23].

4.2 Mitigating Sun Glare and Specular Overexposure

Specular reflection, commonly known as sun glare, presents a direct inverse challenge to deep shadows. This glare frequently occurs on wet asphalt, recently sealed roads, or highly reflective lane markings. Consequently, these intense reflections completely saturate the CMOS sensor of the UAV camera [24]. The resulting localized areas of maximum pixel intensity effectively obliterate any underlying structural data. This saturation generates a severe visual blind spot for the deep learning feature extraction model. Furthermore, extreme gradient shifts at the periphery of glare artifacts frequently trigger false positive detections. These false edges ultimately confuse the bounding box regression heads of standard detection architectures.

Robustness under glare depends heavily on non-linear luminance adjustments and advanced generative reconstructions. Recent architectures, such as the YOLO-ERCD model, specifically address these extreme lighting conditions. This model deploys a Channel-wise Adaptive Gamma Correction (CAGC) module to counteract severe solar irradiance [17]. Rather than applying a static coefficient, CAGC introduces stochastic deviations during the training phase. This dynamic approach effectively models the non-linear response of human perception to sudden light shifts [17]. Additionally, hierarchical auto-encoder networks can mathematically reconstruct underlying road topologies even when input pixels are saturated. By compressing the saturated image into a latent space, these lightweight architectures filter out extreme glare noise. They then decode this latent representation to reveal the road's true structural geometry [25]. These combined techniques prevent the activation functions of the primary detector from prematurely saturating. Consequently, they maintain the continuous gradient flow necessary for identifying defects traversing through reflective patches.

4.3 Correcting UAV Flight Dynamics and Motion Blur

Vehicle-mounted sensors typically maintain a stable planar relationship with the road surface. Conversely, UAVs are subjected to turbulent aerodynamic forces, multi-axis translations, and varying flight speeds. These dynamics induce varying degrees of motion blur within the captured imagery [26]. Mathematically, motion blur acts as a directional low-pass filter. This filter aggressively smears high-frequency edges across multiple contiguous pixels. Certain distresses, such as hairline cracks or pothole edges, are defined by sharp boundaries. Motion blur destroys the discriminative features required by object detectors to identify these structures. Consequently, this degradation directly precipitates a collapse in the overall recall rate [23].

Mitigating motion blur requires a combination of strict operational parameters and advanced computational restoration. Operationally, researchers emphasize controlling camera exposure and optimizing UAV flight speed. For example, maintaining a velocity of 5 m/s at an altitude of 50 m is often recommended. This specifically limits physical sensor smear during the raw data acquisition process [23]. Computationally, traditional linear sharpening filters often fail to recover severely smeared geometries. Consequently, robust contemporary approaches increasingly rely on Generative Adversarial Networks (GANs). These networks are deployed as a preprocessing deblurring layer prior to the primary object detector [26]. A GAN-based motion deblur network leverages a generator to computationally reconstruct smeared edges. This reconstruction is based on the learned statistical distribution of sharp pavement textures. Simultaneously, a discriminator network strictly enforces visual realism on the generated outputs [26]. These generative models explicitly reverse blur degradation before feature extraction occurs. Therefore, they directly restore the mAP metrics that are otherwise lost to flight turbulence. Ultimately, this targeted restoration enables highly accurate and reliable facility safety inspections.

5. Addressing Road Complexity and False Positive Rates

Topographical and functional complexities of operational roadways introduce numerous semantic distractors into UAV imagery. Unlike controlled laboratories, real-world asphalt surfaces are heavily populated with painted markings and transient traffic. Furthermore, tree shadows and highly variable surface states—such as standing water—complicate analysis. To a deep learning object detector, the visual signatures of these elements frequently conflict with genuine distresses. This conflict precipitates a critical inflation of the False Positive Rate (FPR). If left unaddressed, high FPRs render automated detection systems unviable for municipal maintenance prioritization. Mitigating these distractors requires advanced strategies to blend semantic masking and localized noise modulation. Consequently, context-aware spatial filtering becomes essential. In aerial photogrammetry, accurately distinguishing the primary road matrix—an impervious surface—from surrounding heterogeneous environments, such as vegetation and soil, remains a fundamental feature extraction challenge [27]. While ensemble machine learning algorithms, such as the RUSBoost method, have historically demonstrated high efficacy in classifying broad impervious land covers [27, 28], transitioning from macro-level surface classification to microscopic distress localization requires significantly more robust, context-aware spatial filtering. Similar to the successful utilization of UAVs in accurately delineating complex hydrological and drainage networks across varied terrains [29], isolating pavement anomalies requires navigating severe topographical interference. As depicted in Figure 5, topographical complexities and semantic distractors, such as thermoplastic markings and puddles, frequently mimic structural distress signatures.



Fig. 5. Semantic distractors in complex urban topographies contributing to FPR inflation

5.1 Semantic Filtering of Lane Markings and Diverse Objects

UAVs operating highly congested traffic corridors capture high-resolution imagery laden with non-target objects. These objects notably include thermoplastic lane markings, pedestrian crosswalks, and urban debris. The sharp, high-contrast geometric boundaries of newly painted markings generate massive gradient spikes. Traditional spatial preprocessing filters, such as unsharp masking, inadvertently amplify these specific spikes. Consequently, naive CNNs frequently misclassify the edges of these markings. They are often incorrectly labeled as severe longitudinal or transverse cracks.

State-of-the-art frameworks systematically suppress these false positives by incorporating semantic instance segmentation. Recent methodologies explicitly isolate road markings from the primary asphalt tensor. By utilizing bounding box and pixel-level information obtained through initial segmentation, algorithms mask out markings. The extracted markings are processed independently, effectively blinding the primary detector to their high-contrast boundaries. Furthermore, algorithms like Random Sample Consensus (RANSAC) mathematically restore and track marking geometries via affine transformations. This ensures that only raw, unmarked asphalt

regions are subjected to distress analysis [23]. By semantically partitioning the image, these systems significantly reduce the FPR induced by complex urban topographies.

5.2 Distinguishing Structural Voids from Puddles and Wet Roads

Transient surface conditions—particularly standing water, puddles, and globally wet asphalt—introduce profound diagnostic confusion. A puddle reflects ambient light unevenly and is typically bordered by an irregular dark boundary. This renders it visually identical to a severe structural pothole or an area of asphalt raveling. Furthermore, wet asphalt significantly alters the standard reflectance values of the pavement. This darkening effect reduces the relative contrast of subtle hairline cracks within the overall matrix.

Addressing the visual mimicry of puddles requires adaptive preprocessing frameworks capable of specific noise modulation. Recent architectural enhancements, such as the YOLO-ERCD model, implement a Visual Focus Noise Modulation (VFNM) module. This module operates by selectively introducing calculated noise patterns into identified background areas. This suppresses the faint, erratic gradients caused by wet surface reflections. By drowning out reflective interference, the neural network explicitly focuses on deep, consistent textural voids. These voids are highly characteristic of genuine structural damage. Additionally, combining this with edge-preserving bilateral filtering computationally homogenizes the internal textures of water reflections. Conversely, the rigid structural boundary of an actual pothole is strictly preserved. This synergistic filtering allows classification heads to accurately differentiate puddles from critical structural voids based on texture [17].

5.3 Detection Reliability in Snow Covered Environments

Automated pavement inspection in cold regions introduces unique photogrammetric complexities. The presence of snow and ice obfuscates road boundaries and generates extreme surface glare. Furthermore, it visually mimics light-colored structural patching or sealed cracks. In these occluded environments, relying solely on global pixel-intensity transformations often exacerbates the issue. Heavy manipulation can wash out the few visible pavement textures remaining in the capture.

Advanced frameworks bypass heavy photogrammetric manipulation to ensure detection reliability in winter conditions. Recent studies utilizing cold-region specific datasets demonstrate the necessity of expanding the network's contextual awareness. Researchers successfully expanded the receptive field of models by replacing standard convolutions with self-calibrated convolutions. Integrating low-parameter triplet attention modules into the network's neck further enhances this architectural approach. This enables the network to process broader spatial contexts rather than relying on strictly localized pixels. Consequently, the model can infer the trajectory of a crack by analyzing contiguous asphalt structures. It identifies damage surrounding a patch of snow rather than failing when a segment is obscured. This demonstrates that attention mechanisms function as powerful, implicit semantic pre-processors under adverse weather conditions [30].

6. Detection Performance Variations by Damage Typology

The morphological characteristics of pavement distresses vary significantly across different operational environments. These anomalies range from narrow, unidirectional linear cracks to expansive, highly textured surface deformations. Consequently, the efficacy of specific preprocessing filters depends heavily on the targeted damage typology. Furthermore, deep learning feature extraction mechanisms are similarly constrained by these distinct morphological variations. Standardized global datasets typically categorize road surface damages into four primary quadrants. These specific classes include longitudinal cracks (D00), transverse cracks (D10), alligator cracking (D20), and potholes (D40). Each classification requires distinct computational strategies to ensure optimal autonomous detection capabilities. The following subsections delineate specific performance variations and optimal algorithmic approaches for these distinct damage types [31]. These distinct damage categories and their respective morphological characteristics are visually represented in Figure 6, providing a categorical framework for the subsequent performance evaluations.

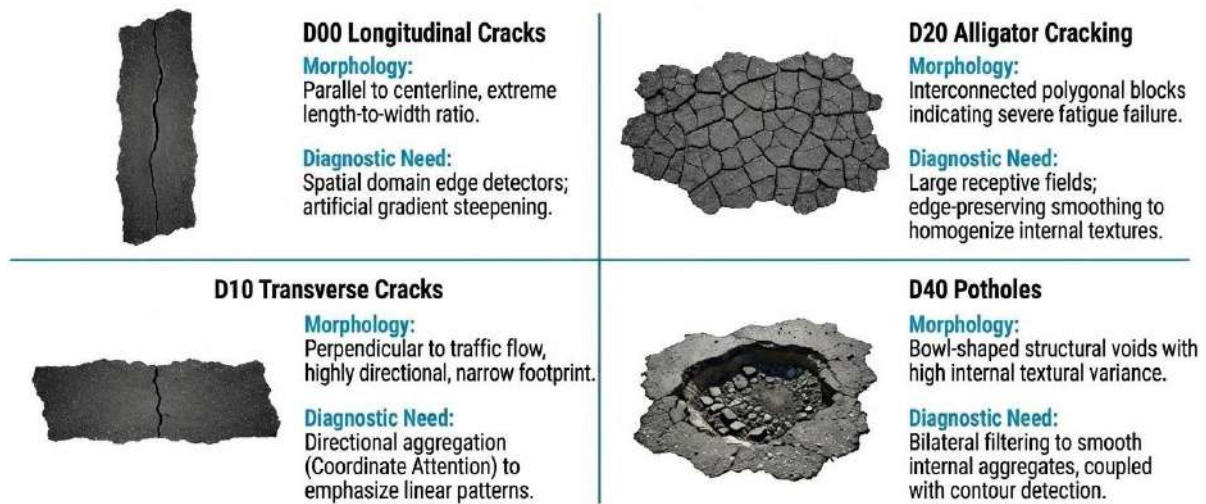


Fig. 6. Morphological typologies and classification quadrants of road surface damages (D00, D10, D20, and D40) in automated detection framework

6.1 Algorithmic Sensitivity to Linear Longitudinal and Transverse Cracks

Longitudinal cracks often run parallel to the road centerline following wheel paths. Transverse cracks are similarly characterized by specific directional propagation. Both are fundamentally defined by their extreme length-to-width ratios [32]. In high-altitude UAV imagery, these linear distresses present a minuscule spatial footprint. They sometimes span only a few pixels in total width. Consequently, they are highly susceptible to resolution loss and inappropriate image smoothing algorithms. To successfully detect these features, spatial domain edge detectors are strictly required. Furthermore, high-frequency amplification filters are necessary to accurately define the structural boundaries of the damage [33].

Extensive evaluations of spatial filters applied to UAV-acquired images demonstrate notable findings. Specifically, the Laplacian of Gaussian (LoG) filter provides exceptional sensitivity for fine linear cracks. This filter operates by calculating the second spatial derivative of the image intensity. Consequently, the LoG filter achieves a reported accuracy of 92% and a precision of 88%. This successfully isolates micro-cracks with widths as narrow as 0.1 millimeters in rapid processing times [34]. Similarly, Unsharp Masking techniques are frequently coupled with Gaussian low-pass filters. This combination effectively steepens the gradient transitions of faint, hairline cracks prior to feature extraction [35]. These approaches artificially emphasize the high-frequency components of the captured image. Additionally, they often utilize techniques like Otsu's thresholding for global image binarization. Consequently, neural networks receive a much sharper structural boundary representation of the linear defect [36]. Furthermore, the efficacy of these spatial filters is inextricably linked to foundational photogrammetric flight parameters. Most notably, the Ground Sample Distance (GSD) plays a critical role in this preprocessing phase. Variations in UAV flight altitude, velocity, and sensor focal length directly dictate the GSD. Consequently, these factors determine the initial pixel density of the captured structural distresses. Within this research context, elevated flight altitudes or suboptimal sensors often yield a coarser GSD. In such cases, high-frequency edge enhancers become strictly mandatory to artificially reconstruct gradient transitions. These algorithms successfully recover sub-millimeter cracks (D00/D10) prior to the neural feature extraction stage.

Architecturally, modern single-stage detectors have adapted to these distinct linear features. They achieve this by incorporating specialized, mathematically rigorous attention mechanisms. The integration of Coordinate Attention (CA) or spatial attention modules is particularly effective. This enables the network to aggregate horizontal and vertical spatial information independently. This directional aggregation is crucial for detecting both longitudinal and transverse cracks. It forces the model to mathematically emphasize continuous linear patterns. Simultaneously, it suppresses background distractors like painted lane markings or longitudinal shadows [6]. Quantitative assessments of optimized YOLOv8 models on specific crack classes reveal notable performance

metrics. They demonstrate precision scores of 84% for longitudinal cracks (D00) and 69% for transverse cracks (D10). This confirms the high sensitivity of attention-driven frameworks to linear, directional anomalies [37].

6.2 Filtering Strategies for Complex Alligator Cracking and Potholes

Alligator cracking (D20) manifests as an interconnected network of polygonal blocks indicating severe fatigue failure. This provides a stark contrast to the highly directional morphology of linear cracking. Furthermore, potholes (D40) appear as bowl-shaped surface depressions. These specifically result from the progressive structural collapse of the pavement matrix [32]. These macroscopic defects exhibit high internal textural variance alongside distinctly rough edges. Additionally, they often contain internal debris and demonstrate profound spatial complexity [38].

Applying aggressive high-frequency edge enhancers to D20 and D40 damages is often counterproductive. Filters such as LoG or standard Unsharp Masking artificially amplify coarse asphalt aggregates inside potholes. Similarly, they inappropriately amplify the fragmented blocks characteristic of alligator cracks. Consequently, this leads to severe over-segmentation and significant bounding-box confusion for the detector. Instead, edge-preserving smoothing filters are definitively optimal for these highly complex topologies. Applying Bilateral Filtering or median blur mathematically homogenizes chaotic internal textures within the damaged region. Simultaneously, these techniques strictly preserve the macroscopic bounding perimeter of the structural void [39]. Figure 7 demonstrates the quantitative superiority of bilateral filtering over standard Gaussian smoothing for complex topologies, as it effectively preserves structural boundaries while homogenizing internal textures.

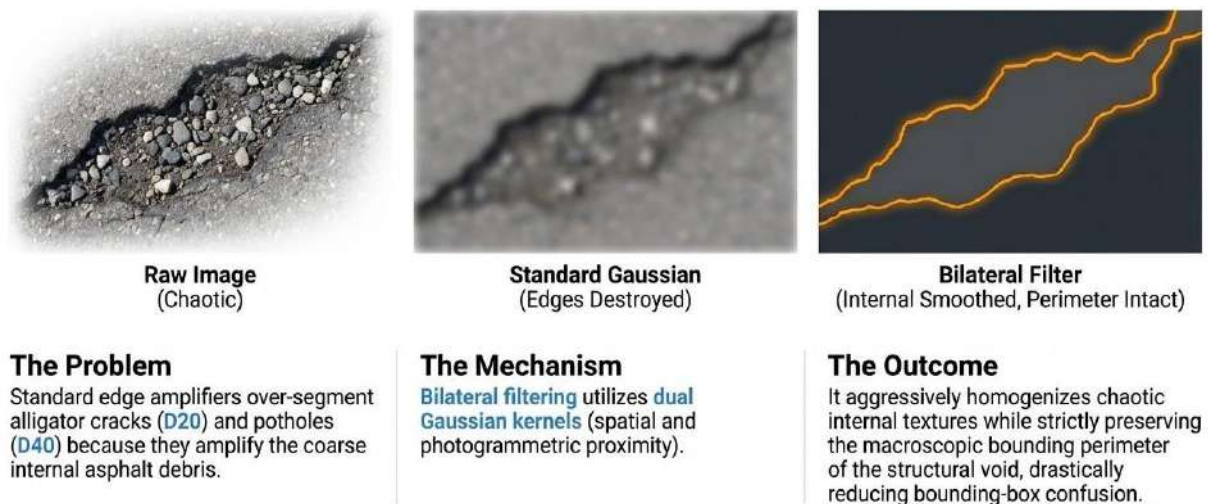


Fig. 7. Comparison of edge-preserving bilateral filtering and standard smoothing on alligator cracking and potholes

Once the region is computationally smoothed, morphological operations such as erosion and dilation are applied. Subsequently, contour detection algorithms effectively categorize the damage based on holistic size constraints. This geometric regression drastically reduces false positive rates typically triggered by complex pavement backgrounds [40].

Within deep learning frameworks, the expansive nature of D20 and D40 damages presents unique challenges. Models must possess a large receptive field capable of fully grasping the global context. Introducing advanced attention modules, such as CBAM or Swin Transformer blocks, strengthens multi-scale feature fusion. Consequently, this allows networks to evaluate the spatial relationships of entire damaged regions simultaneously [41]. Integrating these global feature extractors yields highly robust quantitative results for complex macroscopic damages. For instance, specifically enhanced YOLO architectures have recorded notable recall scores in recent evaluations. They successfully achieved 83% for alligator cracking and 74% for potholes. This demonstrates that balancing local texture smoothing with global contextual awareness is absolutely critical.

Ultimately, this balance accurately quantifies severe, macroscopic pavement degradation across automated networks [37].

7. Applications and Ablation Studies on Standardized Datasets

The theoretical advantages of specific image preprocessing filters and architectural optimizations must be rigorously validated. This validation strictly requires empirical testing on expansive, highly annotated datasets. Consequently, the introduction of standardized global benchmarks has catalyzed rapid advancements in automated pavement analysis. These datasets provide a common ground for objective, quantitative comparisons of algorithmic performance. Specifically, they enable the standardized evaluation of metrics such as Mean Average Precision (mAP), precision, recall, and F1-scores. Furthermore, ablation studies systematically add or remove specific enhancement modules or architectural layers. These targeted evaluations offer profound insights into the efficacy of isolated computational mechanisms. Ultimately, they clearly demonstrate how these mechanisms directly influence road damage detection within UAV imagery.

The quantitative impact of these isolated preprocessing mechanisms is comprehensively summarized in Table 2. Specifically, regional contrast enhancements like CLAHE demonstrate substantial empirical improvements for fine distresses. Furthermore, high-frequency filters like Unsharp Masking explicitly elevate Mean Average Precision (mAP) for microscopic features. These targeted techniques successfully optimize neural network inputs while strictly preventing global noise amplification.

Table 2. Quantitative impact of specific preprocessing algorithms on detection metrics

Preprocessing Technique	Target Damage Typology	Baseline Metric (Raw)	Enhanced Metric	Reference
CLAHE	Fine Cracks (D00/D10)	88.0% Accuracy	95.0% Accuracy	Freitas et al., 2025 [14]
Unsharp Masking	Hairline Cracks	74.2% mAP	86.97% mAP (+12.77%)	Aina et al., 2025 [37]
LoG Filter	Micro-Cracks (0.1 mm)	82.5% Precision	88.0% Precision	Dorafshan et al., 2018 [33]
MSR / LIME	Low Light / Deep Shadows	78.8% mAP	91.1% Precision	Freitas et al., 2025 [14]
CAGC	Sun Glare / Reflections	0.62 F1-Score	0.74 F1-Score	Li X. et al., 2026 [17]

7.1 Benchmarking on the RDD2022 Global Dataset

The CRDDC 2022 challenge facilitated the release of the expansive RDD2022 dataset. This dataset currently serves as the definitive global benchmark for automated road condition monitoring. It encompasses 47,420 high-resolution images gathered across six distinct nations. These nations include Japan, India, the Czech Republic, Norway, the United States, and China. The dataset contains over 55,000 expertly annotated damage instances. These are systematically categorized into longitudinal cracks, transverse cracks, alligator cracking, and potholes [5]. Crucially for photogrammetric research, RDD2022 incorporates a specialized "China Drone" subset. This ensures deep learning models are explicitly tested against unique spatial scales and rotational variations. Furthermore, it rigorously tests models against motion-blur artifacts inherent to UAV captures [6].

Ablation studies conducted exclusively on the RDD2022 dataset have quantified immense values. Specifically, integrating advanced preprocessing algorithms and attention-based filters into detection pipelines proves crucial. For instance, the RDD-YOLO framework integrated a Simple Attention Mechanism (SimAM) into its backbone. This integration effectively filters out complex environmental noise. Additionally, researchers coupled this with replacing nearest interpolation with bilinear interpolation. This modification resulted in a 2.5% increase in mAP@0.5 over the baseline YOLOv8 model. Furthermore, it achieved a 5.2% improvement in the stringent

mAP@0.5:0.95 metric. The overall F1-score on the test set ultimately reached 69.6%. These results robustly affirm the absolute necessity of sophisticated feature enhancement [7]. Similarly, the SEA-YOLO v8 architecture incorporates an Efficient Multi-Scale Attention (EMA) mechanism. This allows the network to selectively focus on critical structural areas while suppressing background shadows. Consequently, this advanced architecture achieved a robust mAP@0.5 of 63.2% on the RDD2022 dataset [4].

Furthermore, researchers have actively explored generative models as an alternative to traditional spatial preprocessing. The Joint Training of Image Generator and Detector (JTGD) framework exemplifies this approach. It utilizes adversarial networks to artificially synthesize hard-example damage representations on flawless road textures. Ablation studies confirmed the efficacy of exposing the detector to these generated anomalies during training. These generated anomalies specifically include complex lighting scenarios and synthetic motion blur. Consequently, the JTGD model outperformed highly complex Faster Swin Transformer baselines in F1-scores across all evaluated countries. Notably, this generative framework achieved these superior metrics while utilizing less than 20% of competing parameter counts [10].

7.2 Specialized Validations on UAV Specific Datasets

RDD2022 provides an excellent generalized baseline for automated detection frameworks. However, specialized high-altitude datasets are strictly necessary for comprehensive photogrammetric validation. These specialized datasets directly evaluate the specific degradation factors inherent to aerial sensing. The UAV-PDD2023 dataset has emerged as a premier resource in this specific domain. It consists of 2,440 ultra-high-resolution UAV captures. These captures detail over 11,158 pavement distress instances across complex urban and rural topologies [42].

Evaluations on UAV-PDD2023 consistently highlight a major architectural paradigm shift. Researchers increasingly favor architectures performing implicit, mathematically integrated preprocessing. For example, the Dynamic Scale-Aware Fusion Detection Model (RT-DSAFDet) addresses these issues directly. It was explicitly designed to automatically remove common background interference. This specifically includes filtering out tree shadows and highly reflective lane markings. Experimental results on UAV-PDD2023 demonstrated an impressive mAP@0.5 of 54.2%. This is remarkably 11.1% higher than the baseline YOLOv10-m architecture. Simultaneously, it reduced the parameter count by 88% to a mere 1.8M [3]. Similarly, the ALC-Net framework utilized this dataset to rigorously validate its internal Focus module. This module performs initial down-sampling and channel-wise concatenation. Consequently, it strips away high-resolution noise without requiring external resizing filters. Ablation studies demonstrated consecutive and significant metric enhancements. They proved that tailored internal attention mechanisms provide a superior operational trade-off. This approach optimizes real-time processing speeds and accuracy against unaltered raw captures [6].

Additional validations utilize datasets like CrackTinyNet to substantiate cross-dataset generalization. This is particularly crucial for accurately localizing microscopic linear defects. Models like OBC-YOLOv8 were rigorously evaluated on custom UAV datasets. They integrated Omni-Dimensional Dynamic Convolution (ODConv) blocks into their architecture. This dynamically adjusts to the profound feature variations of input images. This architectural filter resulted in a 1.8% increase in mAP@0.5. Furthermore, it generated a 1.6% increase in the overall F1-score. This proves the model is exceptionally capable under highly complex illumination conditions [9].

7.3 Summary of Recent Architectural Innovations

The culmination of recent literature points toward a distinct architectural paradigm shift. The field is rapidly moving away from explicit, external digital image processing (DIP) techniques. Instead, it strongly favors implicit, architecturally integrated feature enhancement strategies. Edge-deployed UAV platforms impose highly stringent computational and latency constraints. To meet these demands, researchers embed mathematical equivalents of traditional filters directly into networks.

One prominent innovation is the deployment of Cross-Scale Feature Fusion. This is perfectly exemplified by the Generalized Feature Pyramid Network (GFPN). Architectures like CSGEH-YOLO utilize this specifically for advanced remote sensing. By dynamically connecting different layers and scales, GFPN acts as an internal spatial filter. It significantly enhances the transfer of damage features across multi-level scales. Consequently, it mitigates the resolution loss typically associated with traditional down-sampling. This mechanism yielded a 3.1% improvement in mAP on UAV imagery. Furthermore, it successfully reduced overall computational complexity to 78% of the baseline [3]. Crucially, this transition toward implicit preprocessing directly addresses the severe hardware constraints of UAV-based edge computing platforms. While explicit, external digital image processing filters inherently bottleneck real-time inference, implicit attention mechanisms execute these enhancements directly. These modules operate within the GPU-accelerated convolutional pipeline. This architectural integration guarantees sustained high Frames Per Second (FPS) rates and minimal inference latency. Ultimately, this ensures rapid, real-time diagnostic capabilities without sacrificing spatial precision.

Furthermore, integrating hybrid attention mechanisms has transformed standard photogrammetric workflows. It has largely superseded the need for global radiometric transformations like standard Histogram Equalization. Modules like CBAM, Coordinate Attention (CA), and LFPA dynamically calculate attention weights. They explicitly emphasize structural damage characteristics while suppressing redundant background channels [6, 7]. These mechanisms mathematically emulate the noise-suppression attributes of Bilateral filtering. Simultaneously, they replicate the gradient-steepening effects of Unsharp Masking directly within feature maps. The synthesis of these incredible innovations confirms a clear technological trajectory. The future of UAV-based road inspection strictly requires highly optimized, attention-driven networks. These lightweight frameworks will autonomously execute spatial and radiometric corrections in real-time.

Table 3 presents a comparative benchmarking of these recent architectural innovations on UAV-specific datasets. The data clearly illustrates the superior computational efficiency of implicit preprocessing strategies. For example, models such as RT-DSAFDet significantly reduce overall parameter counts compared to standard baselines. Remarkably, they achieve this 88% parameter reduction while simultaneously generating double-digit increases in mAP metrics. This explicitly confirms their optimal suitability for real-time, edge-deployed UAV inspection operations.

Table 3. Comparative analysis of recent UAV-based road damage detection architectures.

Model Architecture	Validation Dataset	mAP@0.5 (%)	Parameter Count (Millions)	Implicit Preprocessing / Innovation	Reference
Baseline YOLOv10-m	UAV-PDD2023	43.1	~20.0	Standard Convolution	Zhang W. et al., 2025 [3]
RT-DSAFDet	UAV-PDD2023	54.2	1.8	Dynamic Scale-Aware Fusion	Zhang W. et al., 2025 [3]
SEA-YOLO v8	RDD2022	63.2	~3.2	EMA Attention Mechanism	Britto et al., 2025 [4]
ALC-Net	UAV-PDD2023	56.8	2.4	Lightweight Focus Module	L. Chen et al., 2025 [6]
RDD-YOLO	RDD2022	69.6 (F1)	~3.0	SimAM & Bilinear Interpolation	X. B. Chen et al., 2024 [7]

To visually contextualize these architectural advancements, Figure 8 illustrates the inverse relationship between model complexity and detection accuracy. As shown, implicit preprocessing frameworks achieve superior mAP scores while drastically reducing the computational parameter footprint.

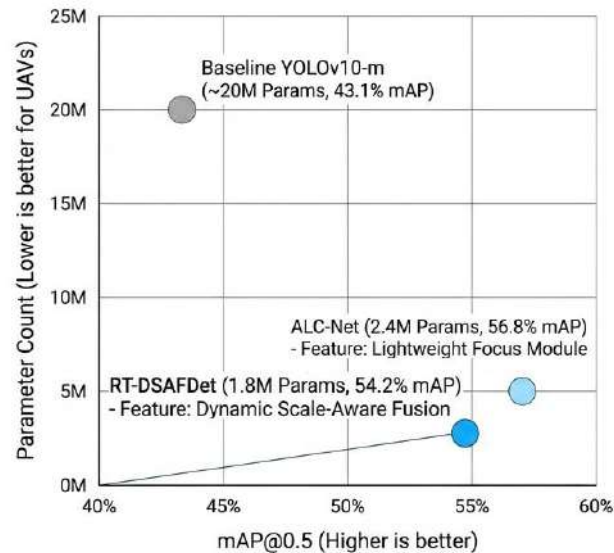


Fig. 8. Comparative benchmarking of state-of-the-art UAV road inspection models, highlighting the efficiency of implicit preprocessing architectures

8. Conclusion

This research has systematically investigated the intersection of aerial photogrammetry, digital image processing, and advanced deep learning architectures for the autonomous detection of pavement distresses. The primary innovative impact lies in the rigorous isolation and quantification of how explicit preprocessing algorithms control the feature extraction capabilities and final Mean Average Precision (mAP) of neural networks. By bridging the critical gap between raw optical capture and convolutional mapping, this study provides a robust mathematical framework for overcoming the severe environmental degradations inherent in high-altitude aerial sensing. The findings demonstrate that the reliability of autonomous monitoring is fundamentally tied to the quality of the optical input field.

The key findings and quantitative results of this study are summarized as follows:

- Feeding raw UAV imagery directly into deep learning object detectors consistently produces inadequate precision due to sensor noise, flight-induced motion blur, and extreme illumination variances.
- Mathematical preprocessing serves as an indispensable prerequisite for normalizing the statistical distribution of captured data and maximizing model reliability in volatile environments.
- Contrast Limited Adaptive Histogram Equalization (CLAHE) improved fine crack detection accuracy from 88% to 95% by significantly enhancing local feature contrast.
- This method allowed the network to identify subtle distresses without the interference of the global noise amplification typically associated with standard histogram equalization.
- High-frequency spatial enhancers, such as Unsharp Masking and Laplacian of Gaussian (LoG) filters, proved critical for sharpening the gradient transitions of microscopic linear defects.
- These spatial strategies achieved a 12.77% improvement in mAP, effectively preventing the loss of sharp structural boundaries during the successive down-sampling stages of feature extraction.
- Edge-preserving smoothing via Bilateral Filtering was identified as the optimal strategy for complex topologies, such as alligator cracking and potholes.
- This approach successfully homogenized chaotic internal textures while strictly preserving the macroscopic bounding perimeters of structural voids, thereby reducing bounding-box confusion.
- Deep topological shadows were effectively resolved using Multi-Scale Retinex (MSR) algorithms, which increased information entropy by 63% in low-light conditions.

- This illumination-invariant transformation ensured a high recognition accuracy of 97.5%, proving its efficacy in shadowed urban corridors and forested environments.
- Channel-wise Adaptive Gamma Correction (CAGC) successfully mitigated the saturating effects of specular reflections on wet asphalt and reflective lane markings.
- By normalizing the pixel-intensity shifts, this module maintained a continuous gradient flow within the activation maps, preventing the formation of visual blind spots.
- Semantic preprocessing and instance segmentation techniques enabled the network to successfully ignore background distractors like thermoplastic markings and urban debris.
- This targeted masking strategy directly lowered False Positive Rates, making automated detection systems viable for real-world municipal maintenance prioritization.
- A distinct paradigm shift toward implicit preprocessing was observed, where mathematical equivalents of traditional filters are embedded directly into the network architecture.
- Advanced frameworks utilizing attention mechanisms and cross-scale fusion reduced parameter counts by up to 88% while increasing mAP by 11.1% over baseline models.

In conclusion, the successful automation of UAV-based road inspection requires the meticulous optimization of the optical input field rather than a singular focus on network depth. By matching specific environmental degradations with targeted mathematical countermeasures, engineers can effectively overcome the physical limitations of aerial photogrammetry. As the field moves toward real-time edge inference on lightweight platforms, the integration of these spatial and radiometric filtering principles will be essential for the deployment of truly autonomous infrastructure monitoring systems.

References

- [1] Liu J, Lemus-Romani J, Rueda EJ, Becerra-Rozas M, Astorga G. Identification of Pathologies in Pavements by Unmanned Aerial Vehicle (UAV): A Systematic Literature Review. *Drones*. 2026;10(2):90. <https://doi.org/10.3390/drones10020090>
- [2] Silva LA, Leithardt VRQ, Batista VFL, González GV, Santana JFDP. Automated Road Damage Detection Using UAV Images and Deep Learning Techniques. *IEEE Access*. 2023; 11:62918-31. <https://doi.org/10.1109/ACCESS.2023.3287770>
- [3] Zhang W, Li X, Wang L, Zhang D, Lu P, Wang L, et al. A Lightweight Method for Road Defect Detection in UAV Remote Sensing Images with Complex Backgrounds and Cross-Scale Fusion. *Remote Sensing*. 2025;17(13):2248. <https://doi.org/10.3390/rs17132248>
- [4] Britto JGM, Satheesh DRN, Adharsh N, Nayab SD. Deep learning-based automated detection of road surface damage using UAV imagery. *International Journal of Engineering Research and Science & Technology*. 2025;21(4):691-8. <https://doi.org/10.62643/ijerst.2025.v21.n4.pp691-698>
- [5] Arya D, Maeda H, Ghosh SK, Toshniwal D, Sekimoto Y. RDD2022: A multi-national image dataset for automatic road damage detection. *Geoscience Data Journal*. 2024;11(4):846-62. <https://doi.org/10.1002/gdj3.260>
- [6] Chen L, Wu P, Tan W, Li H, Chen H, Zeng N. A Novel UAV-Based Road Damage Detection Algorithm with Lightweight Convolution and Attention Mechanism. *International Journal of Network Dynamics and Intelligence*. 2025;4(4):100025. <https://doi.org/10.53941/ijndi.2025.100025>
- [7] Chen XB, Liu C, Chen L, Zhu XD, Zhang YH, Wang CX. A Pavement Crack Detection and Evaluation Framework for a UAV Inspection System Based on Deep Learning. *APPLIED SCIENCES-BASEL*. 2024;14(3). <https://doi.org/10.3390/app14031157>
- [8] Magdy A, Moustafa MS, Ebied HM, Tolba MF. Lightweight faster R-CNN for object detection in optical remote sensing images. *Scientific Reports*. 2025;15(1):16163. <https://doi.org/10.1038/s41598-025-99242-y>
- [9] Zhang S, Liu Z, Wang K, Huang W, Li P. OBC-YOLOv8: an improved road damage detection model based on YOLOv8. *PeerJ Computer Science*. 2025;11: e2593. <https://doi.org/10.7717/peerj-cs.2593>
- [10] Peng K-C. Joint Training of Image Generator and Detector for Road Defect Detection. *arXiv preprint arXiv:250903465*. 2025.

- [11] Alshawabkeh S, Wu L, Dong D, Cheng Y, Li L. A hybrid approach for pavement crack detection using mask R-CNN and vision transformer model. *Computers, Materials, & Continua*. 2025;82(1):561-77. <https://doi.org/10.32604/cmc.2024.057213>
- [12] Karkınlı A, Kesikoğlu A. Enhancement of UAV-Aerial Images Using Weighted Differential Evolution Algorithm. *International Journal of Scientific Research in Computer Science, Engineering and Information Technology*. 2021;7(2):196-206. <https://doi.org/10.32628/CSEIT217248>
- [13] Jiskani SM, Hussain T, Sahito AA, Shaikh F, Kumar L. Improving Insulators Detection Accuracy via Image Enhancement Techniques: Case of Indigenous Aerial Image Dataset. *IEEE Access*. 2024; 12:145582-9. <https://doi.org/10.1109/ACCESS.2024.3474255>
- [14] Freitas G, Mota G, Júnior E, Albuquerque V. Impact of Image Enhancement Filters on Road Defect Detection Using Deep Learning. *Anais do Computer on the Beach*. 2025; 16:213-8.
- [15] Zhang F, Zhang C, Yang H, Zhao L. Point Cloud Denoising with Principal Component Analysis and a Novel Bilateral Filter. *Traitement du Signal*. 2019; 36:393-8. <https://doi.org/10.18280/ts.360503>
- [16] Nguyen NHT, Perry S, Bone D, Le HT, Nguyen TT. Two-stage convolutional neural network for road crack detection and segmentation. *Expert Systems with Applications*. 2021; 186:115718. <https://doi.org/10.1016/j.eswa.2021.115718>
- [17] Li X, Chu Y, Chan T, Lo WL, Fu H. YOLO-ERCD: An Upgraded YOLO Framework for Efficient Road Crack Detection. *Sensors*. 2026;26(2):564. <https://doi.org/10.3390/s26020564>
- [18] Sun W, Liu X, Lei Z. A Tunnel Crack Detection Method Based on an Unmanned Aerial Vehicle (UAV) Equipped with a High-Speed Camera and Crack Recognition Algorithm Using Improved Multi-Scale Retinex and Prewitt-Otsu. *Drones*. 2025;9(6):393. <https://doi.org/10.3390/drones9060393>
- [19] Yao Z, Xu J, Hou S, Chuah MC. Cracknex: a few-shot low-light crack segmentation model based on retinex theory for uav inspections. 2024 IEEE International Conference on Robotics and Automation (ICRA); May 13-17, 2024; Yokohama, Japan: IEEE. p. 11155-62. <https://doi.org/10.1109/ICRA57147.2024.10611660>
- [20] Li C, Teng Y, Li C, Huang Z, Jiang L. Lightweight detection of grapevine downy mildew using UAV based on image information enhancement and adaptive dual-path fusion attention. *BMC Plant Biology*. 2025;26(1):16. <https://doi.org/10.1186/s12870-025-07849-0>
- [21] Kesikoglu MH. Comparative analysis of R/SWIR, NIR/SWIR, and NDSI classification methods for determination of snow cover change on Erciyes Mountain, Türkiye using Landsat satellite images. *Research on Engineering Structures and Materials*. 2025;11(4):1653-65. <https://doi.org/10.17515/resm2025-524ml1110rs>
- [22] Kesikoglu MH. Enhancing Snow Detection through Deep Learning: Evaluating CNN Performance Against Machine Learning and Unsupervised Classification Methods. *Water Resources Management*. 2025;39(12):6053-71. <https://doi.org/10.1007/s11269-025-04240-4>
- [23] Nie Y, Wu W, Shan J, Peng H, Guo F, Liu Y, et al. Road Marking Distress Detection and Assessment Based on UAV Imagery. *Materials*. 2026;19(5):992. <https://doi.org/10.3390/ma19050992>
- [24] Esfahani M, Wang H. Robust Glare Detection: Review, Analysis, and Dataset Release 2021 [Available from: <https://arxiv.org/abs/2110.06006>].
- [25] Shim S, Kim J, Lee S-W, Cho G-C. Road surface damage detection based on hierarchical architecture using lightweight auto-encoder network. *Automation in Construction*. 2021; 130:103833. <https://doi.org/10.1016/j.autcon.2021.103833>
- [26] Lee J-H, Gwon G-H, Kim I-H, Jung H-J. A Motion Deblurring Network for Enhancing UAV Image Quality in Bridge Inspection. *Drones*. 2023;7(11):657. <https://doi.org/10.3390/drones7110657>
- [27] Kesikoglu MH, Ozkan C, Kaynak T. The impact of impervious surface, vegetation, and soil areas on land surface temperatures in a semi-arid region using Landsat satellite images enriched with Ndaisi method data. *Environmental Monitoring and Assessment*. 2021;193(3):143. <https://doi.org/10.1007/s10661-021-08916-3>

- [28] Kesikoglu MH, Atasever UH, Ozkan C, Besdok E. The Usage of RusBoost Boosting Method for Classification of Impervious Surfaces. *Int Arch Photogramm Remote Sens Spatial Inf Sci*. 2016; XLI-B7:981-5. <https://doi.org/10.5194/isprs-archives-XLI-B7-981-2016>
- [29] Günen MA, Atasever ÜH, Taşkanat T, Beşdok E. USAGE OF UNMANNED AERIAL VEHICLES (UAVs) IN DETERMINING DRAINAGE NETWORKS. *Nature Sciences*. 2019;14(1):1-10. <https://doi.org/10.12739/NWSA.2019.14.1.4A0062>
- [30] Zhu L, Luo Z, Bi Y, Zhang Z. Cold Region Road Surface Defect Detection Using an Intelligent Machine Vision Method. *Journal of Cold Regions Engineering*. 2025;39(3):04025024. <https://doi.org/10.1061/JCRGEI.CRENG-874>
- [31] Tang Z, Chamchong R, Pawara P, editors. A comparison of road damage detection based on YOLOv8. 2023 International Conference on Machine Learning and Cybernetics (ICMLC); 09-11 July 2023. Adelaide, Australia: IEEE. <https://doi.org/10.1109/ICMLC58545.2023.1032799>
- [32] Li D, Duan Z, Hu X, Zhang D, Zhang Y. Automated classification and detection of multiple pavement distress images based on deep learning. *Journal of Traffic and Transportation Engineering (English Edition)*. 2023;10(2):276-90. <https://doi.org/10.1016/j.jtte.2021.04.008>
- [33] Dorafshan S, Thomas RJ, Maguire M. Comparison of deep convolutional neural networks and edge detectors for image-based crack detection in concrete. *Construction and Building Materials*. 2018; 186:1031-45. <https://doi.org/10.1016/j.conbuildmat.2018.08.011>
- [34] Dorafshan S, Thomas RJ, Maguire M. SDNET2018: An annotated image dataset for non-contact concrete crack detection using deep convolutional neural networks. *Data in Brief*. 2018; 21:1664-8. <https://doi.org/10.1016/j.dib.2018.11.015>
- [35] Maslan J, Cicmanec L. A System for the Automatic Detection and Evaluation of the Runway Surface Cracks Obtained by Unmanned Aerial Vehicle Imagery Using Deep Convolutional Neural Networks. *Applied Sciences*. 2023;13(10):6000. <https://doi.org/10.3390/app13106000>
- [36] Liu W, Ren G, Yu R, Guo S, Zhu J, Zhang L. Image-adaptive YOLO for object detection in adverse weather conditions. *Proceedings of the 36th AAAI Conference on Artificial Intelligence*; 2022. p. 1792-800. <https://doi.org/10.1609/aaai.v36i2.20072>
- [37] Aina J, Haghi N, Famewo B, Lambert T, Owolabi D, Efe S. Real-Time Road Damage Detection Using YOLOv8. *International Conference on Transportation and Development* 2025. p. 411-8. <https://doi.org/10.1061/9780784486191.036>
- [38] Zhang X, Wang M, Liu C, Xu H, Yang Y. Crack Detection and Quantification of Rural Cement Concrete Pavements Based on Deep-Learning Fusion Model. *Journal of Transportation Engineering, Part B: Pavements*. 2026;152(1):04025062. <https://doi.org/10.1061/JPEODX.PVENG-1969>
- [39] Kim Y-M, Kim Y-G, Son S-Y, Lim S-Y, Choi B-Y, Choi D-H. Review of Recent Automated Pothole-Detection Methods. *Applied Sciences*. 2022;12(11):5320. <https://doi.org/10.3390/app12115320>
- [40] Schiopu I, Saarinen J, Kettunen L, Tabus I. Pothole detection and tracking in car video sequence. 39th International Conference on Telecommunications and Signal Processing (TSP); 27-29 June 2016; Vienna, Austria: IEEE; 2016. <https://doi.org/10.1109/TSP.2016.7760975>
- [41] Fan J, Bocus MJ, Hosking B, Wu R, Liu Y, Vityazev S, et al., editors. Multi-Scale Feature Fusion: Learning Better Semantic Segmentation For Road Pothole Detection. 2021 IEEE International Conference on Autonomous Systems (ICAS); 2021 11-13 Aug. 2021; Montreal, QC, Canada: IEEE. <https://doi.org/10.1109/ICAS49788.2021.9551165>
- [42] Yan H, Zhang J. UAV-PDD2023: A benchmark dataset for pavement distress detection based on UAV images. *Data in Brief*. 2023; 51:109692. <https://doi.org/10.1016/j.dib.2023.109692>

Performance assessment of sustainable one-part alkali-activated concrete: An experimental and machine learning approach

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Abstract

One-part alkali-activated concrete (OPAAC) is regarded as a sustainable substitute for traditional concrete and demonstrates significant potential in practical field constructions. This paper explores the properties of OPAAC made using a binder system that consists of ground granulated blast furnace slag, sodium metasilicate (SMS), and calcium hydroxide. Three mix designs for grades M15, M20, and M25 were formulated. The performance evaluations revealed that higher SMS content and water-to-binder ratio favored the workability of mixes. The hardened-state tests suggested that OPAAC showed an enhancement in compressive strength (CS), flexural strength (FS), and split-tensile strength (STS) with an increase in SMS content. The highest CS, FS, and STS recorded were 32.72, 6.20, and 3.44 MPa, respectively. Various machine-learning models were tested to estimate the strength of developed concrete. The gradient boosting model showed the best performance among others with an average test R^2 of 0.9684 and was able to successfully predict the strength. OPAAC also exhibited crucial environmental and economic advantages, including carbon reduction by 55-59% and cost savings by 12-16% compared to conventional concrete. Therefore, the outcomes show that OPAAC does not only present a comparable material performance but also an eco-friendly and less expensive option for the field construction activities.

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1. Introduction

In the construction sector, concrete is the most utilized construction material on the earth, second only to water. This is mainly because of its abundance, affordability, and resilient nature. It is estimated that the annual production of concrete may rise from current 14 billion m^3 to 20 billion m^3 by 2050 [1]. But due to its dependency on traditional cement, there are serious environmental issues. Cement manufacturing is a very energy-consuming process that accords to about 8% of total man-made CO_2 emissions in the world with an amount of emission close to 0.80-0.95 kg of CO_2 -equivalent per kilogram of ordinary Portland cement (OPC) [2]. In order to reach net-zero emission targets, the industry is increasingly using blended cements, such as Portland slag cement, Portland pozzolana cement, and composite cement. Composite cement (CC) is a more sustainable variant of blended cement with 23-47 % lower carbon emissions and it is largely used in various construction applications in Indian construction industry [3]. This has been largely due to the higher level of ground granulated blast furnace slag (GGBS) and fly ash (FA) and reduced content of clinker of 35-65 % [4]. In spite of the current developments, OPC continues to be a major contributor to carbon emission, thus explaining the need to focus on other binders that possess the latent to decrease carbon emissions and reduce the energy requirements. At the same time, the alternative binders should be locally available and economical with comparable strength and durability. Over the past decades, alkali-activated binders (AABs) were identified to be more sustainable substitute of the

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conventional OPC. AABs utilize the industrial wastes such as FA, GGBS or metakaolin, as alumina-silicate precursors. These precursors are triggered by alkaline solutions (referred to as two-part AABs), which leads to the generation of stable alumina-silicate gels that act as a binding agent for aggregates and provide concrete with the normal strength properties. Traditional two-part AABs have already proved to have huge reductions in embodied carbon and their life-cycle analysis shows that they emit 40-80 % less CO₂ than OPC [5,6]. These values are mainly due to the avoidance of clinker and the use of industrial by-products. Apart from the enormous reductions in emissions, they happened to possess excellent mechanical properties and longevity compared to OPC [7,8]. Though two-part AABs are slowly gaining acceptance and several countries have already developed standards for their usage. Their application is currently hampered by the necessity to manage concentrated activator liquids, which is a safety, storage, and quality control issue on site [9]. To overcome the above concerns, one-part alkali activated binders (one-part AABs), also referred as “just-add-water” materials, have recently been developed [10]. These binders offer the sustainability and durability benefits associated with conventional AABs, while at the same time offering handling and placement similar to OPC. Recent researches have demonstrated that the concrete made using one-part AAB has excellent performances, which are high mechanical strength and greater durability in chemically hostile circumstances [11–13].

One-part alkali-activated concrete (OPAAC) renders as an eco-friendly and a practical substitute to traditional concrete. Simultaneously, OPAAC mitigates the challenges of managing corrosive liquid activators needed by two-part alkali-activated concrete and provides a better mechanical property and durability. The only difference that exists between OPAAC and traditional concrete is the composition of the binder. OPAAC requires a binder that composes a solid alumina-silicate materials and powder alkaline activators [14–16]. In production of OPAAC, GGBS is considered a highly reactive precursor because of its enhanced reactivity and amorphous structure. Due to their high CaO, SiO₂, and Al₂O₃ content, the formation of rigid and stable gel structures, majorly in form of calcium alumina silicate hydrate (C-A-S-H) gels occur, that results in high strength at early ages, good prolonged mechanical performance, and high durability [17–19]. But, higher content of GGBS in binder system reduces the workability of OPAAC [20] and, hence, it recommended to use the optimum GGBS content in the mix design of OPAACs. Overall, these superior properties of GGBS make it an excellent material to work within one-part systems [19,21]. On the other hand, sodium metasilicate (SMS) in forms of Na₂SiO₃-Anhydrous, Na₂SiO₃·5H₂O and Na₂SiO₃·9H₂O, has been widely used as solid activators because of their high alkalinity and cementitious properties. Ma et al. [22] investigated the suitability of all three forms of sodium metasilicate in one-part alkali-activated systems. They reported that Na₂SiO₃-Anhydrous has found to be best suited for the enhanced properties of one-part alkali-activated systems. But the usage of metasilicate may speed up the setting leading to efflorescence and cracking and result in lowering mechanical performances when overused beyond 14% of mass of binder system [11,17]. A body of research has indicated that, these problems can be alleviated by addition of supplementary solid activators like Ca(OH)₂, Na₂CO₃, NaOH or NaAlO₂ which help to improve the properties of binders as well as reduce early-age cracking [23–26].

Depending on the mix design, precursor type, activator ratio, and water-to-binder (w/b) ratio, OPAAC typically has a low workability, which requires more compaction efforts similar to conventional approach. It has been shown that the alteration of mix proportion, precursor type, activator ratio and w/b ratio affect the workability of one-part alkali-activated systems [20,27,28]. In same vein, the usage of traditional super-plasticizers improves the workability without affecting mechanical strength [25,29]. Haruna et al. [30] investigated the performance of OPAAC using a binder made of high calcium FA and solid Na₂SiO₃-Anhydrous. They reported that the workability of OPAAC was obtained in the acceptable range of 34–165 mm. An increase in paste-aggregate ratio resulted in a significant improvement in workability. In a recent study [31], the slump values for GGBS-based OPAAC, activated using SMS anhydrous, was reported in the range of 20–230 mm. This suggests that OPAAC with very low to high workability can be developed, which is useful for numerous structural applications from foundations to beam and slabs. The previous literature also indicates that OPAAC is capable of reaching a very high compressive strength (CS) as high as 70 MPa at 28-days of ambient or water curing, the flexural strength (FS) typically fell in the range of

4–7 MPa, and split tensile strength (STS) was in the span of 2–5 MPa at 28 d under similar curing methods [30,32–34]. These strength developments in OPAAC were mainly attributed to the occurrence of compact microstructure containing C-A-S-H gels as a major strength giving phase [17–19,26]. Nevertheless, the major influence on the strength was found to be due to mix formulation, precursor type, activator type, ratio of activators and w/b ratio, and the method of curing [9,15,16]. Research indicates that OPAAC is curable in both ambient and water. Water curing proved to be better in long term retention of strength and in providing better durability in case of OPAAC. This can be ascribed to the phenomenon that the water curing keeps the paste saturated in concrete matrix, which enhances ion diffusion and allows hydration reactions to persist at the interface of the paste and aggregates [18,30]. Furthermore, OPAAC has been found to have better durability compared to traditional concrete, with greater resistance to chemical attack, lower permeability, lower water absorption, and improved long-term performance, as suggested by previous studies [8,35,36].

In previous studies [37–42], the application of machine learning (ML) methods in the field of alkali-activated materials has become a common practice, primarily, to predict CS, identify effective influencing factors, optimize the mix proportions, and guarantee the quality control of mix. In literature review, it is implied that many ML models, including but not restricted to random forest, gradient boost, decision trees, support vector regression, ridge regression, lasso regression, ElasticNet, and artificial neural network (ANN) have been utilized to achieve the above-said. In a recent study [42], ML algorithms, including ANN and support vector machines, were studied on 3D-printed soil-cement composites. The authors also identified the necessity of standard datasets in order to overcome data scarcity and to enhance the reliability of the models. They suggested future research on the development of hybrid ML with physics-based simulations and exploration of environmentally friendly binders, including that these would be able to optimize local resources, improve longevity, and cut down emissions and expenses. Another study [43] has surveyed how digital twins and artificial intelligence (AI) can be used in real-time structural monitoring. These methods make it possible to monitor and identify anomalies earlier, and they turned out to be much more effective than the traditional approaches. Collectively, these studies demonstrate that there is a high potential of ML and AI in the material assessment, quality inspection, and structural health assessment. Nevertheless, the largest figure of studies on ML models was carried out on two-part AABs [37–40]. This makes use of ML models in OPAAC a less exploited area. Harika et al. [44] used five models of ML to estimate the CS of one-part AAB mortars. They train, test and validate the ML models using 135 experimental data. Findings indicated that the random forest model was the most predictive, with gradient boost and decision tree coming next, and support vector regression was the least predictive. In addition, they indicated that the content of the GGBS, activator dosage, curing age, and w/b ratio were the most affecting characteristics. In another study [45], application of models such as ANN and gene expression programming (GEP) in the prediction of OPAAC systems was presented using 171 experiments database. They added that ANN performed better as compared to GEP with regard to prediction efficacy. A sensitivity analysis indicated that the CS would be most affected by the activator and slag contents.

1.1. Research Significance

Although there is an increasing trend towards OPAAC, research in this area is still limited, and most of the research work has been carried out at the paste or mortar level, and not at the concrete level. There exists a gap in knowledge about selection and dosage of solid activators, mix design methods, fresh and hardened properties at concrete level or the application of ML methods in OPAAC. The current research aims to fill these gaps by assessing OPAACs in the M15 to M25 strength grades, which are widely utilized in practice, under different binder ratios and different levels of activators. Chemical activators of industrial grade were used for widespread applicability of binder systems. The proportions of mixes were designed in line with Indian standard mix design method. Such that one-part AABs could be equated with OPC and thus there was no need to develop new mix guidelines. The binders were pre-mixed and packaged for testing in a way similar to regular cement, which makes them easy to handle. Well-established testing methods were followed to determine the fresh and hardened concrete properties. Furthermore, ML models were applied for simultaneous predictions of the CS, FS and STS, which will enable fast formulation of mix

proportions, and guarantee quality control. Additionally, embodied carbon, and cost-effectiveness of OPAAC were assessed and compared with regular concrete. The current study provides detailed information about the application of OPAAC as a sustainable, economical, and practically feasible alternative to conventional concrete.

2. Materials and Methods

2.1. Materials

2.1.1. Solid Precursor and Activators

GGBS was employed as the aluminosilicate precursor in this research. Sodium metasilicate anhydrous (SMS) and calcium hydroxide (CH) mixture was employed as the solid alkaline activator. GGBS was procured from Astrra Chemicals, Chennai, India. Its chemical content was found through X-ray fluorescence (XRF) analysis, which showed that the mineral is predominantly made up of calcium with CaO constituting 42.7 % of the total oxide deposits as shown in Table 1. SMS, in granular form and of industrial grade, was obtained from Akshar Chem Exim Co. Pvt Ltd, Kolkata, India, and the supplier's specifications are listed in Table 1. CH, which was also purchased by the same supplier, was delivered as a fine industrial-grade powder with 96% purity of $\text{Ca}(\text{OH})_2$. The use of industrial-grade activators in this study aimed at enhancing the applicability and feasibility of the proposed binder system. Table 2 shows physical characteristics of precursor and activators. Additionally, the particle size distribution curve of GGBS and activators are presented in Figure 1. As can be noted, SMS was significantly coarser than CH and GGBS. Further, the fineness of CH was higher than that of GGBS.

Table 1. Chemical compositions of GGBS and SMS

Constituents	CaO	SiO ₂	Al ₂ O ₃	MgO	MnO	SO ₃	Fe ₂ O ₃	Na ₂ O	LOI
GGBS (%)	42.7	31.2	15.1	5.71	1.63	1.19	0.68	-	0.26
SMS (%)	-	≥48	-	-	-	-	-	50-52	-

Table 2. Physical properties of GGBS, SMS, and CH

Descriptions	Specific Gravity	Blaine fineness (m ² /kg)	Mean particle size (μm)
GGBS	2.85	420	13.85
SMS	1.05	151	575
CH	2.21	450	11.75

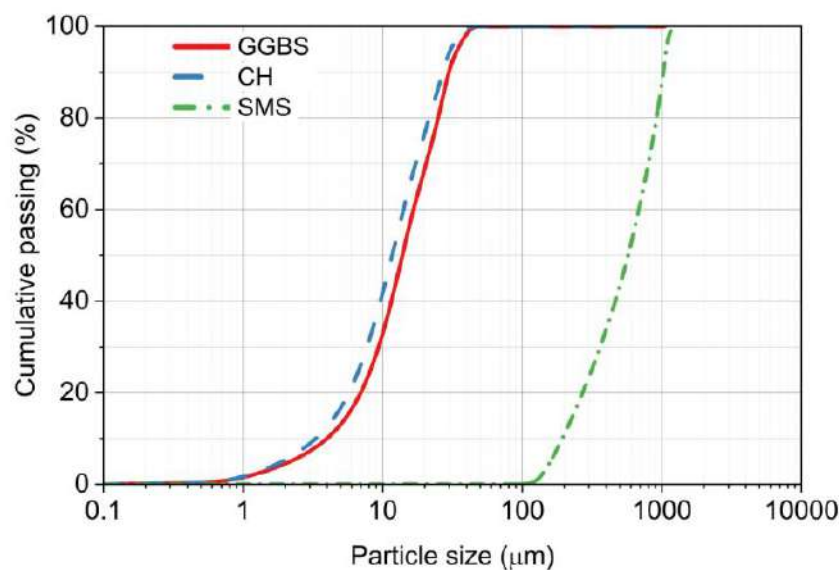


Fig. 1. Particle size distribution curves for GGBS, SMS, and CH

2.1.2. Binders

The current paper analyzed two different, one-part alkali-activated binders (AABs) designed to be used in concrete, with the content of each being 80% GGBS and 20% blended solid activators (SMS and CH). The first binder (AAB -1) had 14 % SMS and 6 % CH, and the second binder (AAB -2) had 12 % SMS and 8 % CH, the GGBS content was kept constant in both formulations. These ratios were fixed based on prior studies [17,26] and preliminary laboratory trials. For comparison purposes, a composite cement (CC) meeting the requirements of IS 16415:2015 [4] was also used. The CC, which is equivalent to 33-grade OPC, is a variant of blended cement made up of OPC, FA, and GGBS. According to the supplier's specification, CC was made up of 32.50% FA, 30.25% GGBS, and 37.25% OPC clinker, but excluding gypsum. The reasons for choosing the binder compositions, as well as the experimental work and performance assessment of the raw materials and binders, have been documented in the authors' previous work [46]. The binder properties are presented in Table 3. The performance analysis revealed that the physical properties of all binders were similar to those of 33-grade OPC and, hence, they were utilized in the design of concrete mixes.

Table 3. Physical properties of binders used in preparation of concrete mixes

Parameters	CC	AAB-1	AAB-2
Fineness (m ² /kg)	348±2	351±5	358±4
Specific gravity	2.84±0.04	2.64±0.02	2.58±0.02
Standard consistency (%)	32	29	32
Initial setting time (minutes)	134±6	55±5	76±2
Final setting time (minutes)	288±8	285±6	328±7
Soundness (mm)	1.2±0.1	2.0±0.2	1.8±0.3
Compressive strength (MPa), 28d	38.52±0.5	40.00±1.1	35.12±1.2

2.1.3. Aggregate and Water

The natural fine aggregate (NFA) used was local river sand, which conformed to Zone-III based on IS 383:2016 [47]. The nominal sizes of 20 mm and 10 mm of the natural coarse aggregates (NCA) were combined in a ratio of 60:40 based on laboratory trials to make a well-graded coarse aggregate of 20 mm nominal size, confirming to IS 383:2016 [47]. The physical attributes of NFA and NCA used in this study are provided in Table 4. Potable water that meets the criteria set out in IS 456:2000 [48] was used to produce all test specimens.

Table 4. Physical characteristics of fine and coarse aggregate

Descriptions		Fine aggregate	Coarse aggregate
Sieve Analysis	Size (mm)	% Passing	% Passing
	40	-	100
	20	-	90.5
	10	100	33.4
	4.75	100	0
	2.36	90.8	-
	1.18	76.2	-
	0.60	62.3	-
	0.30	15.2	-
	0.15	0	-
Nominal maximum size (mm)		4.75	20
Grading & Zoning		Zone-III	Graded aggregate
Specific gravity		2.64±0.01	2.80±0.02
Water absorption (%)		1.00	0.50
Compacted bulk density (kg/m ³)		1621±18	1606±22
Fineness modulus		2.56	6.76

2.2. Mix Proportioning and Sample Preparation

The mixes were formulated to produce three different grades of concrete (M15, M20, and M25), commonly used for residential buildings and general construction purposes, as per IS 10262:2019 [49]. The mixes were designed without using any chemical admixture. Thus, a low workability of concrete mix, target with slump value up to 50 mm, was selected during the mix design. During mix proportioning, the target CS (f'_{ck}) at 28 d for M15, M20 and M25 grades were adopted as 20.775, 26.60, and 31.60 MPa, respectively. It was set higher than the minimum characteristics CS (f_{ck}) of concrete as per the codal provision. Concrete mixes of each strength grade were produced using three different binders- AAB-1, AAB-2, and CC. The same proportions of ingredients and water-to-binder (w/b) ratios designed for AAB-1 binder were maintained across all the other binders of the same strength grade. All the concrete mixtures showed the intended workability, having slump values of 20–55 mm, during the laboratory trials. The common mixture ratios (binder: NFA: NCA) and w/b ratios were the following: M15- 1:1.85:3.64 (w/b = 0.55), M20- 1:1.58:3.07 (w/b = 0.48), and M25- 1:1.40:2.78 (w/b = 0.44). Table 5 shows the proportions of mixes in detail. Within the nomenclature, CC, A1 and A2 represent composite cement, AAB-1 and AAB-2 respectively, with the last two figures representing the grade of concrete strength.

Table 5. Mix proportions of different concrete mixtures used in the study

Mix ID	GGBS (kg/m ³)	SMS (kg/m ³)	CH (kg/m ³)	CC (kg/m ³)	NFA (kg/m ³)	NCA (kg/m ³)	Water (kg/m ³)	W/B ratio
CC15	-	-	-	338.18	625.62	1230.30	186	0.55
A115	270.54	47.35	20.29	-	625.62	1230.30	186	0.55
A215	270.54	40.58	27.06	-	625.62	1230.30	186	0.55
CC20	-	-	-	387.50	614.01	1188.40	186	0.48
A120	310.00	54.25	23.25	-	614.01	1188.40	186	0.48
A220	310.00	46.50	31.00	-	614.01	1188.40	186	0.48
CC25	-	-	-	422.72	591.66	1175.70	186	0.44
A125	338.18	59.18	25.36	-	591.66	1175.70	186	0.44
A225	338.18	50.73	33.81	-	591.66	1175.70	186	0.44



Fig. 2. Freshly prepared concrete mixture, the slump cone test, and prepared test specimens

The concrete was made by first dry mixing the required quantities of binder, NFA, and NCA for 2 min in a laboratory pan mixing machine. After this, the measured quantity of water was added as per the prescribed w/b ratio as mentioned earlier. The mixing was performed for further 3-5 min till the mixture attained a homogeneous consistency. All the mixing and casting were done in a laboratory setting, which was maintained in range of 27 ± 2 °C temperature and $65 \pm 5\%$ relative humidity. The concreting process was followed same as the standard procedure generally adopted for conventional concrete. Figure 2 shows the freshly prepared mixture, the slump test on the fresh mix, and the prepared cubic, cylindrical, and prismatic moulds. Concrete was immediately poured into different moulds: cubic moulds of 150 x 150 x 150 mm, cylindrical moulds of 150 mm diameter x 300 mm height, and prismatic moulds of 500 x 100 x 100 mm. A table vibrator was used to compact each mould for removing entrained air and achieving dense specimens. The compacted mould was, then, covered with wet cloths and left in a laboratory atmosphere for a period of 24 ± 2 h prior to demolding. After demolding, the test samples were submerged in a temperature-controlled curing tank, which was regulated at 27 ± 2 °C. The samples were cured in the curing tank until the time of test.

2.3. Experimental Methods

2.3.1. Test On Fresh and Hardened Properties

The workability of the prepared mixtures was measured using the slump test according to the IS 1199 (Part 2): 2018 [50]. For slump cone test, a common reference mixes with (binder: NFA: NCA) ratio of 1:1.40:2.78, referring M25 grade mix, was selected in order to explore the influence of w/b ratio on the workability. The w/b ratio was altered stepwise (0.40 to 0.60) and the slump tests were conducted on each of the w/b ratios to establish the effects of the water content in the fresh concrete mixture on the workability. For every mix and w/b ratio, three slump tests were performed, and the average slump value was recorded and reported.

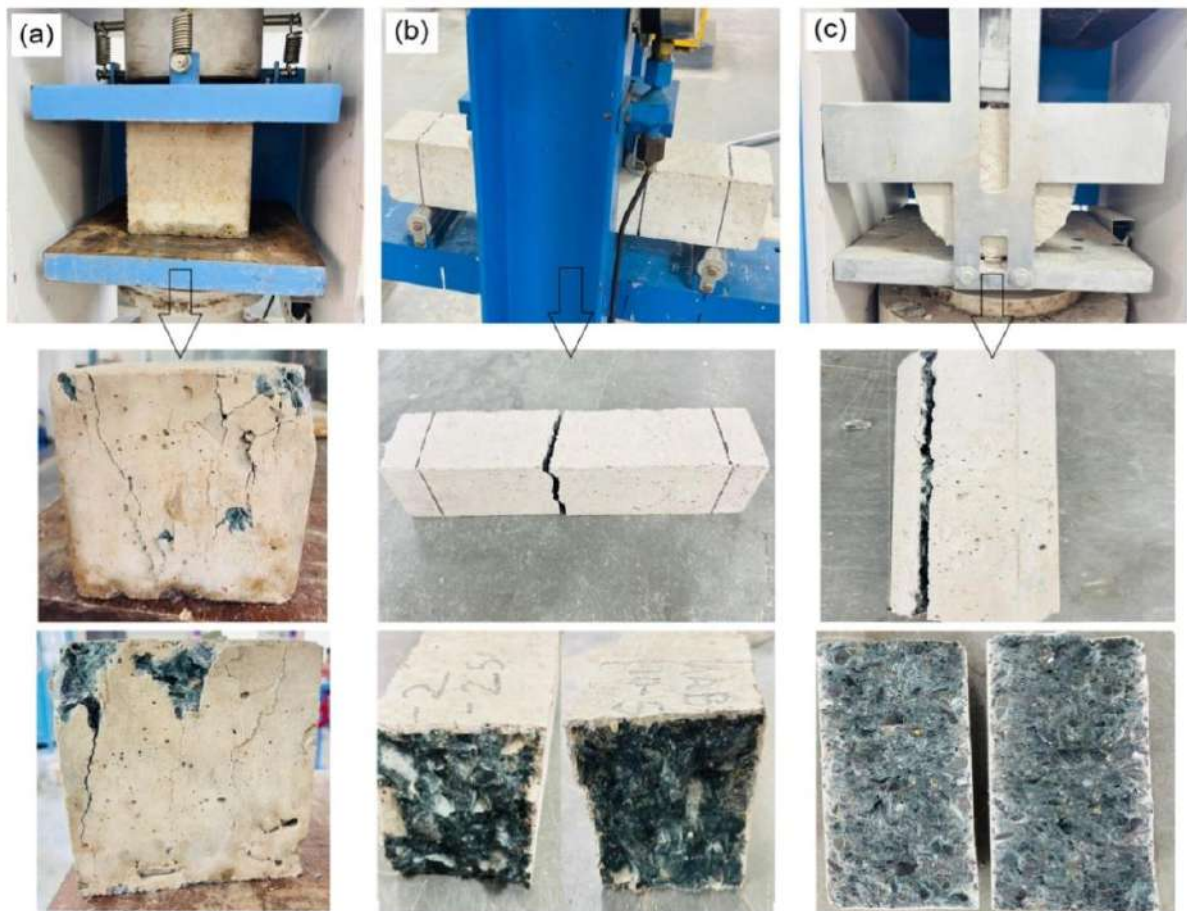


Fig. 3. Test set-up and failure mechanisms of concrete specimens

The CS, FS and STS tests were conducted to identify the mechanical properties of hardened OPAAC as per the methodology suggested in IS 516 (Part 1/Sec 1): 2021 [51]. All experiments were also carried out in a laboratory environment at 27 ± 2 °C temperature and $65 \pm 5\%$ relative humidity. Specimens were tested after 3, 7, and 28 d of water curing to determine the development of strength with time. CS and STS of all specimens were obtained through a digital compression test machine with the associated accessories, and FS was obtained on prismatic specimens through a four-point flexural set-up using a flexural testing machine. The test set-up and failure mechanism of the specimens were akin to that of traditional concrete as shown in Figure 3. Three replicate specimens were tested at every curing period with each mix proportion and mean values were recorded. A total of 81 cube specimens were subjected to CS tests, 81 prismatic beam specimens to FS tests, and 81 cylindrical specimens to STS tests, putting into consideration all mix proportions and the curing periods.

2.4. Machine Learning Methodology

The present modeling study was performed on the basis of the experimental strength results, which were conducted on 09 different mixes of concrete (Table 5). Parameters recorded include CS, FS and STS at the curing age of 3, 7, and 28 d. There were 81 specimens that were tested, inclusive of all curing ages, in each strength test. Therefore, 81 data point featuring 08 input variables (GGBS, SMS, CH, CC, NFA, NCA, w/b ratio, and curing time) and 03 output variables (CS, FS, STS) were prepared for modeling study. Table 6 shows the input (feature) and output (target) variables that were utilized in the modeling with their specifications. Preprocessing of the experimental data was done before modeling to eliminate possible inconsistencies. The data were preprocessed followed by the division of the data into eight input features and three output targets. In order to achieve an objective evaluation of model performance, the data were randomly divided into two sets- 80% training set for model development and 20% testing set for evaluation.

Table 6. Dataset properties of input and output variables

Parameter	Minimum	Mode	Median	Mean	Maximum	Standard deviation
GGBS (kg/m ³)	0	0	270.54	204.16	338.18	147.04
SMS (kg/m ³)	0	0	46.5	33.18	59.18	24.1
CH (kg/m ³)	0	0	23.25	17.86	33.81	13.25
CC (kg/m ³)	0	0	0	127.6	422.72	182.69
NFA (kg/m ³)	591.66	625.62	614.01	610.43	625.62	14.18
NCA (kg/m ³)	1175.7	1230.3	1188.4	1198.13	1230.3	23.47
W/B ratio	0.44	0.55	0.48	0.49	0.55	0.05
Curing time (days)	3	3	7	12.67	28	11.03
CS (MPa)	8.67	10.6	18.2	19.12	34.2	6.7
FS (MPa)	1.01	2.98	3.22	3.27	6.5	1.37
STS (MPa)	0.7	0.79	1.82	1.85	3.62	0.76

Similar to prior studies in the field of ML application in the prediction of concrete properties [37–40,44,45], five supervised learning models were used: Ridge Regression (RR), ElasticNet (EN), Support Vector Regression using radial-basis-function kernel (SVR), Gradient Boosting (GB) and Random Forest (RF). Simultaneously, since the task involved predicting several output variables, multi-output regressor was employed to generalize single-output regressors to multi-target prediction. In algorithms that can make use of feature scaling (RR, EN, SVR), the input variables in a preprocessing pipeline were standardized with a Standard Scaler to provide an appropriate normalization. The model performance was measured through the “repeated K -fold cross-validation”, where 5-folds were repeated twenty times to obtain a robust performance estimate. Evaluation metrics of the models were the “coefficient of determination (R^2)”, “root-mean-square error (RMSE)” and the “mean absolute error (MAE)” [37]. After the procedure of cross-validation (CV), every model was again trained on the entire training set and then tested on the held-out test subset in order to have an unbiased estimate of the predictive generalization. An over-fitting score, which is the difference between the mean training R^2 and the mean CV R^2 was calculated to measure

generalization capacity and detect the possibility of over-fitting. In order to find the share of individual input variables to the strength predictions, feature-importance analysis was conducted on the best model. The best model was applied to hyperparameter optimization using GridSearchCV in order to find the best parameters of learning rate, tree depth, and the number of estimators. This was followed by the utilization of the Leave-One-Out Cross-Validation (LOOCV) of un-tuned and tuned versions of the top model on the complete dataset. This furnishes an additional evaluation of the generalization especially in the case of a small amount of data. The resulting optimized model was ultimately used to provide predictions of new input parameters demonstrating its practical use.

3. Results and Discussion

3.1. Workability

The workability of concrete mix contributes significantly to the abilities of the fresh mixture besides the strength and long-term properties of concrete. Figure 4 displays the slump values of all concrete mixes corresponding to different w/b ratio (0.40–0.60). The results show that the values of slump in all concrete mixes were in the range of 10–85 mm, categorized as very low to medium workability. AAB-1 concrete mixes outperform CC mixes as well as AAB-2 mixes in workability at each level of w/b ratio. The increase in w/b ratios enhanced the slump values by lowering the frictional forces between the particles and thereby, enhancing the flowability of the mix [29]. It is relevant to add that A125 mix had slightly higher slump values in comparison to control mix, CC25 and A225 concrete mix. This is due to greater SMS content (14%) and lower CH (6%) in binder composition of A125 concrete mix. The SMS/CH activator ratio is found to be directly related with the workability of OPAAC mixes. The lower silica modulus of SMS used ($\text{SiO}_2/\text{Na}_2\text{O} \sim 0.9$) has helped in improving the workability by faster dissolution of SMS particles in the fresh state of OPAAC mixes [10]. In a previous work, Oderji et al. [20] investigated FA and GGBS- based OPAAC systems. They reported that SMS anhydrous provided higher flowability and enhance strength compared to other solid activators used. On the contrary, the increase in CH content tends to reduce the fresh workability of OPAAC, which may be attributed to the particle packing influence of finer particles of CH in fresh mixture [24]. These finding collectively suggests that OPAAC has similar performance to CC concrete in terms of workability.

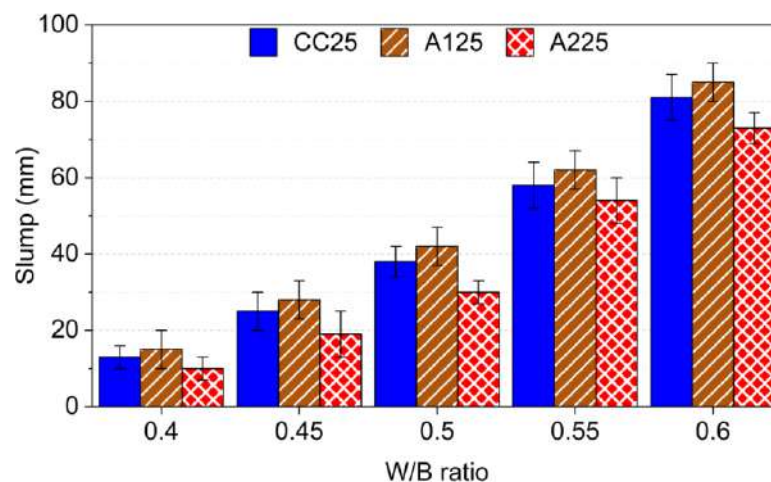


Fig. 4. Slump values of concrete mixes based on different w/b ratios

3.2. Mechanical Performances

3.2.1. Compressive Strength

The CS of concrete is the most essential characteristic on which the quality, effectiveness and adaptability of concrete mixes relies upon. Figure 5 illustrates the CS of all designed concrete mixtures, measured at 3, 7 and 28 d of curing ages. In all mixtures, the CS increased gradually with

age upon continuous hydration under water curing. In every concrete grades, the mixes, prepared using all three binders, exhibited CS well above f_{ck} values, meeting the minimum criteria. In AAB-1 and CC concretes, the mixes not only met but also surpassed f'_{ck} values in all the respective grades. But, AAB-2 concrete mixes had values that were close but fell short of f'_{ck} in all the grades. The SMS/CH activator ratio is found to be directly related with the CS of OPAAC. Higher the SMS content tends to enhance the strength development. AAB-1 concrete mixes always exhibited the highest CS at all the ages compared to both of CC and AAB-2 concretes. The highest CS value obtained for AAB-1 mixes in M15, M20 and M25 grades was 22.38, 27.88 and 32.72 MPa, respectively. The reason may be the higher SMS (14%) and lower CH content (6%) in binder composition of AAB-1. The elevated SMS content resulted in the rapid dissolution of GGBS and yields to the accelerated production of C-A-S-H gels, which are regarded to be the major strength-providing entities in GGBS-based OPAAC [17,18].

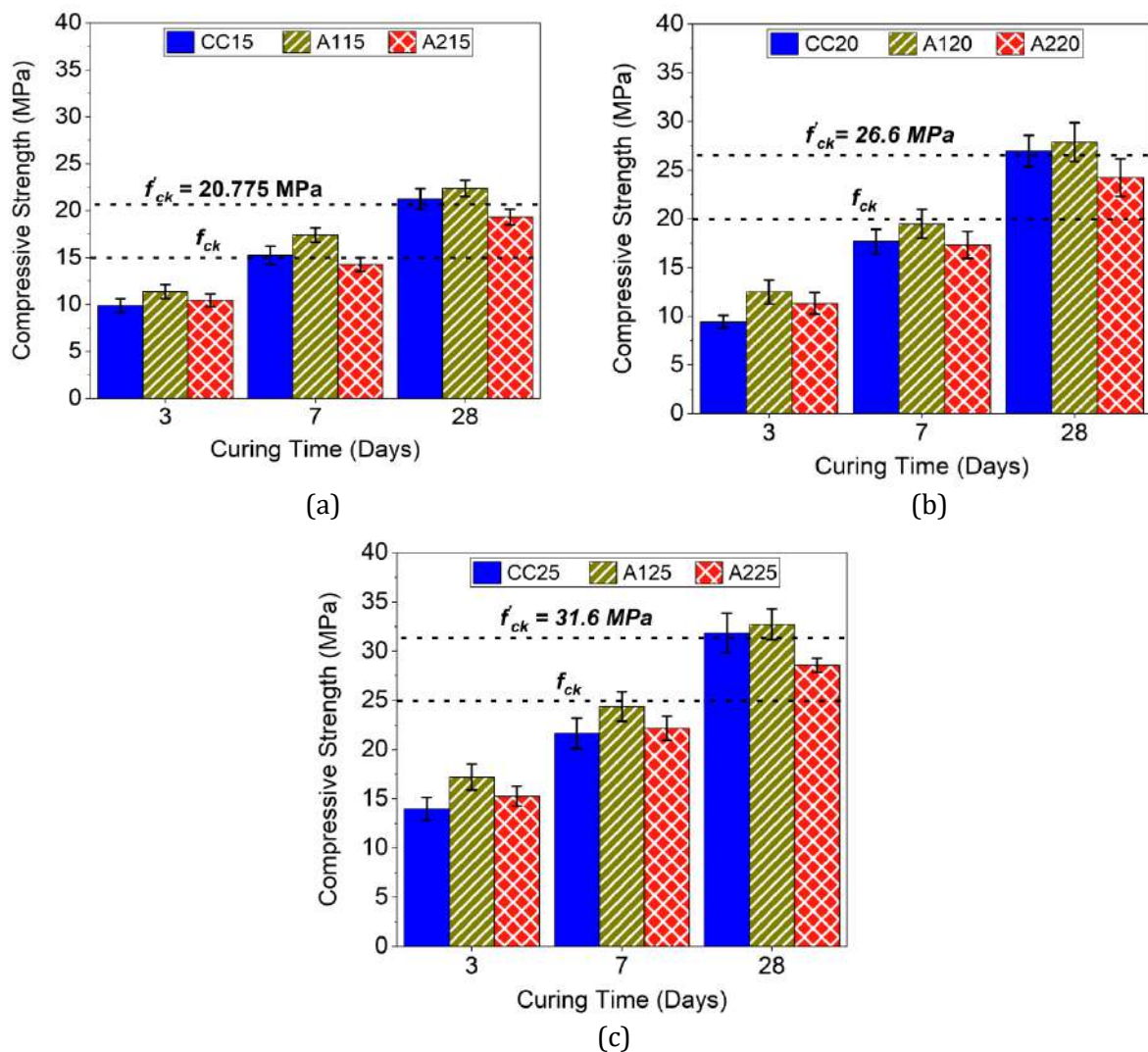


Fig. 5. Compressive strength of M15, M20 and M25 mixes

Additionally, the CH particles present in the binder also contributed to gel formation by making the calcium soluble and, thus, it reacts with the dissolved SiO_2 and Al_2O_3 to achieve a denser and more interconnected pore structure [24]. This did not only enhance the values of CS but also produced test specimens free from common defects such as efflorescence and cracking, as observed from their visual appearance (Figures 2f and 3), indicating improved matrix integrity. It is noticeable that SMS content plays the major influential role in strength developed and complemented by auxiliary activator CH to produced dense concrete products. The strength development rate of OPAAC at early age was more compared to CC concrete. CC concrete reached approximately 42 % and 68 % of its 28 d CS at 3 and 7 d, respectively, whereas OPAAC mixes reached approximately 50 % and 74 % of its 28 d CS, respectively. This is given the fact that despite being very reactive, GGBS

dissolves well with the alkali activators which are known to be catalysts in the initial development of C-A-S-H gels. In a similar research, Ma et al. [22] presented that GGBS-based OPAAC systems show a rapid development of strength at an initial curing age (3 and 7 d), and a decelerated strength development at a higher age (beyond 28 d). Overall, these findings show the potential of OPAAC to develop M15, M25 and M25 grades concrete, applicable to various concreting applications.

3.2.2. Flexural Strength

The FS of concrete is a basic mechanical characteristic utilized to determine its tensile resistance to rupture under flexural stresses. In case of normal concrete, the strength in tension is estimated by the relation between FS and CS that has been given in the IS-456:2000 code [41] as $f_{cr} = 0.7\sqrt{f_{ck}}$. Figure 6 shows the FS of all prepared concretes at curing ages of 3, 7 and 28 d. Similarly, to CS, FS increased with time of curing, due to the ongoing hydration under water curing. All the concrete mixes had reached the desired FS values after 28 d, which is 2.71, 3.13, and 3.50 MPa with M15, M20 and M25 grades, respectively. The FS of AAB-1 concrete was the highest of the mixes, recording the highest values of FS at 4.02, 5.08 and 6.20 MPa for M15, M20 and M25 grades, respectively. These values of AAB-1 concrete were significantly higher than the f_{cr} value, probably as the result of the high-quality binder paste bonding behavior between the binder paste and the aggregates [30].

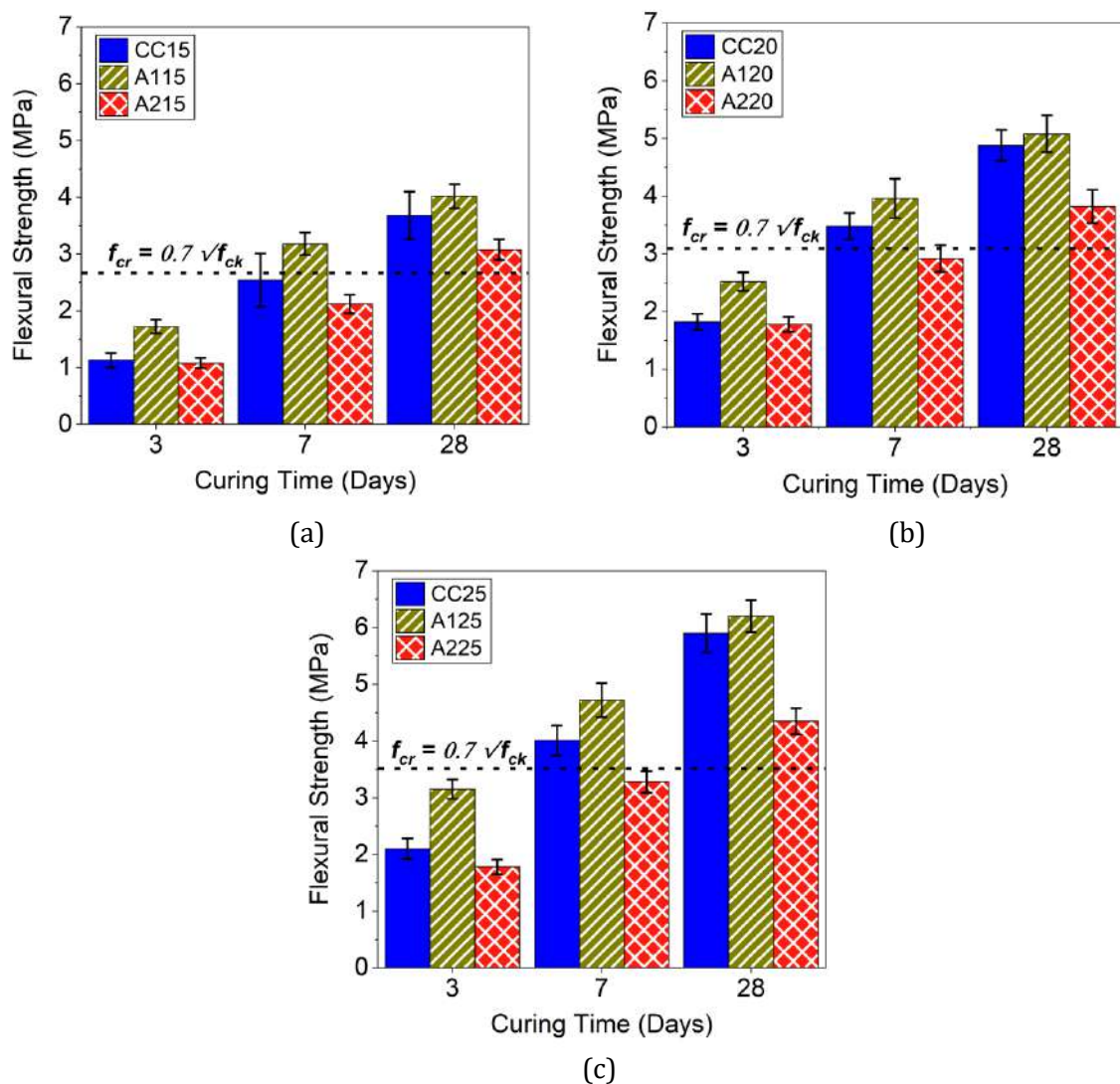


Fig. 6. Flexural strength of M15, M20 and M25 mixes

Figure 3b clearly shows the test specimen failure mode during flexural loading, which shows that the specimen is mostly intact after failure. The high FS values of AAB-1 concrete mixes can be explained by high SMS content (14 %) of the binder mixture composition which predisposes rapid

alkali-activation and creation of high density, interlocking C-A-S-H gels network [18]. Similar to behavior of CS, SMS content stands out as a leading influential factor that is moderated by CH content in the development of FS. When the SMS/CH ratio increases, FS is increased and when the ratio decreases the FS decreases. Lower FS values were observed using the reduced SMS content (12 %) in the AAB-2 concrete mixes than the CC and AAB-1 concretes. The FS/CS ratios at 28 days of curing were 0.17–0.18 in CC, 0.18–0.19 in AAB-1 and 0.15–0.16 in AAB-2 across all grades. These ratios are higher than the standard range of f_{cr} which is about 10–15% of f_{ck} and the FS values were more than that calculated by the relation $f_{cr} = 0.62\sqrt{f_{ck}}$ as specified by ACI 318-05 [52]. Similar to standard concrete, the FS of OPAAC is directly proportional to its CS; the mixes with higher CS values achieve a higher FS and, conversely, as well. On the whole, flexural behavior of OPAAC shows its potential to be used in various construction applications like casting of beams, slabs, and pavements.

3.2.3. Split Tensile Strength

The STS is another vital parameter utilized to measure the direct tension capacity of concrete to withstand cracking. Figure 7 displays the STS of all prepared mixes at 3, 7 and 28 d of water curing. The STS was also a reflection of the overall trend of CS, where strength gained progressively as the curing continued. IS 456: 2000 code does not imply any definite relation to estimate STS, thus, the standard relation to estimate STS on 28 d, $f_{ct} = 0.56\sqrt{f_{ck}}$, as suggested by ACI 318-05 [46], was used.

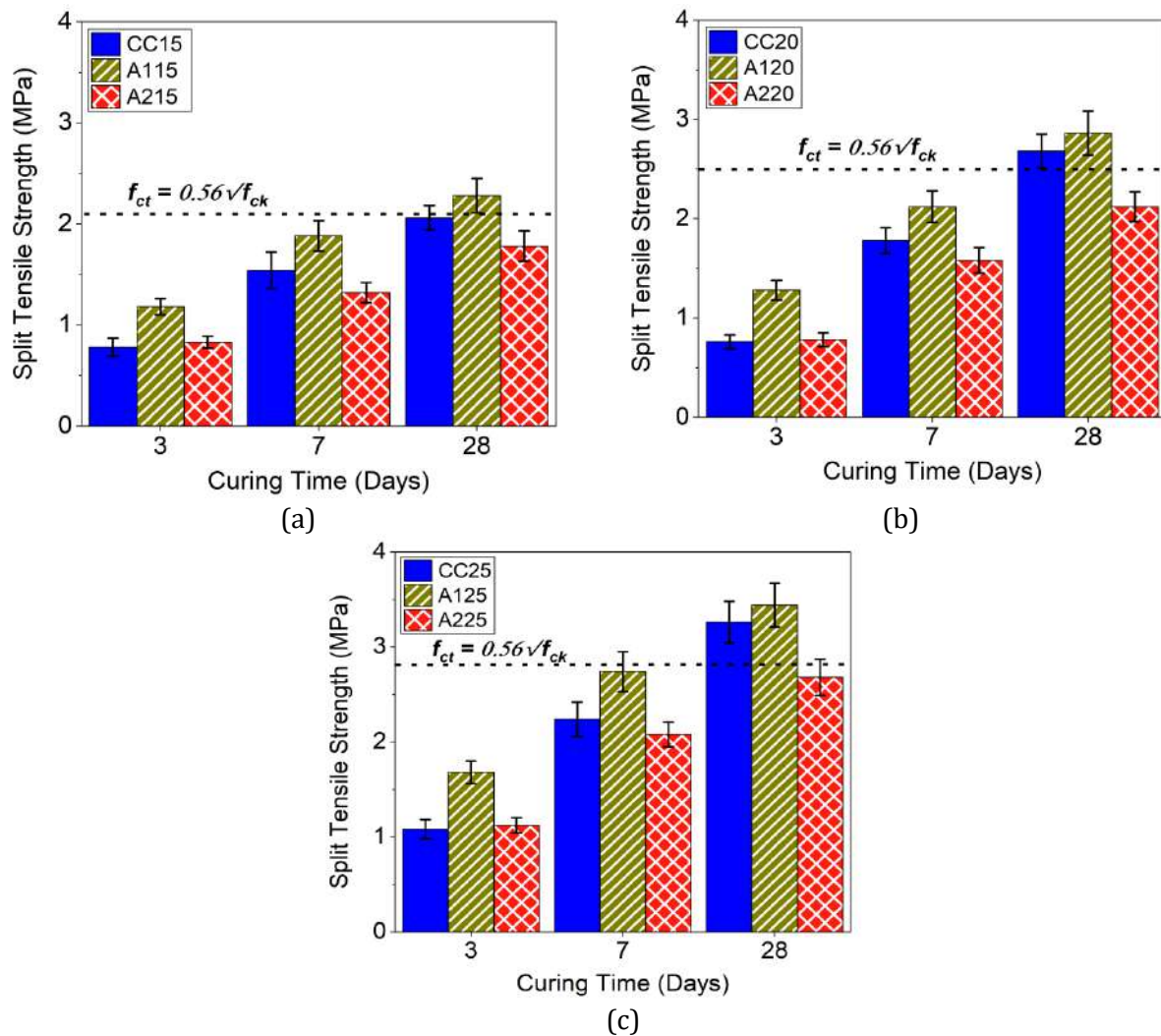


Fig. 7. Split tensile strength of M15, M20 and M25 mixes

These findings show that the AAB-1 and the CC concrete mixes meet the necessary f_{ct} of 2.17, 2.50, and 2.80 MPa at 28 d of water curing for M15, the M20 and the M25 grades, respectively. In AAB-2 concrete mixes, the observed f_{ct} values fall below the thresholds. However, they have STS values

above the general benchmark of 8 % of f_{ck} across all grades. AAB -1 concrete mixes recorded maximum STS values of 2.28, 2.86 and 3.44 MPa at the M15, M20 and M25 grades. The higher values were observed in OPAAC as a result of the good bonding behavior between the binder paste and aggregates [30]. As seen in Figure 3c, failure mode of the split tensile test could be very clear and shows that even upon failure; the specimen is left relatively intact.

In line with the behaviors of CS and FS, the high STS values of AAB-1 concrete mixes may be explained by the high level of the SMS content (14%) in its binder composition that facilitates the rapid alkali-activation and occurrence of a dense and interlocked C-A-S-H gels network [18,31]. Similar to CS and FS, SMS content is a decisive factor that is moderated by CH content in the development of STS. Increasing the SMS/CH ratio increases STS and a decrease result in lower STS values; to this end, the lower SMS content of AAB-2 concrete mixes (12%), provided lower STS values compared to CC and AAB-1 concrete mixes. The ratio of STS/CS between 28 d of curing among all grades was 0.09 -0.10 in CC, 0.10-0.11 in AAB-1, and 0.08-0.09 in AAB-2. Simultaneously with the traditional concrete, STS of OPAAC is directly proportional to its CS; higher CS values of mixes yield higher STS and vice versa. In sum, STS performance of OPAAC is evidenced to have superior resistance to cracking when subjected to tensile stresses, therefore making it a suitable choice for the concrete structures.

3.3. Machine Learning Based Strength Prediction

A comprehensive ML framework was used to forecast multiple concrete strength variables simultaneously. Such that, a single model could be used to analyze CS, FS, and STS with respect to material compositions and curing conditions.

3.3.1. Feature and Target Correlations

Figure 8 presents the feature and target correlation heatmap. In feature-feature correlation, the heatmap shows a number of strongly significant positive and negative relationships between the input features. GGBS, SMS and CH (Ingredients for one-part AAB) have high positive correlations with each other (0.93 to 0.99) and negative correlations with CC (-0.95 to -0.98). This observation shows that there is a purposeful mechanism of substitution that is between the two systems of binders. Independent variation is observed when the one-part AAB (made of GGBS, SMS, and CH) is used in place of CC. Such strong relationships are expected to occur in mixture designs where two systems of binder compete with fixed total binder content, and this poses inherent interdependence of compositions [41]. Similarly, there is a strong inter-correlation of NFA and NCA, and the correlations of these variables to the w/b ratio (0.89 to 0.99) are also high, which shows that aggregate and water content correlate with each other to maintain concrete properties. Curing time is showing no relation with the material compositions, as it is an independent variable rather a compositional variable. To tackle such complex correlations between features, five different models were used in this research for the prediction the strength of concrete.

In feature to target correlation, curing time has the greatest positive influence on CS (0.79), STS (0.74), and FS (0.73), as indicated in the feature-target correlation matrix, thus proving that long curing periods have a significant positive impact. The aggregates and w/b ratio have moderate negative relationships (-0.36 to -0.46), which means that their ratios should be well balanced to produce the best strength. GGBS, SMS and CH (making up the one-part AAB) and CC exhibit lesser lineal correlations, indicating that their roles may rely on nonlinear or synergistic interactions among the binder system components, but participate in a main role in the all-round performance of the materials.

3.3.2. Model Comparison

The relative performances of the five ML models show distinct differences in prediction capability of multi-output strength. Table 7 presents the comparative results of RR, EN, SVR, RF, and GB on the training, validation, and testing data using the R^2 , RMSE, and MAE as the evaluation measures. The most accurate predictive models in the training dataset were those based on ensembles (RF and GB). GB achieved the highest training (R^2 score of 0.9797) and minimum RMSE (0.416 MPa) and MAE (0.3431 MPa) followed by RF ($R^2=0.9739$).

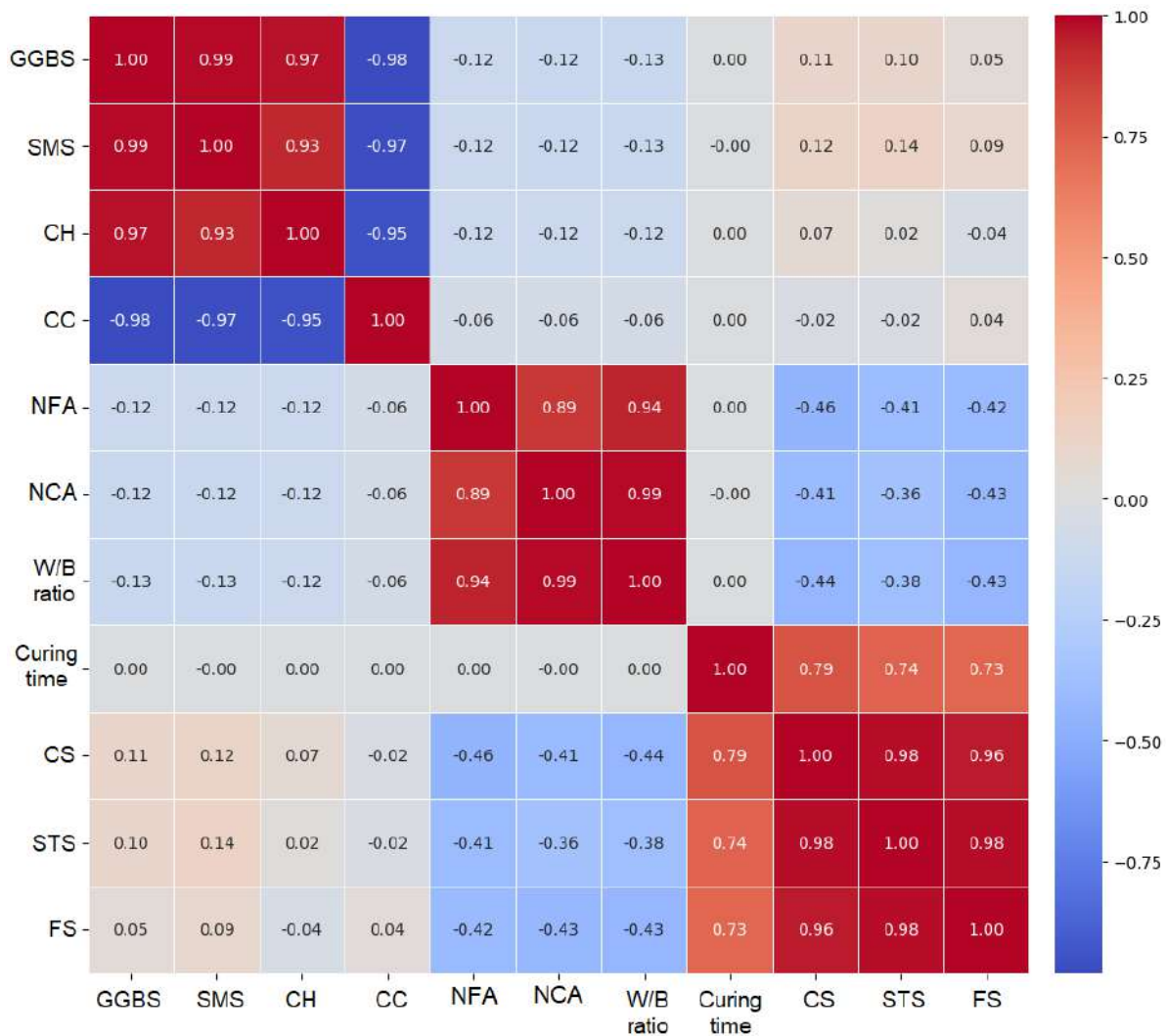


Fig. 8. Feature and target correlation heatmap

Table 7. Comparative results of models on training, validation and testing data

Model	Training data			Validation data			Testing data		
	R ² Score	RMSE (MPa)	MAE (MPa)	R ² Score	RMSE (MPa)	MAE (MPa)	R ² Score	RMSE (MPa)	MAE (MPa)
RR	0.8259	1.1052	0.9269	0.7444	1.2145	1.0285	0.8386	1.5333	1.0437
EN	0.7671	1.1872	0.9821	0.6772	1.295	1.0811	0.7409	1.7018	1.1621
SVR	0.9300	0.8480	0.6415	0.8241	1.1778	0.9997	0.9176	1.4478	0.9145
RF	0.9739	0.4520	0.3742	0.9096	0.7618	0.6314	0.9560	0.8063	0.4764
GB	0.9797	0.4160	0.3431	0.9219	0.7326	0.6184	0.9675	0.7525	0.4360

The capability of good fitting was also shown by SVR ($R^2 = 0.9300$), but not the linear models RR ($R^2 = 0.8259$) and EN ($R^2 = 0.7671$). The high accuracy of GB and RF in training suggests the fact that both models have very strong ability to follow nonlinear trends in the dataset. The results of the validation indicate a weak deterioration in the performance of all models, as expected in the measurement of the generalization ability.

It was observed that GB had the best validation ($R^2 = 0.9219$) and minimum RMSE (0.7326 MPa) which demonstrated that the predictive performance remained stable and reliable across cross-validation folds. RF also exhibited good validation ($R^2 = 0.9096$). On the other hand, SVR had a greater decrease in the R^2 between training and validation (0.9300 to 0.8241) indicating a relatively high sensitivity to the variation of data. The linear models still showed inferior validation results with EN providing the poorest results ($R^2 = 0.6772$).

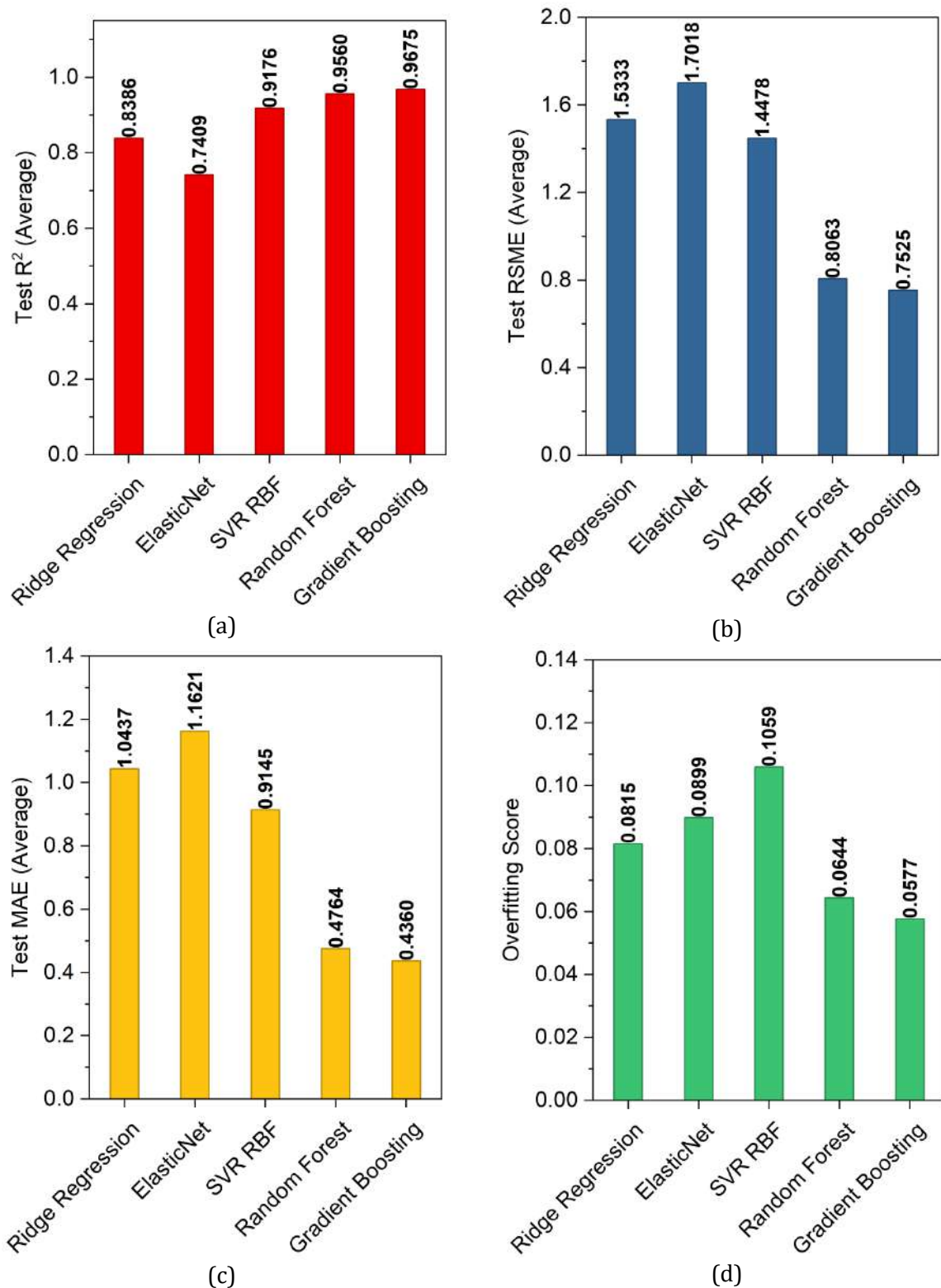


Fig. 9. Comparison of overall test performance across all models

The excellence of ensemble techniques is also supported by testing performance, as displayed in Figure 9. GB has the highest test R^2 of 0.9675 and the smallest RMSE of 0.7525 MPa and MAE of 0.436 MPa. Hence, it exhibits good predictive power and generalization. RF ranked close ($R^2 = 0.9560$) and SVR performed mediocrity ($R^2 = 0.9176$) with greater error measures. RR and EN showed relatively lower test accuracy as they have a small ability to represent complex nonlinear relations. Overfitting analysis further validated these results (Figure 9d). SVR showed the highest overfitting score of 0.1059, which reflects a larger gap between training and validation metrics. In contrast, GB showed the lowest overfitting value of 0.0577, suggesting balanced learning behavior.

and effective generalization without excessive memorization of training data. In general, the outcomes show that ensemble ML methods, especially GB, are more effective in terms of predictive ability in all data partitions. The comparatively low gap between training, validation, and testing measurements of GB and RF suggests that the model complexity can be under control and the generalization of the model can be high, but the high-performance difference between SVR and the linear models suggest the relatively reduced robustness.

Table 8 presents the evaluation metrics of all the assessed models in terms of the test performance with regard to CS, FS and STS. In CS prediction, GB with $R^2 = 0.9669$ had the greatest predictive power with the lowest values of RMSE (1.2677 MPa) and MAE (1.0065 MPa). RF was also close behind ($R^2 = 0.9627$). The SVR demonstrated moderate results ($R^2 = 0.8742$) in comparison with RR and EN that generated relatively lower accuracy and higher error rates. Ensemble methods were once again the best in the estimation of FS. The best R^2 of 0.9639 with the least error measures was achieved by GB and the second best R^2 of 0.9486 was by the RF. SVR also gave a very good performance ($R^2 = 0.9362$) which is much higher than that of the linear models. EN posted the lowest result of this target ($R^2 = 0.6431$). Similarly, in predicting STS, GB performed the best with ($R^2 = 0.9715$) followed by RF (R^2 of 0.9566) and SVR (R^2 of 0.9424). The linear models are also associated with relatively smaller values of R^2 and larger values of error. On the whole, the results show that ensemble methods, in particular, GB and RF, always have better predictive accuracy by all measures of individual strengths. Compared to linear models, SVR is still competitive in FS and STS, but it has a low capacity to model non-linear and complex relationship inherent to the features.

Table 8. Target-specific prediction accuracy of all models

Model	CS test			FS test			STS test		
	R^2	RMSE	MAE	R^2	RMSE	MAE	R^2	RMSE	MAE
	Score	(MPa)	(MPa)	Score	(MPa)	(MPa)	Score	(MPa)	(MPa)
RR	0.8650	2.5603	2.2595	0.8063	0.3714	0.3231	0.8445	0.6006	0.5484
EN	0.8387	2.7989	2.4247	0.6431	0.5041	0.4070	0.7410	0.7752	0.6546
SVR	0.8742	2.4717	2.2608	0.9362	0.2131	0.1717	0.9424	0.3654	0.3111
RF	0.9627	1.3465	1.0556	0.9486	0.1913	0.1407	0.9566	0.3171	0.2330
GB	0.9669	1.2677	1.0065	0.9639	0.1602	0.1192	0.9715	0.2570	0.1824

3.3.3 Performance of The Best Model

The comparative analysis found GB to be the best model among others, and, hence, it was selected to be refined further. To improve its estimation accuracy, hyperparameter tuning was used and the findings are tabulated in Table 9. The predictive accuracy of the tuned GB model had a small increase in comparison with the original one. Test R^2 slightly rose to 0.9684 out of 0.9675, whereas RMSE fell to 0.7412 MPa out of 0.7525 MPa with MAE declining to 0.4353 MPa out of 0.4360 MPa. Moreover, the overfitting measure dropped to 0.0575 by a small margin compared to 0.0577, indicating a slight but significant increase in the level of generalization ability. Though the measured improvements are small, it is a pointer to the fact that the original model had already been highly optimized and stable.

GridSearchCV identified the optimal hyperparameters as estimator learning rate = 0.1, max tree depth = 3, number of estimators = 50 and the best CV Test R^2 score was 0.9222. The overfitting score of tuned GB model was the difference between CV train R^2 of 0.9797 and CV test R^2 of 0.9222. The optimization process was useful in minimizing prediction error and slightly improving robustness without a significant increase in overfitting. In turn, the tuned GB model gives the highly precise and reliable predictions on the estimation of strength.

In the final validation of model, LOOCV showed that tuned GB model had slightly better RMSE and MAE (0.5467 MPa vs 0.5473 MPa in both) compared to the untuned model. These minor enhancement in RMSE and MAE for the tuned GB model suggested that the tuning process indeed led to a more precise and generalizable model performance, even when tested with the most granular cross-validation process. The final model did very well for predicting various strength targets based on the provided material compositions and curing conditions.

Table 9. Hyperparameter tuning effect on gradient boosting model

Descriptions	Test R ² Score	Test RMSE (MPa)	Test MAE (MPa)	Overfitting Score
Untuned GB model	0.9675	0.7525	0.4360	0.0577
Tuned GB model	0.9684	0.7412	0.4353	0.0575
Change in metrics	+0.0009	-0.0113	-0.0007	-0.0002

Furthermore, the feature importance analysis of the best model is displayed in Figure 10. The importance of the feature analysis of the GB model indicated that curing time had the highest predictive power since it showed 69.16% of overall performance. This dominant significance implies that the target variable is very sensitive to the period of curing; this coincides with the established behavior of time-dependent strength development in concrete [30,39]. Out of the compositional parameters, SMS (8.23 %) and NCA (8.12 %) had significant contribution. The rest had a lower contribution, namely NFA (4.68 %), w/b ratio (4.51 %), and CH (4.11%). In contrast, CC (1.02 %) and GGBS (0.17%) indicate little influence due to nature of current mix compositions in the present modeling framework.

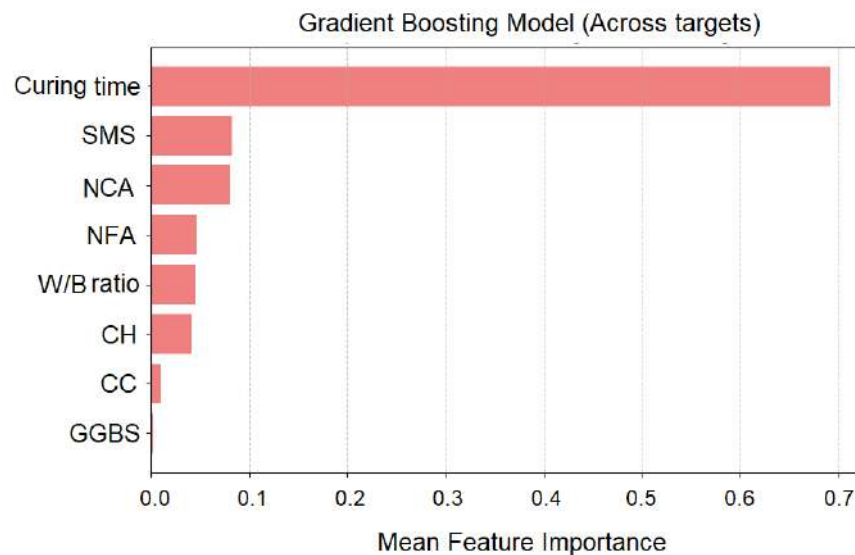


Fig. 10. Mean feature importance for the best model across all targets

Figure 11 present the actual vs predicted strength along with prediction errors of tuned GB model. The model showed excellent predictive power, with respect to all the three mechanical strength as indicated by high score of R². The tuned model had R² of 0.9679, 0.9716 and 0.9656 thereby explaining more than 96 percent of the variance in the experimental scores. FS was the most predictive of the three targets, followed closely by CS and STS, which verify the predictability of the model using the different mechanical responses. The strength of the model is also supported using the error analysis. On CS, the model yielded an RMSE of 1.2480 MPa and an MAE of 1.0072 MPa, which demonstrates a slight average deviation of the model generated and actual values in terms of an average strength range of the entire spectrum. In the case of FS, predictions produced RMSE of 0.2567 MPa and MAE of 0.1816 MPa. Similarly in STS, RMSE of 0.1566 MPa and MAE of 0.1172 MPa, which indicates a very accurate prediction with a very limited variation. which is equal to remarkably small absolute prediction errors.

The low and stable values of RMSE and MAE in all target variables and the high values of R² indicate that the model does not only capture the variance effectively but also maintains a good level of numerical accuracy. The strength of the model further is proven by a close analysis of the errors of predictions. In CS, most of the deviation lies in the range of about ± 2 MPa even at high strength levels of over 30 MPa and this shows consistency in performance at both low and high strengths. The FS predictions are tend to exhibit low deviations of ± 0.4 MPa, whereas the error on STS is about

± 0.2 MPa. Even though some slight overestimations are noted at some higher levels of strength, the errors are symmetrically distributed and they do not indicate any systematic bias or instability.

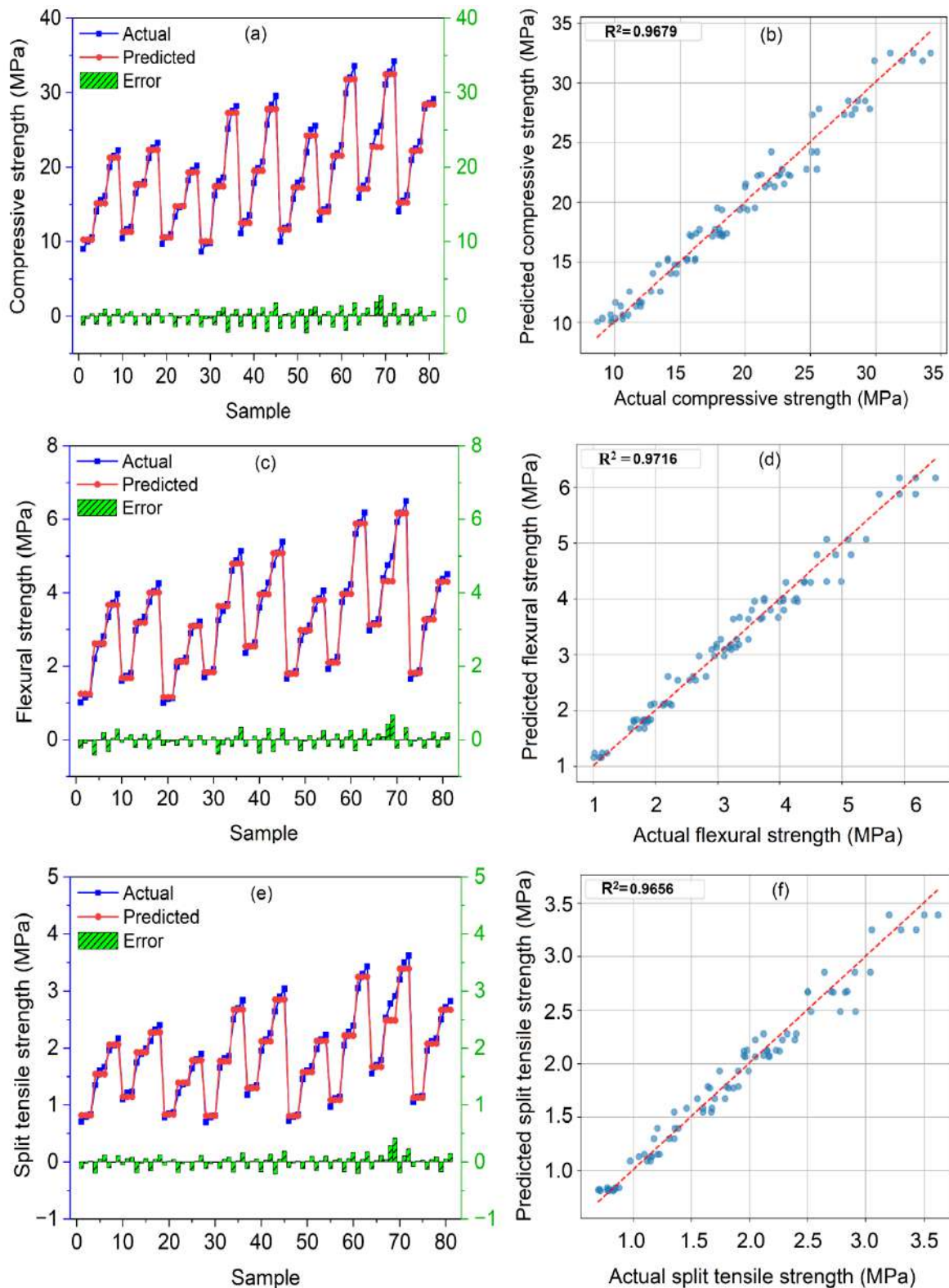


Fig. 11. Actual vs. predicted results of tuned gradient boosting model

Significantly, the model was also able to describe the internal relationship between the mechanical properties. The FS/CS ratio was found to be predicted within the range of 0.11-0.20 and the STS/CS ratio was found between 0.07 to 0.11. The ratios observed was very close to the experimental values too. These intervals are consistent with known mechanical behavior of cementitious composites, and refer to the fact that the model maintains realistic proportionate relationships

among various strengths parameters. The capacity to concurrently forecast various properties and yet retain the physical interdependence of the properties, are further indicators of the high degree of generalization and predictive stability of the GB model. From the point of view of models' applicability limits, the model performs reliably within the range of input values seen during training. For the GB-tuned model, the valid ranges are: GGBS: 0–338.18 kg/m³, SMS: 0–59.18 kg/m³, CH: 0–33.81 kg/m³, CC: 0–422.72 kg/m³, NFA: 591.66–625.62 kg/m³, NCA: 1175.7–1230.3 kg/m³, w/b ratio: 0.44–0.55, and curing time: 3–28 days. Within these limits, the model provides reliable and consistent predictions, while inputs outside these ranges or unseen combinations resulted in less reliable results, this is because of the fact that the model was not designed to extrapolate beyond its training data.

In conclusion, GB model showed excellent accuracy, reliability and strength simultaneously predicting CS, FS, and STS. The large scores of R², the small amounts of the prediction errors, and the evenly distributed values clearly demonstrate that the model has the capability of representing the intricate nonlinear inter-relationships among mix-design variables and mechanical properties. In addition, GB maintains realistic strength ratios, and it is also equally efficient in OPAAC and traditional concrete mixes. Although GB was the best-performing model, the RF model demonstrated high predictive rates as well and took the second place. These outcomes were in line with the prior studies, which details the effectiveness of ensemble techniques, as indicated by Harika et al. [44], they found RF to be the best model when predicting CS in a one-part AAB mortar than GB and decision tree with SVR coming after. Arun et al. [37] reported that the GB model was the best in prediction of CS for geopolymers concrete developed in their work. Hence, the current findings demonstrate the usefulness, flexibility, and stability of ensemble ML methods to provide precise multi-strength predictions of OPAAC in research and practice

3.4. Environmental and Economic Aspects

The benefits in terms of environment were determined by finding the carbon emission value regarding raw materials used in the mix compositions. The carbon emissions factors, in kg CO₂-eq per kg of material, were taken from existing sources of literature [17,46,53] and are presented in Table 10. The calculation was restricted to the material production phase of concrete using the formulated mix design, and did not consider the transportation, construction, service life, and end-of-life phases. This allows for a direct comparison between OPAAC and CC concrete, which is already available in the market and well-known for its lower embodied carbon content ~44-47% lesser than that of conventional 100% OPC-based concrete. This is necessary to point that such significant cut of carbon emissions is supported with relatively small economic benefits since CC is usually offered at a lower price than OPC, thus making it a viable low-carbon option. The quantity of total emissions per cubic meter (kg CO₂-eq/m³) was calculated as the product of the quantity of each component (Table 5) and its emission factor (Table 10). The calculations (Figure 12a) suggested that OPAAC mixes result in an additional 20-22% embodied-carbon saving compared to CC concrete mixes. In case of same mix ratios of OPC (emission factor ~ 0.85 kg CO₂-eq/kg), when OPC is used as a binder rather than CC, the overall carbon emissions of the OPC-based concrete mixes (M15, M20, and M25) were estimated to be 380.25, 419.50, and 447.68 kg/m³ of concrete respectively. When comparing the outcomes obtained using OPC-based conventional concrete, it is possible to note that the implementation of OPAAC leads to the general decrease in carbon emissions by 55-59%.

A parallel analysis was also done on the costs of ingredients of the concrete mixes formulated. The unit cost of the materials (Table 10) was chosen based on the prevailing market prices in India and estimates as given in the previous research works [11,46,54]. Despite the fact that the cost of materials may change depending on the supplier, geographical location, distance of transportation, and quantity of purchase, the chosen costs are average market prices appropriate for comparative purposes. The total cost per m³ of the concrete was calculated by multiplying amounts of materials in Table 5 by their unit costs in Table 10. Figure 12b shows the total unit volume cost of each mix grade and type of binder. OPAAC mixes showed a cost-saving potential of about 10–13% in comparison to the use of CC concrete. Using the same mix ratios, the OPC (Unit cost ≈ 0.0861 \$/kg) as the binder rather than the CC, the estimated cost of the OPC-based concrete of M15, M20, and

M25 mixes were 44.12, 47.91, and 50.70 \$/kg per m³ of concrete, respectively. When compared to OPC-based conventional concrete, OPAAC leads to the overall reduction in cost by about 12-16%. In a practical perspective, the carbon-emission reduction along with the cost-saving consideration highlights the massive potential of OPAAC mixes as an economically feasible and environmentally friendly alternative to ordinary or even a standard grade of concrete.

Table 10. Emission factor and unit cost of ingredients of concrete

Material	Emission factor (kg CO ₂ -eq/kg)	Cost rate* (\$/kg)
CC	0.35	0.0828
GGBS	0.07	0.0243
SMS	0.80	0.3091
CH	0.94	0.1325
NFA	0.05	0.0055
NCA	0.05	0.0094
Water	0.00	0.0000

*Cost of ingredients as per Indian scenario converted from INR to US\$ [1 US\$ ~ 90.60 INR (₹) in Feb 2026].

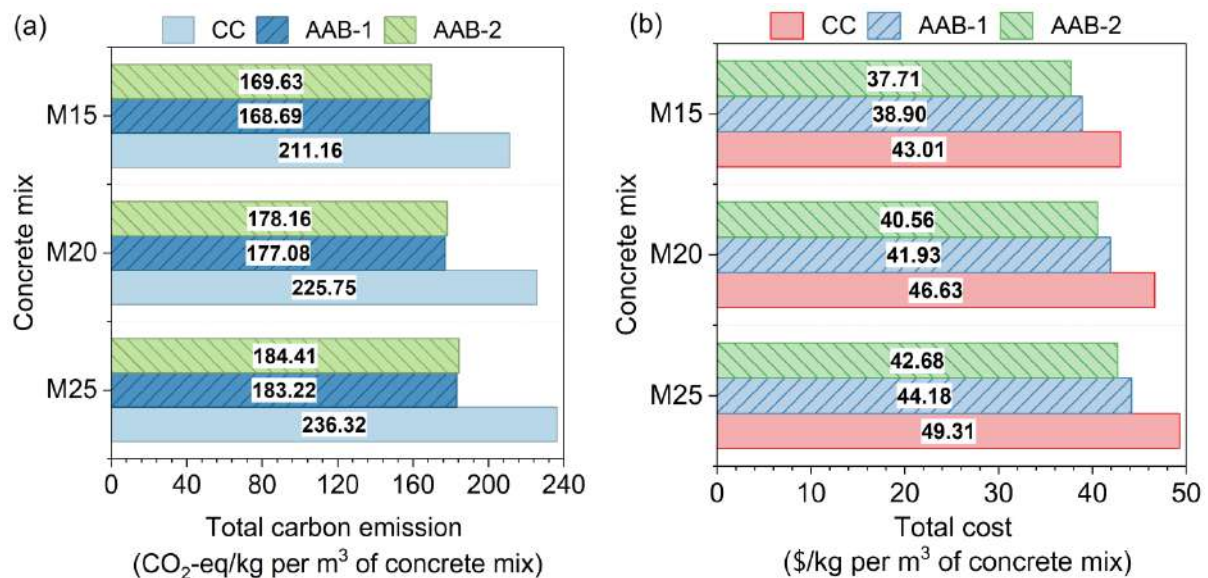


Fig. 12. Carbon emissions and total cost of developed concrete mixes

3.5. Comparison with Recent OPAAC Studies

Table 11 reveals the latest researches on one-part alkali-activated/ geopolymer concrete (OPAAC) that uses solid precursors, powder alkali activators, natural aggregates, water and superplasticizer. The review of these studies reveals that the most common solid precursor in OPAAC was either GGBS or a binary mixture of FA and GGBS.

The most widespread activators were the silicate-based activators that could be utilized individually or could be combined with both additional activators to control the system alkalinity and silica modulus. This has been widely used to enhance rapid reaction and enhance early development of strength. The comparative analysis further indicates that most of the previous studies have mainly been concerned with achieving the high to ultra-high CS by means of optimization of mix design parameters, such as precursor composition, quantity of the activators, w/b ratio, and curing methods. Mostly employed curing regimes were ambient and water curing, which highlights the increasing importance of focusing on the OPAAC systems that can be as efficient without the usage of high-temperature treatment. These studies also suggested that the workability of OPAAC can be improved as per the requirement by utilizing superplasticizers.

Table 11. Performance comparison of OPAAC developed in recent studies

One-part Binder (B)		Aggregate	Mix grade/ ratio (B:NFA:NCA)	Curing Method	Strength of concrete at 28 d (MPa)			Ref.
Precursor	Activator				CS	FS	STS	
GGBS	SHMP	NCA, NFA	1:1.59:2.83 W/B-0.45	Ambient	33.48	4.27	2.31	[32]
GGBS	SHMP	NCA, NFA	1:1.59:2.83 W/B-0.45	Water	32.00	4.53	2.38	[32]
FA, GGBS	K ₂ CO ₃	NCA, NFA	1:1.48:2.84, W/B-0.3, SP- 1.4% wt of B	Ambient	11.40	-	-	[55]
FA, GGBS	SS, Na ₂ CO ₃	NCA, NFA	M40	Ambient	47.50	6.30	4.60	[33]
FA, GGBS	SS, Na ₂ CO ₃	NCA, NFA	M50	Ambient	61.00	7.10	5.10	[33]
GGBS	SMS Pentahydrate	NCA, NFA	SMS/GGBS-0.16, NFA/GGBS-2.26, NCA/NFA-0.66, W/GGBS-0.45	Water	56.11	6.54	3.33	[21]
GGBS	SMS anhydrous	NCA, NFA	M30	Water	36.24	-	-	[34]
GGBS	SMS anhydrous	NCA, NFA	M50	Water	55.28	-	-	[34]
FA, GGBS	SMS anhydrous	NCA, NFA	M40	Ambient	51.60	-	-	[31]
FA, GGBS	Crystalline hydrated sodium silicate	NCA, NFA	M30-M50	Ambient	30.40 to 47.10	3.60 to 5.40	3.34 to 4.70	[29]
FA, GGBS	SMS Anhydrous	NCA, NFA	FA/GGBS-0.33, SMS/GGBS-0.18, W/FA+GGBS-0.4, NFA/FA+GGBS- 0.86, NCA/NFA-0.85, SP-1.3% (FA+GGBS)	Ambient	76.70	7.20	~4.6	[56]
GGBS	SMS Anhydrous	NCA, NFA	GGBS:NFA:NCA- 1:2.2:4 SMS/GGBS-0.163, W/B-0.41 SP/GGBS-0.02	Ambient	47.97	~4.7	4.80	[11]
FA	SMS Pentahydrate, NaOH (16M)	NCA, NFA	NFA/FA-0.84 NCA/FA-1.93 SMS/FA-0.093 NaOH/FA-0.09 W/FA-0.22	Ambient	68.60	-	6.10	[57]
FA, GGBS	SMS Anhydrous	NCA, NFA	1:1.42:2.72 W/B-0.344 SP/B-0.016	Ambient	37.10	-	-	[13]
GGBS	SMS anhydrous, Ca(OH) ₂	NCA, NFA	M15 (1:1.85:3.64) W/B-0.55	Water	22.38	4.02	2.28	Current study
GGBS	SMS anhydrous, Ca(OH) ₂	NCA, NFA	M20 (1:1.58:3.07) W/B-0.48	Water	27.88	5.08	2.86	Current study
GGBS	SMS anhydrous, Ca(OH) ₂	NCA, NFA	M25 (1:1.40:2.78) W/B-0.44	Water	32.72	6.20	3.44	Current study

Notations:- B= (Precursor + Activator), SMS-Sodium Metasilicate, SHMP-Sodium Hexametaphosphate, SS-Sodium Silicate, FA-Fly Ash, NFA-Natural Fine Aggregate, NCA-Natural Coarse Aggregate, W/B- Water to Binder Ratio, SP- Superplasticizer

The CS in the current study was relatively lower as compared to those established in some high-performance OPAAC systems [11,21,29,31,33,34,56,57], but was similar to some moderate-strength mixtures [13,32,34]. It is necessary to mention that the aim of this research was not to make the maximum CS but to create structurally viable mixes to be used in the practical applications in the low- to medium-strength concrete. Interestingly, the CS that was achieved was lower, but the tensile strength values were similar to those in other literature. The behavior can be attributed to the phenomenon that alkali-activated paste leads to stronger interfacial bonding with aggregates and this leads to higher tensile performance and resistance to cracks. The mix proportioning in the current study was done through the conventional Indian Standard mix design method (IS 10262:2019 [49]), that is used in OPC concrete, unlike most of the earlier studies which had used tailored or experimentally optimized mix design methodologies that was unique to alkali-activated systems. However, few studies [11,32] developed OPAAC with similar Indian Standard mix design method in their studies earlier. In current study, the alkali-activated binders (AAB-1 and AAB-2) were considered to be equivalent to OPC 33-grade in terms of physical properties and mix design procedure was carried out in a similar way as conventional concrete. This was deliberately selected as a way of testing the viability of implementing OPAAC into the current construction operations without the need to make significant changes to the current design procedures. The CS goals were restricted to the following grades- M15, M20, and M25; developed without using any chemical admixture. This was a practical decision, since a large percentage of the current construction activity within the local environment, especially residential construction, rigid pavements, and general-purpose structural constructions, needs concrete grades of no more than M25. As such, this research lays focus on applicability, constructability, and adaptability in the current construction requirements, and does not only consider high-strength performance. This kind of strategy promotes the possibility of implementing OPAAC at the field level since it is an environmentally friendly and user-friendly substitute to conventional concrete.

4. Conclusions

The current study determined the fresh workability and hardened strength for OPAAC of M15, M20 and M25 grades and determined predictive modeling by applying machine-learning processes. On the basis of the experimental and analytical studies, the following conclusions are made:

- Workability of OPAAC increased as proportions of sodium metasilicate and calcium hydroxide increased, and further increase of the workability was detected with high levels of w/b ratios. The slump values of OPAAC mixes were equivalent to the control mix.
- OPAAC has a relatively higher mechanical strength compared to conventional concrete. Among the mixes, AAB-1 concrete had the best results in compressive, flexural, and split tensile tests when compared to its control mix and AAB- 2. In particular, the CS ranges between 22.38-32.72 MPa, FS between 4.02-6.20 MPa and STS between 2.28-3.44 MPa, thus, indicating that OPAAC can provide similar strength to that of conventional concrete in terms of field construction requirements.
- Ensemble-based ML models, such as gradient boosting (GB) and random forest (RF), were effective in multi-strength prediction of OPAAC. The GB model was able to predict CS, FS and STS at the same time with R^2 scores of 0.9679, 0.9716 and 0.9656, respectively, with low magnitudes of RMSE and MAE. Overall GB model exhibited an average R^2 of 0.9684 with very low errors.
- The environmental and economic benefits of OPAAC with 55-59 % lesser carbon emissions and cost savings of 12-16 %, which validates the idea that the material can serve as a low-carbon and low-cost binder.

Overall, the developed OPAAC is offering similar levels of workability, better mechanical performance, significant carbon savings, cost-saving, and predictable mechanical performance through machine-learning. It also defeats the limitations of liquid activators within a two-part system, allowing conventional mix-design solutions, as well as increasing field applicability. All of these benefits make OPAAC a feasible and viable alternative to traditional concrete in field construction works. Even though the results are promising, the current study is limited to small range of strength grades, particular material mixtures, and laboratory-level performance checks.

Future research may investigate formulation of the high strength concrete mix design, usage of chemical admixtures, durability assessment, microstructural evaluations, application of ML models on larger dataset, and the full life cycle assessment. These efforts may enhance the credence in the performance, durability and the widespread applicability of OPAAC in large construction works.

References

- [1] GCCA United Kingdom. Concrete Future – Roadmap to Net Zero. Global Cement and Concrete Association; 2021. Accessed February 12, 2026. Available from: <https://gccassociation.org/concretefuture/wp-content/uploads/2022/10/GCCA-Concrete-Future-Roadmap-Document-AW-2022.pdf>
- [2] Nejad BM, Enferadi S, Andrew R. A comprehensive analysis of process-related CO₂ emissions from Iran's cement industry. *Cleaner Environmental Systems*. 2025;16:100251. <https://doi.org/10.1016/j.cesys.2024.100251>.
- [3] Karadumpa CS, Pancharathi RK. Study on energy use and carbon emission from manufacturing of OPC and blended cements in India. *Environ Sci Pollut Res*. 2024;31:5364–5383. <https://doi.org/10.1007/s11356-023-31593-3>.
- [4] Bureau of Indian Standards. IS 16415: 2015 Composite Cement-Specification. New Delhi: BIS; 2015.
- [5] Lanjewar BA, Chippagiri R, Dakwale VA, Ralegaonkar RV. Application of Alkali-Activated Sustainable Materials: A Step towards Net Zero Binder. *Energies*. 2023;16(2):969. <https://doi.org/10.3390/en16020969>.
- [6] Meshram RB, Kumar S. Comparative life cycle assessment (LCA) of geopolymer cement manufacturing with Portland cement in Indian context. *Int J Environ Sci Technol*. 2022;19:4791–4802. <https://doi.org/10.1007/s13362-021-03336-9>.
- [7] Aiken TA, Gu L, Kwasny J, Huseien GF, McPolin D, Sha W. Acid resistance of alkali-activated binders: A review of performance, mechanisms of deterioration and testing procedures. *Constr Build Mater*. 2022;342:128057. <https://doi.org/10.1016/j.conbuildmat.2022.128057>.
- [8] Mohamed OA. A Review of Durability and Strength Characteristics of Alkali-Activated Slag Concrete. *Materials*. 2019;12(8):1198. <https://doi.org/10.3390/ma12081198>.
- [9] Mahendra K, Narasimhan MC. One part alkali-activated materials for construction – A review. *Mater Today Proc*. 2023;93:182–188. <https://doi.org/10.1016/j.matpr.2023.07.116>.
- [10] Luukkonen T, Sreenivasan H, Abdollahnejad Z, Yliniemi J, Kantola A, Telkki VV, et al. Influence of sodium silicate powder silica modulus for mechanical and chemical properties of dry-mix alkali-activated slag mortar. *Constr Build Mater*. 2020;233:117354. <https://doi.org/10.1016/j.conbuildmat.2019.117354>.
- [11] Yankesh, Pal R, Singh N, Priyadarshree A, Rana D, Kumar V. Performance Evaluation of One-Part Geopolymer Concrete. *Procedia Struct Integrity*. 2025;70:311–318. <https://doi.org/10.1016/j.prostr.2025.07.058>.
- [12] Rostami M, Nasrollahzadeh K, Zwicky D. Durability of alkali-activated slag concrete with sodium metasilicate powder as activator under nitric acid attack. *Adv Cem Res*. 2025:1–13. <https://doi.org/10.1680/jadcr.24.00227>.
- [13] Jahandari S, Tao Z, Rahmani A, Alim MA. Durability of one-part geopolymer concrete in aggressive environments. *Constr Build Mater*. 2025;490:142510. <https://doi.org/10.1016/j.conbuildmat.2025.142510>.
- [14] Kumar A, Kumar V, Prasad B. Strength development and flexural behavior of reinforced concrete beam using one-part alkali-activated binder. *Constr Build Mater*. 2021;281:122619. <https://doi.org/10.1016/j.conbuildmat.2021.122619>.
- [15] Reddy RRK, Yaragal SC, Srinivasa AS. One-part eco-friendly alkali-activated concrete–An innovative sustainable alternative. *Constr Build Mater*. 2023;408:133741. <https://doi.org/10.1016/j.conbuildmat.2023.133741>.
- [16] Ghafoor MT, Rakhimova N, Shi C. One part geopolymers: A comprehensive review of advances and key challenges. *J Build Eng*. 2025;111:113112. <https://doi.org/10.1016/j.jobbe.2025.113112>.
- [17] Rashid K, Raoof MN, Daud M, Wang Y, Ju M. One-part alkali activated binder activated by sodium metasilicate and ternary-factored sustainability of structural block. *Structures*. 2024;68:107108. <https://doi.org/10.1016/j.istruc.2024.107108>.
- [18] Alhamoud A, Riahi HT, Ataei A. Mechanical properties and stress-strain relationship of slag-based one-part geopolymer concrete: A comparative study. *Results Eng*. 2024;24:102952. <https://doi.org/10.1016/j.rineng.2024.102952>.
- [19] Ren J, Sun H, Li Q, Li Z, Ling L, Zhang X, et al. Experimental comparisons between one-part and normal (two-part) alkali-activated slag binders. *Constr Build Mater*. 2021;309:125177. <https://doi.org/10.1016/j.conbuildmat.2021.125177>.

- [20] Oderji SY, Chen B, Ahmad MR, Shah SFA. Fresh and hardened properties of one-part fly ash-based geopolymer binders cured at room temperature: Effect of slag and alkali activators. *J Clean Prod.* 2019;225:1–10. <https://doi.org/10.1016/j.jclepro.2019.03.290>.
- [21] Abdulkarim A, Riahi HT, Ataei A. Comparative study of mechanical properties of one-part alkali-activated slag concrete using deformed and recycled tire steel fibers. *Case Stud Constr Mater.* 2025;22:e04124. <https://doi.org/10.1016/j.cscm.2024.e04124>.
- [22] Ma C, Long G, Shi Y, Xie Y. Preparation of cleaner one-part geopolymer by investigating different types of commercial sodium metasilicate in China. *J Clean Prod.* 2018;201:636–647. <https://doi.org/10.1016/j.jclepro.2018.08.060>.
- [23] Oderji SY, Chen B, Ahmad MR, Shah SFA. Fresh and hardened properties of one-part fly ash-based geopolymer binders cured at room temperature: Effect of slag and alkali activators. *J Clean Prod.* 2019;225:1–10. <https://doi.org/10.1016/j.jclepro.2019.03.290>.
- [24] Ouyang S, Chen W, Zhang Z, Li X, Zhu W. Experimental study of one-part geopolymer using different alkali sources. *J Phys Conf Ser.* 2020;1605:012155. <https://doi.org/10.1088/1742-6596/1605/1/012155>.
- [25] Alrefaei Y, Wang YS, Dai JG, Xu QF. Effect of superplasticizers on properties of one-part $\text{Ca}(\text{OH})_2/\text{Na}_2\text{SO}_4$ activated geopolymer pastes. *Constr Build Mater.* 2020;241:117990. <https://doi.org/10.1016/j.conbuildmat.2019.117990>.
- [26] Askarian M, Tao Z, Samali B, Adam G, Shuaibu R. Mix composition and characterisation of one-part geopolymers with different activators. *Constr Build Mater.* 2019;225:526–537. <https://doi.org/10.1016/j.conbuildmat.2019.07.083>.
- [27] Gao H, Al-Damad IMA, Siddika A, Kim T, Foster S, Hajimohammadi A. Enhancing the workability retention of one-part alkali activated binders by adjusting the chemistry of the activators. *Cem Concr Compos.* 2025;157:105928. <https://doi.org/10.1016/j.cemconcomp.2025.105928>.
- [28] Ali N, Soliman AM. Influence of mixing protocols on flow retention of one-part alkali activated slag systems. *Constr Build Mater.* 2024;450:138467. <https://doi.org/10.1016/j.conbuildmat.2024.138467>.
- [29] Pham TT, Nguyen NL, Nguyen TT, Nguyen TT, Pham TH. Effects of Superplasticizer and Water–Binder Ratio on Mechanical Properties of One-Part Alkali-Activated Geopolymer Concrete. *Buildings.* 2023;13(7):1835. <https://doi.org/10.3390/buildings13071835>.
- [30] Haruna S, Mohammed BS, Wahab MMA, Liew MS. Effect of paste aggregate ratio and curing methods on the performance of one-part alkali-activated concrete. *Constr Build Mater.* 2020;261:120024. <https://doi.org/10.1016/j.conbuildmat.2020.120024>.
- [31] Sun D, Lee J, Mohyeddin A, Migunthanna J. Mechanical Properties and Reproducibility of One-Part Ambient-Cured Slag and Fly Ash-Based Geopolymer Concrete. *Buildings.* 2026;16(4):707. <https://doi.org/10.3390/buildings16040707>.
- [32] Kumar V, Kumar A, Prasad B. Mechanical behavior of non-silicate based alkali-activated ground granulated blast furnace slag. *Constr Build Mater.* 2019;198:494–500. <https://doi.org/10.1016/j.conbuildmat.2018.11.282>.
- [33] Neupane K. Evaluation of environmental sustainability of one-part geopolymer binder concrete. *Cleaner Materials.* 2022;6:100138. <https://doi.org/10.1016/j.clema.2022.100138>.
- [34] Sharma D, Singh RB. Strength-based design mix methodology of one-part geopolymer concrete using response surface methodology. *Multiscale Multidiscip Model Exp Des.* 2025;8:120. <https://doi.org/10.1007/s41939-024-00713-y>.
- [35] Mahendra K, Narasimhan MC, Rathod S, Das AK, Prakash GB. Durability performance of one-part alkali-activated self-compacting concrete mixes under aggressive and elevated temperature conditions. *Sustain Chem Pharm.* 2025;45:102025. <https://doi.org/10.1016/j.scp.2025.102025>.
- [36] Karabaş C, Niş A. Durability of Alkali-Activated Concretes Under 5% Sulfuric Acid Environment. *J Sustain Constr Mater Technol.* 2025;10. <https://doi.org/10.29187/2458-973X.1203>.
- [37] Arun BR, Srishaila JM, S MK, Algur V, K K, Tanu H. Prediction of machine learning application in the development of novel sustainable self-compacting geopolymer concrete. *Res Eng Struct Mater.* 2024. <https://doi.org/10.17515/resm2024.354ma0716rs>.
- [38] Mahajan L, Bhagat S. Machine learning approaches for predicting compressive strength of concrete with fly ash admixture. *Res Eng Struct Mater.* 2023;9(2):431–456. <https://doi.org/10.17515/resm2022.534ma0927>.
- [39] Hasan MM, Xu J, Uddin MN, Yang J, Tonny NJ. An interpretable machine learning approach to predict the compressive strength of fly ash-based geopolymer concrete. *Innov Infrastruct Solut.* 2025;10:500. <https://doi.org/10.1007/s41062-025-02307-0>.
- [40] Khan AQ, Naveed MH, Rasheed MD, Miao P. Prediction of Compressive Strength of Fly Ash-Based Geopolymer Concrete Using Supervised Machine Learning Methods. *Arab J Sci Eng.* 2024;49:4889–4904. <https://doi.org/10.1007/s13369-023-08283-w>.

- [41] Huseien GF, Baghban MH, Faridmehr I, Dong K. Optimizing Mix Design for Alkali-Activated Concrete: A Comprehensive Review of Critical Selection Factors. *CivilEng.* 2025;6(3):43. <https://doi.org/10.3390/civileng6030043>.
- [42] Fareghian M, Afrazi M, Armaghani DJ, Akhoundan M, Asghari N, Yazdani M. Machine Learning in the Modeling of 3D Printed Soil-Cement Materials: A Short Overview. *J Soft Comput Civil Eng.* 2026;10. <https://doi.org/10.22115/scce.2025.1966>.
- [43] Afrazi M, Armaghani DJ, Afrazi H, Fattahi H, Samui P. Real-time monitoring of tunnel structures using digital twin and artificial intelligence: A short overview. *Deep Undergr Sci Eng.* 2025:1–16. <https://doi.org/10.1002/dug2.70029>.
- [44] Harika R, Venkateswara Rao S, Ramujee K, Keerthan B. Analytical and experimental investigation on strength characteristics of one part geopolymer using machine learning models. *Asian J Civ Eng.* 2025. <https://doi.org/10.1007/s42107-025-01602-6>.
- [45] Abdel-Mongy M, Iqbal M, Farag M, Yosri AM, Alsharari F, Yousef SEAS. Artificial Intelligence Prediction of One-Part Geopolymer Compressive Strength for Sustainable Concrete. *CMES.* 2024;141:525–543. <https://doi.org/10.32604/cmcs.2024.052505>.
- [46] Kumar S, Sinha AK. Development of eco-friendly concrete paving blocks utilizing one-part alkali-activated binders. *Res Eng Struct Mater.* 2026;12. <https://doi.org/10.17515/resm2025-963ma0612rs>.
- [47] Bureau of Indian Standards. IS 383-2016 (Reaffirmed 2021) Coarse and Fine Aggregate for Concrete Specification (Third Revision). New Delhi: BIS; 2016.
- [48] Bureau of Indian Standards. IS 456: 2000 (Reaffirmed 2021) Plain and Reinforced Concrete Code of Practice (Fourth Revision). New Delhi: BIS; 2000.
- [49] Bureau of Indian Standards. IS 10262: 2019 Concrete Mix Proportioning Guidelines (Second Revision). New Delhi: BIS; 2019.
- [50] Bureau of Indian Standards. IS 1199 (Part 2): 2018 Fresh Concrete - Methods of Sampling, Testing and Analysis Part 2 Determination of Consistency of Fresh Concrete (First Revision). New Delhi: BIS; 2018.
- [51] Bureau of Indian Standards. IS 516 (Part 1/Sec 1): 2021 Hardened Concrete Methods of Test Part 1 Testing of Strength of Hardened Concrete Section 1 Compressive, Flexural and Split Tensile Strength (First Revision). New Delhi: BIS; 2021.
- [52] ACI Committee 318. Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05). Farmington Hills, MI: American Concrete Institute; 2005.
- [53] Jameel M, Suteerasak T, Chumpol P, Puttiwongrak A, Sae-Long W, Sukontasukkul P. Properties of one-part geopolymer pedestrian blocks made using 100 % waste materials. *Case Stud Constr Mater.* 2025;22:e04538. <https://doi.org/10.1016/j.cscm.2025.e04538>.
- [54] Sengupta J, Dhang N, Deb A. Life cycle sustainability assessment of one-part alkali-activated concrete activated with industrial grade soda ash and hydrated lime-A comparative study. *Sustain Chem Pharm.* 2025;48. <https://doi.org/10.1016/j.scp.2025.102254>.
- [55] Askarian M, Tao Z, Adam G, Samali B. Mechanical properties of ambient cured one-part hybrid OPC-geopolymer concrete. *Constr Build Mater.* 2018;186:330–337. <https://doi.org/10.1016/j.conbuildmat.2018.07.160>.
- [56] Mahendra K, Narasimhan MC, Prakash GB, Das AK. Experimental investigation and optimization of one-part alkali-activated self-compacting concrete mixes. *Case Stud Constr Mater.* 2024;21:e04062. <https://doi.org/10.1016/j.cscm.2024.e04062>.
- [57] Wibowo YN, Piscesa B, Tajunnisa Y, Sutrisno W. Investigation on the mechanical behavior of high-calcium fly ash geopolymer concrete using one-part method. *Constr Build Mater.* 2025;490:142436. <https://doi.org/10.1016/j.conbuildmat.2025.142436>.

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Functional analysis of high-dosage crumb rubber modified bitumen

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Abstract

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This study evaluates the performance and aging resistance of high-dosage crumb rubber modified bitumen (CRMB) by analyzing physical and rheological properties of viscosity grade-30 (VG-30) binder modified with 0%, 10%, 15%, 20%, and 24% crumb rubber (CR) before and after short-term aging. Conventional tests (Penetration, softening point, storage stability) and rheological tests (complex modulus, phase angle, rutting factor, elastic recovery using dynamic shear rheometer (ER-DSR), multiple stress creep recovery (MSCR), frequency sweep and zero shear viscosity (ZSV)) were carried out. Elastic recovery improved with increasing CR content and showed strong correlation ($R^2 = 0.79$) and ($R^2 = 0.96$) with percent recovery obtained from MSCR test for unaged and short-term aged CRMB respectively. Aging index (AI) based on complex modulus decreased 1.85 (VG-30) to 1.03 (24% CR) at 60°C, while ZSV-based AI showed substantial reduction, from 3.22 (VG-30) to 1.44 (24% CR), indicating improved durability and resistance to oxidative aging. The ZSV increased with CR content, indicating improved resistance to permanent deformation due to stronger internal network formation. The relaxation time (λ) reached a maximum at 20% CR with balanced viscoelastic behavior. Accordingly, 20% CR was identified as the optimum rubber content based on ZSV, λ , n , and elastic recovery, since beyond this level further rubber addition does not result in significant improvements in elastic recovery and ZSV. It is concluded from the study that addition of 20% CR in VG-30 binder reduces oxidative aging and improves binder rutting performances, resulting in sustainable and environmentally friendly binder for flexible pavements.

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1. Introduction

The blending of bitumen has provided valuable insight in its utilization in the road construction. A range of waste materials, including waste plastic bags, rubber from used tyres, and pyrolysis char have been added to bitumen or directly into bituminous mixes, depending on the case. Successful application of these materials requires a detailed analysis of sources and the characteristics. In India, approximately 2.5 million metric tons of tyres are produced annually, with 2.0 million metric tons of waste tyres generated each year. Disposal of scrap tyres has become a major environmental concerns. To address the large volume of tyres disposal, recycling efforts focuses on improving product properties and expanding the scope of applications. While scrap tyres are used as fuel in energy facilities and paper mills, this contributes to carbon footprint and raises additional environmental concerns [1].

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Safe approach for disposing of waste tires involves grinding them into crumb rubber (CR). CR is produced through scrap tyres using cryogenic or ambient mechanical grinding. In ambient grinding, tyres are shredded and ground with metal and fiber separated. Cryogenic grinding uses liquid nitrogen at around -80°C , involving four stages: size reduction, chilling, separation, and grinding, to produce high-quality CR. The process includes hammer mills, magnetic screens, and sifters to remove contaminants and ensure the purity of the final product [2, 3].

The 30 mesh CR outperformed the 60 mesh CR while preparing the CRMB with 22.5% CR by weight. The addition of CR improved physical, rheological, and elastic recovery properties of binder. The smaller CR particles tend to swell more than the large particles due to the more surface area. There is also a possibility that the finer particles may undergo de-polymerization and dissolve in the bitumen [4]. Researchers have used varying gradation and dosage of CR based on their specific research criteria, grade of base binder and desired performance outcomes, indicating a flexible approach to modify bituminous mixtures [2, 4–6]. The Binder's physical properties are improved (reduction in penetration and an increment in softening point and elastic recovery) after CR addition. This change can significantly enhance the performance characteristics of the CRMB, making it more durable and resistant to cracking [7–9]. Dynamic Shear Rheometer (DSR) is utilized to evaluate elastic recovery, this value is also termed as ER-DSR. Authors have been established correlation between the ER-DSR and the percent recovery values obtained from the Multiple Stress Creep Recovery (MSCR) test for polymer modified bitumen (PMB)[10–12]. Bitumen aging affects all essential properties specifically performance and durability. Aging that occurs during transport, mixing, and laying is termed as short-term aging, whereas the aging that takes place during the pavement's service life is termed as long-term aging. These two aging influences overall quality and stability of bituminous pavements [13]. The evaporation of lighter components during production and laying at high temperatures leads to binder's short-term aging. Long-term aging results from exposure to heat, air, moisture and sunlight. The durability of the mix decreases due to binder hardening, which is influenced by its composition and additives [14]. The particle structure of the CR is complete, preventing the lighter components from volatilizing through the internal network structure, which also explains why CRMB shows distinct aging behaviour. Researcher has found that the CRMB are less affected by the aging at higher temperatures, showing improved durability under such conditions. On the other hand CRMB exhibits high viscosity and do not fully coat the bottle flow at 163°C , which necessitates the use of special aging protocols [5, 15]. The study shows that RTFO aging temperature of 16°C in the case of CRMB binders is not adequate due to limited film formation in the RTFO bottle caused by the higher viscosity of CRMB. CRMB containing 10–20% crumb rubber requires a higher aging temperature, typically between 173°C to 193°C , to achieve oxidation levels comparable to actual short-term aging[16]. Storage stability using the CIGAR tube at high temperatures is key for assessing phase separation and modifier compatibility in modified binder. Higher CR concentrations reduce particle movement, preventing phase separation and ensuring a stable blend[4, 11, 17].

The complex modulus indicates overall stiffness of binder, whereas balance between binder's elastic and viscous characteristics are indicated by phase angle. The incorporation of CR improves the complex modulus and decreases the viscous component, as evidenced by decrease in the phase angle [4, 8]. The complex modulus (G^*), phase angle (δ), and rutting factor ($G^*/\sin \delta$) are commonly used to assess the effectiveness of modifier matrix as an antioxidant and to evaluate aging parameters in terms of the aging index (AI). The increasing content of CR also enhances the elasticity of CRMB and percent recovery, while lowering non-recoverable creep compliance. The aging effect can be evaluated by assessing non-recoverable creep compliance at 3.2 kPa for both unaged and short-term aged binders. A lower AI value indicates minimal changes in the rheological properties, helping to prevent the breakdown of lighter components [5,15,18–20]. The non-Newtonian behavior in fluids results from changes in particles and their alignment along the flow direction. When the binder is subjected to a very low shear rate over an extended period, corresponding to steady-state flow, its viscosity at this stage is called Zero Shear Viscosity (ZSV). ZSV serves as an indicator of the binder's stiffness properties and its resistance to permanent deformation under prolonged or extended loading [21–23]. The ZSV-based AI is determined by

the ratio of ZSV after and before aging, where a lower value signifies less change in ZSV as a result of aging [20].

Short-term aging may lead to oxidative and thermal degradation of CRMB, along with the hardening of the binder due to crosslinking and polymerization of CR particles at high temperatures. These changes significantly impact the long-term performance and durability of bituminous pavements. This study explored the aging resistivity potential of CRMB by analyzing its physical properties like penetration, softening point before and after aging. The aging behavior has been studied in detail in terms of AI using complex modulus, phase angle, rutting factor, non-recoverable creep compliance. The ER-DSR is an indicator of the binder's resistance to thermal and load-related cracking, which is also addressed in this study [10–12]. The aging resistivity potential also depends on the broad range of frequencies that pavements typically encounter in real-world conditions, and it can be determined by the ratio of the rutting factor before and after aging. Lower frequencies correspond to slow-moving loads and higher frequencies correspond to fast-moving loads. This aspect has been studied by various researchers for multiwall carbon nanotubes and organo-modified nanoclay, but it has not yet been addressed for CRMB [18, 24]. The addition of CR also affects the ZSV, as changes in the molecular structure and an increase in molecular weight occur during short-term aging. Therefore, it was felt that there is need for further research of CRMB to assess the impact of short-term aging on ZSV for obtaining a durable CRMB for sustainable pavements.

The novel aspects of this study is to assess the aging resistivity potential of CRMB using ER-DSR, which has not been reported in the existing literature. A correlation between ER-DSR and percent recovery obtained from MSCR test has also been established for better understanding of elastic behavior of CRMB at varying CR content, which contributes the knowledge to the existing literature. Furthermore, this study assesses the aging resistivity potential across a broad range of frequencies, and author has not come across any such study in case of CRMB. Additionally, the impact of short-term aging on ZSV, which determines the long term pavement performance, has also been investigated in case of CRMB. All these aspect has not been comprehensively addressed in the previous research on CRMB, therefore evaluating these aspects in this study provides the significant contribution to the existing literature database with regard to CRMB at varying CR content levels.

The main objective of this study is to evaluate the changes in the physical and rheological properties of high-content CRMB and their relationship before and after aging in detail. The storage stability properties are assessed using the CIGAR tube test. Additionally, the study also investigates in detail the short-term aging effects by evaluating the AI based on rheological properties such as complex modulus, phase angle, rutting factor, varying frequencies, ZSV and non-recoverable creep compliance.

2. Materials and Methodology

2.1. Materials and Binder Preparation

VG-30 grade bitumen, procured from the Panipat refinery of Indian Oil Corporation Limited, is used to prepare CRMB blends. Base binder viscosity was found 2816 Poise at 60°C. The CRMB was prepared by adding varying CR dosages (10%, 15%, 20%, 24%) by weight of the blend. The blending process was carried out using mechanical stirrer at 200-300 rpm for one hour at temperatures ranging from 175-185°C, to ensure a uniform mixture [2, 25]. Crumb rubber was made from waste tyres through mechanical shredding and was processed at ambient temperature, with particle dimension varying 0 to 0.425 mm (40 mesh). The material was carefully processed to be free from steel, fibers, and other foreign contaminants commonly found in rubber tyres. The particle size gradation of CR is presented in Fig.1.

2.2. Aging Procedure

The rolling thin film oven (RTFO) test, performed in accordance with ASTM D2872 [26], is a widely adopted laboratory method to simulate binder's short-term aging, which occurs during processes

such as plant production, transportation, laying and compaction. The glass bottle is filled with thirty-five grams of binder and positioned in circular carriage inside a controlled oven. The temperature of the oven was maintained at 175 °C for 85 minutes instead of 163 °C, as the blending temperature of VG-30 bitumen with CR is maintained between 175–185 °C. The conventional RTFO aging at 163 °C is not sufficient for CRMB due to its higher viscosity and limited film formation. Hence, the RTFO aging was carried out at 175 °C to better simulate the short-term aging conditions [5, 15-16]. Whereas the bottle is rotated to form a uniform film of 1.25 mm thickness. To accelerate the aging process, hot air is injected into the bottle at a rate of 4000 ml/min.

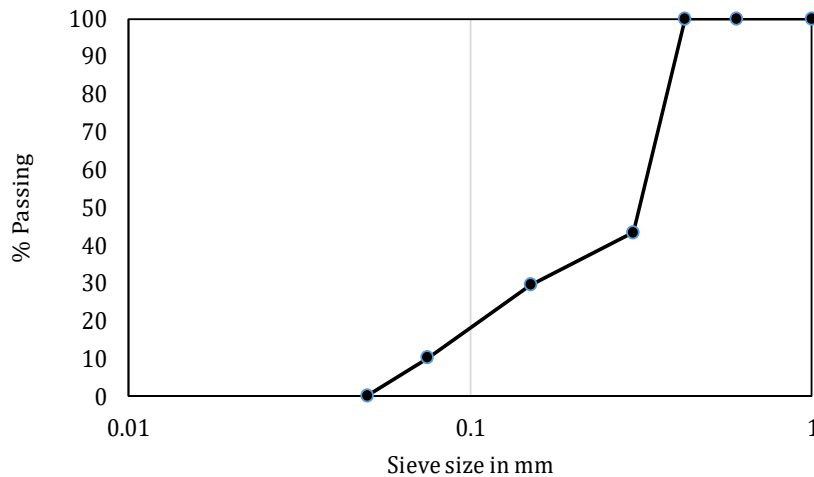


Fig. 1. Particle size gradation of crumb rubber

2.3. Conventional Tests

The penetration value of bitumen, determined through an empirical test, indicates its consistency. This test measures the depth to which a standard needle, loaded with 100 grams, penetrates into a bitumen sample for 5 seconds at a temperature of 25°C. Each 0.1 mm of penetration in the bitumen is considered one unit of penetration [27]. The softening point is the temperature at which bitumen achieves a specific level of softening. The temperature increase is maintained at 5°C per minute during the test. A higher value of the softening point indicates better resistance of bitumen at higher temperatures [28]. The penetration and softening point values were determined for VG-30 and all CRMB samples under both unaged and short-term aged conditions. To evaluate the effect of short-term aging, the difference in softening point values between unaged and short-term aged is assessed.

2.4. Storage Stability Potential

Storage stability of all the blends was analyzed using softening point approach. The CIGAR tube test is for evaluation of storage stability property. The CIGAR tube setup consists of an aluminum tube having 25.4mm dia and a height of 136.7 mm. These tubes are filled with bitumen and kept at $163 \pm 5^\circ\text{C}$ for 48 hours. Afterward, tubes were conditioned in a freezer for 4 hours. Samples are then collected from the top third and bottom third portions of CIGAR tube to perform the softening point test [29].

2.5. Rheological Test

ER-DSR was conducted in accordance with AASHTO TP 123 at 25°C on unaged and short-term aged blends, using 8-mm diameter parallel plate and a 2-mm gap test geometry. The sample was tested at 25°C for ER-DSR at a constant strain rate of $2.315\% \text{ s}^{-1}$ until a strain of 277.8% is achieved. Subsequently, a shear stress of 0.0 kPa was sustained for 30 minutes to facilitate the recovery phase. Stress (τ , Pa) and strain (γ , %) are recorded for one data point every two seconds throughout the entire test [30]. The ER-DSR is calculated using Eq. (1):

$$ER-DSR = \frac{\text{(Recovered strain at the end of relaxation time)}}{\text{Strain at the end of loading time}} \quad (1)$$

Where, ER-DSR is elastic recovery value using dynamic shear Rheometer.

DSR was utilized to access rheological parameters including complex modulus (G^*), phase angle (δ), and rutting factor ($G^*/\sin \delta$) for both unaged and short-term aged binders. The temperature sweep test was performed at frequencies of 1.59 Hz and 1% strain level at 50°C, 60°C, and 70°C. The test was conducted using 25mm dia DSR plates, maintaining 1 mm gap between the two plates. Additionally, a frequency sweep test was performed over broad range of frequency 0.1 to 100 rad/s at 60°C to evaluate behavior of binders across a wide frequency spectrum. The AI, based on the rheological parameters such as G^* , δ , $G^*/\sin \delta$ and J_{nr} was calculated using Eq. (2,3,4 & 5) respectively.

$$AI \text{ Value based on } G^* \text{ value} = \frac{G^*_{\text{short term aged}}}{G^*_{\text{unaged}}} \quad (2)$$

$$AI \text{ Value based on } \delta \text{ value} = \frac{\delta_{\text{short term aged}}}{\delta_{\text{unaged}}} \quad (3)$$

$$AI \text{ Value based on } G^*/\sin \delta \text{ value} = \frac{G^*/\sin \delta_{\text{short term aged}}}{G^*/\sin \delta_{\text{unaged}}} \quad (4)$$

$$AI \text{ Value based on } J_{nr} \text{ value} = \frac{J_{nr \text{ at } 3.2 \text{ kPa unaged}}}{J_{nr \text{ at } 3.2 \text{ kPa short term aged}}} \quad (5)$$

Where, G^* is the complex modulus (kPa); δ is the phase angle (degree); $G^*/\sin \delta$ is rutting factor (kPa); and J_{nr} is non recoverable creep compliance (kPa⁻¹).

ZSV was determined by conducting a steady shear test with shear 0.01 s⁻¹ to 10 s⁻¹ at a constant temperature of 60°C. This viscosity represents limiting value observed in Newtonian region at very low shear rates. Carreau model is commonly applied to characterize steady shear rheological behavior of binders and their flow properties [31, 32]. The AI using ZSV is expressed as a ratio of ZSV of short-term aged binder to that of the unaged binder [20]. Carreau model, represented by Eq. (6) is used for computation of ZSV.

$$\eta = \eta_o + (\eta_o + \eta_\infty)(1 + (\lambda\gamma)^2)^{[(n-1)/2]} \quad (6)$$

Where η is the steady state viscosity (Pa.s); η_o is the Zero shear viscosity (Pa.s); η_∞ is the infinite shear viscosity (Pa.s); γ is the shear rate (s⁻¹); λ and n are the fitting parameter (relaxation time, power law index).

3. Results and Discussion

3.1. Penetration and Softening Point

Three tests were carried out for penetration, softening point, and storage stability for each sample. The reported values correspond to the arithmetic mean of the three replicates, and variability is expressed as standard error. The penetration of the VG-30 blend decreased from 58 to 20 with 24% CR in the unaged condition, as shown in Fig.2. Similarly, the penetration decreased from 46 to 20 in the short-term aged condition. The consistent penetration value for both unaged and short-term aged 24% CR binder implies that higher CR content improves the binder's resistance to aging.

The softening point is 47.3°C for VG-30 and increases to 62.6°C, 65.5°C, 67.8°C, and 72.6°C for unaged binders with 10%, 15%, 20%, and 24% CR content, respectively. Fig.3 shows, the softening point is 51.7°C for VG-30 and increases to 66.5°C, 69.2°C, 71.2°C, and 74.9°C for short-term aged binders. This increase in softening point can be attributed to the increased stiffness and improvement in high-temperature flowing properties of the blends, making it more stable under load and enhancing rutting resistance and overall performance durability. The decrease in the

softening point for both unaged and short-term aged binders was 4.4°C for the VG-30 binder, which reduced to 3.9°C, 3.7°C, 3.4°C, and 2.3°C for the modified binders with 10%, 15%, 20%, and 24% CR content, respectively. The CR particles absorb the lighter fraction of bitumen and swell multiple times to their original size. Some part of CR dissolves into the bitumen, increasing the aromatic and resin and generating smaller molecular components. The absorption of maltenes and swelling of rubber particles contributes to stiffening of binder [33, 34].

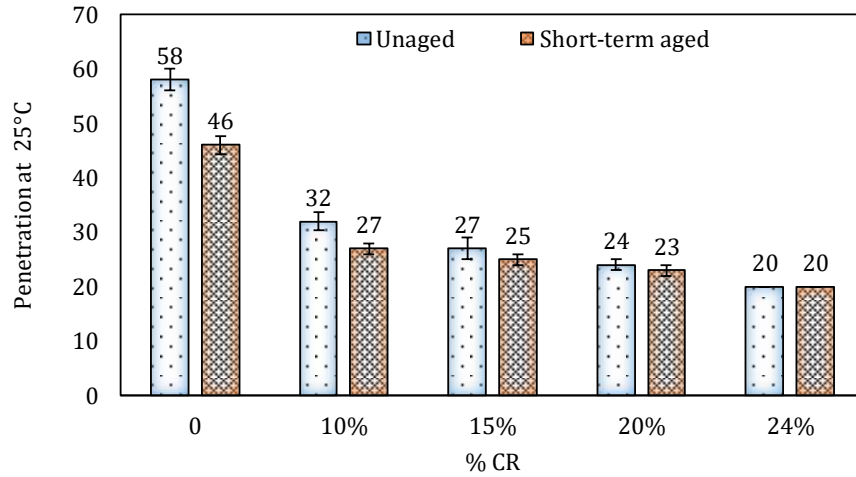


Fig. 2. Penetration of unaged and short-term aged VG-30 and CRMB

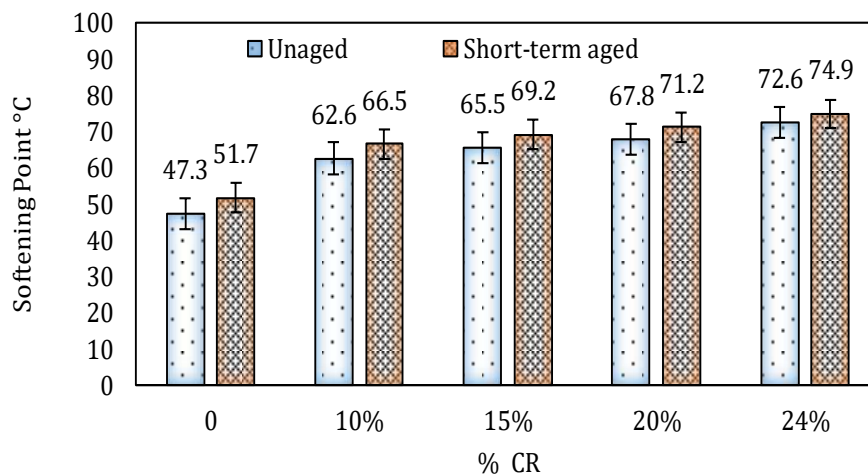


Fig. 3. Softening point of unaged and short-term aged VG-30 and CRMB

3.2. Storage Stability of CRMB

The softening points of VG-30 and various CRMB blends from the top and bottom portions of the CIGAR tube are shown in Fig.4. The bottom portion of the CIGAR tube exhibited a higher softening point in various CRMB blends, indicating the settlement of CR particles at the bottom. The difference in softening point between bottom and top portions was observed to be 3.7°C, 3.5°C, 2.4°C, and 1°C for 10%, 15%, 20%, and 24% CR content, respectively. The difference in softening point of less than 4 °C indicates uniform dispersion of CR particles in the VG-30 bitumen. This could be due to the limited space for CR particles, with higher specific gravity, to shift and settle at the bottom of the tube during storage in the oven. Similar trends were also observed in studies with higher CR content [4, 18, 35].

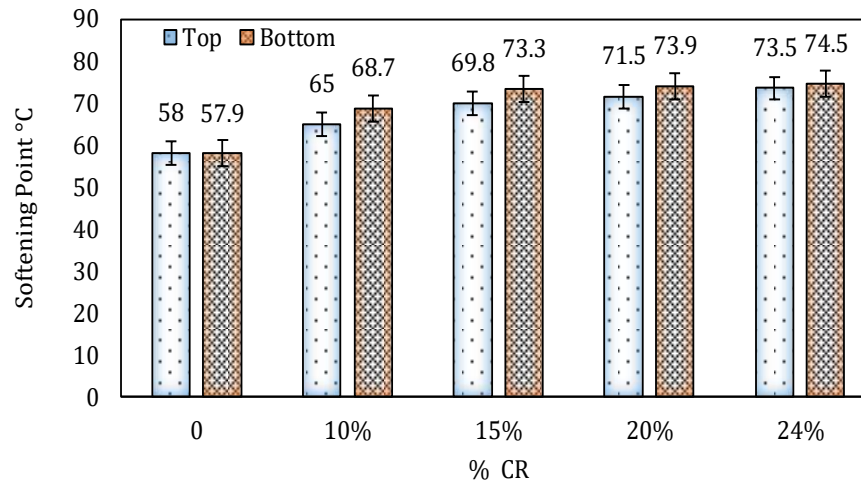


Fig. 4. Variation in softening point between top and bottom samples of the CIGAR tube

3.3. Unaged and Short-Term Aged Rheological Properties

3.3.1 Elastic Recovery Analysis

The ER-DSR was calculated using Equation (1). The test results of ER-DSR for unaged and short-term aged VG-30, along with various CRMB are presented in Fig.5. It can be seen that elastic recovery is 14.2% for VG-30 and 57.8%, 65.6%, 73.2%, and 71.3% for 10%, 15%, 20%, and 24% CR content, respectively for unaged binders. Similarly, elastic recovery is 8.6% for VG-30 and 44.2%, 55.2%, 63.1%, and 70.2% for 10%, 15%, 20%, and 24% CR content, respectively for short-term aged binders. There is significant increase in elastic recovery with the addition of CR for both aged and short-term aged binders.

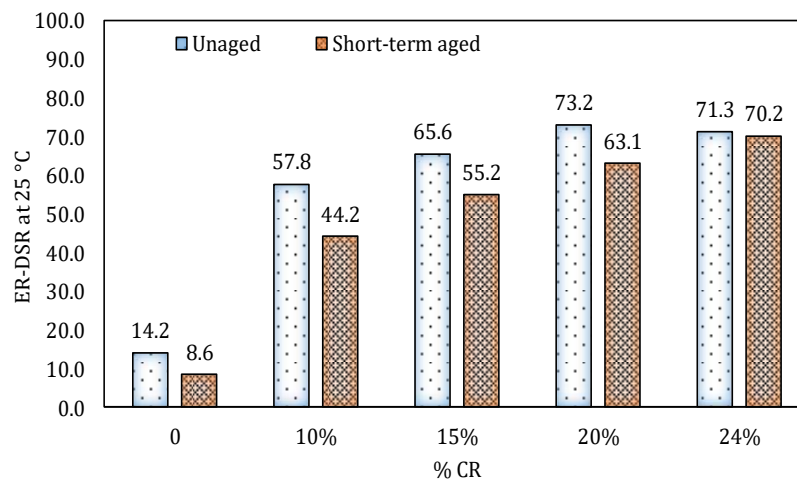


Fig. 5. ER-DSR of unaged and short-term aged VG-30 and CRMB

The ER-DSR value for the 20% CR bitumen is higher than that of the 24% CR bitumen, indicating that increasing the crumb rubber content beyond 20% does not result in further improvement in the elastic recovery properties of CRMB. It is evident that the elastic recovery shows lower values for short-term aged binders compared to unaged binders; this may be due to the aging of the binder. As the CR content increases, the difference between unaged and short-term aged binder's ER-DSR decreases, indicating that an increasing CR content is reducing the aging effect. Similar trends have also been observed by other researchers [5]. Correlation has been established between MSCR percent recovery at 3.2 kPa and ER-DSR for both unaged and short-term aged CRMB, as presented in Fig.6. It can be observed from Fig.6 that the MSCR percent recovery at 3.2 kPa and ER-DSR exhibit good correlation with correlation coefficient is 0.79 and 0.96 for unaged and short-term aged CRMB respectively. This may be due to the initial interaction between

bitumen and CR particles in the unaged state, where the absorption of lighter fractions and the swelling of rubber are still in progress. CRMB undergoes stiffening, stabilizing the rubber–binder interaction and leading to a stronger correlation between the two parameters after short-term aging.

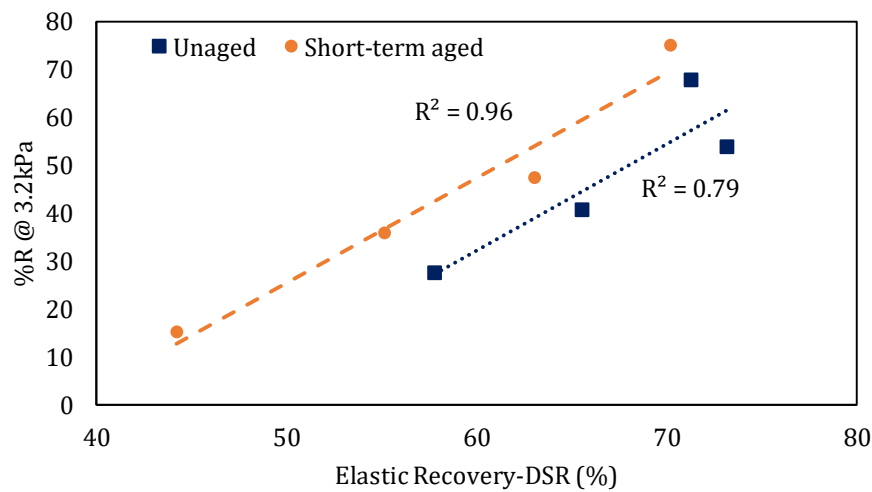


Fig. 6. Relationship between percent Recovery and ER-DSR of unaged and short-term aged VG-30 and CRMB

3.3.2 Complex Modulus Analysis

The test results for the complex modulus of VG-30 and various CRMB blends at temperatures ranging from 50°C, 60°C, and 70°C for unaged and short-term aged are presented in table 1. The complex modulus of bitumen measures its resistance to deformation, reflecting stiffness through elastic and viscous components. It is crucial for assessing binder performance under varying temperatures and loads. The complex modulus increases with higher CR content at all temperatures, indicating that addition of CR enhances the stiffness of binder as can be seen from table 1. Similar trends in complex modulus have also been reported by other researchers for CRMB, Styrene Butadiene Styrene (SBS) modified binders, and other combinations of binders with CR [5, 36]. The complex modulus values for short-term aged binders show an upward shift (increase) due to binder's aging, as can be seen from table 1. AI based on complex modulus was calculated using Equation (2). AI values at 50°C, 60°C, and 70°C are presented in Fig. 7. It is observed that the AI value decreases as the CR content increases. Similarly, the AI of VG-30 bitumen is 1.85, which reduces to 1.23, 1.19, 1.11, and 1.03 for 10%, 15%, 20%, and 24% CR content respectively at 60°C. This indicates that the presence and higher concentration of CR particles retard the upward shift of the complex modulus due to short-term aging and contribute to delay in overall oxidation process. However, minor deviations from the decreasing trend are observed at 50°C and 70°C for some binders as can be seen from Fig. 7, this may be due to temperature-dependent viscoelastic behavior of the binder–rubber system.

Table 1. Complex modulus of unaged and short-term aged VG-30 and CRMB

Binder/ Temperature	Complex Modulus in kPa, unaged			Complex Modulus in kPa, aged		
	50°C	60°C	70°C	50°C	60°C	70°C
VG-30	28.20	7.58	2.27	52.80	14.00	4.00
10% CR	50.50	20.00	8.33	62.70	24.50	10.20
15% CR	67.50	23.70	9.53	72.10	28.30	11.60
20% CR	68.50	27.00	10.90	73.20	30.00	13.00
24% CR	75.70	33.10	14.10	79.50	34.20	16.10

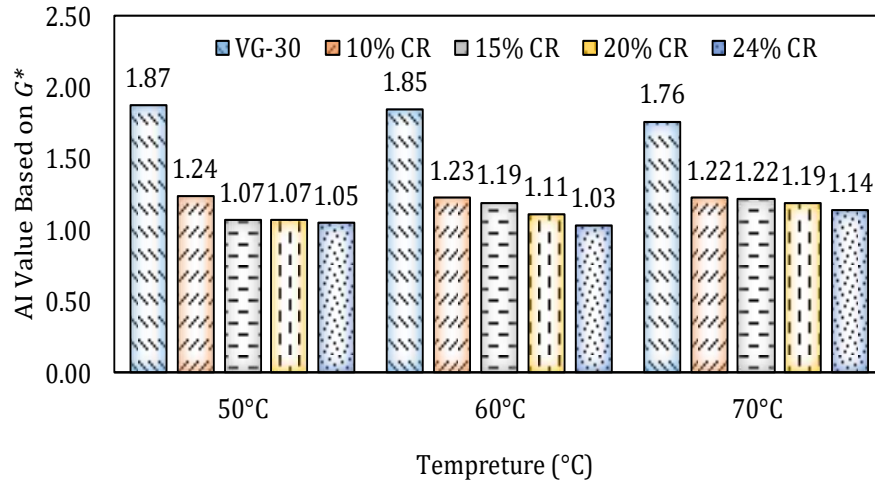


Fig. 7. Aging index based on complex modulus at different temperatures for VG-30 and CRMB

3.3.3 Phase Angle Analysis

The phase angle of unaged and short-term aged bitumen provides insight into its resistance to deformation and ability to recover after stress. A lower phase angle indicates more elastic behavior and higher energy storage. Additionally, a lower phase angle reflects increased resistance to rutting and improved durability. The phase angle for unaged and short-term aged binders at 50°C, 60°C, and 70°C is presented in table 2. AI index based on phase angle was calculated using Equation (3). AI values at 50°C, 60°C, and 70°C are presented in Fig.8. As the CR content increases, binder's phase angle decreases, indicating more elastic behavior, which can be seen from Table 2. The phase angle also decreases after short-term aging of binder, as presented in Table 2.

Table 2. Phase angle of unaged and short-term aged VG-30 and CRMB

Binder/ Temperature	Phase angle, unaged			Phase angle, aged		
	50°C	60°C	70°C	50°C	60°C	70°C
VG-30	81.9	85.1	87.3	77	81.5	85
10% CR	63.5	64.3	65.2	59.5	60.2	61.3
15% CR	63.1	63.7	64.6	58.9	59.5	60.5
20% CR	58.8	59.3	59.7	54.8	55.2	55.7
24% CR	54.2	54.7	54.8	49.8	50	50.3

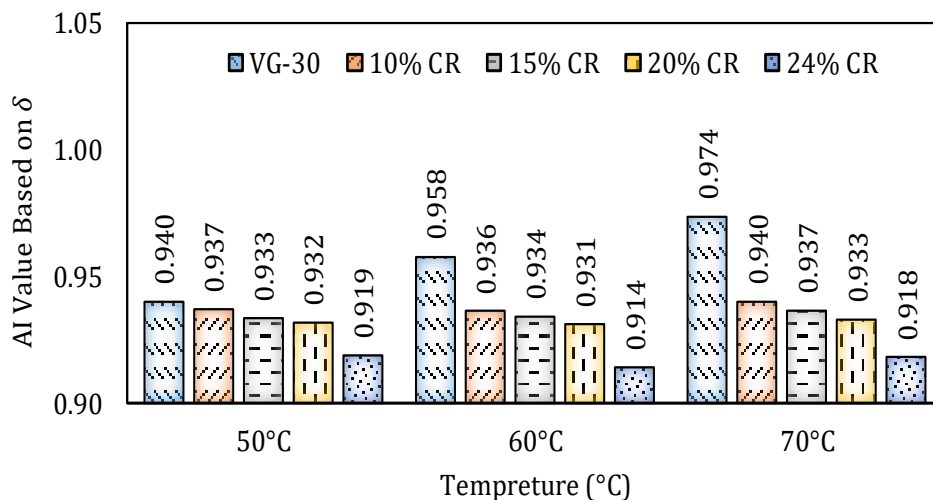


Fig. 8. Aging index based on phase angle at different temperatures for VG-30 and CRMB

The AI based on phase angle at 50°C, 60°C, and 70°C is presented in Fig.8. It is observed that as the CR content increases the AI value decreases. Specifically, the AI of VG-30 is 0.958, which decreases to 0.936, 0.934, 0.931, and 0.914 for 10%, 15%, 20%, and 24% CR content respectively at 60°C. This indicates that the presence and higher concentration of CR particles retard the downward shift of the phase angle due to short-term aging and retard aging of binder. Similar trends in the AI have also been reported by other researchers for SBS-modified binders for different aging protocols [19, 37].

3.3.4 Rutting Factor Analysis

The rutting factor is utilized to evaluate the bitumen's resilience to permanent deformation at elevated temperatures. The rutting factor values for unaged and short-term aged VG-30 and various CRMB blends at 50°C, 60°C, and 70°C are presented in table 3. The rutting factor is lower for the VG-30 and increases progressively as the CR content increases from 10 to 24%. This trend is consistent for both unaged and short-term aged binders as reported in available literature [6]. It is also observed that as the test temperature increases, rutting factor values for VG-30 and various CRMB blends (for unaged and short-term aged) decrease for all test temperatures. This reduction is attributed to the increasing dominance of the viscous component at higher temperatures. The decline in the rutting factor gradually slows down, indicating that the performance of VG-30 and CRMB deteriorates with rising temperature. AI index based on rutting factor was calculated using Equation (4). AI values at 50°C, 60°C, and 70°C are presented in Fig.9. The AI based on rutting factor at 50°C, 60°C, and 70°C is presented in Fig.9. It is observed that the AI value reduces as the CR content increases. The AI of VG-30 bitumen is 1.86, which reduces to 1.27, 1.24, 1.16, and 1.10 for 10%, 15%, 20%, and 24% CR content respectively at 60°C. The AI decreases as the CR content increases, which indicates a reduction in temperature susceptibility and an improvement in aging properties. The resistance to oxidative aging and thermal degradation is also represented by a lower AI [36].

Table 3. Rutting factor ($G^*/\sin \delta$) of unaged and short-term aged VG-30 and CRMB

Binder/ Temperature	Rutting factor in kPa, unaged			Rutting factor in kPa, aged		
	50°C	60°C	70°C	50°C	60°C	70°C
VG-30	28.5	7.6	2.3	54.2	14.2	4.0
10% CR	56.4	22.2	9.2	72.8	28.2	11.6
15% CR	75.7	26.4	10.5	84.2	32.8	13.3
20% CR	80.1	31.4	12.6	89.6	36.5	15.7
24% CR	93.3	40.6	17.3	104.1	44.6	20.9

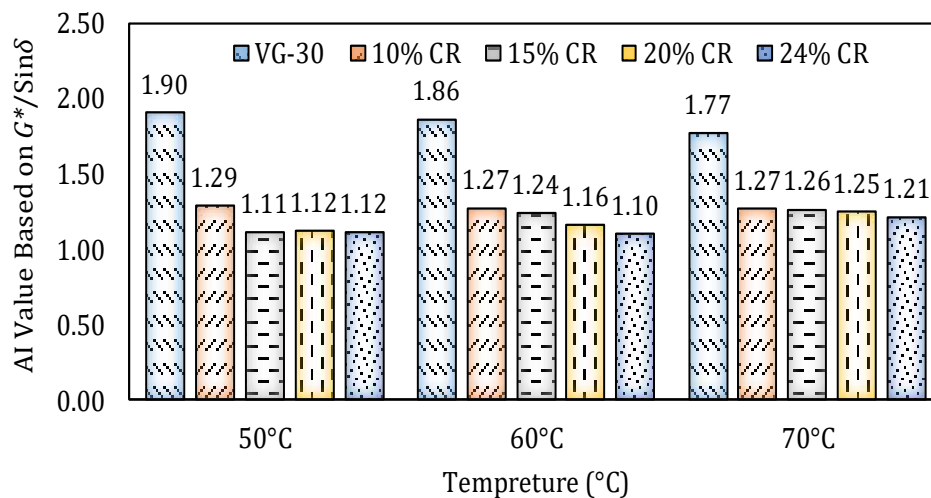


Fig. 9. Aging index based on rutting factor ($G^*/\sin \delta$) at different temperatures for VG-30 and CRMB

3.3.5 AI Based on Non-Recoverable Creep Compliance

The AI based on J_{nr} 3.2 kPa was calculated using Equation (5). This stress level more accurately simulates real-world loading conditions, especially under heavy traffic. It provides the reliable and comparable percent recovery results for PMB and CRMB, ensuring a better assessment of their elastic behavior and rutting resistance. The test results of aging index based on J_{nr} value at 3.2 kPa is presented in Fig.10. The AI of VG-30 bitumen is 2.35, which reduces to 2.02, 1.82, 1.52, and 1.01 for 10%, 15%, 20%, and 24% CR content respectively at 70°C. The AI decreases as the CR content increases, indicating that higher CR content in CRMB makes it less susceptible to aging. Similar trends have also been observed by other researchers. [15].

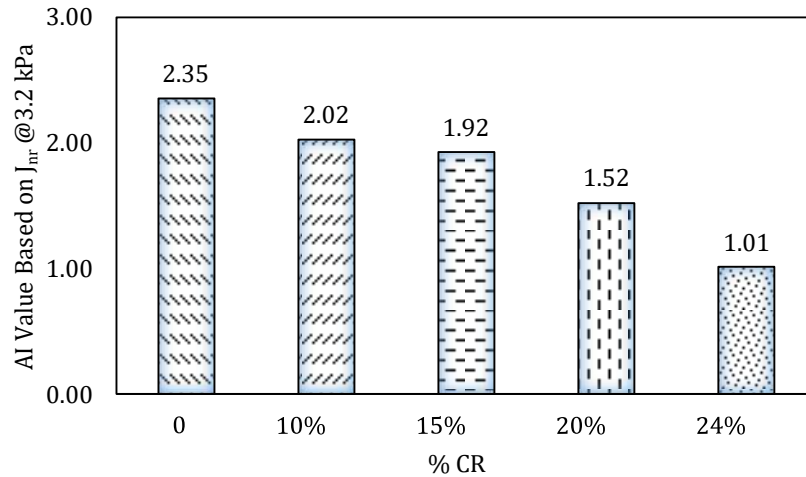


Fig. 10. AI based on J_{nr} at 3.2 kPa for VG-30 and CRMB

3.3.6 Analysis of AI Using Broad Range of Frequencies

The variation across a broad range of frequencies (0.1 to 100 rad/s) at 60°C is presented in Fig.11 for VG-30 and various CRMB blends. The lower frequencies correspond to slow-moving loads, where the viscous component of the binder plays a significant role. On the other hand, higher frequencies correspond to fast-moving loads, where the elastic component of the binder dominates. The AI of VG-30 is observed 2.08 at 0.1 rad/s and 1.46 at 100 rad/s, while the binder with 20% CR content showed an AI of 1.50 at 0.1 rad/s and 1.04 at 100 rad/s, respectively. A similar pattern of AI is observed in other CRMB. The decrease in the AI value at low frequencies indicates an improvement in the binder's resistance to cracking due to the addition of CR. This trend is rational, as lower frequencies represent higher temperatures, while higher frequencies correspond to lower temperatures. Other researchers have also reported similar trends with different types of modifiers [20, 24].

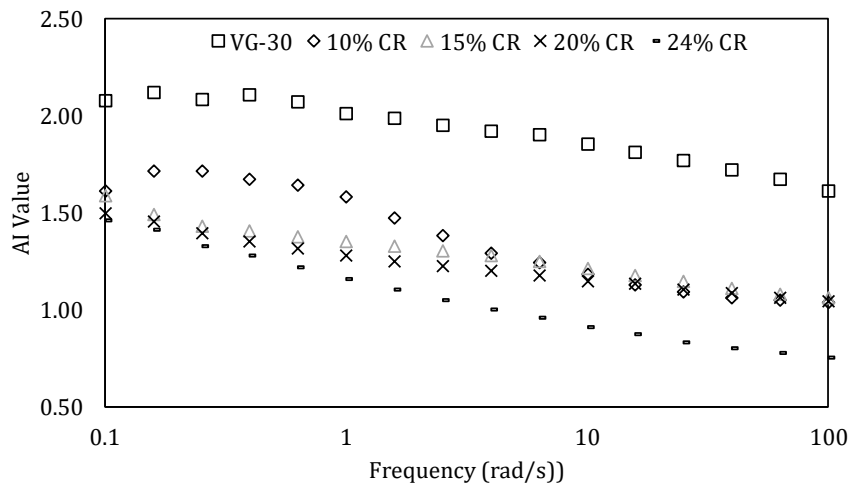


Fig. 11. Variation of aging index at 60°C over a broad range of frequencies for VG-30 and CRMB

3.3.7 ZSV Analysis

The test results of ZSV is also considered an important parameter for assessing binder's ability to resist permanent deformation caused by high temperatures and heavy axle loads under prolonged loading condition. The test results of ZSV at 60 °C for unaged and short-term aged VG-30 and CRMB binders are presented in Table 4. For unaged binders, the ZSV of VG-30 bitumen is 7.68×10^2 , which increases to 6.26×10^4 at 24% CR content. Similarly, for short term aged binders, the ZSV of VG-30 increases from 2.48×10^3 to 8.99×10^4 at 24% CR content. The ZSV increases with increasing CR content, indicating enhanced resistance to permanent deformation due to the development of a stronger internal network structure. The value of λ increases from 0.94 to 6.26 for the unaged binder and from 0.58 to 6.34 for the short-term aged binders when comparing VG-30 and 24% CR binders. The 20% CR binder exhibits the maximum λ value i.e. 11.11 s, indicating the formation of the most effective elastic network and delayed stress relaxation. Although the 24% CR binder exhibits a higher ZSV, no corresponding improvement in λ is observed, indicating that the additional CR does not further enhance the elastic response.

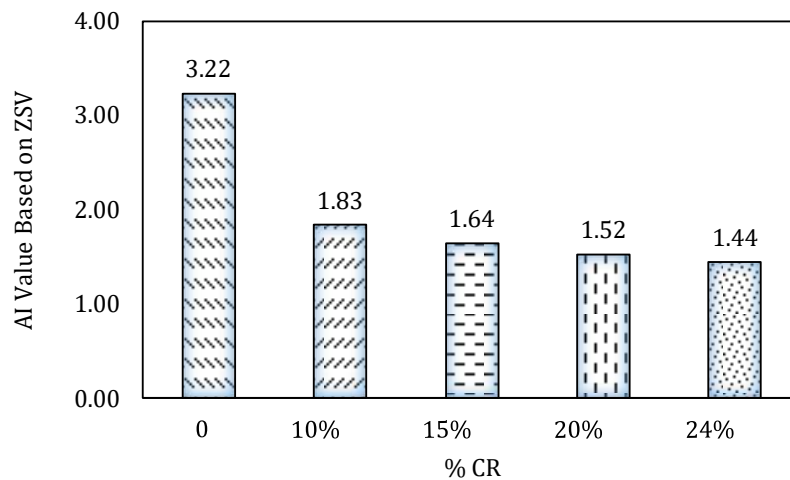


Fig. 12. Aging index based on ZSV for VG-30 and CRMB

The n value decreases from 0.93 to 0.56 for the unaged binders and from 0.75 to 0.44 for the aged binder when comparing VG-30 with 24% CR binders, confirming the increasing non-Newtonian and viscoelastic nature of CRMBs. The n values for the 20% CR binder are 0.63 for unaged condition and 0.50 for short-term aged condition, representing a balanced viscoelastic response with adequate stiffness and workability. In contrast, the 24% CR binder exhibits a much lower n value of 0.44 in the short-term aged condition, indicating excessive stiffness that may lead to compaction and constructability issues.

Table 4. Carreau model parameters of unaged and short-term aged VG-30 and CRMB

Binder/ Temperature	Unaged				Aged			
	η_0 (Pa.s)	λ (s)	N	R ²	η_0 (Pa.s)	λ (s)	N	R ²
VG-30	7.68×10^2	0.94	0.93	0.90	2.48×10^3	0.58	0.75	0.99
10% CR	4.34×10^3	5.23	0.77	0.99	7.96×10^3	7.48	0.74	0.99
15% CR	1.35×10^4	3.70	0.67	0.93	2.22×10^4	5.17	0.65	0.97
20% CR	2.43×10^4	11.11	0.63	0.99	3.69×10^4	4.54	0.50	0.94
24% CR	6.26×10^4	6.26	0.56	0.99	8.99×10^4	6.34	0.44	0.95

The high R^2 values (0.90–0.99) across all samples confirm excellent fit of the Carreau model to the experimental data, supporting the reliability of the predicted ZSV. Although the 24% CR binder exhibits a higher ZSV, no corresponding improvement in λ is observed, indicating that the additional CR does not further enhance the elastic response. Based on the combined evaluation of ZSV, relaxation time (λ), and flow behavior index (n), the 20% CR content is identified as the optimum dosage, providing superior resistance to permanent deformation while maintaining a

balanced viscoelastic response and practical workability. The AI based on the ZSV is derived by calculating the ratio of short-term aged ZSV to unaged ZSV. The results of ZSV based AI are shown in Fig.12. The AI of VG-30 bitumen is 3.22, which reduces to 1.83, 1.64, 1.52, and 1.44 for 10%, 15%, 20%, and 24% CR content respectively at 60°C. The AI value decreases with an increase of CR content, signifying a decrease in temperature sensitivity and an enhancement in aging resistance. Similar trends of reduction in the AI have also been reported for other modifiers [20].

Table 5. Summary of Aging Index for VG-30 and CRMB based on rheological and ZSV parameters

Binder/ Temperature	AI based on G^*			AI based on δ			AI based on $G^*/\sin \delta$			AI base d on J_{nr} 70°C	AI based on ZSV 60°C
	50°C	60°C	70°C	50°C	60°C	70°C	50 °C	60 °C	70 °C		
VG-30	1.87	1.85	1.76	0.940	0.958	0.974	1.9	1.86	1.77	2.35	3.22
10% CR	1.24	1.23	1.22	0.937	0.936	0.940	1.29	1.27	1.27	2.02	1.83
15% CR	1.07	1.19	1.22	0.933	0.934	0.937	1.11	1.24	1.26	1.82	1.64
20% CR	1.07	1.11	1.19	0.932	0.931	0.933	1.12	1.16	1.25	1.52	1.52
24% CR	1.05	1.03	1.14	0.919	0.914	0.918	1.12	1.1	1.21	1.01	1.44

Table 5 presents the summary of test results of AI for VG-30 and CRMB based on rheological and zero shear viscosity at different temperatures. The AI based on complex modulus (G^*), phase angle (δ), $G^*/\sin \delta$, non-recoverable creep compliance (J_{nr} at 3.2 kPa), and ZSV at 60°C presents changes in binder property due to short-term aging. VG-30 exhibited the highest aging indices among all the parameters, which shows greater susceptibility to aging. CRMB with 10 to 24 % CR content, demonstrated improved aging resistance, as AI is decreasing with the increase of CR content as evidenced by lower AI values. The AI based on ZSV is decreasing with the increase of CR content, indicating that CR enhances the long-term stability of the binder. These findings suggest that the incorporation of crumb rubber can lower the aging effects in terms of improvements in durability of bituminous binders on elevated temperatures.

4. Conclusion

Based on the detailed experimental results the following conclusions can be drawn:

The addition of CR in VG-30 reduces penetration, increases softening point and improves the binder's stiffness, thermal stability, durability and aging characteristics. Elastic recovery values increased with the increase in CR content, indicating improved recovery and flexibility. This was validated through a strong correlation between elastic recovery and percent recovery at 3.2 kPa, with correlation coefficients of 0.79 and 0.96 for unaged and short-term aged binders, respectively. The optimum rubber content is observed at a 20% CR level, as beyond this percentage, rubber does not contribute significantly to the elastic recovery. However, higher dosage of CR may provide improved aging resistance. The difference between the unaged and short-term aged elastic recovery reduced with the increase in CR content. From this it can be inferred that aging reduces with the increase in CR content.

It can be inferred that storage stability is not an issue with the use of 24% CR as the difference in softening point value between the top and bottom portion after storage stability test was less than 4°C.

The addition of CR showed lower phase angle even at higher temperatures indicating the elastic behaviour of the CR modified binders. The aging index parameter showed better resistance against oxidative aging with the increase in CR content.

The percent recovery after short-term aging indicated the elastic behaviour of the CR modified binders, especially when the CR content was higher. Therefore, it can be inferred from this study that higher CR can result in a rut resistant binder with minimal binder aging.

The ZSV increases with increasing CR content, indicating enhanced resistance to permanent deformation due to stronger internal network formation. The 20% CR binder shows the maximum relaxation time reflecting the most effective elastic network and delayed stress relaxation. Its corresponding n values indicate a balanced viscoelastic response with adequate stiffness and workability, while 24% CR exhibits excessive stiffness. Therefore, based on ZSV, λ , n , and elastic recovery, 20% CR is identified as the optimum dosage at the binder level. Although 24% CR further improves aging indices, its excessively low n -value and increased stiffness may adversely affect workability.

The present study is limited to binder-level evaluation; therefore, mixture-level investigations are required to confirm performance in future studies.

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References

- [1] Amirkhanian SN, Corley M. Utilization of rubberized asphalt in the United States. Proceedings of the International Symposium on Pavement Recycling, Sao Paulo, Brazil; 2005.
- [2] Bilema M, Yuen CW, Alharthai M, Al-Saffar ZH, Al-Sabaei A, Yusoff NIM. A review of rubberised asphalt for flexible pavement applications: production, content, performance, motivations and future directions. Sustainability. 2023;15. <https://doi.org/10.3390/su151914481>
- [3] Lo Presti D. Recycled tyre rubber modified bitumens for road asphalt mixtures: a literature review. Construction and Building Materials. 2013;49:863–881. <https://doi.org/10.1016/j.conbuildmat.2013.09.007>
- [4] Jamal M, Martinez-Arguelles G, Giustozzi F. Effect of waste tyre rubber size on physical, rheological and UV resistance of high-content rubber-modified bitumen. Construction and Building Materials. 2021;304:124638. <https://doi.org/10.1016/j.conbuildmat.2021.124638>
- [5] Zhao Z, Wang L, Wang W, Shanguan X. Experimental investigation of the high-temperature rheological and aging resistance properties of activated crumb rubber powder/SBS composite-modified asphalt. Polymers. 2022;14. <https://doi.org/10.3390/polym14091905>
- [6] Lee SJ, Akisetty CK, Amirkhanian SN. The effect of crumb rubber modifier on performance properties of rubberized binders in HMA pavements. Construction and Building Materials. 2008;22:1368–1376. <https://doi.org/10.1016/j.conbuildmat.2007.04.010>
- [7] Cao WD, Liu ST, Cui XZ, Yu XQ. Effect of crumb rubber particle size and content on properties of crumb rubber modified asphalt. Applied Mechanics and Materials. 2011;99–100:955–959. <https://doi.org/10.4028/www.scientific.net/AMM.99-100.955>
- [8] Zhang B, Chen H, Zhang H, Kuang D, Wu J, Zhang X. Physical and rheological properties of rubberized bitumen modified by different methods. Materials. 2019;12. <https://doi.org/10.3390/ma12213538>
- [9] Kumar G, Sahoo UC, Ramachandra Rao K, Bose S. Design and evaluation of stone matrix asphalt using stiffer grade crumb rubber modified bitumen. Roads and Bridges. 2019;18:151–165. <https://doi.org/10.7409/rabdim.019.010>
- [10] Clopotel CS, Bahia HU. Importance of elastic recovery in the DSR for binders and mastics. Engineering Journal. 2012;16:3–5. <https://doi.org/10.4186/ej.2012.16.4.99>
- [11] Kumar A, Choudhary R, Kandhal PS, Julaganti A, Behera OP, Singh A, Kumar R. Fatigue characterisation of modified asphalt binders containing warm mix additives. Road Materials and Pavement Design. 2020;21:519–541. <https://doi.org/10.1080/14680629.2018.1507921>
- [12] Morshed MMT, Hossain Z, Chen D, Baumgardner G. Alternatives to elastic recovery and fatigue tests for modified binders. International Journal of Pavement Research and Technology. 2020;13:630–636. <https://doi.org/10.1007/s42947-020-6009-2>
- [13] Hofko B, Hospodka M. Rolling thin film oven test and pressure aging vessel conditioning parameters. Transportation Research Record. 2016;2574(1):111–116. <https://doi.org/10.3141/2574-12>
- [14] Petersen JC. Chemical composition of asphalt as related to asphalt durability. Transportation Research Record. 1984;13–30.
- [15] Wang H, Liu X, Apostolidis P, van de Ven M, Erkens S, Skarpas A. Effect of laboratory aging on chemistry and rheology of crumb rubber modified bitumen. Materials and Structures. 2020;53:1–15. <https://doi.org/10.1617/s11527-020-1451-9>

- [16] Jin T, Feng Y, Li M, Liu L, Yuan J, Sun L. Laboratory short-term aging of crumb rubber modified asphalt: RTFOT temperature optimization and performance investigation. *Journal of Cleaner Production*. 2024;434:140327. <https://doi.org/10.1016/j.jclepro.2023.140327>
- [17] Vigneswaran S, Yun J, Lee MS, Lee SJ. Effect of nanocomposite clays on storage stability of rubberized binders under different curing and blending conditions. *International Journal of Pavement Research and Technology*. 2025. <https://doi.org/10.1007/s42947-025-00499-3>
- [18] Ashish PK, Singh D. Functional characteristics of carbon nanotube modified asphalt binder. *International Journal of Pavement Engineering*. 2020;21:1069–1082. <https://doi.org/10.1080/10298436.2018.1519190>
- [19] Zhang D, Zhang H, Shi C. Aging performance of SBS modified asphalt under various aging methods. *Construction and Building Materials*. 2017;145:445–451. <https://doi.org/10.1016/j.conbuildmat.2017.04.055>
- [20] Kumar A, Choudhary R, Kumar A. Aging characteristics of asphalt binders modified with tire and plastic chars. *PLoS One*. 2021;16. <https://doi.org/10.1371/journal.pone.0256030>
- [21] Ren S, Liu X, Xu J, Lin P. Role of swelling-degradation degree of crumb rubber on CR/SBS modified asphalt. *Construction and Building Materials*. 2021;300:124048. <https://doi.org/10.1016/j.conbuildmat.2021.124048>
- [22] Wang H, Liu X, Apostolidis P, Scarpas T. Non-Newtonian behaviour of crumb rubber-modified bitumen. *Applied Sciences*. 2018;8. <https://doi.org/10.3390/app8101760>
- [23] Ashish PK, Singh D. High and intermediate temperature performance of CNT modified binders. *Journal of Materials in Civil Engineering*. 2018;30. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0002106](https://doi.org/10.1061/(ASCE)MT.1943-5533.0002106)
- [24] Ashish PK, Singh D, Bohm S. Influence of nanoclay addition on rheological performance of asphalt binder. *Road Materials and Pavement Design*. 2017;18:1007–1026. <https://doi.org/10.1080/14680629.2016.1201522>
- [25] Mashaan NS, Ali AH, Karim MR, Abdelaziz M. Effect of crumb rubber concentration on physical and rheological properties of rubberised bitumen. *International Journal of Physical Sciences*. 2011;6:684–690. <https://doi.org/10.5897/IJPS11.113>
- [26] ASTM D2872. Standard test method for effect of heat and air on asphalt (rolling thin-film oven test). ASTM International, West Conshohocken, PA; 2022.
- [27] IS 1203. Method of testing tar and bituminous material: penetration. BIS, New Delhi, India; 2022.
- [28] IS 1205. Method of testing tar and bituminous material: softening point, ring-and-ball method. BIS, New Delhi, India; 2022.
- [29] ASTM D7173. Standard practice for determining separation tendency of polymer in modified asphalt. ASTM International, West Conshohocken, PA; 2020.
- [30] AASHTO TP-123. Method for measuring binder yield energy and elastic recovery using DSR. AASHTO, Washington DC; 2020.
- [31] Biro S, Gandhi T, Amirkhanian S. Zero shear viscosity of warm mix binders. *Construction and Building Materials*. 2009;23:2080–2086. <https://doi.org/10.1016/j.conbuildmat.2008.08.015>
- [32] Parvez MA, Al-Abdul Wahhab HI, Shawabkeh RA, Hussein IA. Asphalt modification using acid-treated waste oil fly ash. *Construction and Building Materials*. 2014;70:201–209. <https://doi.org/10.1016/j.conbuildmat.2014.07.045>
- [33] Wang H, Liu X, Zhang H, Apostolidis P, Erkens S, Skarpas A. Micromechanical modelling of complex shear modulus of crumb rubber modified bitumen. *Materials and Design*. 2020;188:108467. <https://doi.org/10.1016/j.matdes.2019.108467>
- [34] Gao Y, Cao R. Interaction theory of asphalt and rubber. *Journal of Wuhan University of Technology*. 2010;25:853–855. <https://doi.org/10.1007/s11595-010-0107-y>
- [35] Ibrahim B, Wiranata A, Zahrina I, Sentosa L, Nasruddin N, Muharam Y. Phase separation study on storage of natural-rubber modified bitumen. *Applied Sciences*. 2024;14. <https://doi.org/10.3390/app14083179>
- [36] Li Y, Abdelmagid AAA, Qiu Y, Yang E, Chen Y. Aging mechanism and microstructure of rice-husk-ash and crumb-rubber modified asphalt. *Polymers*. 2022;14. <https://doi.org/10.3390/polym14101969>
- [37] Zhao X, Wang S, Wang Q, Yao H. Rheological and structural evolution of SBS modified asphalt under natural weathering. *Fuel*. 2016;184:242–247. <https://doi.org/10.1016/j.fuel.2016.07.018>

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Recycling polypropylene waste to use as material in a 3D printer device

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Abstract

This study investigates the feasibility of mechanically recycling post-industrial polypropylene (PP) waste into homogeneous pellets and 3D-printable filaments for fused deposition modeling (FDM) without the use of chemical additives. The primary objective is to evaluate whether recycled polypropylene can retain sufficient mechanical, thermal, and structural integrity for additive manufacturing applications. The recycled PP was processed using single-screw extrusion to produce pellets and filaments, followed by fabrication of test specimens through injection molding and FDM printing. Mechanical performance was evaluated through tensile testing in accordance with ASTM D638, while thermal behavior and structural integrity were assessed using melting point analysis, Fourier transform infrared spectroscopy (FTIR), and scanning electron microscopy (SEM). The results demonstrate that pellet-based recycled polypropylene retained approximately 90% of the tensile strength of virgin polypropylene, whereas filament-based recycled polypropylene retained approximately 80%. FTIR analysis confirmed preservation of the polypropylene chemical backbone with no significant evidence of oxidative degradation, while SEM observations revealed minor micro voids and interlayer features associated with FDM processing. The successful fabrication of dimensionally stable 3D-printed components further confirms the processability of recycled polypropylene under standard FDM conditions. These findings indicate that mechanically recycled polypropylene can be effectively utilized in additive manufacturing, offering a simplified and sustainable pathway for polymer reuse without chemical modification.

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1. Introduction

Polypropylene (PP) is one of the most extensively produced thermoplastic polymers due to its low density, balanced mechanical properties, chemical resistance, thermal stability, and processing versatility. These characteristics have enabled its widespread application in automotive components, packaging systems, consumer products, medical devices, and prosthetic and orthotic applications [1,2]. The continuous growth in polypropylene consumption has consequently led to a substantial increase in polymer waste generation worldwide. The environmental persistence of polypropylene, attributed to its low biodegradability, has intensified concerns regarding solid waste accumulation and long-term ecological impact. Recycling has therefore become a critical strategy for mitigating plastic pollution, conserving raw materials, reducing energy demand, and

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lowering greenhouse gas emissions associated with virgin polymer production [3–6]. Among available waste-management approaches, mechanical recycling remains the most industrially adopted method for polypropylene recovery. Extensive research has evaluated the influence of recycling on the mechanical, thermal, and structural behavior of polypropylene. Previous investigations have shown that repeated thermal processing may induce chain scission, reduce molecular weight, and gradually deteriorate tensile strength and stiffness [8–10,13]. At the same time, polypropylene recycling technologies have been reported to significantly contribute to solid waste reduction and environmental sustainability [11]. Furthermore, studies have demonstrated that recycled polypropylene can be reintegrated into engineering and construction-related applications when appropriate processing strategies are implemented to maintain thermal stability and mechanical resistance [12]. Additional research has examined crystallinity evolution, degradation indicators, emissions during recycling, and composite reinforcement strategies, confirming the continued industrial relevance of recycled polypropylene [14–19].

Although the feasibility of recycled polypropylene has been established across various conventional manufacturing routes, including injection molding, extrusion into bulk products, and composite fabrication, many reported approaches rely on chemical modification, stabilizers, compatibilizers, or filler incorporation to compensate for degradation effects. While these strategies may enhance performance, they increase process complexity and economic cost, which can limit their applicability in simplified or resource-constrained recycling systems. In contrast, the direct use of mechanically recycled polypropylene as feedstock for additive manufacturing—particularly fused deposition modeling (FDM)—has received comparatively limited systematic investigation. Additive manufacturing requires stable melt flow behavior, consistent filament geometry, and sufficient interlayer bonding performance, all of which are sensitive to molecular degradation and thermal history. Therefore, evaluating whether recycled polypropylene can retain adequate structural and mechanical integrity for FDM applications without additional chemical modification is of significant scientific and practical importance. Accordingly, the present study investigates the feasibility of converting post-industrial polypropylene waste into pellets and subsequently into 3D printing filaments through mechanical recycling without incorporating chemical additives or secondary processing treatments. The recycled materials are characterized in pellet and filament forms using tensile testing, melting point analysis, Fourier transform infrared spectroscopy (FTIR), and scanning electron microscopy (SEM), and their properties are compared with those of virgin polypropylene. The applicability of the recycled filament in additive manufacturing is further demonstrated through the fabrication of functional printed models. By integrating mechanical recycling with additive manufacturing, this work aims to evaluate a simplified and sustainable pathway for polypropylene reuse and to contribute to circular manufacturing strategies that reduce environmental impact while maintaining functional material performance.

2. Materials and Method

The raw material used in this study was post-industrial polypropylene (PP) waste generated during manufacturing processes, including rejected pump molding parts and production remnants. This type of waste was selected due to its known origin, high purity, and absence of foreign polymer contamination, making it suitable for controlled mechanical recycling. The collected polypropylene waste used as the raw material in this study is shown in (Fig. 1).

Virgin polypropylene was also used as a reference material to enable a direct comparison between recycled and non-recycled PP. The fundamental properties of polypropylene reported in the literature, such as density and melting temperature, were used as baseline values for comparison with the recycled material. The fundamental physical properties of virgin polypropylene used as reference material are summarized in Table.1, based on supplier data and literature-reported values.



Fig. 1. The collected polypropylene waste used as the raw material

Table 1. Basic properties of virgin polypropylene used as reference material

Property	Value
Density	0.90–0.91 g/cm ³
Melting temperature	160–170 °C
Polymer type	Homopolymer PP
Physical form	Injection molding grade

2.1. Process of Recycling Polypropylene and Turning it into Homogeneous Spherical Pellets

The recycling of polypropylene waste into homogeneous spherical pellets is achieved through a series of mechanical processing steps designed to convert plastic waste into reusable raw material. The process involves collection, washing, shredding, extrusion, cooling, and pellet formation to obtain uniform pellets suitable for manufacturing applications. The complete sequence of these stages is presented in (Fig. 2).

2.2 Procedures of Recycling Polypropylene to Make Filament to Use in 3D Printing

The recycling of polypropylene waste into filaments suitable for 3D printing involves a series of controlled mechanical processing steps, including shredding, melting, extrusion, cooling, and filament diameter control. The main stages of this process are illustrated in Fig. 3. The overall mechanical recycling process of polypropylene waste, including melting, extrusion, and the production of pellets or filaments, is illustrated in (Fig. 4).

2.3 Tensile Test

The tensile tests were conducted using a universal testing machine (Testometric), as shown in (Fig. 5). Three different types of tensile specimens were prepared for mechanical characterization, namely virgin polypropylene as the reference material (Sample A), recycled polypropylene processed into filament form and fabricated by 3D printing (Sample B), and recycled polypropylene processed into pellet form, re-melted, and subsequently fabricated by 3D printing (Sample C). All tensile specimens were designed and manufactured in accordance with the ASTM D638 standard using Type I specimen geometry to ensure consistency and comparability of the mechanical test results. For Samples B and C, the tensile specimens were fabricated using fused deposition modeling (FDM) 3D printing technology. For Samples B and C, tensile specimens were fabricated using fused deposition modeling (FDM) technology with controlled and reproducible processing parameters.

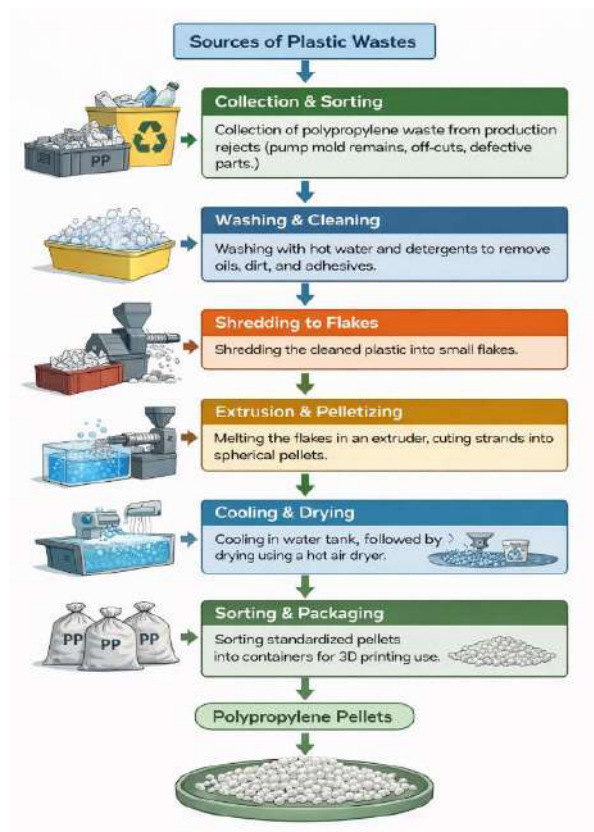


Fig. 2. Flowchart of the mechanical recycling process of polypropylene waste for the production polypropylene pellets

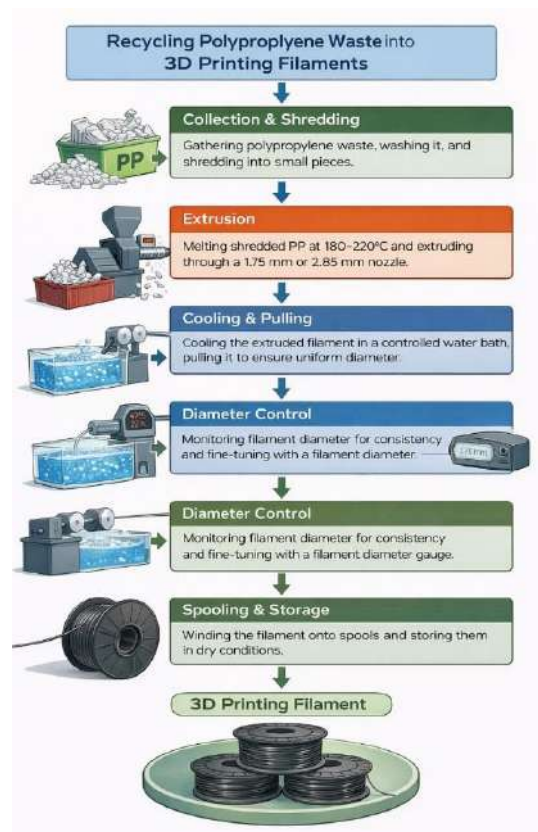


Fig. 3. Flow diagram illustrating the main stages of recycling polypropylene waste into 3D printing filaments



Fig. 4. Stages of recycling polypropylene waste into pellets or 3D-printing filaments

A nozzle diameter of 0.4 mm was used. The extrusion (nozzle) temperature was maintained at 205 °C (within a controlled range of 200–210 °C) to ensure adequate polymer melt flow and interlayer diffusion bonding. The build platform temperature was set at 95 °C to minimize warpage and enhance first-layer adhesion. Layer thickness was fixed at 0.2 mm to promote consistent layer stacking and interfacial fusion. Printing speed was maintained at 45 mm/s, corresponding to a controlled material deposition rate compatible with the selected nozzle diameter. A 100% infill density was used, and the printing direction was aligned parallel (0°) to the tensile loading axis to ensure maximum load transfer efficiency and minimize anisotropic strength reduction typically observed in FDM-printed specimens. Cooling was achieved under natural convection without forced fan cooling to allow gradual solidification and improved interlayer bonding. These parameters were selected to optimize interlayer adhesion and ensure repeatability of mechanical performance evaluation. Virgin polypropylene tensile specimens were produced using injection molding under controlled processing conditions. The mold cavity conformed to ASTM D638 Type I geometry. The standard specimen dimensions used in this study are illustrated in Fig. 6. The injection temperature was maintained between 200–220 °C, while the mold temperature was controlled at approximately 40 °C. Injection pressure was approximately 80–100 MPa with moderate injection speed to ensure uniform cavity filling. Cooling time was maintained at 30–40 seconds before specimen ejection. These parameters were selected to minimize residual stresses and ensure dimensional consistency. The printed specimens can be seen in (Fig. 7).



Fig. 5. The Testometric device

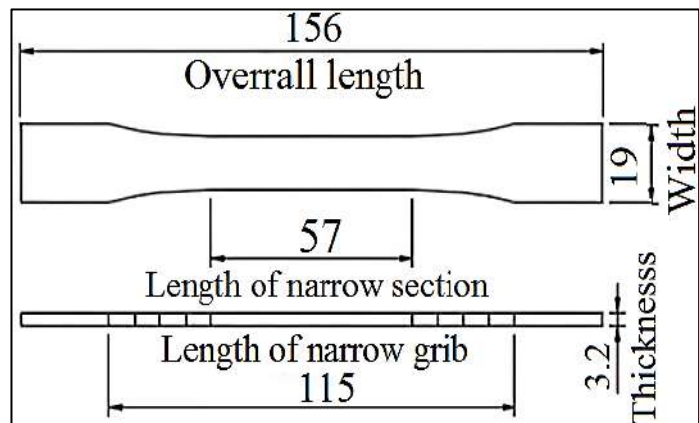


Fig. 6. The standard dimensions according to ASTM D638

While injection-molded virgin polypropylene specimens exhibited smooth and continuous surfaces, the 3D-printed recycled specimens showed clearly distinguishable layer-by-layer deposition patterns. These visible interlayer boundaries are characteristic of FDM processing and contribute to the slight reduction in tensile strength observed in filament-based samples.

Sample A-Raw Polypropylene (Raw PP): Symbolises the initial, which is not recycled. **Sample B-Filament recycled Polypropylene:** The plastic waste in the form of polypropylene was recycled into filaments that could be used in 3D printing. The test specimens were printed as 100% infill filaments. **Sample C- Recycled Polypropylene (Pellet form):** The recycled polypropylene was moulded into pelletized spherical form, which was subsequently mixed and re-melted as the feed material to extrude the test specimens with an infill density of 100 per cent by use of 3D printing.

To ensure its accuracy and reproducibility, each of the types of samples was tested at least three times under the same conditions of testing, such as the same tensile speed, ambient temperature, and the same specimen size. Some of the mechanical parameters that were recorded during the testing process are stress, strain, ultimate tensile strength, Young's modulus and the elongation at break. These parameters gave thorough insight into the mechanical behavior of the material before and after recycling. The printed tensile specimens clearly exhibited visible layer deposition lines

characteristic of fused deposition modeling, confirming the additive manufacturing origin of Samples B and C.

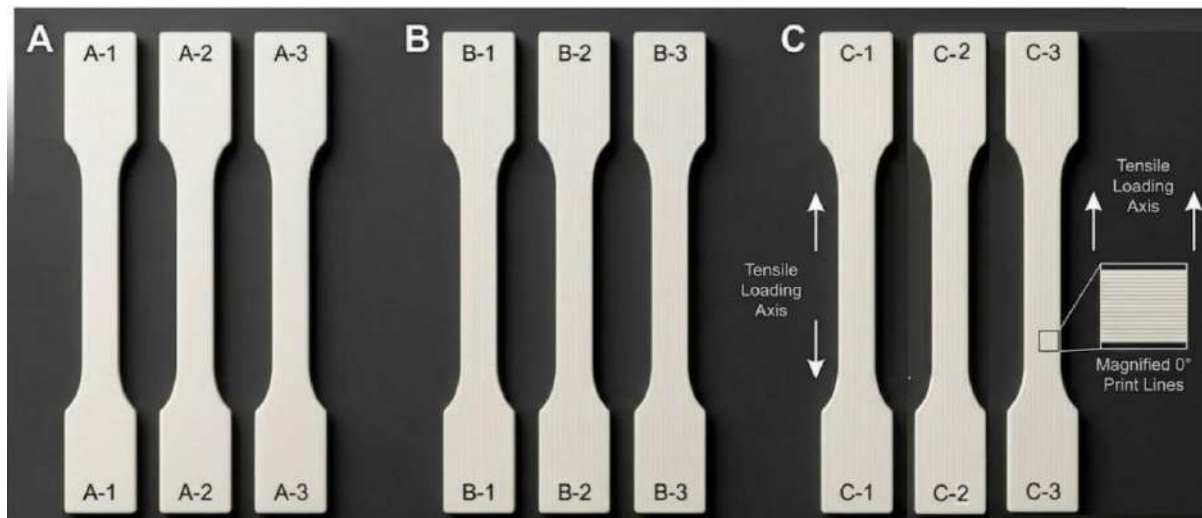


Fig. 7. Tensile specimens for the three groups to be tested

2.4 Test the Melting Point of Recycled Polypropylene by the Stuart SMP1 Device

This test aims to find out the melting point of a recycled polypropylene (PP) and also how much the thermal property is maintained after the recycling process. This breakdown will be critical in knowing whether the material can be reused in the form of thermal manufacturing processes like 3D printing. Equipment Used:

- -Melting point apparatus (Stuart SMP1) as shown in Fig. 8.
- - Fine capillary tubes in which the sample will be placed.
- - The device contains an in-built thermometer.



Fig. 8. Stuart SMP1

A minute amount (in the form of pellets or fine powder) of a recycled polypropylene was removed and inserted into a capillary tube compatible with the Stuart SMP1 device. The heating rate was made slowly, around C/min 5-10. Heating was commenced, and it was observed manually through the device observation window. The temperature at which the material started changing its state from solid to a liquid was recorded. Other samples were used in repeating the test to demonstrate the accuracy of the results.

2.5 Test Using FTIR of Recycled Polypropylene

FTIR analysis was performed to evaluate the chemical structure of recycled polypropylene and to detect any potential chemical degradation or oxidation resulting from the recycling process.

Recycled polypropylene samples were prepared as thin films and analyzed using an FTIR spectrometer over the spectral range of $4000\text{--}500\text{ cm}^{-1}$. The obtained spectra were compared with those of virgin polypropylene to identify characteristic absorption peaks and assess structural integrity

2.6 Scanning Electron Microscopy (SEM) Analysis

The surface and fracture morphologies of virgin polypropylene and recycled polypropylene (pellet-based and filament-based) samples were examined using scanning electron microscopy (SEM). Small specimens were mounted on aluminum stubs using conductive carbon tape and sputter-coated with a thin layer of gold to enhance electrical conductivity. SEM observations were carried out at accelerating voltages between 10 and 15 kV, with magnifications ranging from $100\times$ to $2000\times$, allowing detailed assessment of surface texture, porosity, and internal morphology.

3. Results

Representative optical images of the recycled pellets and extruded filaments are shown in (Fig. 9) and (Fig. 10). The pellets exhibited uniform geometry and homogeneous surface texture, indicating stable extrusion and cutting processes. The produced filaments demonstrated consistent diameter and smooth surface continuity suitable for FDM processing.



Fig. 9. The particle of polypropylene recycled



Fig. 10. The filament of polypropylene recycled

The tensile properties of virgin polypropylene and recycled polypropylene in pellet and filament forms were evaluated using uniaxial tensile testing. Representative stress–strain curves for all tested materials are presented in Fig. 11, while the corresponding mechanical properties are summarized in Table 2. Virgin polypropylene exhibited the highest mechanical performance, with an average yield strength of approximately 22 MPa, an ultimate tensile strength of 36 MPa, and a Young's modulus of about 1.6 GPa. Grain or small pellet recycled polypropylene showed a slight reduction in mechanical properties, recording an average yield strength of approximately 19 MPa and an ultimate tensile strength of 34 MPa. Recycled polypropylene in filament form exhibited a further reduction, with an average yield strength of approximately 16 MPa and an ultimate tensile strength of 33 MPa. Despite the observed reductions, both recycled materials retained a substantial portion of the mechanical properties of virgin polypropylene. The standard deviation values for all measured properties were low, indicating good repeatability and consistency of the experimental results across the tested specimens.

To statistically evaluate the differences in mechanical properties between virgin polypropylene and recycled polypropylene samples, a one-way analysis of variance (ANOVA) was performed using a significance level of $p < 0.05$. The analysis revealed a statistically significant difference in yield strength among the three groups ($p < 0.05$). Post-hoc comparison indicated that filament-based recycled polypropylene exhibited a significant reduction compared to virgin polypropylene. However, the difference between virgin polypropylene and pellet-based recycled polypropylene was less pronounced, indicating that mechanical degradation remains controlled after single-cycle

mechanical recycling. These findings confirm that the observed reductions in tensile properties are not due to experimental variability but are statistically supported.

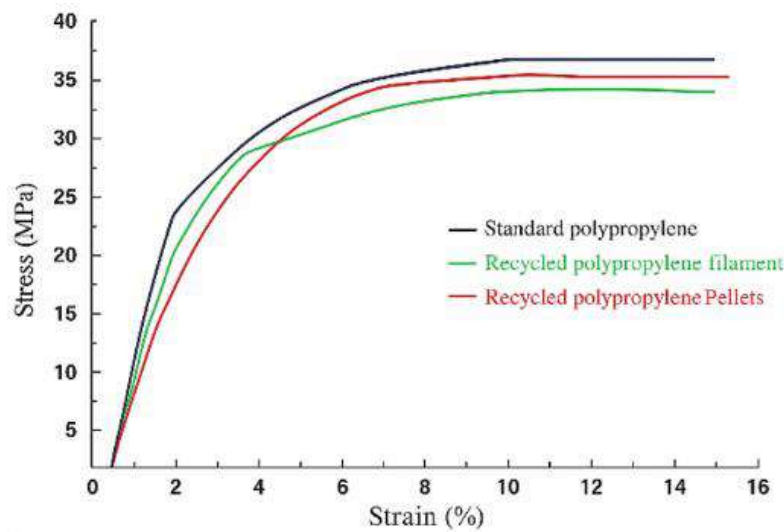


Fig. 11. Stress-Strain curve of standard and recycled polypropylene

Table 2. Summary of mechanical properties of standard and recycled polypropylene

Group	Material Type	Sample	Yield Strength (MPa)	Ultimate Tensile Strength (MPa)	Young's Modulus (GPa)
A	Standard Polypropylene	1	22.1	36.2	1.6
		2	21.9	35.9	1.59
		3	22	36	1.61
		Mean $\pm$ SD	22 $\pm$ 0.1	36 $\pm$ 0.2	1.6 $\pm$ 0.01
B	Recycled Polypropylene Filament	1	19	34.2	1.31
		2	19.1	33.7	1.29
		3	18.9	34.1	1.3
		Mean $\pm$ SD	19 $\pm$ 0.1	34 $\pm$ 0.2	1.3 $\pm$ 0.01
C	Recycled Polypropylene Pellets	1	16.1	33.2	1.21
		2	15.9	32.9	1.19
		3	16	33	1.2
		Mean $\pm$ SD	16 $\pm$ 0.1	33 $\pm$ 0.15	1.2 $\pm$ 0.01

The tensile strength values obtained for virgin polypropylene in this study fall within the typical range of 30–38 MPa reported in the literature for injection-molded homopolymer polypropylene [10,13]. The slight reduction observed in recycled filament-based specimens is consistent with previously reported degradation effects during reprocessing cycles. Nasir et al. [13] reported gradual decreases in tensile strength and modulus due to chain scission and molecular weight reduction during repeated recycling. Similarly, Infurna et al. [10] observed mechanical performance deterioration in recycled polypropylene blends. The results of the present study align well with these findings, confirming that mechanical recycling without stabilizers may induce moderate but predictable strength reduction.

Table 2 shows yield, ultimate tensile, and Young's modulus of the virgin and recycled polypropylene (PP) materials. The groups were subjected to three tests ($n = 3$). The findings have revealed that the raw PP recorded the best mechanical properties, whereas the recycled materials (PP) gave a slightly lower value that depended on the process of recycling. The low standard deviation values are a measure of good repeatability and prove the statistical reliability of the results.

The melting temperatures of recycled polypropylene samples were measured using a Stuart SMP1 melting point apparatus. The recorded melting temperatures for both pellet-based and filament-

based recycled polypropylene were in the range of 160–170 °C. These values are comparable to those typically reported for virgin polypropylene, indicating that the recycling process did not cause a significant shift in the melting behavior of the polymer. Repeated measurements showed minimal variation, confirming the reliability of the obtained results.

FTIR spectra of virgin polypropylene and recycled polypropylene in pellet and filament forms are presented in Fig. 12. All samples exhibited the characteristic absorption peaks of polypropylene. Strong absorption bands were observed in the range of 2950–2870 cm^{-1} , corresponding to C–H stretching vibrations of the $-\text{CH}_2$ and $-\text{CH}_3$ groups. Additional characteristic peaks were detected between 1450 and 1375 cm^{-1} , which are associated with bending and deformation vibrations of these functional groups. The FTIR spectra of recycled polypropylene closely matched that of virgin polypropylene, with no additional peaks indicating the formation of new chemical groups. Minor variations in peak intensity were observed between samples but no significant changes in peak position were detected.

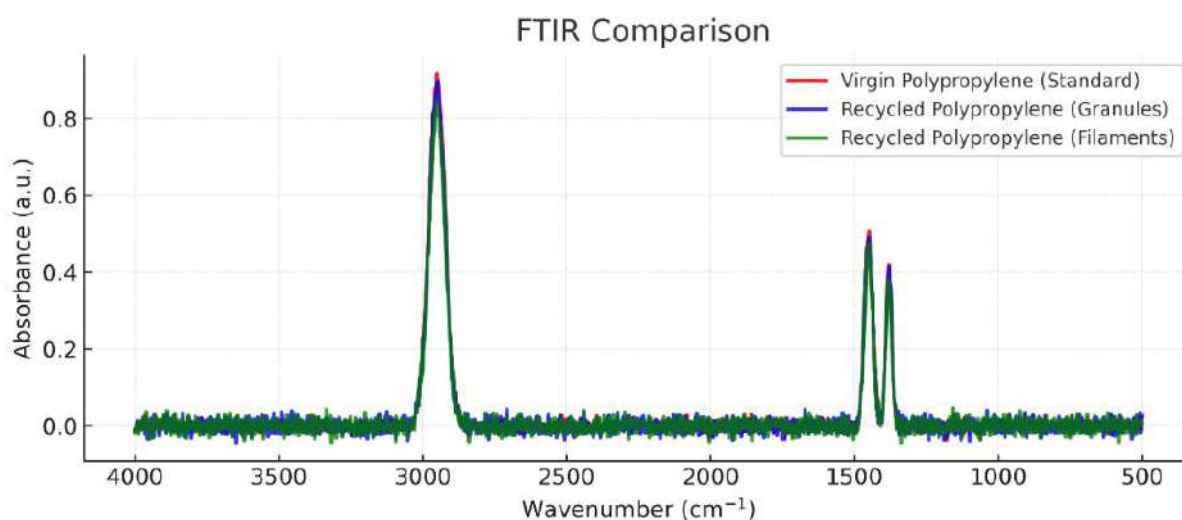


Fig. 12. FTIR spectrum comparison between virgin and recycled polypropylene

The FTIR spectra of recycled polypropylene did not reveal significant new absorption peaks associated with carbonyl groups ($\sim 1700\text{--}1750\text{ cm}^{-1}$), which are commonly attributed to thermo-oxidative degradation during processing. This observation suggests limited oxidative damage during the applied recycling procedure. Similar findings were reported by Polachova et al. [8], who indicated that controlled thermoplastic reprocessing does not necessarily introduce severe oxidation if processing temperatures are maintained within stable ranges. The preservation of characteristic PP peaks (e.g., CH_2 and CH_3 stretching vibrations) further confirms chemical structure stability. SEM micrographs revealed relatively homogeneous fracture surfaces in injection-molded specimens, whereas filament-printed samples showed characteristic interlayer boundaries. Comparable fracture morphologies have been documented in FDM-printed polypropylene systems, where interlayer bonding plays a critical role in tensile performance. Previous studies on recycled polypropylene have also reported increased surface roughness and micro void formation associated with thermal history and molecular degradation [15]. The morphological features observed in the present work are therefore consistent with established degradation mechanisms reported in the literature.

Fig. 13 presents SEM micrographs of three polypropylene samples: virgin polypropylene (Sample A), recycled polypropylene in pellet form (Sample C), and recycled polypropylene in filament form (Sample B). The virgin polypropylene sample exhibited a smooth and dense surface morphology with a nearly circular cross-section and no observable voids, with an approximate diameter in the range of 1.70–1.80 mm. The pellet-based recycled polypropylene sample showed a slightly rougher surface with minor surface irregularities and small voids. The cross-section remained nearly circular, with an approximate diameter of 1.65–1.84 mm, indicating good structural continuity after recycling. The filament-based recycled polypropylene sample exhibited increased surface roughness and the presence of micro void-like features, along with a somewhat irregular cross-

sectional geometry. The measured diameter ranged approximately between 1.60 and 1.70 mm. In addition to SEM analysis, the fractured tensile specimens were visually examined at the macroscopic level.

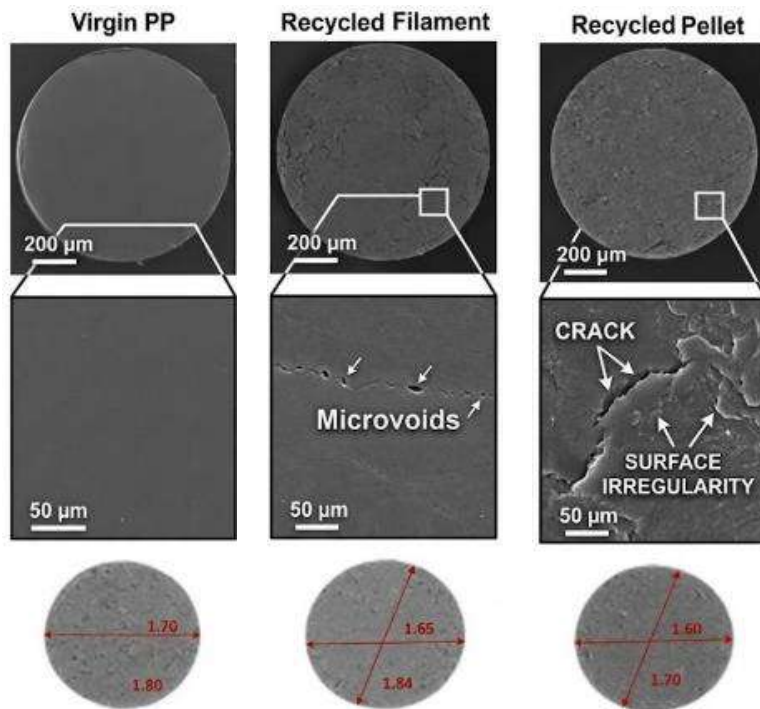


Fig.13. The SEM of virgin PP and recycled PP



Fig. 14. 3D printed model of recycled polypropylene

The fracture surfaces of virgin polypropylene showed pronounced necking and ductile deformation prior to failure, which is characteristic of semi-crystalline thermoplastics. Recycled polypropylene specimens, particularly filament-based samples, exhibited visible stress whitening and localized plastic deformation at the fracture zone. The presence of neck formation confirms ductile failure behavior rather than brittle delamination between layers. Although minor interlayer separation was observed in filament-printed specimens, catastrophic interfacial failure was not detected, indicating acceptable interlayer bonding under the selected FDM parameters. To experimentally validate the processability of recycled polypropylene, the material was extruded into filament and subsequently used in fused deposition modeling (FDM) 3D printing to fabricate a cosmetic hand prototype of a prosthetic limb, as shown in Fig. 14. The printing process was completed successfully without interruption, filament breakage, or dimensional instability. The fabricated prototype exhibited satisfactory shape fidelity and surface continuity, demonstrating the suitability of recycled polypropylene filaments for complex geometries produced by additive manufacturing. This application serves as a proof of concept highlighting the potential of integrating polypropylene recycling with 3D printing technologies for non-load-bearing and demonstrative prosthetic components.

Dimensional stability of the printed specimens was assessed through visual inspection and diameter measurements of printed tensile samples. Filament diameter variation remained within ± 0.05 mm, and no significant warpage or corner lifting was observed during printing or post-cooling. The maintained geometric integrity indicates controlled shrinkage behavior and acceptable dimensional predictability of recycled polypropylene under the selected FDM conditions. Although precise shrinkage percentage was not quantified, the successful fabrication of geometrically stable components suggests reliable dimensional performance for non-load-bearing applications. The use of recycled polypropylene instead of virgin polypropylene offers notable environmental benefits, particularly in terms of energy consumption and greenhouse gas emissions, as reported in life cycle assessment studies. It should be clarified that the energy consumption and emission values presented in this section were not experimentally measured

within the scope of the present study. Instead, they are derived from previously published life-cycle assessment (LCA) studies and international environmental reports addressing polypropylene production and recycling systems [16–18]. These literature-based data are included solely for comparative and contextual purposes to highlight the potential environmental benefits of mechanical recycling relative to virgin polymer production. According to published data, the production of virgin polypropylene requires approximately 80 MJ/kg of energy, whereas mechanical recycling of polypropylene into pellets or filaments typically consumes between 25 and 35 MJ/kg. Similarly, carbon dioxide emissions associated with virgin polypropylene production, estimated at around 2.0 kg CO₂/kg, are significantly reduced to approximately 0.6–0.8 kg CO₂/kg when recycled polypropylene is used. In addition, recycled polypropylene can partially replace virgin material, with studies indicating that 50–70% of material demand can be fulfilled using post-consumer polymers. These reported reductions correspond to energy and emission savings of approximately 60–65%, highlighting the potential of polypropylene recycling to lower material demand, reduce plastic waste generation, and decrease the overall carbon footprint of polymer-based manufacturing processes [20–21]. Therefore, the presented environmental comparison serves as a qualitative sustainability indicator rather than a direct experimental outcome of this work.

4. Discussion

The tensile results showed that recycled polypropylene in pellet form retained approximately 86–90% of the tensile strength of virgin PP, while filament-based recycled PP retained around 80–85%. In fused deposition modeling (FDM), mechanical performance is strongly governed by interlayer diffusion bonding rather than bulk material continuity. During deposition, molten polypropylene filaments fuse through thermal diffusion across adjacent layers. The effectiveness of this bonding depends on melt temperature, cooling rate, and polymer chain mobility at the interface. The slight reduction in tensile strength observed in filament-based recycled polypropylene can therefore be attributed to limited interlayer diffusion and the presence of microscopic interfacial discontinuities. Since load transfer in FDM-printed components occurs across these bonded interfaces, any reduction in molecular interpenetration reduces effective stress transfer efficiency. The consistent printing parameters and full infill structure applied in this study minimized structural variability, suggesting that the observed strength reduction is primarily governed by intrinsic interlayer bonding limitations rather than macroscopic defects. The slight reduction in yield strength and Young's modulus can be attributed to thermal reprocessing effects such as chain scission and minor oxidative degradation during extrusion and FDM printing. Similar behavior was reported by Douiri, L. et al., who investigated multiple recycling loops of polypropylene and observed a gradual decrease in tensile strength due to molecular weight reduction and crystallinity changes during repeated thermal cycles. Their study demonstrated that mechanically recycled PP typically maintains 75–90% of its original tensile properties after limited recycling stages, which aligns closely with the retention ratios observed in the present work [22].

The melting temperature of recycled polypropylene (160–170 °C) remained within the characteristic range of virgin PP, indicating preservation of crystalline phase stability after recycling. Although Differential Scanning Calorimetry (DSC) was not performed in the present study, the measured melting temperature range (160–170 °C) remained consistent with typical semi-crystalline polypropylene behavior, suggesting preservation of the crystalline phase structure after recycling. Furthermore, the relatively stable Young's modulus values (1.2–1.3 GPa for recycled samples compared to 1.6 GPa for virgin PP) indicate that no drastic alteration in crystallinity occurred. Since stiffness and shrinkage behavior in polypropylene are strongly correlated with crystalline fraction, the moderate reduction in modulus suggests limited structural disruption rather than significant phase transformation. Future investigations will incorporate DSC analysis to quantify crystallinity changes and correlate them directly with mechanical and dimensional performance. The stability of melting behavior suggests that the recycling process did not significantly alter lamellar crystal structure. Comparable findings were reported by Mannheim and Simenfalvi, who demonstrated through life-cycle and thermal evaluation studies that mechanically recycled polypropylene generally preserves its melting transition unless severe

oxidative degradation occurs. The thermal stability observed in the present study therefore supports the suitability of recycled PP for repeated thermal processing such as FDM printing [23].

FTIR spectra of recycled PP closely matched those of virgin PP, with characteristic peaks at 2950–2870 cm^{-1} and 1450–1375 cm^{-1} preserved. No additional carbonyl peaks ($\approx 1700\text{--}1750\text{ cm}^{-1}$) were detected, indicating minimal oxidative degradation. Similar spectral stability was observed by Nasir, N.H. et al., who reported that mechanically recycled polypropylene composites retained their fundamental chemical structure with only minor intensity variations in FTIR spectra [24]. Furthermore, Polachova, A. et al. indicated that early-stage thermal degradation of polypropylene is often detectable by slight peak intensity changes rather than the formation of new functional groups, which corresponds with the observations in the current study. These findings confirm that the recycling procedure did not significantly alter the polymer backbone structure [25]. Although Gel Permeation Chromatography (GPC) analysis was not performed to directly quantify molecular weight distribution, several indirect indicators suggest limited chain degradation during recycling. The retention of approximately 80–90% of tensile strength and the absence of significant carbonyl peak formation in FTIR spectra indicate that severe chain scission or oxidative breakdown did not occur. In polypropylene, substantial molecular weight reduction typically results in pronounced decreases in tensile strength and modulus. The moderate mechanical reduction observed in this study therefore suggests partial but controlled molecular weight alteration rather than extensive polymer backbone degradation. Future work will incorporate GPC measurements to quantitatively correlate molecular weight evolution with mechanical performance. Although Melt Flow Index (MFI) testing was not performed in this study, extrusion stability, absence of nozzle clogging, and consistent filament diameter (1.60–1.75 mm) provide indirect indicators of acceptable melt viscosity. Since viscosity behavior is directly related to molecular weight evolution during recycling, future work will include quantitative MFI measurements to establish a direct correlation between rheological changes and mechanical performance. Thermogravimetric Analysis (TGA) was not conducted in this study to quantify decomposition onset temperature and thermal stability margins. However, polypropylene typically exhibits thermal degradation onset above 300 °C, which is substantially higher than the processing temperatures used in extrusion and FDM (200–210 °C). The absence of discoloration, bubbling, or flow instability during repeated processing cycles suggests that no premature thermal decomposition occurred. Nevertheless, future studies will incorporate TGA measurements to evaluate long-term thermal aging resistance and degradation kinetics under multiple recycling and manufacturing cycles.

SEM analysis of recycled polypropylene specimens revealed a rougher and more porous fracture surface compared to the relatively smooth morphology of virgin PP, especially in filament-based samples that underwent extrusion and 3D printing. These morphological irregularities, such as micro voids and interlayer gaps, can weaken interfacial bonding and act as stress concentrators, leading to the reduction in tensile strength observed in recycled specimens. A similar effect was reported in a recent study on recycled polypropylene for additive manufacturing, where the fracture surfaces of 3D-printed recycled PP showed pronounced interlayer cavities and porosity, which were directly linked to diminished mechanical performance due to reduced effective load-bearing area and poorer interlayer adhesion. This confirms that microstructural defects induced during processing (e.g., extrusion and printing) contribute to the mechanical degradation of recycled PP parts [26–28]. Although SEM observations were primarily qualitative, comparative analysis of fracture surfaces indicates that filament-based recycled polypropylene exhibited a higher density of micro void-like features relative to pellet-based samples. The presence of these voids reduces the effective load-bearing cross-sectional area and can act as localized stress concentrators during tensile loading. While quantitative porosity measurements were not performed, the observed morphological differences correlate consistently with the measured reduction in tensile strength and modulus. Future investigations will include image-based porosity quantification and microstructural statistical analysis to establish a direct numerical relationship between void fraction and mechanical durability. The present study intentionally excluded the use of antioxidants or thermal stabilizers in order to evaluate the baseline mechanical and structural performance of mechanically recycled polypropylene. While stabilizer integration is commonly employed in industrial recycling to mitigate oxidative degradation and extend long-term durability,

the objective here was to assess intrinsic material retention without chemical modification. The retained mechanical performance (80–90%) suggests that limited degradation occurred under the selected processing conditions. Future optimization studies will investigate the incorporation of stabilizing additives to further enhance long-term structural reliability and storage stability of recycled polypropylene filaments.

It should be acknowledged that the present study focused primarily on tensile characterization. For prosthetic and orthotic applications, materials are typically subjected to cyclic loading, bending stresses, and occasional impact forces during real-world use. Therefore, additional mechanical evaluations such as Charpy or Izod impact testing and three-point bending tests are recommended to comprehensively assess long-term structural reliability. The current tensile results demonstrate promising baseline strength retention; however, future investigations will incorporate impact resistance and flexural behavior analysis to further validate suitability for load-bearing biomedical components.

5. Conclusions

This study demonstrated the feasibility of mechanically recycling polypropylene waste into pellets and filaments suitable for fused deposition modeling (FDM) 3D printing without the use of chemical additives. The recycled polypropylene retained a substantial proportion of the mechanical properties of virgin polypropylene, with pellet-based recycled material showing slightly higher performance than filament-based recycled material due to reduced thermal reprocessing. This observation highlights the influence of repeated thermal cycles on the mechanical stability of thermoplastic polymers and emphasizes the importance of minimizing unnecessary processing steps during recycling.

Thermal analysis confirmed that the melting behavior of recycled polypropylene remained within the typical range of virgin polypropylene, indicating preservation of the polymer's crystalline structure after recycling. FTIR results showed that the characteristic chemical structure of polypropylene was maintained, with no evidence of significant chemical degradation. These findings suggest that the mechanical recycling method applied in this study did not lead to severe polymer chain scission or oxidation reactions. SEM observations revealed increased surface roughness and micro voids in recycled samples compared to virgin polypropylene; however, no critical structural defects were observed that would significantly compromise the integrity of the printed components.

The successful fabrication of 3D-printed specimens and a cosmetic prosthetic hand prototype further demonstrated the processability of recycled polypropylene using standard FDM conditions, serving as a proof of concept for additive manufacturing applications. The printed components showed stable extrusion behavior and acceptable dimensional consistency, indicating that recycled polypropylene filament can be processed reliably under typical printing parameters. These results highlight the potential of integrating recycled thermoplastics into additive manufacturing workflows for practical engineering and biomedical prototype applications.

From an environmental perspective, the use of recycled polypropylene provides clear sustainability advantages. Recycling plastic waste reduces the need for virgin polymer production, which is associated with significant energy consumption and greenhouse gas emissions. Reported literature data indicate that the use of recycled polypropylene can significantly reduce energy consumption, carbon dioxide emissions, and virgin material demand compared to conventional polypropylene production. Therefore, converting plastic waste into functional 3D printing filament supports circular economy principles and contributes to more sustainable materials management.

Although the results are promising, some limitations should be acknowledged. Advanced characterization techniques such as melt flow index analysis, differential scanning calorimetry for detailed crystallinity quantification, and molecular weight distribution measurements were beyond the scope of the present study. Future research could incorporate these analyses to better understand the rheological behavior, crystallization mechanisms, and long-term stability of recycled polypropylene during repeated processing cycles.

Overall, the findings confirm that recycled polypropylene is a viable material for additive manufacturing of non-load-bearing components, offering a practical route to reduce plastic waste and support more sustainable polymer processing while maintaining acceptable mechanical and structural performance.

References

- [1] Caudle B, Nguyen TT, Kataoka S. Evaluation of three solvent-based recycling pathways for circular polypropylene. *Green Chemistry*, 2025; 27: 1667-1678. <https://doi.org/10.1039/D4GC02646B>
- [2] Teklemuz TG, Gereziher AG, Tesfay AG. Enhancing the machinability of recycled polypropylene composites reinforced with waste cotton and pine sawdust through milling parameters optimization. *Journal of Thermoplastic Composite Materials*, 2025; 38: 08927057251331588. <https://doi.org/10.1177/08927057251331588>
- [3] Kouranou D, Galanopoulou C, Korres DM, Tzani A, Detsi A, Vouyiouka S. Restabilization of post-consumer recycled polypropylene with natural antioxidants from spent coffee residue. *Journal of Applied Polymer Science*, 2025; 142: e56494. <https://doi.org/10.1002/app.56494>
- [4] Karahan M, Özyurt İ, Atalay ŞÖ, Turan ŞY, Haji A, Karahan N. Comparative study of virgin and recycled polyethylene terephthalate and polypropylene intermingled thermoplastic composites. *Polymer Composites*, 2025; 46: 3820-3836. <https://doi.org/10.1002/pc.29209>
- [5] Lee JE, Lee D, Lee J, Park YK. Current methods for plastic waste recycling: Challenges and opportunities. *Chemosphere*, 2025; 370: 143978. <https://doi.org/10.1016/j.chemosphere.2024.143978>
- [6] Dehghani S, Salehiyan R, Srithep Y. Mechanical recycling of polyamide 6 and polypropylene for automotive applications. *Polymer Engineering & Science*, 2025; 65: 250-257. <https://doi.org/10.1002/pen.27006>
- [7] Oladele IO, Falana OS, Okoro CJ, Onuh LN, Akinbamiyori I, Akinrinade SO, Adegun MH, Odemona ET. Sustainable composites reinforced with glass fiber and bio-derived calcium carbonate in recycled polypropylene. *Hybrid Advances*, 2025; 8: 100357. <https://doi.org/10.1016/j.hybadv.2024.100357>
- [8] Polachova A, Cisar J, Novak M, Dusankova M, Sedlarik V. Effect of repeated thermoplastic processing of polypropylene matrix on the generation of low-molecular-weight compounds. *Polymer Degradation and Stability*, 2025; 238: 111337. <https://doi.org/10.1016/j.polymdegradstab.2025.111337>
- [9] Dib A, Fantuzzi N, Agnelli J, Paleari L, Bragaglia M, Nanni F, Pierattini A. Parametric characterization of recycled polymers with nanofillers. *Polymer Composites*, 2025; 46: 1080-1089. <https://doi.org/10.1002/pc.27663>
- [10] Infurna G, Romani A, Riccelli MC, Levi M, Incarnato L, Dintcheva NT. Recycling of personal protective equipment: Sanitisation, degradation and polymer blend formulations. *Polymer Degradation and Stability*, 2025; 234: 111249. <https://doi.org/10.1016/j.polymdegradstab.2025.111249>
- [11] Abdurakhimov FF, Omonov BM. Sustainable approaches for polypropylene recycling: Challenges and future perspectives. *IMRAS*, 2024; 7: 163-166.
- [12] Dokmai W, Ratanawilai T. Improved mechanical, thermal, and flame-retardant properties of hybrid composites with different recycled plastics. *Polymer Bulletin*, 2024; 81: 8119-8139. <https://doi.org/10.1007/s00289-023-05089-6>
- [13] Nasir NH, Usman F, Woen EL, Ansari MN, Sofyan M. Effect of fly ash on mechanical properties and microstructural behaviour of injected moulded recycled polypropylene composites. *Construction and Building Materials*, 2024; 447: 138130. <https://doi.org/10.1016/j.conbuildmat.2024.138130>
- [14] Kasper JB, Parker LA, Postema S, Höppener EM, Leighton AH, Finnegan AM, Rutten SB, Nijman J, Larasati A, Soares AC, van Eijk MC. Losses and emissions in polypropylene recycling from household packaging waste. *Waste Management*, 2025; 191: 230-241. <https://doi.org/10.1016/j.wasman.2024.11.029>
- [15] Douiri L, Jdidi H, Kordoghli S, Sleiman GE, Béreaux Y. Degradation indicators in multiple recycling processing loops of impact polypropylene and high density polyethylene. *Polymer Degradation and Stability*, 2024; 219: 110617. <https://doi.org/10.1016/j.polymdegradstab.2023.110617>
- [16] Sambyal P, Najmi P, Sharma D, Khoshbakhti E, Hosseini H, Milani AS, Arjmand M. Plastic recycling: Challenges and opportunities. *Canadian Journal of Chemical Engineering*, 2025; 103: 2462-2498. <https://doi.org/10.1002/cjce.25531>
- [17] Lacourt C, Mukherjee K, Garthoff J, O'Sullivan A, Meunier L, Fattori V. Recent and emerging food packaging alternatives: Chemical safety risks, current regulations, and analytical challenges. *Comprehensive Reviews in Food Science and Food Safety*, 2024; 23: e70059. <https://doi.org/10.1111/1541-4337.70059>
- [18] Grubb CA, Mokhtarnejad M, Greene J, Misasi J, Keffer DJ, Kardos M, Mainka H, Harper DP. Development of an automotive-relevant recycling process for paper fiber-reinforced polypropylene composites. *Recycling*, 2024; 9: 126. <https://doi.org/10.3390/recycling9060126>
- [19] Chia E, Nguyen HB, Le KN, Bi K, Pham TM. Performance of hybrid basalt-recycled polypropylene fibre reinforced concrete. *Structures*, 2025; 75: 108711. <https://doi.org/10.1016/j.istruc.2025.108711>

- [20] Mannheim V, Simenfalvi Z. Total life cycle of polypropylene products: Reducing environmental impacts in the manufacturing phase. *Polymers*, 2020; 12(9): 1901. <https://doi.org/10.3390/polym12091901>
- [21] Alsabri A, Tahir F, Al-Ghamdi SG. Environmental impacts of polypropylene (PP) production and prospects of its recycling in the GCC region. *Materials Today: Proceedings*, 2022; 56: 2245-2251. <https://doi.org/10.1016/j.matpr.2021.11.574>
- [22] Douiri L, Jdidi H, Kordoghli S, Sleiman GEH, Béreaux Y. Degradation indicators in multiple recycling processing loops of impact polypropylene and high density polyethylene. *Polymer Degradation and Stability*, 2024; 219: 110617. <https://doi.org/10.1016/j.polymdegradstab.2023.110617>
- [23] Mannheim V, Simenfalvi Z. Total life cycle of polypropylene products: Reducing environmental impacts in the manufacturing phase. *Polymers*, 2020; 12(9): 1901. <https://doi.org/10.3390/polym12091901>
- [24] Nasir NHM, Usman F, Woen EL, Ansari MNM, Sofyan M. Effect of fly ash on mechanical properties and microstructural behaviour of injected moulded recycled polypropylene composites. *Construction and Building Materials*, 2024; 447: 138130. <https://doi.org/10.1016/j.conbuildmat.2024.138130>
- [25] Polachova A, Cisar J, Novak M, Dusankova M, Sedlarik V. Effect of repeated thermoplastic processing of polypropylene matrix on the generation of low-molecular-weight compounds. *Polymer Degradation and Stability*, 2025; 238: 111337. <https://doi.org/10.1016/j.polymdegradstab.2025.111337>
- [26] Polline M, Mutua JM, Mbuya TO, Ernest K. Recipe development and mechanical characterization of carbon fibre reinforced recycled polypropylene 3D printing filament. *Open Journal of Composite Materials*, 2021; 11(3): 47-61. <https://doi.org/10.4236/ojcm.2021.113005>
- [27] Ejakiran C, Kumar SKVSTL, Rani MS. Flexural behavior of ternary blended concrete beams with GFRP rebars strengthened with steel fiber and polypropylene fiber. *Research on Engineering Structures and Materials*, 2026; 12(1): 407-424. <http://dx.doi.org/10.17515/resm2025-961me0612rs>
- [28] Malla CS, Mukunda RD, Nandipati S. Performance and environmental impact analysis of alkali activated slag and OPC concretes with polypropylene fiber in sustainable construction. *Research on Engineering Structures and Materials*, 2025; 11(6): 3289-3307. <https://doi.org/10.17515/resm2025-921ma0524rs>

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Evaluation of the thermal performance of foamed glass cladding panels in mitigating the cooling load for the buildings in hot dry zone

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Abstract

Excess heat through external walls in the summer time can be eliminated by replacing common cladding sheets by effective thermal insulation panels made of foamed glass. These materials have several interested features as well like: acoustic insulation, water proofing, anti-corrosion, durability and sustainable environmental effects. The current work introduces thermal behavior of foamed glass in reducing the cooling load which has been compared to panels made of plastics that used as common external reflective sheets. The specimens have been processed manually in the labs as well as experimental measurements for man samples of several blowing agent content. The aim is to determine density, porosity, expansion ratio and thermal conductivity coefficient. Images of SEM have been taken for selected samples to recognize the morphology and structure of the foamed glass. The results show that minimum density of the foam was 0.3 g/cm³ at 3% wt of blowing agent, and maximum porosity of the foam was 89%. Also, the thermal conductivity of the foamed glass can be decreased by 90% due to the expansion by 4 times with respect to the solid glass. The cooling load for a typical building that use foam glass panels has expected an energy saving by 41% with respect to the conventional one.

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1. Introduction

The Foamed glass is the cellular form of the glass. It is fabricated by heating an amount of glass grains after the addition of a blowing agent, which is usually a chemical powdered formulation. During the melting process, a gas is released from the agent, which is responsible for the foaming in the glass. The product is a rigid material with gas-filled into closed-cell pores with some open cells. The density is between 120-200 kg/m³. The porosity is usually between 75-90%, with a pore size between 0.4-5 mm. This material has many applications in buildings, pavements, roads, industrial and agricultural. In the building section, the material considers as light weight, high strength, thermal and acoustic insulation, fireproofing and waterproofing [1-2].

Nowadays glass foam manufacturers use about 98% of various glass waste and only 2% of pure glass. The processing begins by crushing the glass into particles, and grinding into a powder less than 0.1 mm size. The powder is mixed with the foaming materials, such as silica carbide (SiC). The glass mixture is then passed through a furnace. The furnace heats the powder to a temperature of 800-900 °C. This causes the glass mass to expand to 2-5 times its original size and it subsequently

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cools and hardens into foamed glass. Usually, 92% of foamed glass is composed of air vacancies. Rapid cooling leads to break the foam into aggregates [3-4].

Regarding the blowing agents, many formulations can be implemented to release gases required for foaming during the heating period. These formulations are classified as redox, neutralization agents and decomposing agents. Redox and neutralization additives include non-oxide materials (carbides or nitrides), while decomposing additives include sulfates and carbonates [5-8].

By reviewing the available studies within this field for the last decade, it can be noticed that majority of the works were experimental based with a number of theoretical studies including mathematical models, numerical analysis or computational processing. However, numerous of review studies [9-14] have been conducted to introduce a general overview upon the development in foam glass in the aspects of processes and applications. They propose some examples of the current applications and available data. The current review studies focus upon various configurations, such as:

- Select suitable methods and procedures to form the glass foam.
- Select suitable combinations of waste or virgin glass materials and their corresponding effects on the glass foam.
- The effect of different blowing agents on the glass foam.
- The effect of melting point and reaction temperature on the glass foam.
- The effect of the released gas on the glass foam.
- The development in the manufacturing and production line.
- Properties of glass foam.
- Applications of glass foam.

The literature involves the applications of foamed glass, especially thermal insulation and soundproofing, with some examples of their thermal and acoustic performances depending on available data. The studies focus upon various configurations for the production of foam glass as aggregates, rocks, bricks, panels and composite materials. These research works included developed suggestions for energy conservation purposes in buildings. They show reliable research works focusing upon the improving in the characteristics. The studies looked into the selecting of proper materials depending on the resultant structure or to satisfy optimum heat insulation features.

There are many research studies involving the manufacture of foamed glass for thermal insulation purposes primarily, and it may be accompanied by other purposes such as sound insulation, waterproofing. Early studies [15-17] focused on the method of preparing and grinding glass cullet, mixing it with a blowing agent, and placing it in the oven in a traditional manner. Subsequent research works [18-23] have dealt with the development of the blowing agent, control of its components and addition ratios, as well as controlling the foaming step. Some studies [24-25] have used microwaves to improve the foaming process. Numerous investigations [26-28] have introduced mathematical relationships to connect between microstructure of the foam and the corresponding thermo-physical properties, as well as certain design parameters; like the compaction ratio [29]. The evaluation of glass foam for practical engineering applications was the goal for some works [30-32]. Recent works still look for developing the composing and manufacturing of foam glass by involving advanced materials combined with the main component (waste glass), such as nanoparticles [33] and aerogel [34]. Some orientations look to combine the glass with ceramic materials that have Alkali activation [35], marble waste [36], general ceramics [37], or even red soil [38]. A few studies suggested mixing the waste glass with ash of agricultural waste such as rice husk [39], and sugarcane bagasse [40].

Thermal conductivity of a foamed panel is related to the pore structure; as the porosity increases, the effective thermal conductivity of a material decreases due to the low thermal conductivity of air comparing to solid material. Also, small pores generally reduce conductivity via convection heat transfer. Pores of polygon shapes have less conductivity than spherical ones [41].

Iraq is located in a hot arid region, where the summer season is extremely hot and long. Therefore, cooling of the building by air-conditioning system consumes the majority of the power supplied. In

order to minimize the energy consumption, there is a need to look for ways to enhance the performance of buildings using appropriate and sustainable insulation techniques.

The aim of the current research is manufacturing of foamed glass panel made of waste glass cullet that mixed with certain amount of blowing agents. The contribution of this study includes many factors: the addition of graphene oxide (GO), the processing method and the connection between the foam cladding and cooling load when these panels are suggested as thermal insulation for a typical building in hot air region (Iraq).

2. Materials and Methods

The cullet glass served as raw material for the preparation of foam glass, where pieces of waste glass (broken window glass) have been collected locally. The pieces then cleaned; crushed and sieved to be particles of less than 75 micrometer or mesh 200 (recommended [19]). According to the chemical analysis that conducted by CAC laboratories in Baghdad, the composition of the particles was as follows: 69% SiO₂, 13% Na₂O, 5% CaO, 4% Al₂O₃, 3% K₂O, 3% MgO, 2% BaO, 1% B₂O₃. An amount of the glass particles (95-99 %) is mixed with 1-5 % of the foaming agent (blowing agent). Silicon carbide (SiC) was served as a blowing agent which is commonly used in high-temperature ceramic processing [42]. Furthermore, graphene oxide (GO) nano-powder has been added to enhance the blowing since graphene oxide (GO) acts as a versatile agent in foaming, functioning as a nucleation site and sometimes even generating gas, leading to reduced density, increased strength, and improved thermal insulation in materials [43]. The ratio of GO was 0.5%, which is compensated from the blowing agent content (For example, 3% blowing agent has included 2.5% SiC + 0.5% GO).

The method of obtaining the foamed glass is based on the sintering of the glass particles. The mixture was heated under pressure optimized to 250 kPa [44], and with an initial relative humidity of 40% to ensure better foaming stability [45]. The oven utilized for the study is the common atmospheric Muffle Furnace which is assigned for metals and ceramics. The oven is manually modified to employ higher pressures by connecting its chamber to a compressor. The processing started by putting the mixture of particles in the oven in an ambient condition, and well-closed. The initial pressure inside the chamber was atmospheric. Then the mixture is subjected to a controlled pressure in the oven (200 kPa). As the temperature increased the internal pressure raised reaching approximately 250 kPa. The heating process led to increase the temperature up to 850 °C for 20 minutes (with heat rate of 10 °C/min) to ensure the foaming. After the forming of foamed glass, the pressure should be released to atmospheric pressure again. The sample is left to cool slowly inside the oven to ensure sufficient crystalline structure, and allows the material to develop higher mechanical strength. Figure 1 clarifies the main procedures in the process of the foamed glass.

The total number of specimens is five (one for each blowing agent content) plus the solid glass one (reference). Some undesired samples were ignored due to irregular morphology appeared and arbitrary distribution of the structure. The SEM observation allowed seeing clearly the structure of the foamed glass. In this study, VEGA-II TESCAN device was used to provide SEM images for the samples. This test has been conducted according to ISO 13322-1:2014. The samples have been weighted to determine the mass and corresponding bulk density. The porosity can be measured by subtracting the volume of the porous parts in the foamed sample from the original volume of the reference sample (solid glass). The expansion occurred to the initial glass is also determined.

To measure the thermal conductivity of the foamed glass, the samples were cut into square shapes of 50x50 mm² with a thickness of 8 mm, according to ASTM C177 for thermal conductivity test. The sampling required some treatments, such as removing useless ends or smoothening the frames. Thermal conductivity has been measured using MED-103 thermal conductivity meter. The sample has inserted in a space between two sides: hot one connected to the heater and cold one connected to an extension surface. Values of temperatures on the both sides have been recorded at steady-state condition. These values substituted in modified Fourier equation (one-dimensional steady-state heat equation) assigned for this device. The thermal conductivity measurement has been repeated two times for each sample and took the average to satisfy the reliability of the results. The

whole steps in the preparation and manufacturing of the samples, as well as the tests have been conducted at the Laboratory of Materials Tests at Mustansiriyah University.

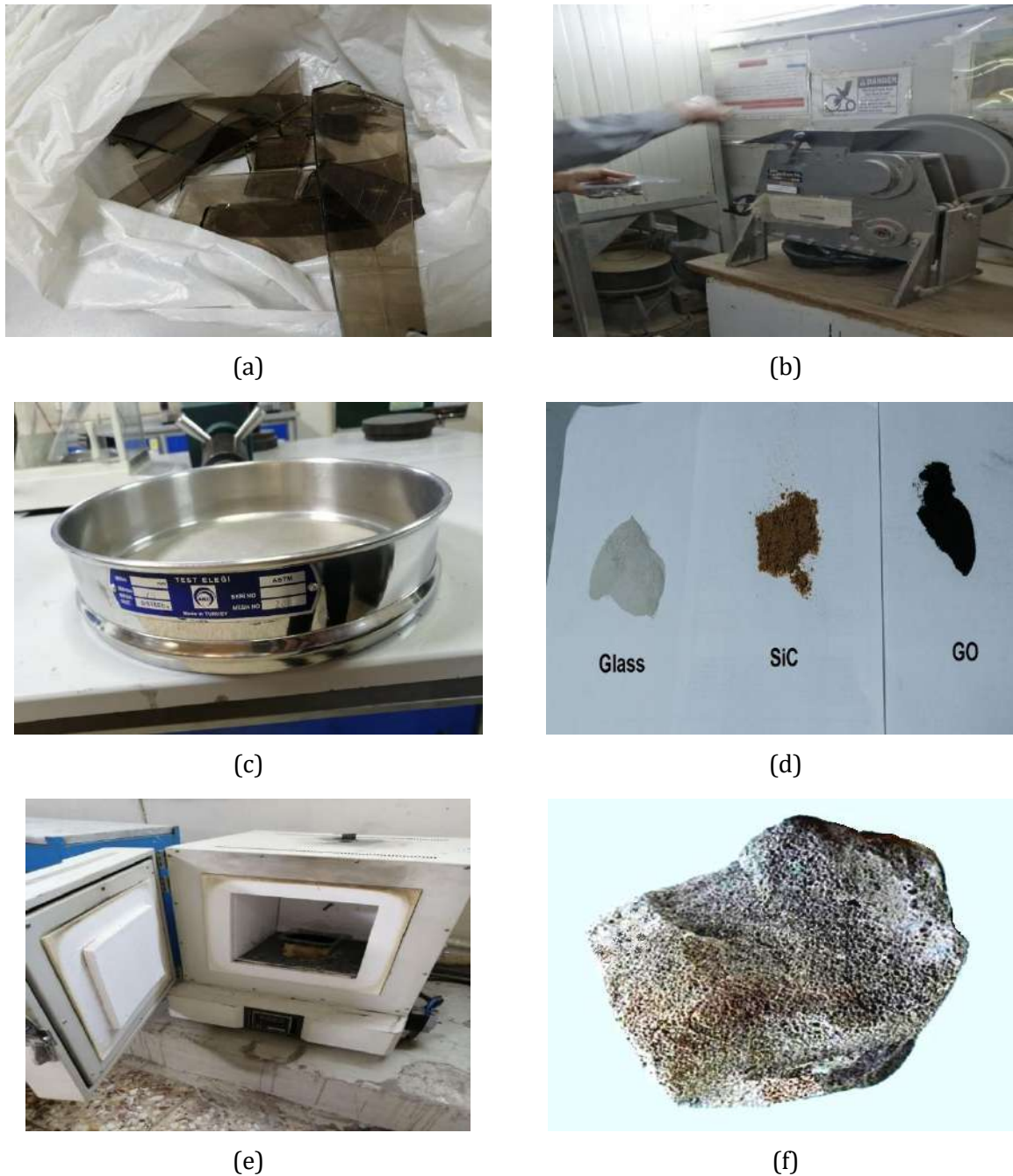


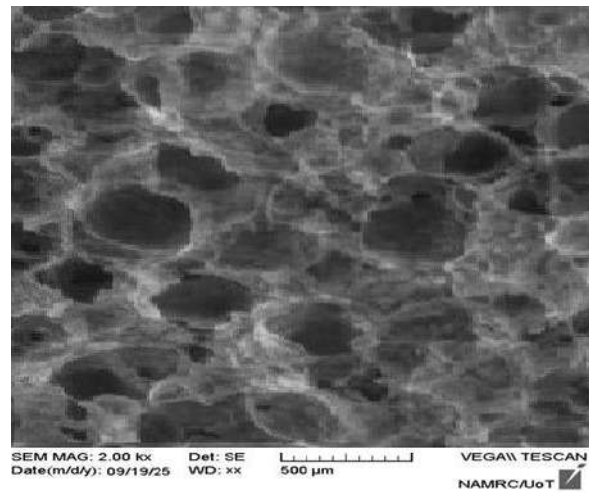
Fig. 1. Process of the manufacturing of the foamed glass (a) Collecting pieces of waste glass, (b) Grinding the glass, (c) Sieving used, (d) Preparation of tiny particles, (e) Heating the mixture in the oven, and (f) Final shape of the foam glass

3. Results and Discussion

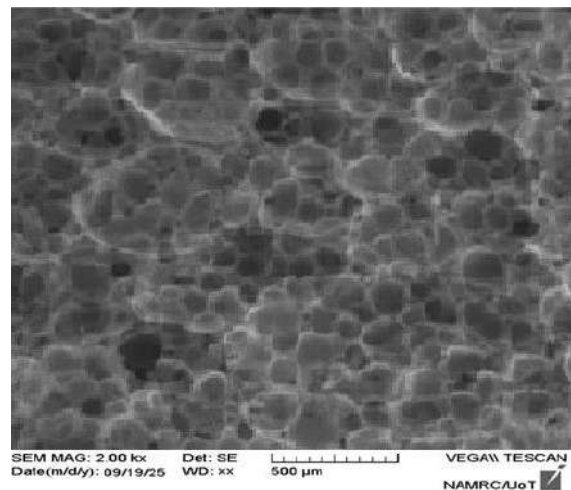
3.1 SEM Images

The SEM image represents the sample's morphology for internal parts taken from the specimen, as shown in Figure 2 for some samples (1, 3 and 5% wt). Generally, the images illustrate connected crystalline cells characterized as packed and gathered. The structure is quite uniform and clear, and has homogenous distribution with micro-sized pores. Foam glass has many sizes and types of pores and distribution. The large pores are surrounded by medium and small pores. The average size of the pore is about 250 mm. The pores are primarily open-cell with some closed-cells,

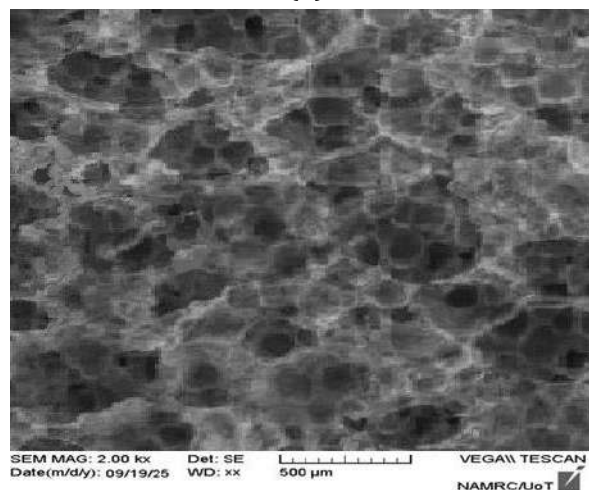
uniformly spherical (or hexagonal). This is due to the good dispersion of the foaming agent into the melted glass particles and as a result of the small particle size. The regional analysis between the images clarified the variation in cell size and corresponding structures.



(a)



(b)



(c)

Fig. 2. SEM images of the foamed glass (a) Foaming agent content (1% wt), (b) Foaming agent content (3% wt), and (c) Foaming agent content (5% wt)

3.2 Porosity and Pore Sizes

The average diameters and distribution of pores in 10 mm³ volume for the selected samples according to the amount of agent added were computed from the images of SEM, as presented in Table 1. In general, the quantity and the size of the pores have augmented via the rise of the weight percentage of blowing agent, and this can be ascribed to the phenomena of coalescence, which being preferred via a rise in the concentration of blowing agent. Eventually, this lowers the apparent density.

Table 1. Diameters and distribution of pores in 10 mm³ volume

Sample (foaming agent%)	Distribution	Diameter of pore (μm)			
		<100	100-250	250-500	>500
0%	Quantity	-	-	-	-
	Percentage	-	-	-	-
1%	Quantity	90	25	4	4
	Percentage	74%	20%	3%	3%
2%	Quantity	110	40	5	6
	Percentage	70%	25%	3%	2%
3%	Quantity	150	60	9	6
	Percentage	66%	27%	4%	3%
4%	Quantity	120	45	6	6
	Percentage	68%	26%	3%	3%
5%	Quantity	130	40	5	5
	Percentage	72%	22%	3%	3%

The increase of blowing agent motivates the release of gas due to decomposed moles at elevated temperatures. This raises the internal gas pressure, which aid the expansion of pores [23]. The porosity is the ratio of the volume of air inside the sample to the total volume of the material [46]. By determining the volume of the pores (estimated as spherical shapes), the porosity can be found. Where, this is done by calculating the pore size within the foam and multiplying it by number of pores for each range of pore sizes mentioned in Table 1. For example, the total porosity of 1% wt sample is calculated as:

$$(90 \times 4\pi/3r_a^3) + (25 \times 4\pi/3r_b^3) + (4 \times 4\pi/3r_c^3) + (4 \times 4\pi/3r_d^3) = 5.2 \quad (1)$$

Where: r_a , r_b , r_c and r_d are the average radii for each range of pore sizes mentioned in Table 1; i. e., 70, 150, 400 and 600 μm. The result (5.2 mm³) is divided by the total volume of the solid sample that employed in SEM (10 mm³), then the total porosity is 0.52 (52%). And so on for other samples. Therefore, the total porosity has increased from 0% for the solid glass to 52, 75, 89, 78 and 66% for the foamed glass with 1, 2, 3, 4 and 5% addition of the blowing agent, respectively.

3.3 Density and Expansion

Before measuring the density, it is important to determine the range of expansion happening to the solid glass due to the addition of the foaming agent. This will give reliability to the resultant value of the density, where the expansion in the specimen is proportional similarly to the reduction in the density as overall. Therefore, the expansion ration was measured for several specimens with the agent contents of (0, 1, 2, 3, 4 and 5%), as shown in Table 2. The dimensions of the solid glass sample under consideration were 100x100x2 mm³.

The results of the expansion ratio show that the foaming agent takes a significant part in glass expansion. The material is expanded as four times as its original size by adding 3% agent. However, the expansion ratio in the foamed glass could reach to 5 times [17]. Now, bulk density of the samples of the foamed glass was obtained as a function of foaming agent content, as shown in Figure 3. As expected, due to the expansion happened by foaming agent, the bulk density has decreased from (2.4 g/cm³) for the solid glass to (0.3 g/cm³) for the foamed glass sample by adding

(3%) of blowing agent. The reduction is attributed to the presence of pores inside the material. This reduction in the density behaves closely to that obtained by other results [15-20]. However, the density of the samples introduced in the mentioned studies can be reduced down to 0.2 g/cm³ [19].

Table 2. Measured values of expansion due to foaming agent

Blowing agent content (%)	Expansion ratio (%)
0	-
1	128
2	233
3	398
4	270
5	210

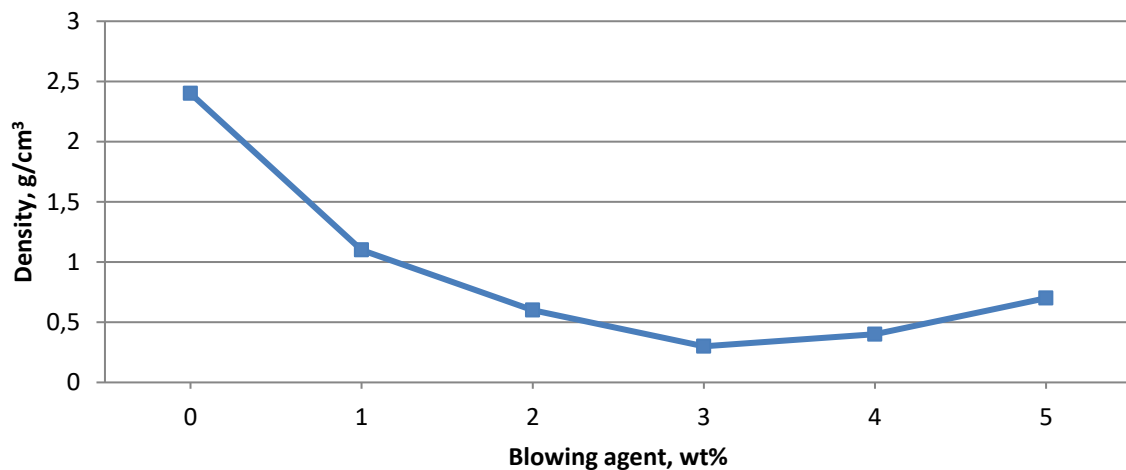


Fig. 3. Density in terms of the foaming content

3.4 Thermal Conductivity

These measurements serve to determine the thermal conductivity of the foamed glass for different combinations. The results of the thermal conductivity (k-value) for the selected specimens are shown in Table 3.

Table 3. Thermal conductivity values

Specimen	Combination	k-value (W/m.K)
1	Solid glass	0.92
2	Foamed glass with 1% of blowing agent	0.31
3	Foamed glass with 2% of blowing agent	0.17
4	Foamed glass with 3% of blowing agent	0.09
5	Foamed glass with 4% of blowing agent	0.14
6	Foamed glass with 5% of blowing agent	0.22

The shown data fundamentally revealed the decreasing in the k-value of the foamed glass comparing to the solid due to the presence of pores. The minimum k-value was 0.09 W/m.K (90% reduction) at 3% content of blowing agent. By increasing the loading of agent content beyond this percentage, the k-value gets higher which is undesirable and out of the goal. The limited decreasing in the k-value beyond this ratio is due the agglomeration of glass particles and thus irregular forming and displacing of pores; which is required to pronounce the reduction in thermal conductivity effective [47]. By looking at the distribution of the pores, it can be seen less generation of small size pores which is necessary to reduce radiation heat transfer [48]. An excessive foaming agent content leads to the increase of water absorption of the sample; thus, the convective heat transfer of the gas phase increases in thermal conductivity term [23]. However, the resources have

advised to incorporate blowing agent with a concentration of no more than 2% [18]. Data from sources have shown that k-value can be as low as 0.03 W/m.K [15].

The values of thermal conductivity can be analyzed in perspective to the images taken by SEM as following. In the SEM image for the 1% wt blowing agent content, the structure was less uniform comparing to other images and the pores with different sizes were distributed randomly. The structure has less number of pores, and most of them were small sizes. Therefore, k-value was lower. Note that the air content plays the major role in decreasing the overall k-value of the sample. With the increasing of blowing agent to 3% wt, the structure become more homogenous, and has higher number of pores with small and medium sizes. This will help to satisfy less k-value since air has less thermal conduction that solid material. Beyond this percentage, the structure return to be less organized with less pores slightly. This can be attributed to coalescence phenomena. The thermal conductivity went higher again. Also, the thermal conduction behavior become ineffective due to the presence of relatively large pores which have higher convection and radiation heat transfer comparing to small pores. Note that the quality of SEM images can be affected by accelerating voltage of the beam and spot size during the test [49].

3.5 Comparison

By comparing the results of many studies including the current study, as shown in Table 4, it can be noticed the following points:

- The most effective blowing agents are: SiC, CaCO₃, NaCO₃ MnO₂ with some additives like ash, acid and biodiesel. The addition of foaming agent is mostly between (1-5%).
- The furnace temperature is commonly between 700-900 °C. There is once case that temperature was more than 1000 °C [24], and one case the temperature was less than 600 °C [37].
- The values of thermal conductivity usually range between 0.05-0.1 W/m.K.
- The values of density usually range between 0.3-0.5 g/cm³.
- The values of porosity usually range between 60-90%.
- The values of pore size usually less than 500 µm.

Table 4. Comparison some parameters between selected studies

[Ref.]	Blowing agent (%wt)	Furnace temperature (°C)	Thermal conductivity (Wm.K)	Density (g/cm ³)	Porosity (%)	Pore size (µm)
Current	SiC + graphene (1-5)	850	0.09-0.31	0.3-1	52-89	50-750
[15]	CaCO ₃ (1)	850	0.03-0.05	0.25	85	100-300
[18]	Na ₂ CO ₃ (1-7) + biodiesel	850	0.24-0.40	0.47	50	< 500
[20]	CaCO ₃ (1-3) + graphite	850	0.05-0.10	0.3-0.5	75-85	< 50
[21]	SiC (1-2)	920	0.04-0.06	0.25-0.4	50	< 1000
[22]	MnO ₂ (2.5)	800	0.08-0.1	0.2-0.3	80-90	< 1000
[24]	SiC (2)	> 1000	0.03-0.04	0.2	93	< 1000
[25]	SiC (3.5-4.5)	850	0.05-0.06	0.2-0.25	90	< 900
[27]	SiC (4-5) + coal fly ash	975	0.05-0.07	0.3-0.36	82-86	> 1000
[33]	Nano-carbon (2-10)	700-900	0.11-0.28	0.2-0.3	60-70	< 500
[37]	Boric acid (2)	400-500	0.05-0.06	0.2-0.25	-	-
[39]	CaCO ₃ (6) + rice husk ash	750-850	0.03	0.25-0.3	85	< 500

3.6 Thermal Performance

Energy consumption for cooling of traditional building in extremely hot region such as Iraq might reach up to 725 kWh/m² annually [50]. Therefore, effective thermal insulation is necessary. Thermal performance of the foamed glass can be estimated by simulating a typical building in a hot arid region which is insulated with foamed glass and comparing it to a conventional one. A simulation program called Iraq Passive Planning Package (IPPP) adopted locally [51] and based on ASHRAE relations [52] has been used to calculate the cooling load for typical single floor building of 100 m² in Baghdad with and without the use of cladding panels. The total cooling load can be determined by:

$$Load = Q_{walls} + Q_{roof} + Q_{floor} + Q_{window} + Q_{door} + Q_{vent} \quad (2)$$

Where; Q represents the heat transfer through a certain element (wall, roof, floor, window and door), and it can be found by:

$$Q = U A \Delta T \quad (3)$$

Where, U is the overall heat transfer coefficient through the element, and A is the exposed surface area, while ΔT is the temperature difference across the element. The ventilation losses is given by:

$$Q = ACH V \rho C_p \Delta T \quad (4)$$

Where, ACH is the air-change in the room per hour. And, V is the volume of the room. Also, ρ and C_p are the density and the heat capacity of air, respectively. Materials, elements and limits assumed during the calculation are listed in Table 5.

Table 5. Features of the typical room assumed to determine the cooling load

Part	Characteristics
Construction	Typical space built of 100 m ² in Baghdad.
Walls (in to out)	1 cm gypsum, 25 cm Brick and 1 cm cement plaster. There're (3) internal walls and merely one external wall.
Roof (in to out)	1.5 cm gypsum, 18 cm reinforced concrete, 8 cm soil, and 5 cm concrete tiles.
Floor	Marble tiles.
Window	2 m ² of UPVC frame and south orientation.
Door	2.5 m ² of wooden structure and internal.
Air quality	The indoor temperature is 24°C, and the ACH for ventilation is 0.8.
Outdoor conditions	The highest range of temperature is within 39-50°C at daytime, and the minimum range of temperature is 23-32°C overnight. Humidity is within 20-45% with average radiation within 750-1100 W/m ² .

The common cladding panels in Iraq are made of plastic cores (4 mm thickness) and covered by reflective layers. The panel should be attached to the external wall by spacing of 5 cm of air gap. These panels can be developed by involving the foamed glass instead of the plastic. The results, shown in Figure 4 revealed that the average cooling load for a typical building in Baghdad in summer months (May to September) is 362 kWh/m². By using cladding panels made of foamed glass, the average cooling load will be 213 kWh/m². This means that there is an energy saving by 41% comparing to the case without cladding. Comparing with the results obtained from Mussa [53], it is observed that this ratio is higher than that recorded by using of cladding panel made of plastic (33%); and higher than that recorded by using of cladding panel made of wood plastic composite (37.5%). The layering of foamed glass panels will reduce the three forms of heat transfer. Where, there is an additional thermal resistance (2.05 m².K/W) due to the presence of 4 mm cladding panel plus 5 cm air gap. The air gap itself removes extra heat transfer by natural convection upward where there are some assigned openings in the frame. Also, the reflective layer

on the cladding panel will eliminate the radiation heat transfer up to 20% [54]. Note that the contribution of air gap as well as the reflective layer is included in the simulation.

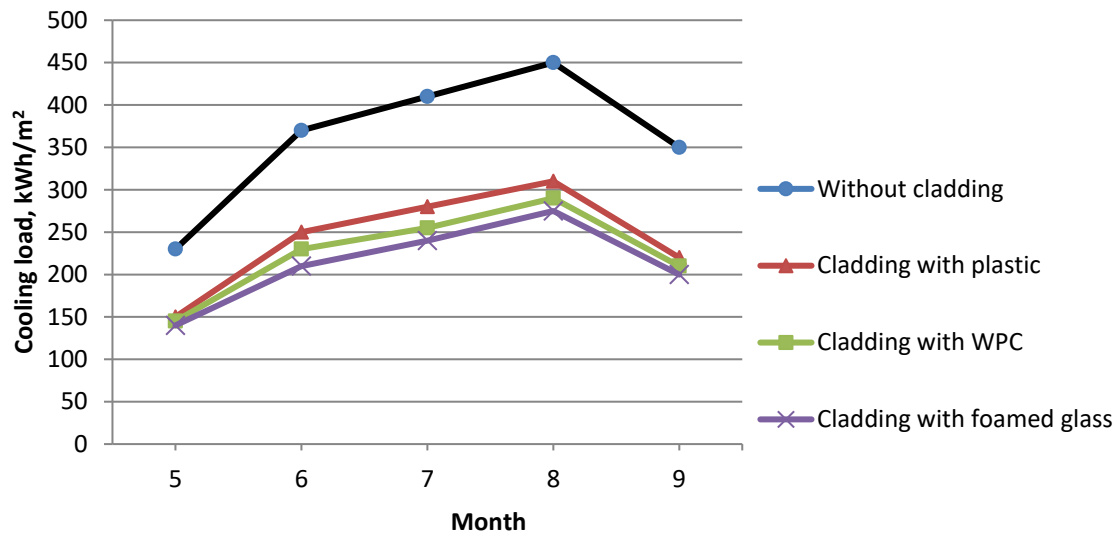


Fig. 4. Comparison of cooling load for different panels during the summer in Iraq

4. Conclusions

The role of foamed glass in the reduction of cooling load in hot arid region (Iraq) is investigated in the current study. Specimens of porous glass were manufactured by the experimental work with many combinations of blowing agent as well as a modified production method. The study included many factors: the addition of graphene oxide (GO), the processing method that employed a controlled pressure and the contribution of the glass foam cladding in the cooling load. Many parameters have been measured in the laboratory, such as: density, porosity expansion ratio and coefficient of thermal conductivity. The morphology of the internal parts of the foam has been analyzed based on SEM images. The thermal performance of foamed glass panels has been compared with common panels made of plastics that used for cladding. The results lead to the following conclusions:

- Minimum density of the foam was 0.3 g/cm³
- Maximum porosity of the foam was 89%.
- Maximum volume expansion was almost 4 times its original volume.
- Minimum k-value was 0.09 W/m.K (reduction by 90% comparing to solid glass).
- Optimum ratio of blowing agent was 3% including 0.5% of graphene oxide. This ratio satisfied the minimum density and thermal conductivity.
- The cooling load for a typical building that use foamed glass cladding panels has expected an energy saving by 41%, which is higher than that recorded by using of plastic cladding (33%).

Furthermore, future works may consider many factors such as: the behavior of foamed glass with the variation of temperature or humidity, durability of the layers, performance of sound proofing or fireproofing, and mechanical features. New technologies can be implemented in the process like 3D printing [55]. Note that the main limitation of the current work is the consideration of the foamed glass as cladding panels for the building. Also, foamed glass is a highly brittle, rigid cellular material, and it easily cracks or breaks under stress, therefore it should install carefully.

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References

- [1] Zhang H. Heat-insulating Materials and Sound-absorbing Materials. In: Building Materials in Civil Engineering. Woodhead Publishing Series in Civil and Structural Engineering. Woodhead Publishing; 2011. p. 304-423. <https://doi.org/10.1533/9781845699567.304>
- [2] EPA. Foam Glass Insulation from Waste Glass. U.S. Environmental Protection Agency; 2013.
- [3] PCU. Foamglas Industrial Insulation Handbook. Pittsburgh Corning Europe, Belgium; 1992.
- [4] Scarinci G, Brusatin G, Bernardo E. Glass Foams. In: Scheffler M, Colombo P, editors. Cellular Ceramics: Structure, Manufacturing, Properties and Applications. 2005. <https://doi.org/10.1002/3527606696.ch2g>
- [5] Bernardo E, Cedro R, Florean M, Hreglich S. Reutilization and stabilization of wastes by the production of glass foams. Ceram Int. 2007;33:963-968. <https://doi.org/10.1016/j.ceramint.2006.02.010>
- [6] Llaudis A, Tari M, Ten F, Bernardo E, Colombo O. Foaming of flat glass cullet using Si₃N₄ and MnO₂ powders. Ceram Int. 2009;35:1953-1959. <https://doi.org/10.1016/j.ceramint.2008.10.022>
- [7] Lv D, Li X, Wang L, Du J, Zhang J. Effect of carbon as foaming agent on pore structure of foam glass. Adv Mater Res. 2010;105-106:765-768. <https://doi.org/10.4028/www.scientific.net/AMR.105-106.765>
- [8] Abdollahi S, Yekta B. Prediction of foaming temperature of glass in the presence of various oxidizers via thermodynamics route. Ceram Int. 2020;46(16):25626-25632. <https://doi.org/10.1016/j.ceramint.2020.07.037>
- [9] Cui S, Zhang J, Tian Y, Sun S, Wu Z, Liu W. Generation review on the production line development of Foam glass at home and abroad. Adv Mater Res. 2014;915-916:524-531. <https://doi.org/10.4028/www.scientific.net/AMR.915-916.524>
- [10] Yao Z, Ling T, Sarker P, Su W, Liu J, Wu W, et al. Recycling difficult-to-treat e-waste cathode-ray-tube glass as construction and building materials: A critical review. Renew Sust Energ Rev. 2018;81:595-604. <https://doi.org/10.1016/j.rser.2017.08.027>
- [11] Zhang J, Liu B, Zhang S. A review of glass ceramic foams prepared from solid wastes: Processing, heavy-metal solidification and volatilization, applications. Sci Total Environ. 2021;781:146727. <https://doi.org/10.1016/j.scitotenv.2021.146727>
- [12] Ibrahim M, Mustaffar M, Ismail S, Ismail A. A review of porous glass-ceramic production process, properties and applications. J Phys: Conf Ser. 2022;2169:012042. <https://doi.org/10.1088/1742-6596/2169/1/012042>
- [13] Makarov D, Manakova N, Suvorova O. Production of rock-based foam-glass materials (Review). Glass Ceram. 2023;79:411-417. <https://doi.org/10.1007/s10717-023-00522-8>
- [14] Wang W, Chi J, Niu S. On the application of foam glass in interior design. E3S Web Conf. 2023;406:04033. <https://doi.org/10.1051/e3sconf/202340604033>
- [15] Stiti N, Ayadi A, Lerabi Y, Benhaoua F, Benzerga R, Legendre L. Preparation and characterization of foam glass based waste. Asian J Chem. 2011;23(8):3384-3386.
- [16] Kim J, Lee C. Preparation of foamed glass block from recycled soda-lime-silicate glasses by chemical composition control. Appl Chem Eng. 2013;24(4):382-390.
- [17] Bai J, Yang X, Xu S, Jing W, Yang J. Preparation of foam glass from waste glass and fly ash. Mater Lett. 2014;136:52-54. <https://doi.org/10.1016/j.matlet.2014.07.028>
- [18] Sooksaen P, Sudyod N, Thongtha N, Simsomboonphol R. Fabrication of lightweight foam glasses for thermal insulation applications. Mater Today Proc. 2019;17:1823-1830. <https://doi.org/10.1016/j.matpr.2019.06.219>
- [19] Panov Y, Ermolaeva E, Nikonov A, Timakov E. High-performance thermal insulation material based on waste glass. IOP Conf Ser: Mater Sci Eng. 2020;896:012074. <https://doi.org/10.1088/1757-899X/896/1/012074>
- [20] Liu Y, Xie J, Hao P, Shi Y, Xu Y, Ding X. Study on factors affecting properties of foam glass made from waste glass. J Renew Mater. 2021;9(2). <https://doi.org/10.32604/jrm.2021.012228>
- [21] Osfouri M, Simon A. Study on the thermal conductivity and density of foam glass. Pollack Period. 2022;18(1):126-131. <https://doi.org/10.1556/606.2022.00591>
- [22] Cozzarini L, De Lorenzi L, Barago N, Sbaizero O, Bevilacqua P. Expanded glass for thermal and acoustic insulation from recycled post-consumer glass and textile industry process waste. Materials. 2023;16:1721. <https://doi.org/10.3390/ma16041721>
- [23] Zhang Y, Xie J, Sun M, Wang S, Xu T, Xu Y, et al. Preparation and study of borosilicate foam glass with high thermal insulation and mechanical strength. J Renew Mater. 2023;11(5). <https://doi.org/10.32604/jrm.2023.025127>
- [24] Dragoescu M, Axinte S, Paunescu L, Fiti A. Foam glass with low apparent density and thermal conductivity produced by microwave heating. Eur J Eng Technol. 2018;6(2).
- [25] Axinte S, Paunescu L, Dragoescu M. Manufacturing the cellular glass from glass waste by an original unconventional technique. Acad J Manuf Eng. 2021;19(4):54-60.

- [26] Østergaard M, Zhang M, Shen X, Petersen R, König J, Lee P, et al. High-speed synchrotron X-ray imaging of glass foaming and thermal conductivity simulation. *Acta Mater.* 2020;189:85-92. <https://doi.org/10.1016/j.actamat.2020.02.060>
- [27] Paunescu L, Axinte S, Dragoescu M, Cosmulescu F. Adequate correlation between the physical and mechanical properties of glass foam. *J La Multiapp.* 2021;2(4):14-26. <https://doi.org/10.37899/journallamultiapp.v2i4.415>
- [28] Safonchik D, Maksimovich S. The correction of foam glass composition based on its principal parameters. *MATEC Web Conf.* 2021;350:00009. <https://doi.org/10.1051/mateconf/202135000009>
- [29] Mustafa W, Szendefy J, Nagy B. Thermal performance of foam glass aggregate at different compaction ratios. *Buildings.* 2023;13:1844. <https://doi.org/10.3390/buildings13071844>
- [30] Wierzbicki S, Siennicki M, Gizejowski M. Assessment of long-term performance of foam glass as an insulating sub-base in varying humidity and temperature conditions. *Arch Civ Eng.* 2022;68(3).
- [31] Sha B, Xiong H, Zheng H, Yuan K, Wen M, Zhang Y. Analysis of the temperature field and deformation characteristics of foam glass thermal insulating decorative integrated board system. *Case Stud Therm Eng.* 2022;38:102299. <https://doi.org/10.1016/j.csite.2022.102299>
- [32] Kugler A, Trivelpiece C, Brigmon R. Foamed glass ceramics - an upcycled scaffold for microbial biofilm development. *Biotechnol Lett.* 2023;45:225-233. <https://doi.org/10.1007/s10529-022-03332-0>
- [33] Kim E, Kim K, Song O. Properties of foamed glass upon addition of nanocarbon and sintering temperatures. *J Asian Ceram Soc.* 2020;8(1):123-129. <https://doi.org/10.1080/21870764.2020.1712798>
- [34] Song Z, Zhao Y, Yuan M, Huang L, Yuan M, Cui S. Thermal insulation and moisture resistance of high-performance silicon aerogel composite foam ceramic and foam glass. *Adv Eng Mater.* 2022;24:2101508. <https://doi.org/10.1002/adem.202101508>
- [35] Mazzi A, Sciarrone M, Bernardo E. Environmental performance of glass foam as insulation material from waste glass with the alkali activation process. *Heliyon.* 2023;9:e19001. <https://doi.org/10.1016/j.heliyon.2023.e19001>
- [36] Babisk M, Vilela T, Tabelini D, Gadioli M, Vieira C. Foam glass synthesized exclusively from wastes. *RGSA - Rev Gest Soc Ambient.* 2024;18(2):1-15. <https://doi.org/10.24857/rgsa.v18n2-049>
- [37] Yu Q. Application of foam glass-ceramic composite thermal insulation material in traditional buildings. *J Chem.* 2022;2022:9662805. <https://doi.org/10.1155/2022/9662805>
- [38] Sadji A, Boudchicha M, Ziouche A, Filali A, Achour S. Glass-ceramic foams from waste glass and natural red soil. *J Aust Ceram Soc.* 2023;59:425-436. <https://doi.org/10.1007/s41779-023-00852-4>
- [39] Fernandes F, Costa D, Martin C, Rossignolo J. Vitreous foam with thermal insulating property produced with the addition of waste glass powder and rice husk ash. *Sustainability.* 2023;15:796. <https://doi.org/10.3390/su15010796>
- [40] Fernandes F, Fernandes T, Rossignolo J. Production of glass foam in a microwave oven using agro-industrial waste as raw material. *Buildings.* 2024;14:1643. <https://doi.org/10.3390/buildings14061643>
- [41] Muhammad A, Mohammed T, Resan K. Thermal conductivity of porous plastics manufactured by 3D printing: Controlling the design of the cavities and corresponding effects. *J Ther Eng.* 2025;11(1):226-239. <https://doi.org/10.14744/thermal.0000915>
- [42] Ren P, Zhou R, Liu H, Huo Y, Wang Y. Preparation of coal gangue based foamed ceramics with SiC as blowing agent and study on its thermal insulation performance. *Ceram Int.* 2024;50(17B):31680-31690. <https://doi.org/10.1016/j.ceramint.2024.05.476>
- [43] Xu L, Jiang S, Li B, Hou W, Li G, Memon M, et al. Graphene oxide: a versatile agent for polyimide foams with improved foaming capability and enhanced flexibility. *Chem Mater.* 2015;27(12):4358-4367. <https://doi.org/10.1021/acs.chemmater.5b00981>
- [44] Østergaard M, Petersen R, König J, Bockowski M, Yue Y. Foam glass obtained through high-pressure sintering. *J Am Ceram Soc.* 2018;101:3917-3923. <https://doi.org/10.1111/jace.15574>
- [45] Kim D, Dutton B, Hrma P, Pilon L. Effect of furnace atmosphere on E-glass foaming. *J Non-Cryst Solids.* 2006;352(50-51):5287-5295. <https://doi.org/10.1016/j.jnoncrysol.2006.08.035>
- [46] Mohammed T. The combined effects of thermal and acoustic parameters on the sound absorption of aluminum foam panels. *RJAV.* 2023;20(1).
- [47] Abdulhameed J, Mohammed T. Practical investigation to evaluate the effect of using nanocomposite layer within double-glazing evacuated window in reducing the solar heat gain and the cooling load for a residential building in a hot arid region. *Arab J Sci Eng.* 2024;49:2269-2283. <https://doi.org/10.1007/s13369-023-08205-w>
- [48] Hameed O, Mohammed T. Thermal performance of nano-structured foam as building insulation in hot arid region: A mathematical modeling. *Math Model Eng Probl.* 2026;13(1). <https://doi.org/10.18280/mmep.130205>
- [49] Yardimci A, Acikbas Y, Capan R. SEM images of carbon nanotubes (CNTs): effect of voltage and spot size on image resolutions. *Res Des.* 2024;1(1):43-51. <https://doi.org/10.17515/rede2024-004en1116rs>

- [50] Abdulsada G, Salih T. Experimental and theoretical study for the performance of new local thermal insulation in Iraqi building. In: Renewable Energy in the Service of Mankind Vol I. Springer, UK; 2015. https://doi.org/10.1007/978-3-319-17777-9_44
- [51] Salih TWM. Theoretical and experimental study to evaluate the performance of passive house in Iraq [PhD Thesis]. Al-Mustansiriyah University, Iraq; 2015.
- [52] ASHRAE. Chapter 27: residential cooling and heating load calculations. ASHRAE Handbook SI (Fundamentals); 1997.
- [53] Mussa H, Salih T. Thermal insulation performance of wood-plastic composite panels as alternative to Alucobond cladding sheets for buildings. JEASD. 2020;24(Special):425-433. <https://doi.org/10.31272/jeasd.conf.1.47>
- [54] Hernández-Pérez I, Álvarez G, Xamán J, Zavala-Guillén I, Arce J, Simá E. Thermal performance of reflective materials applied to exterior building components-A review. Energy Build. 2014;80:81-105. <https://doi.org/10.1016/j.enbuild.2014.05.008>
- [55] Syed R, Rahul B, Lingeshwaran N, Pratheba S, Balamurugesan T, Priyanka M, et al. Recent advances in precast and 3D sandwich wall panels. Res Eng Struct Mater. 2026. <http://dx.doi.org/10.17515/resm2025-1059st0802rv>

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Development of high-performance lightweight concrete through incorporation of polypropylene fibers into polyurethane foam concrete

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Abstract

The increasing emphasis on sustainable and energy-efficient construction materials has heightened interest in structural lightweight concrete (LWC) due to its potential to reduce dead load and enhance thermal performance. Conventional concrete, with a density of around 2400 kg/m³, contributes substantially to the self-weight of structures, whereas incorporating lightweight aggregates can lower density by nearly 20 %. Polyurethane-foam (PUF) concrete, with densities between 1700 and 1950 kg/m³, offers significant weight reduction but suffers from brittleness and reduced strength caused by interconnected voids. To overcome these limitations, this study integrates polypropylene (PP) fibers into PUF concrete, leveraging their high tensile strength, corrosion resistance, and economic viability. Six mixes were prepared by varying foam content (10 % and 15 %) and PP fiber dosage (0–1 %). Mechanical, durability, and microstructural properties were evaluated. The optimal mix containing 10 % foam and 1 % fiber achieved a 25 % reduction in dry density, 30 % improvement in compressive strength, and 29 % decline in chloride permeability. Microstructural analysis revealed compact calcium-silicate-hydrate (C-S-H) gel formation around intact fibers. The findings demonstrate that PP-fiber-reinforced PUF concrete effectively combines lightweight characteristics, mechanical strength, and durability suitable for sustainable structural applications.

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1. Introduction

Structural concrete accounts for a large portion of the dead load in buildings and bridges, and its density typically lies between 2300 and 2500 kg/m³. Such mass contributes to high seismic forces and increases material consumption and embodied energy. Lightweight concrete (LWC) aims to mitigate these issues by replacing some of the dense aggregates with lightweight alternatives or by incorporating air voids. Polystyrene-foam lightweight concrete beams, for example, achieve weight reductions of about 30 % compared with normal-weight concrete while maintaining structural integrity [1]. Moreover, the energy required to transport and erect LWC elements is lower, leading to improved sustainability [2].

In recent years, polyurethane-foam (PUF) concretes have emerged as promising LWCs. Preformed PUF, with a density of around 35 kg/m³, is dispersed into the cement matrix to create a closed-cell foam. PUF concretes with foam contents of 10 – 60 % exhibit hardened densities ranging from 2354 kg/m³ down to 1842 kg/m³. Standards such as ACI 213R define structural lightweight aggregate concretes as having densities between 1120 and 1920 kg/m³ [3], while European

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standard EN 206 limits structural LWC to densities below 2000 kg/m³ [2]. However, increasing foam content in PUF concretes causes a decline in compressive strength: control concretes reach about 30.5 MPa at 28 days, whereas mixes with 10 % and 30 % foam reach 28.7 MPa and 22.9 MPa, respectively [3]. Brittleness and crack propagation are the dominant drawbacks of foam concretes, highlighting the need for toughening mechanisms.

Fiber reinforcement has long been recognized as an effective way to enhance concrete toughness and crack resistance. Steel fibers provide high strength and stiffness, glass fibers contribute good tensile properties but can be susceptible to alkali attack, basalt fibers offer excellent chemical stability, and synthetic fibers such as polypropylene (PP) and polyethylene improve ductility and control micro-cracking [4]. Combining fibers with foam concretes or with lightweight aggregates has been shown to improve load-carrying capacity. For instance, beams cast with polystyrene-foam lightweight concrete and strengthened with carbon- or glass-fiber-reinforced polymers exhibited increased ultimate load and reduced deflection compared with unreinforced beams [1]. Nevertheless, most fiber-reinforced foam concretes employ steel or glass fibers and there is little information on the use of PP fibers in PUF concretes.

Polypropylene fibers possess several attributes that make them attractive for lightweight concrete. They are chemically inert, resist corrosion, and have densities around 0.91 g/cm³, which is significantly lower than that of steel. PP fibers melt at 160 – 170 °C, allowing them to form channels during heating that release steam and mitigate explosive spalling. Studies on lightweight foamed concrete reinforced with PP fibers (length 12 mm, dosage 1–3 kg/m³) report enhancements in tensile and flexural strengths, compressive strength, and drying-shrinkage resistance. The fibers bridge micro-cracks, restrain crack opening, and improve post-peak toughness [4]. However, workability may decrease due to fiber clustering, and the optimum dosage must be determined to balance ductility and workability [5]. Despite these advantages, the combined use of PUF and PP fibers in structural LWC remains underexplored. Most studies on PUF concretes either omit fibers or use steel fibers. Conversely, investigations on PP-fiber-reinforced concretes focus on normal or foamed concretes without lightweight aggregates [4,5]. A systematic study examining mechanical properties, durability, shrinkage, and microstructure of PUF concrete reinforced with PP fibers at different dosages has not been reported.

This research aims to fill that gap by developing high-performance lightweight concretes through the incorporation of PP fibers into PUF concrete. The objectives are to: (1) design mixes with different foam and fiber contents to achieve densities within the structural lightweight range; (2) evaluate fresh properties and mechanical strengths (compressive, flexural, split-tensile); (3) assess durability (water absorption, permeability, chloride migration) and shrinkage; (4) analyze microstructural features via scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDX); and (5) identify the optimum foam and fiber combination that maximizes strength and durability while maintaining low density. The findings are discussed in light of existing literature on fiber-reinforced LWCs and provide practical recommendations for precast and marine applications. While [6] investigated how incorporation of polypropylene fibres affects the permeability of concrete under freeze–thaw cycles and mechanical loading, focusing primarily on durability and transport properties of conventional concrete under environmental stress, the present work extends beyond permeability concerns by examining a multi-component polyurethane foam concrete system that integrates nano-scale mineral additives and polymer fibres. This enables a comprehensive evaluation of mechanical performance enhancement and microstructural behavior in engineered foam concrete, offering new insights into tailored mix design for strength and durability improvements that are not addressed in the earlier study. Previous studies on polypropylene fibre reinforcement have mainly focused on conventional foamed or normal-weight concretes, with emphasis on flexural behavior, shrinkage control, or permeability under specific environmental actions [4]. In contrast, the present work introduces a systematic experimental investigation of polyurethane-foam-based concrete reinforced with polypropylene fibres, providing new insights into the coupled effects of foam-induced cellular structure and fibre bridging on mechanical performance, shrinkage behavior, durability indicators, and microstructural evolution, which have not been jointly examined in earlier studies.

2 Materials, Mix Design and Methods

2.1 Materials

All mixes used ordinary Portland cement (OPC) 43 grade conforming to IS 8112:2013. Natural sand complying with IS 383:2016 and having a maximum size of 4.75 mm was used as fine aggregate. Pumice was selected as the coarse aggregate due to its low density and particle size up to 12 mm. Preformed polyurethane foam with a density of approximately 35 kg/m³ was produced and added at 10 % or 15 % by volume of the fresh mix. Polypropylene fibers (length 12 mm, diameter $\approx$ 20 μ m, aspect ratio $\approx$ 600) were incorporated at dosages of 0, 0.5 and 1.0 % of the total volume. The fibers are non-corrosive and have a melting point between 160 °C and 170 ° [4]. In order to enhance durability, 25 % of the cement was replaced with Class F fly ash and 7 % with silica fume. A polycarboxylate-ether (PCE) superplasticizer (0.5 % of cement by weight) was used to achieve self-compact ability. Potable water was used for mixing and curing.

Polypropylene fibres were used as discrete reinforcement in the polyurethane foam concrete. The fibres had a length of 30 mm and a diameter of 0.5 mm, corresponding to an aspect ratio (L/D) of 60. The selected aspect ratio is consistent with previous studies reporting enhanced crack bridging and flexural toughness in lightweight concrete systems.

2.2 Mix Design

Six mixes were designed. The control mix (M0) contained no foam or fibers and had a water–cement ratio of 0.35. Mixes M1 and M2 incorporated 10 % foam with fiber dosages of 0 % and 0.5 %, respectively. Mix M3 contained 10 % foam and 1 % fibers. Mixes M4 and M5 incorporated 15 % foam with fiber dosages of 0.5 % and 1.0 %, respectively. The binder content remained constant, and the aggregate proportions were adjusted to maintain the same water–cement ratio.

Table 1. Mix proportions and target densities for polyurethane-foam concrete reinforced with polypropylene fibers

Mix ID	Foam (%)	PP Fiber (%)	Target density (kg/m ³)	w/c	SP (%)
M0	0	0	2400	0.35	0.5
M1	10	0	1950	0.35	0.5
M2	10	0.5	1920	0.35	0.5
M3	10	1.0	1890	0.35	0.5
M4	15	0.5	1820	0.35	0.5
M5	15	1.0	1780	0.35	0.5

The mix design strategy presented aligns with the study's objective of developing a lightweight yet structurally efficient concrete. As summarized in Table 1, the systematic variation of foam (10–15 %) and polypropylene (PP) fiber content (0–1 %) allowed tuning of the target density from 2400 kg/m³ down to 1780 kg/m³, bringing the material within the range of structural lightweight concrete. The selected polyurethane foam contents (10% and 15%) and polypropylene fibre dosages (0.5% and 1.0%) were established based on preliminary trial mixes conducted to identify ranges that ensured stable foam dispersion, acceptable workability, and measurable improvements in mechanical and durability performance. Mechanistically, the inclusion of foam introduces a controlled cellular structure that lowers unit weight, while the dispersed PP fibers compensate for the loss in stiffness by bridging microcracks and improving load transfer between paste and aggregate phases. Similar density optimization strategies were reported by [2] and [3], who observed that 10–15 % foam yields a balance between weight reduction and strength retention.

2.3 Mixing and Curing

This section outlines the procedure adopted for combining the constituent materials and for maintaining controlled curing conditions to achieve uniformity and repeatability across all concrete mixes. Uniform dispersion of polyurethane foam within the fresh concrete matrix, as given in Figure 1, shows effective mixing and stable foam incorporation prior to casting. Dry materials

such as cement, fly ash, silica fume, sand and pumice were blended in a pan mixer for two minutes. Approximately 70 % of the mixing water and superplasticizer were then added while mixing continued for another two minutes. Preformed foam was gently folded into the mortar to avoid bubble collapse, followed by gradual addition of polypropylene fibers to ensure uniform dispersion. The remaining water and superplasticizer were then added. The fresh mix was cast into moulds for cubes (100 mm), prisms (100 × 100 × 500 mm) and cylinders (100 × 200 mm), and compacted using vibration. Specimens were demoulded after 24 h and water-cured at 27 ± 2 °C for 7, 28, 56 and 90 days.



Fig. 1. Fresh polyurethane foam concrete mix during laboratory mixing, showing uniform foam dispersion within the cementitious matrix

2.4 Experimental Program

Fresh properties were measured using slump flow and density tests on self-compacting concrete. Hardened properties included compressive strength (IS 516:2018) on cubes, flexural strength on beams, and split-tensile strength on cylinders. Drying shrinkage was monitored on prisms stored in laboratory air for 90 days. Impact resistance was assessed using the drop-weight test (ASTM D7136). Durability was characterized by water absorption, sorptivity, and rapid chloride penetration test (RCPT) per ASTM C1202. Freeze-thaw permeability tests were not performed in this work; however, literature indicates that polypropylene fibers restrict macro-crack width, increase tortuosity, and reduce permeability under freeze-thaw conditions [5]. Microstructural analysis was conducted using scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDX) to identify the distribution of pores, fiber-matrix bonding and elemental composition. Statistical analyses such as regression and correlation were performed to establish relationships between density and strength and to identify optimum mixtures. The drying shrinkage strain was calculated by measuring the change in length of prism specimens at specified ages relative to their initial reference length, using a calibrated length comparator, and expressing this change as strain in micro-strain units. All measurements were taken under controlled laboratory temperature and humidity conditions, and the reported shrinkage values represent the average of at least three specimens to ensure accuracy and repeatability.

2.5 Workflow of the Experimental Program

The overall experimental sequence is illustrated in Figure 2. It begins with material preparation and mix proportioning, followed by casting and curing. After curing, specimens undergo mechanical, durability and microstructural testing, and finally the data are analyzed to determine the optimum mix.

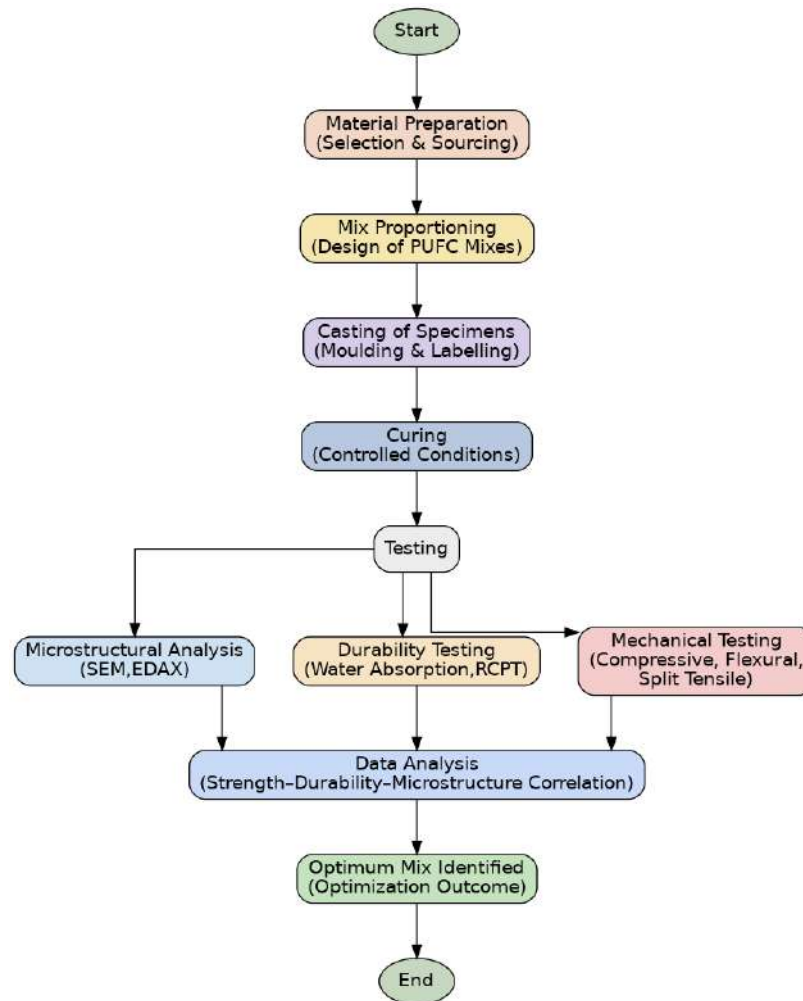


Fig. 2. Workflow of the experimental program for PUFC mix design and evaluation

The workflow adopted in this study ensures a systematic transition from mix design through performance evaluation, thereby anchoring each experimental step in a coherent sequence. As shown in Figure 4, we begin with formulation of foamed concrete blends with varying foam and fiber contents, followed by specimen casting, curing, and sequential testing of mechanical, durability, and microstructural responses. Mechanistically, this pipeline allows us to trace how foam dispersion and fiber bridging influence pore morphology, crack propagation, and load transfer within the cellular matrix, consistent with prior findings that fiber addition enhances tensile and compressive performance in foamed concrete systems [7]. In comparison to earlier works which treated fiber reinforcement and foaming separately, our integrated scheme fills a methodological gap by coupling both effects and enabling direct correlation across multiple property domains [8].

3. Results and Discussion

3.1 Fresh Properties

The addition of polyurethane foam reduced the fresh density of the mixes from 2400 kg/m^3 for the control concrete to 1950 kg/m^3 for 10 % foam mixes and to 1780 kg/m^3 for 15 % foam mixes as illustrated in Table 2. Slump flow decreased with increasing fiber content, with the slump values for mixes containing 0, 0.5 and 1.0 % fibers were 180 mm, 170 mm and 160 mm for 10 % foam mixes (M1–M3) and 150 mm and 145 mm for 15 % foam mixes (M4–M5). The reduction of 10–15 % in workability is consistent with observations that polypropylene fibers restrict the flow of fresh concrete and increase the pore connectivity [5]. However, all mixes remained self-compacting due to the use of superplasticizer.

Table 2. Fresh Properties of Polypropylene-Fiber-Reinforced Polyurethane Foam Concrete Mixes

Mix ID	Foam (%)	PP Fiber (%)	Slump (mm)	Fresh density (kg/m ³)
M0	0	0	200	2400
M1	10	0	180	1950
M2	10	0.5	170	1920
M3	10	1.0	160	1890
M4	15	0.5	150	1820
M5	15	1.0	145	1780

The results demonstrate a clear influence of foam and polypropylene fiber dosage on the fresh behavior of the mixtures. As shown in Table 2, the slump gradually decreased with increasing fiber content from 200 mm for the control to 145 mm for the mix with 15 % foam and 1 % fiber indicating reduced workability due to fiber clustering and increased internal friction. Mechanistically, the entrapped air from polyurethane foam and the random fiber network restricts particle mobility but enhance cohesion and segregation resistance, leading to stable self-compacting behavior. These findings align with those of [5], who reported that PP fibers increase viscosity and modify pore connectivity without compromising flow when coupled with superplasticizers.

3.2 Compressive Strength

The mechanical properties of the polyurethane foam concrete mixes were assessed through compressive strength testing to evaluate the influence of foam content and polypropylene fibre addition on strength development at different curing ages.



Fig. 3. Mechanical strength tests being conducted in the laboratory

Mechanical strength parameters being tested in the laboratory are shown in Figure 3. The compressive strengths of the mixes at 7, 28 and 90 days, are presented in figure 5. At 28 days, the control concrete (M0) reached 40 MPa, whereas mixes with 10 % foam and no fibers (M1) achieved 38 MPa. Incorporating 0.5 % and 1 % fibers increased the strength to 42 MPa (M2) and 46 MPa (M3), respectively. A further increase in foam content to 15 % lowered the strength to 44 MPa (M4) and 43 MPa (M5). These results demonstrate that polypropylene fibers effectively bridge micro-cracks and densify the matrix, leading to strength improvements of up to 30 % relative to the unreinforced foam concrete. Similar trends were reported for foamed concretes reinforced with PP fibers, where tensile and flexural strengths increased significantly and drying shrinkage decreased. Beyond 1 % fiber, workability issues and fiber clustering may negate further strength gains [5].

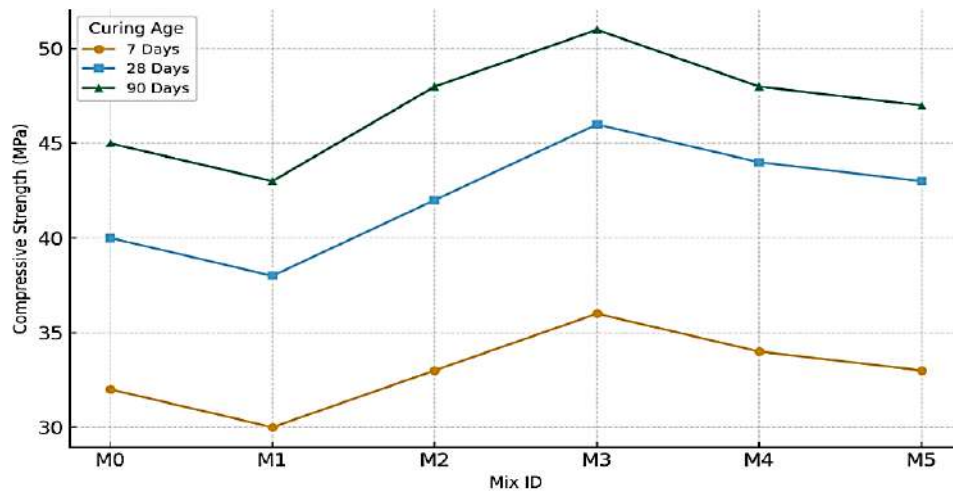


Fig. 4. Variation of compressive strength with polypropylene fiber content at different curing ages

The results displayed in Figure 4 reveal a relation between fiber content and compressive strength, suggesting an optimal dosage beyond which strength gains taper off. Mechanistically, the fibers appear to bridge microcracks in the cementitious matrix, delaying crack coalescence and enhancing post-peak ductility, but excessive fiber insertion may disrupt paste continuity and introduce weak zones. Interpreting these trends in light of previous works, similar strength enhancements with polypropylene, glass, or hybrid fibers have been reported in foamed concrete systems by [9] and [10] showing porosity reduction and strength improvement with fibers.

3.3 Flexural and Split-Tensile Strengths

Flexural and split-tensile strengths show similar improvements with fiber addition. Figure 3 illustrates the flexural strength trend at 7, 28 and 90 days. The control concrete exhibited a 28-day flexural strength of about 5 MPa. With 10 % foam, the strength decreased slightly due to increased porosity, but the inclusion of 0.5 % and 1.0 % fibers raised the values to approximately 5.5 MPa and 6.0 MPa, respectively. Figure 4 shows that split-tensile strength followed the same pattern, where in the control mix reached 3.4 MPa at 28 days, whereas mixes M2 and M3 achieved 3.8 MPa and 4.1 MPa. These enhancements are attributed to fiber bridging, which transfers tensile stress across cracks and increases energy dissipation [4]. Similar behavior has been reported for polypropylene-fiber-reinforced concretes subject to mechanical loading and freeze–thaw cycles, where fibers restrict macro-crack widths and increase tortuosity [5].

The data trend in Figure 5 shows a clear improvement in flexural strength as polypropylene fiber content increases, especially noticeable from 28 to 90 days of curing. This suggests that the fiber network progressively engages in crack-bridging and load transfer as the matrix stiffens with age. Mechanistically, the PP fibers act to arrest microcrack initiation and redistribute stress across micro fissures, thereby delaying full crack coalescence and improving ductility. These findings are consistent with prior studies, for instance, Foamed Concrete reinforced with PP fibers exhibited up to 40 % gain in bending strength. [11] and [12] reported enhanced flexural toughness via combined PP–PVA fiber blends. However, most existing work focuses on fiber effect alone, seldom addressing the interaction of fiber geometry, foam content, and long-term durability, thus leaving a gap in understanding the fiber–foam synergies in flexure.

The variation of split-tensile strength with fiber content demonstrates a clear reinforcing trend. In Figure 6, the tensile strength rises as polypropylene (PP) fiber dosage increases up to about 1 %, highlighting the beneficial effect of fiber bridging in delaying microcrack coalescence and enabling stress redistribution. Mechanistically, the dispersed PP fibers enhance interfacial bonding and act as micro-bridges across developing cracks, restraining crack propagation and thus increasing ductility and residual tensile capacity, which is a mechanism described similarly in studies on PP-reinforced foamed concretes by [13] and [14] in LFC reinforced with PTBF fibers. This behavior

aligns with reports in the literature, such as the improvements in bending and splitting strength in PP-fiber foamed concretes found in [11], yet it also diverges when fiber content is excessive, as clustering and poor fiber dispersion reduce effectiveness.

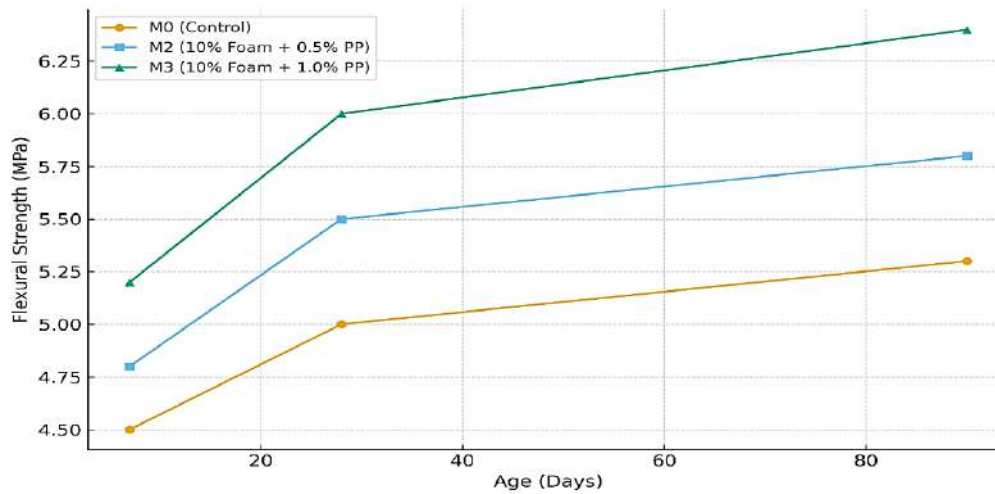


Fig. 5. Variation of flexural strength with polypropylene fiber content at different curing ages

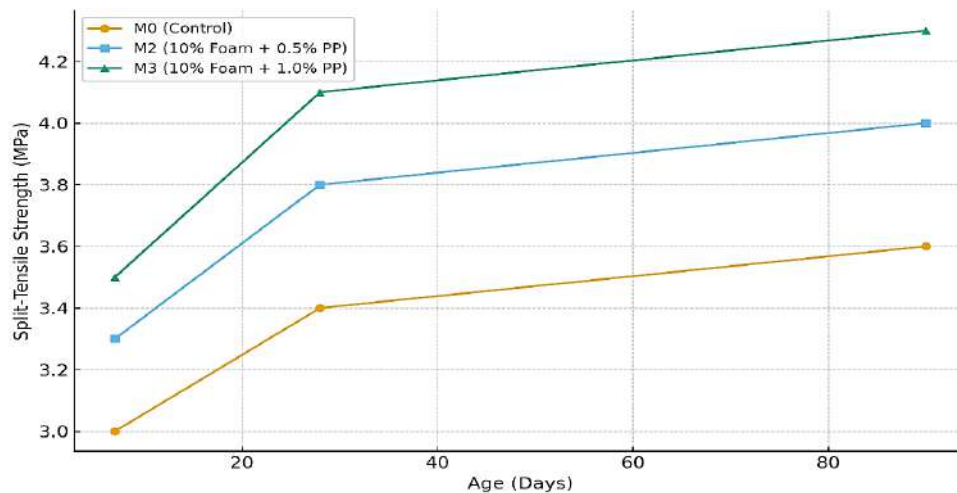


Fig. 6. Variation of split-tensile strength with polypropylene fiber content at different curing ages

3.4 Durability Performance

Durability indicators include water absorption and chloride ion permeability, which are presented in Table 3. Water absorption decreased from 7.0 % in the control concrete to 4.1 % in the mix with 15 % foam and 1 % fiber. The rapid chloride penetration test (RCPT) results show a reduction in charge passed from 3100 C (M0) to 2200 C (M5). These improvements reflect the reduced pore connectivity and crack widths in PP-reinforced PUF concrete. Previous studies on fiber-reinforced concretes have shown that PP fibers restrict macro-cracks, increase the tortuosity of cracks and surface roughness, and consequently reduce permeability. However, excessive fiber content or poor dispersion can create channels that increase matrix permeability [5]. The present results confirm that up to 1 % PP fiber, the net effect is beneficial.

The durability response of the mixes reflects the combined influence of pore refinement and fiber reinforcement in enhancing long-term performance. As summarized in Table 3, both water absorption and chloride permeability decreased consistently with increasing polypropylene fiber dosage and foam content, with the lowest values of 4.1 % and 2200 C recorded for mix M5. Mechanistically, the inclusion of PP fibers bridges microcracks, improves crack tortuosity, and restricts moisture ingress, while the foam-induced closed-cell structure minimizes interconnected

capillary pores. Comparable findings by Zeng et al., (2024) and Mydin and Soleimanzadeh, [4] confirmed that PP fibers reduce permeability and enhance resistance to chloride migration under thermal and cyclic loading. However, the long-term coupling effects of carbonation, freeze-thaw cycles, and fiber dispersion uniformity remain insufficiently explored, suggesting a future scope for integrating advanced permeability models and micro-CT analysis to quantify transport mechanisms in polyurethane-foam-based fiber concretes.

Table 3. Durability properties of polypropylene-fiber-reinforced polyurethane foam concrete mixes

Mix ID	Water absorption (%)	RCPT (Coulombs)
M0	7.0	3100
M1	6.5	2800
M2	5.9	2600
M3	5.0	2400
M4	4.3	2300
M5	4.1	2200

3.5 Shrinkage and Crack Resistance

Drying shrinkage results are shown in Figure 5. The control concrete exhibited a shrinkage strain of about 0.06 % after 90 days. Incorporating 10 % foam and 0.5 % PP fibers reduced shrinkage to 0.055 %, while the combination of 10 % foam and 1.0 % fibers (M3) achieved the lowest shrinkage of 0.045 %. For 15 % foam mixes, shrinkage slightly increased but remained below the control. Polypropylene fibers mitigate shrinkage by bridging micro-cracks and restraining movements at the fiber-matrix interface [4]. This crack-bridging effect contributes to better crack distribution and smaller crack widths, leading to improved durability. Nonetheless, the presence of foam and fibers leads to increased creep at high fiber contents and thus an optimal balance is necessary.

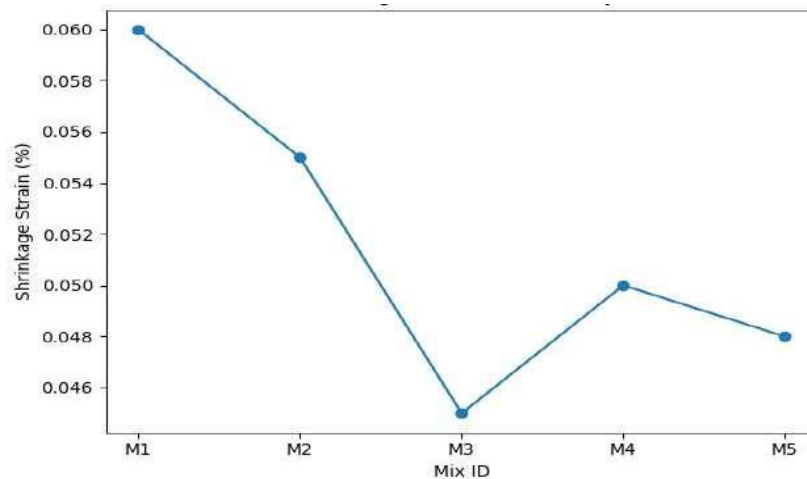


Fig. 7. Drying shrinkage strain of polyurethane foam concrete mixes over 90 days

The trend in Figure 5 demonstrates that as the fiber dosage increases, both early-age and long-term shrinkage strain decrease appreciably, revealing the fibers' restraining influence on volumetric contraction. Figure 7 shows a clear reduction in drying shrinkage with increasing polypropylene fibre content, which can be attributed to fibre bridging that restrains microcrack initiation and limits shrinkage-induced tensile strains within the cementitious matrix. This behavior is consistent with earlier observations in fibre-reinforced lightweight and foamed concretes, where distributed polypropylene fibres were reported to reduce shrinkage by improving internal stress redistribution and crack control. Compared with published ranges for foamed concretes, the magnitude of shrinkage reduction observed in the present study indicates that the combined action of polyurethane foam and fibre reinforcement is particularly effective in

mitigating volumetric instability. This reduction suggests that the fibers act as internal reinforcement, restraining shrinkage-induced tensile stresses and reducing the tendency for microcrack initiation and propagation. Mechanistically, the fibers impose local constraint against capillary tension, create micro-bridging across nascent shrinkage cracks, and interrupt continuous shrinkage strain fields analogous to findings in polypropylene fiber studies where shrinkage mitigation was attributed to fiber network stiffening and moisture retention [15] and [16] in ultra-high-performance concretes using micro- and microfibers. The observed behavior also aligns with results in steel fiber concrete, where distributed fibers reduced drying shrinkage and delayed crack formation [17], albeit other works caution that beyond optimal fiber contents, packing defects or fiber clustering may reduce effectiveness. Yet, a clear knowledge gap persists in quantifying how fiber orientation, spacing, and local moisture gradients collectively influence shrinkage restraint.

3.6 Microstructure and Interface Analysis

SEM micrographs as shown in Figure 6 reveal that the control concrete contains large and interconnected pores, whereas PUF concretes with PP fibers exhibit a more refined pore structure. At 28 days, fibers are embedded within the matrix and surrounded by a dense C-S-H gel. After 90 days, the C-S-H layer thickens, and the fiber-matrix bond becomes more intimate. EDX analysis, as shown in Table 4 indicates that the calcium (Ca) content decreased from 60 % at 28 days to 55 % at 90 days, while the silicon (Si) content increased from 20 % to 25 %.

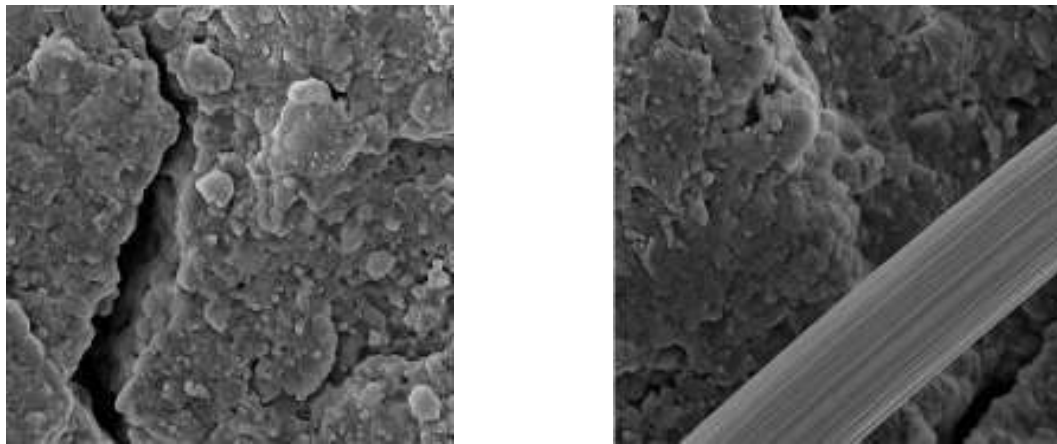


Fig. 8. SEM microstructure of polypropylene-fiber-reinforced polyurethane foam concrete matrix

The relatively constant oxygen (O) content suggests stable hydration products. These changes reflect continued hydration and pozzolanic reactions with fly ash and silica fume, leading to a denser microstructure. Polypropylene fibers show no signs of degradation, which is consistent with their chemical inertness and high melting point [4]. In the micrographs, It can be clearly observed that there are finer pore walls and tighter interfaces in fiber-reinforced specimens. Figure 8 reveals that the control matrix contains large, loosely bound voids, whereas the polypropylene-fiber-reinforced PUF samples show densified microstructure and bridging across microcracks by fibers. Mechanistically, the polypropylene fibers appear to act as crack-arresters and stress-transfer bridges, thereby restricting pore coalescence and promoting additional nucleation of secondary hydration products at the fiber-matrix interface. These insights align with observations by Hazlin et al., (2017), who reported improved microstructural compactness with PP fibers in foamed concrete, and also it resonates with recent findings in foamed PU systems showing that fiber addition leads to enhanced cell wall integrity and mechanical reinforcement [19]. Nevertheless, a key research gap remains in quantifying how interfacial chemistry and nanoscale hydration kinetics evolve over long durations under adverse exposure.

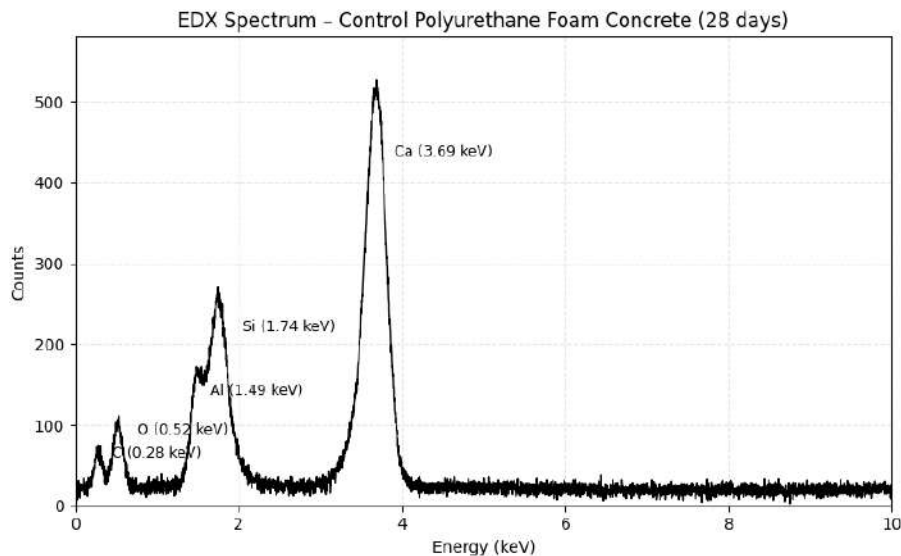


Fig. 9. EDX spectrum of control polyurethane foam concrete at 28 days, showing dominant Ca, Si, Al, O, and C peaks, indicative of hydration products and a relatively calcium-rich cementitious matrix

Figure 9 shows the EDX spectrum of the control polyurethane foam concrete at 28 days, where the elemental composition indicates a relatively calcium-rich and silica-poor matrix compared with the optimum mix. Quantitatively, the atomic percentages reveal Ca at $\sim 22.8\%$ and Si at $\sim 17.1\%$, giving a Ca/Si ratio of ~ 1.33 for the control specimen, which is significantly higher than that of the optimum mix (Figure 6.2, Ca/Si ~ 1.00) and typical of a matrix dominated by portlandite and primary C-S-H with limited secondary gel formation. The oxygen content ($\sim 50.1\%$) remains high, consistent with abundant oxide hydration phases, while carbon ($\sim 5.8\%$) reflects the presence of polymeric components rather than carbonation. Mechanistically, the elevated Ca/Si ratio in the control indicates less extensive pozzolanic conversion of calcium hydroxide into additional C-S-H, resulting in a coarser pore structure and weaker microstructural integration, which explains the lower compressive and flexural strengths reported earlier. This trend aligns with findings in similar lightweight and fibre-reinforced concretes where higher Ca/Si ratios correlate with increased permeability and reduced mechanical performance (e.g., [4]; [7]; [5]). In contrast to some reports of anomalous elemental spikes in aged foamed concretes, the control EDX pattern here does not show such irregularities, confirming a consistent hydration progression but limit secondary reactions. Overall, the numerical shift toward higher calcium relative to silicon in Figure 6.1 supports the narrative that the control mix's microstructure is comparatively less refined and that introducing polypropylene fibres and SCM synergy in the optimum mix drives more favorable microchemical evolution and performance gains.

Figure 10 (optimum mix) indicates a chemically "leaner-calcium / richer-silica" matrix compared with the control, which is the fingerprint of continued secondary C-S-H formation and pore refinement in foamed systems containing SCMs. Quantitatively, the EDX composition shows Ca decreasing from 60% (28 d) to 55% (90 d) while Si increases from 20% to 25%, with O remaining $\sim 20\%$, this shifts the Ca/Si ratio from ~ 3.0 (60/20) at 28 d to ~ 2.2 (55/25) at 90 d, a direction typically associated with consumption of $\text{Ca}(\text{OH})_2$ and growth of additional C-S-H/C-A-S-H gels, which densify the paste and strengthen the ITZ around fibres. Mechanistically, the 10% foam + 1% PP fibre mixture (M3) benefits from a dual action: foam introduces a controlled cellular structure (which can otherwise weaken the matrix), while PP fibres arrest microcrack opening and stabilize the cell walls, allowing hydration and pozzolanic reactions to "convert" more of the available Ca into binding gels rather than leaving a calcium-rich, leachable phase; this aligns with the observed performance balance where the optimum mix achieves $\sim 1890 \text{ kg/m}^3$ density and $\sim 46 \text{ MPa}$ compressive strength, indicating that matrix refinement is sufficient to compensate for foam-induced porosity.

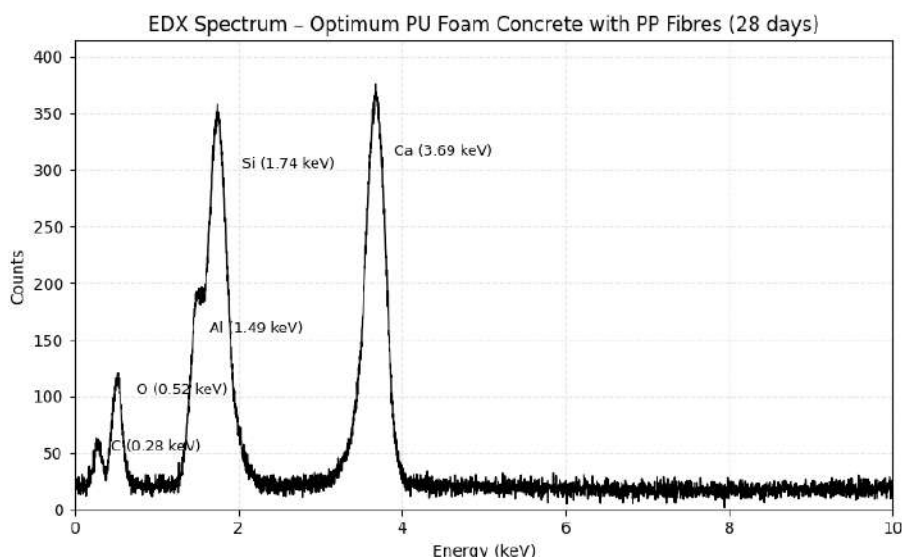


Fig. 10. EDX spectrum of the optimum polyurethane foam concrete incorporating 10% foam and 1% polypropylene fibres at 28 days, showing increased silicon content and reduced calcium intensity, indicative of enhanced C–S–H formation and a denser cementitious matrix

A clear trend is therefore visible (and not a reversal): with age, Ca drops and Si rises, consistent with progressive gel development rather than stagnation; importantly, the lack of any reported anomalous peaks or compositional “spikes” also supports chemical inertness of PP fibres and suggests the improvement is microstructure-driven rather than due to new crystalline phases. Compared with prior fibre-reinforced lightweight/foamed concrete studies that report similar densification signatures under extended curing, the present trend appears more pronounced in the optimum PU-foam system because the fibre bridging reduces crack-connected pathways and helps retain moisture for longer-term reactions, strengthening the link between microchemistry and durability indicators such as reduced ingress pathways.

Table 4. Elemental composition from EDX analysis of polyurethane foam concrete at different curing ages

Mix ID	Ca (%)	Si (%)	O (%)
28-day	60	20	20
90-day	55	25	20

The microstructural assessment provides valuable insights into the progression of hydration and the stability of the fiber–matrix interface over time. As presented in Table 4, the calcium content slightly decreased from 60 % to 55 %, while the silicon fraction increased from 20 % to 25 % between 28 and 90 days, indicating continued pozzolanic activity and the formation of additional C–S–H gel. The Ca/Si ratio derived from Table 4 provides insight into matrix densification, as a lower Ca/Si ratio indicates enhanced formation of secondary C–S–H gel, which is directly associated with improved strength and durability performance. The observed reduction in Ca content with a corresponding increase in Si suggests progressive pozzolanic reactions, leading to a denser microstructure and improved fibre–matrix bonding. SEM observations further support this by showing reduced pore connectivity and more compact hydration products in fibre-reinforced mixes, which explains the lower chloride permeability and enhanced mechanical performance. Mechanistically, this transformation reflects the ongoing reaction of calcium hydroxide with silica fume and fly ash, producing a denser matrix and improved interfacial bonding around the polypropylene fibers. Similar microstructural densification trends were reported by Mydin and Soleimanzadeh [4], who observed enhanced C–S–H development in fiber-reinforced lightweight concretes exposed to prolonged curing. Nevertheless, further nano- scale investigations using TEM or XPS could clarify the evolution of hydration products and fiber– matrix

interactions, addressing the current research gap in linking elemental composition to durability and mechanical performance in polyurethane foam concretes.

3.7 Correlation and Optimization

The relationship between compressive strength and density is illustrated in Figure 7. A clear trend emerges: as density decreases from 2400 kg/m³ to about 1800 kg/m³, compressive strength initially drops but then recovers when fibers are added. The optimum mix (M3, with 10 % foam and 1 % PP fiber) achieves a density of 1890 kg/m³ and a compressive strength of 46 MPa. This balance satisfies the structural LWC criteria and delivers high strength. The fitted regression line ($R^2 > 0.95$) confirms a strong correlation between strength and density. Density reductions beyond 15 % foam (i.e., below 1780 kg/m³) may compromise strength and promote foam collapse. Thus, foam contents greater than 15 % are not recommended. Additionally, fiber contents above 1 % tend to reduce workability and may lead to fiber balling [5].

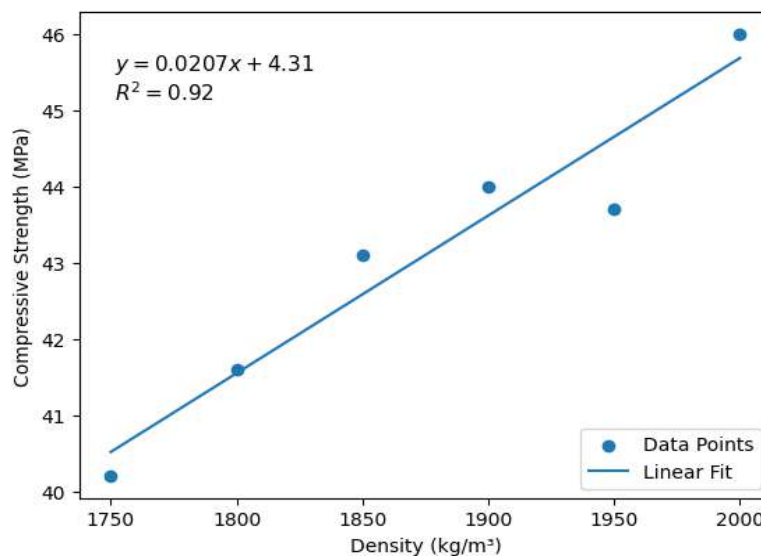


Fig. 11. Correlation between compressive strength and density of polyurethane foam concrete mixes

The trend observed between compressive strength and density of the PU foam concrete incorporated with PP fiber points to a clear trade-off i.e., as density drops through foam incorporation, strength declines. However, with judicious addition of PP fibers, performance can be improved. In Figure 11, it can be observed that beyond a critical density threshold, the strength-density curve becomes steeper, indicating that fibers are effectively compensating for porosity-induced weakening. Mechanistically, PP fibers act as crack-bridging elements that restrain microcrack opening, enhance stress redistribution around voids, and densify the matrix locally, much like what has been reported for fiber-reinforced lightweight concretes [20]. The positive slope of the correlation is consistent with results in foamed-concrete literature, where denser foams yield higher strengths because of reduced void [21]. However, whereas most prior works focus simply on plain foam concretes, our fiber-augmented composite fills a gap in understanding how fiber dosage modulates the strength-density linkage under varying densities.

4. Conclusions

This study has demonstrated that incorporating polypropylene fibers into polyurethane-foam lightweight concrete can produce structural LWCs that exhibit superior strength and durability while maintaining reduced density. A mix with 10 % foam and 1 % fibers achieved a dry density of approximately 1890 kg/m³, representing a 22 % reduction compared with normal concrete. The same mix attained a 28-day compressive strength of 46 MPa, which is around 30 % higher than the unreinforced foam concrete. Water absorption decreased from 7.0 % to 4.1 %, and chloride ion

permeability fell by about 29 %, indicating enhanced resistance to fluid ingress. Drying shrinkage was reduced by roughly 25 % owing to the crack-bridging action of fibers. Microstructural analysis showed dense hydration products around the fibers and no evidence of fiber degradation. These improvements confirm that PP-fiber-reinforced PUF concrete is a durable and economical material suitable for precast panels, bridge decks, marine structures and retrofitting applications. The findings demonstrate that polypropylene fibres effectively compensate for the strength and durability losses typically associated with polyurethane foam incorporation by bridging microcracks, stabilizing the foam-induced cellular structure, and promoting matrix densification. The synergistic interaction between fibres and foam enables the development of lightweight concrete with balanced mechanical performance, reduced shrinkage, and improved resistance to fluid ingress. While the results confirm the effectiveness of the selected mix ranges, the study is limited to laboratory-scale evaluation under controlled conditions, and future work should examine long-term performance under aggressive exposures and explore fibre-foam interactions using advanced quantitative pore analysis techniques. Further research should also explore hybrid fiber systems combining polypropylene with steel or basalt, investigate long-term performance under freeze-thaw and carbonation, and apply finite-element modelling to optimize fiber distribution and foam stability. Life-cycle assessment could also quantify the environmental benefits of this lightweight system.

References

- [1] Abedi M, Kanstad T, Jacobsen S, Ji G. Shrinkage properties of steel fiber reinforced concrete- A review. *Appl Eng Sci.* 2025;23:100244. <https://doi.org/10.1016/j.apples.2025.100244>
- [2] Amran M, Fediuk R, Vatin N, Huei LY, Murali G, Ozbakkaloglu T, et al. Fibre-Reinforced Foamed Concretes: A Review. *Materials.* 2020;13(19):4323. <https://doi.org/10.3390/ma13194323>
- [3] Androuët C, Charron JP. Shrinkage mitigation of an ultra-high performance concrete submitted to various mixing and curing conditions. *Materials.* 2021;14(14):3982. <https://doi.org/10.3390/ma14143982>
- [4] Hazlin AR, Iman A, Mohamad N, Goh WI, Sia LM, Samad AAA, et al. Microstructure and tensile strength of foamed concrete with added polypropylene fibers. *MATEC Web Conf.* 2017. <https://doi.org/10.1051/mateconf/201710301013>
- [5] Hedjazi S, Castillo D. Utilizing polypropylene fiber in sustainable structural concrete mixtures. *CivilEng.* 2022;3(3):562-72. <https://doi.org/10.3390/civileng3030033>
- [6] Jiang P, Wang F, Li N, Wang W, Wang B, Yu P. Polyurethane foam lightweight concrete: Preparation, CO₂ fixation properties and mechanism. *J CO₂ Util.* 2025;97:103109. <https://doi.org/10.1016/j.jcou.2025.103109>
- [7] Montaser WM, Shaaban IG, Zaher AH, Khan SU, Sayed MN. Structural behaviour of polystyrene foam lightweight concrete beams strengthened with FRP laminates. *Int J Concr Struct Mater.* 2022;16(1). <https://doi.org/10.1186/s40069-022-00549-1>
- [8] Mydin MAO, Abdullah MMAB, Razak RA, Nawawi MNM, Risdanareni P, Puspitasari P, et al. Study on polypropylene twisted bundle fiber reinforced lightweight foamed concrete. *Buildings.* 2023;13(2):541. <https://doi.org/10.3390/buildings13020541>
- [9] Mydin MAO, Soleimanzadeh S. Effect of polypropylene fiber content on flexural strength of lightweight foamed concrete at ambient and elevated temperatures. *Adv Appl Sci Res.* 2012;3:2837-46.
- [10] Othman R, Jaya RP, Muthusamy K, Sulaiman M, Duraisamy Y, Abdullah MMAB, et al. Relation between density and compressive strength of foamed concrete. *Materials.* 2021;14(11):2967. <https://doi.org/10.3390/ma14112967>
- [11] Othuman Mydin MA, Mohd Nawawi MN, Omar R, Dulaimi A, Najm HM, Mahmood S, et al. Mechanical, durability and thermal properties of foamed concrete reinforced with synthetic twisted bundle macro-fibers. *Front Mater.* 2023;10:1158675. <https://doi.org/10.3389/fmats.2023.1158675>
- [12] Rodacka M, Domagała L, Szydlowski R. Assessment of properties of structural lightweight concrete with sintered fly ash aggregate in terms of its suitability for use in prestressed members. *Materials.* 2023;16(15):5429. <https://doi.org/10.3390/ma16155429>
- [13] Roobankumar R, Pandian S. Investigating the correlation between ultrasonic pulse velocity and compressive strength in polyurethane foam concrete. *Sci Rep.* 2025;15. <https://doi.org/10.1038/s41598-025-07666-3>
- [14] Rudziewicz M, Maroszek M, Setlak K, Góra M, Hebda M. Optimization of foams-polypropylene fiber-reinforced concrete mixtures dedicated for 3D printing. *Materials.* 2024;17(16):4106. <https://doi.org/10.3390/ma17164106>

- [15] Shankar AN, Chopade S, Srinivas R, Kumar Mishra N, Eftikhaar HK, Sethi G, et al. Physical and mechanical properties of foamed concrete, a literature review. *Mater Today Proc.* 2023. <https://doi.org/10.1016/j.matpr.2023.10.105>
- [16] Shi X, Ning B, Wang J, Cui T, Zhong M. Improving flexural toughness of foamed concrete by mixing polyvinyl alcohol-polypropylene fibers: An experimental study. *Constr Build Mater.* 2023;400:132689. <https://doi.org/10.1016/j.conbuildmat.2023.132689>
- [17] Wen C, Shen D, Luo Y, Wang W, Liu C, Li M. Early-age autogenous shrinkage and tensile creep of concrete reinforced with polypropylene macro fiber. *J Sustain Cem Based Mater.* 2023;12(10):1255-69. <https://doi.org/10.1080/21650373.2023.2214135>
- [18] Xu Y, Yao L, Yu X. Effect of polypropylene fibers on mechanical and wetting properties of overall superhydrophobic foamed concrete. *Constr Build Mater.* 2024;448:138207. <https://doi.org/10.1016/j.conbuildmat.2024.138207>
- [19] Zeng W, Wang W, Wang Q, Li M, Zhang L, Tong Y. Influence of polypropylene fiber on concrete permeability under freeze-thaw conditions and mechanical loading. *Materials.* 2024;17(12):2945. <https://doi.org/10.3390/ma17122945>

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Influence of ultrafine blast furnace slag and nano silica on the engineering characteristics of self-compacting concrete

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Abstract

Self-compacting concrete (SCC) mix design conventionally requires a high binder content to achieve the required flowability and segregation resistance, thereby increasing the risk of shrinkage at both early and later ages. The present study explores the synergistic effect of incorporating nano silica (NS) and ultrafine blast furnace slag (UFBFS) on the fresh and hardened properties of M60 grade SCC, including durability and shrinkage performance, by optimizing the binder content without compromising SCC performance requirements. Preliminary experimental investigations revealed that UFBFS enhances workability and strength properties, while NS increases paste viscosity and uniformity. The results of the experiments showed that a 5–8% replacement of the UFBFS-to-OPC ratio could be considered optimal for enhancing compressive strength. Integrating UFBFS and NS allowed for optimizing the binder content at 565 kg/m³ while meeting SCC flow requirements, reducing it from 600 kg/m³. The modified SCC demonstrated a 28-day compressive strength of 70 MPa and exhibited better durability, such as high resistance to water penetration, low water absorption, and low chloride permeability. The drying shrinkage was reduced by 38% in the modified SCC, and cracking due to plastic shrinkage was avoided in the modified SCC. The work clearly illustrates how UFBFS and NS can be used to prepare a durable SCC at lower binder content and with good shrinkage resistance.

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1. Introduction

The application of self-compacting concrete (SCC) is considered a solution for the construction of reinforced concrete structures (RCC) that pose casting difficulties [1]. Difficulties can be due to 1) the unique shape of the structural element, 2) rebar congestion, 3) limited access for placing concrete effectively, and 4) the need for a higher number of resources in the form of skilled manpower and equipment required for placing and compaction [2–4]. In some situations, the project execution team may encounter multiple difficulties, especially a shortage of skilled manpower required for casting. It could be a challenge for the execution team to cast such RCC Structures without any blemishes while meeting the time-bound execution plan [5,6]. Experience across various parts of the world shows that the preference for speedy construction can lead to improper casting of RCC elements, thereby adversely affecting the durability of RCC structures. Criticalities, as mentioned above, add to the challenge of achieving durability in time-bound project execution. Hence, a concrete mix with the flowing ability of a fluid was considered to address these difficulties [7,8]. Okamura first conceptualized a fluid-like concrete material in 1986 in Japan, where a shortage of skilled labor made durable construction a challenge, leading to the development of self-compacting concrete. Ozawa and Maekawa conducted studies on the development of self-compacting concrete with appropriate workability and other hardened concrete properties at the University of Tokyo. Subsequently, a prototype SCC application was

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made in 1988 [9]. The approach was to make fresh concrete fluid enough to remove entrapped air to the maximum extent possible, thereby eliminating the need for manual compaction.

While a flowing concrete mix containing varying aggregate sizes is required to pass through rebars and fill the formwork, it is expected to remain uniform without segregation. Hence, SCC is designed to meet requirements such as flowing ability, passing ability, segregation resistance, and optimal viscosity [10,11]. Dependence on a higher binder content to make a fresh SCC mix robust has been the conventional approach since the earliest days of SCC. Okamura and Ouchi state that limiting the coarse aggregate content in SCC to levels lower than those in normal concrete is essential to reduce the energy required for the flow of fresh concrete. The concept is to increase the space between aggregate particles to reduce the internal stress caused by collisions and contacts between aggregates. A reduction in coarse aggregates and an increase in fines could provide a better-flowing medium, helping the SCC mix remain uniform whilst flowing to fill and occupy the structure, even in the presence of congested rebars, and to take shape without requiring any external compaction. Reduction in coarse aggregates has thus become an invariable feature of an SCC mix [12]. However, reducing the aggregate/binder ratio compared with normal concrete would increase the concrete's shrinkage probability at both early and later ages. Although the mechanical properties of the SCC are superior to those of the conventional concrete, the shrinkage of SCC is significantly higher [13]. Farhad Aslani and Shami Nejadi quote based on other researchers that "because the SCC has a higher paste volume (or higher sand to aggregate ratio) to achieve high workability and high early strength, several researchers have claimed larger shrinkage of the SCC for precast, prestressed concrete, resulting in larger prestress losses" [14].

Various research into chemical admixtures led to the development of a viscosity-modifying agent (VMA). As the name implies, VMA increases the viscosity of the liquid medium, i.e., the slurry, making it more viscous [15]. VMA is found to be highly effective in underwater concrete, enhancing its anti-washout properties. A fresh mix with higher workability yet remains very viscous enough to resist washing out when placed underwater is possible with the addition of VMA [16,17]. This principle has been extended to SCC as well. An increase in slurry viscosity is found to moderately counter segregation in the SCC mix. Gettu R et al. concluded from an experimental study that the use of VMA in SCC mix enhanced the uniformity and robustness of the fresh concrete mix [18]. The incorporation of VMA into the SCC mix has also reduced binder content [19]. The reason is that the role of the increased binder content in making the slurry more viscous is now partially offset by VMA. The industry worldwide has started using liquid-based VMA in SCC, the last ingredient in the batching sequence added to the concrete mixer. As far as India is concerned, its availability is limited due to import dependency and a lack of manufacturers. Apart from a lack of availability, SCC mixes with VMA have been reported to yield mixed results, including an adverse impact on strength and durability. After reviewing various research works on the different behavior of VMA, Beata Lazniewska-Piekarczyk reported that "due to the controversial data dealing with the relationship between the viscosity-modifying agents and strength of concrete, the experiments should be performed to find how the type of VMA alters mechanical properties". Based on the experimental findings, it was concluded that the VMA type also significantly affects the air content in high-performance self-compacting concrete (HPSCC) [20]. Hence, an alternative solution is required in the Indian context. Attempts are also being made worldwide to optimize the binder content in SCC using suitable alternative materials that are finer than cement and locally available. Research has been conducted on the use of fine limestone powder, along with other supplementary cementitious materials (SCM) such as fly ash or GGBS, to improve the deformability and stability of fresh SCC [21–24].

In India, ultrafine blast furnace slag (UFBFS) has been used for more than a decade, especially in high-grade applications [25]. It is considered an alternative to micro silica. Micro silica has been used in India for more than 25 years and is imported, originally from Europe and later mainly from China. Micro silica fineness is known to be extremely high, i.e., more than 12,000 m²/kg, and can reach 25,000 m²/kg. Micro silica's fineness contributes to a reduction in bleeding and segregation to some extent when added to concrete. The addition of micro silica also increases admixture demand due to its extreme fineness, as concluded by Roncero J et al. in their study of cement pastes [26]. The main purpose of incorporating micro silica is to enhance concrete's mechanical

properties. It has been found that compressive strength, split tensile strength, and static modulus of elasticity increased with the addition of micro silica [27]. However, as reported by Yun K K et al., the probability of plastic shrinkage increases with the addition of micro silica due to its extreme fineness, leading to an increase in capillary pressure [28]. Another important challenge is its densified form. Densification is required, as handling silica fume is impractical due to its very low bulk density. Thus, densification becomes necessary to pack, transport, and store MS without requiring a much greater volume effect [29]. It is important to note that densified micro silica should be fully de-densified during the mixing process in the concrete mixer. In regular concrete mixing, substantial amounts of micro silica agglomerates are almost always retained in the concrete. Sidney Diamond and Sadananda Sahu state that “thus the assumption that the densification process is somehow ‘reversible’ is not generally warranted. The sizes of undispersed agglomerates remaining in concrete after mixing often exceed those of Portland cement particles, thereby limiting the potential benefits of the fine particle filler effect. Large undispersed grains always appear to undergo chemical reactions in concrete, but such reactions may induce alkali silica reaction (ASR) damage only under especially unfavorable circumstances [30]. In such a condition, MS could be counterproductive.

UFBFS is understood to have contributed to the mechanical properties of concrete, similar to micro silica. The amorphous nature and high fineness of UFBFS are found to be the factors behind the enhancement of concrete strength and durability. Ground granulated blast furnace slag (GGBS) is produced by grinding granular slag aggregates that are made by the controlled quenching of molten slag, which is released as a by-product during steel manufacturing. When these granular slag aggregates, which are amorphous in nature, are ground much finer, they become UFBFS. While the fineness of GGBS is around 4,000 m²/kg, that of UFBFS exceeds 10,000 m²/kg. Its high fineness is found to contribute to the secondary reaction with calcium hydroxide and the formation of secondary calcium silicate hydrate (C-S-H) gel [31]. As in the case of micro silica, UFBFS is found to have a higher pozzolanic reactivity index and can therefore replace a higher percentage of OPC. The advantage of UFBFS is that it is produced in India. It is also important to note that densification is not required for UFBFS. The product is used in its “as produced” form. Based on these facts, UFBFS is a suitable alternative to micro silica as a tertiary binder to optimize OPC content.

Another material under consideration in various research on concrete is nano silica (NS), among other nanoparticles. The influence of NS on both fresh and hardened properties has been analyzed and presented by many researchers. Niewiadomski et al have mentioned that “based on the studies conducted, it can be concluded that the addition of nanoparticles improves the microstructure of self-compacting concrete. Analysis of the porosity results proved that concretes modified with the addition of nanoparticles are characterized by lower porosity” [32]. On the mechanical and rheological properties of SCC with nanoparticles, Faez A et al have shared that “the increase in nanoparticles increases the homogeneity and consistency of concrete. Due to their higher effective surface area and greater water absorption, Al₂O₃ Nano Particles increase Yield Stress and reduce flowability [33]. In the study by Quercia et al., it was reported that nano silica significantly increases the water demand of cement paste, resulting in reduced workability [34]. It can be inferred that adding NS increases the viscosity of the concrete mix. Dispersion of NS, like other nanoparticles, has been a topic of particular interest, as agglomeration can occur in slurry media due to their extreme fineness. The use of a poly carboxylate ether (PCE)-based superplasticizer is found to be effective for dispersing nanoparticles. Du et al., in their review, citing various studies, noted that the use of superplasticizers helps overcome the difficulty of dispersing nanoparticles in cement-based materials [35]. Khan et al. have highlighted the beneficial effect of PCE-based admixture in effectively dispersing NS [36]. Since the PCE-based superplasticizer is used in the SCC mix to impart flowability, the challenge of NS dispersion can be addressed.

Recent studies have reported the individual effects of ultrafine blast furnace slag (UFBFS) and nano silica (NS) on concrete properties. The high fineness and pozzolanic reactivity of UFBFS have been proven to enhance strength and durability. At the same time, the NS is known to influence the rheological behavior and refinement of the cementitious matrix. However, the existing literatures mainly focus on the individual application of these materials or on conventional concrete systems. Very few studies have investigated the application of UFBFS in combination with nano silica in self-

compacting concrete, especially to optimize binder content while minimizing shrinkage potential without compromising fresh and hardened concrete performance. The availability of the two aforementioned finer materials, one ultra-fine and the other of nano-scale, provides a good opportunity to consider them in designing a uniform and robust SCC mix with reduced binder content as an alternative to the conventional approach of using higher binder content. Hence, the current research aims to study the combined effect of UFBFS and NS on the fresh and mechanical properties of SCC, its durability, and its shrinkage potential, to formulate a reliable SCC with an optimized binder content and lower shrinkage. The research objectives of the present study are:

- To investigate the integrated effect of UFBFS and nano silica on the fresh properties of SCC.
- To evaluate the effect of UFBFS and NS on the mechanical and durability features of SCC
- To examine the influence of UFBFS and NS on the plastic and drying shrinkage behavior of SCC.
- To develop a robust SCC mix with optimized binder content and improved shrinkage resistance.

2. Materials

2.1. Aggregates

Coarse aggregates (CA) with 20 mm and 12.5 mm as nominal maximum size (NMS) are chosen for the initial experimental works to evaluate the efficiency of UFBFS, whereas CA with 12.5 mm as NMS are chosen for experimental works on SCC. Coarse and fine aggregates belong to the granite type of rock. Particle size distribution is given in Figs. 1 and 2.

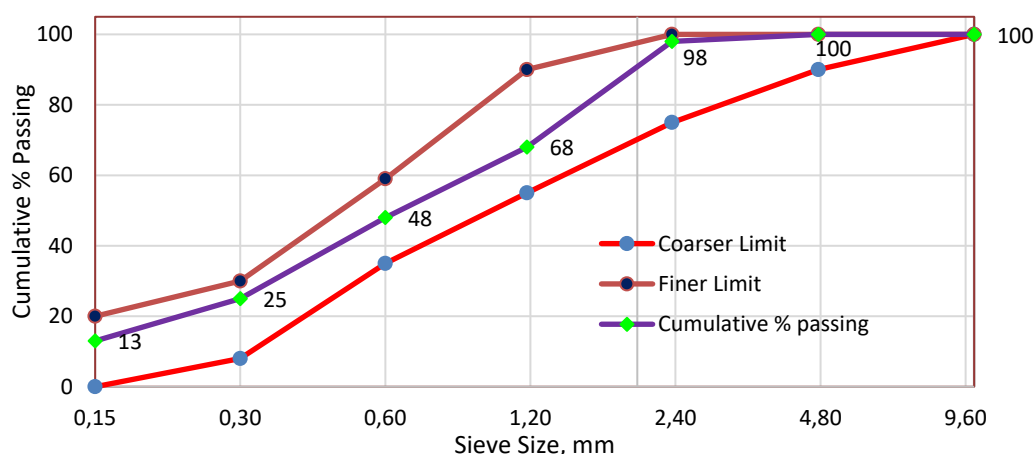


Fig. 1. Particle size distribution of crushed sand stone

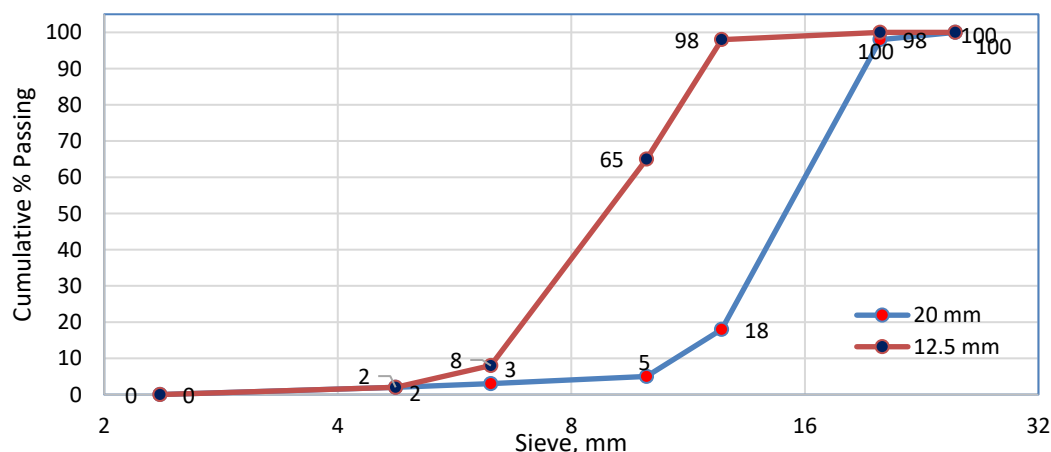


Fig. 2. Particle size distribution of coarse aggregates used in this study

The crusher unit located at Walaja, near Chennai, has been identified as the source of CA, with a three-stage crushing process comprising a vertical shaft impact (VSI) crusher installed as the third-stage crusher. Fine Aggregates (FA) used in the experiments are the product of crushed stone aggregates processed through a VSI Crusher, which provides better particle shape. The same crusher unit from which CA is sourced is also used to procure crushed stone sand (CSS).

2.2. Ordinary Portland cement

Ordinary Portland Cement (OPC) of 53 Grade is chosen for the experimental work. OPC from a single source, i.e., the UltraTech Cement factory located near Chennai, has been ensured throughout the study period. The sample has been tested for its Physical and Chemical Properties; these are presented in Tables 1 and 2.

Table 1. Physical properties of binders used in the study

Physical Properties		OPC	Fly ash	UFBFS	NS
Blaine's Permeability, m ² /kg		274	326		
Fineness (BET Method), m ² /kg				2730	202,000
Average Particle size, um in %				1.5	0.017
Retention on 45 microns			29 %		
Normal Consistency		27.0 %			
Setting Time	Initial Setting Time (min)	90			
	Final Setting Time (min)	225			
Sp. Gravity	Le-Chatelier Flask	3.15	2.10	2.90	2.4
Soundness	Le-Chatelier (mm)	0.15			
Compressive Strength	72 hours (MPa)	32.9			
	168 hours (MPa)	43.2			
	672 hours (MPa)	57.8			
28-day Compressive Strength w.r.t. OPC			85%	127%	

Table 2. Chemical properties of the binders employed in the study

Chemical Properties		OPC	Fly ash	UFBFS	NS
Aluminum oxide, percent by mass		4.9			
Ferric oxide, percent by mass		4.9			
Insoluble residue percent by mass		3.0		0.11	
Magnesium oxide (MgO), in %		1.8		7	
Manganese oxide (MnO), in %				0.35	
Sulphate (SO ₃), in %		2.8	0.19	0.26	
Sulphide Sulphur				0.3	
Total loss on ignition percent by mass		2.8	0.19	0.11	0.66
Silica, percent by mass		20.4	64.2		99.9
Calcium oxide, percent by mass		59.7	2.11		
Total chloride content, percent by mass		0.010	0.011	0.015	0.010
Equivalent Sodium Oxide, percent by mass			0.26		
Glass Content, in %				96	
CaO-0.7SO ₃ /2.8SiO ₂ +1.2Al ₂ O ₃ +0.65Fe ₂ O ₃		0.90			
Silicon dioxide (SiO ₂) + Aluminum oxide (Al ₂ O ₃) +Iron oxide (Fe ₂ O ₃) in % by mass			95.8		
(CaO+MgO+1/3Al ₂ O ₃)/(SiO ₂ +2/3Al ₂ O ₃), in %				1.12	
(CaO+MgO+Al ₂ O ₃)/(SiO ₂), in %				1.8	

2.3. Fly Ash

Fly ash collected from the electrostatic precipitator is used in the experimental studies. The thermal power plant located in North Chennai is identified for this study, and the fly ash available at this

source belongs to 'Type F'. The physical and chemical property test results of the fly ash employed in this study are presented in Tables 1 & 2, respectively.

2.4. Ultrafine Blast Furnace Slag (UFBFS)

UFBFS particles are finely ground blast furnace slag with a D (0.95) less than 10 microns. The UFBFS used is Alccofine, a product of Gujarat Ambuja Limited, and is governed by IS 16715:2018 (amended 2019). Its properties (physical & chemical) are presented in Tables 1 and 2, respectively.

2.5. Nano Silica

Nano silica with particle sizes ranging from 15 to 80 nm and containing more than 90% of silicon dioxide (SiO_2) is sourced for the experimental study. The properties of the nano silica, as given in Tables 1 & 2, indicate that NS is highly reactive. Nano silica was incorporated as the final ingredient during mixing to enhance the uniformity of the fresh SCC mix. Sonication was not employed in this present work. The adopted procedure was based on Marsh Cone flow test findings, which showed that flow time increased with increasing NS addition. The images of SCC trial mixes, shown before and after the addition of NS, show a fresh SCC mix becoming uniform and cohesive after NS is added.

2.6. Superplasticizer

A PCE-based superplasticizer admixture is selected for the trial mixes in this experimental study. High-range water-reducing (HRWR) CONXL PCE 2635 from Chemcon TecSys, Chennai, is used in the trial mixes, which is Type 'G' as per ASTM C494 standards.

3. Mix Design Methodology

Methods as per IS 10262:2019 and IS 9103:1999 are used for proportioning normal concrete mixes. "European Guidelines for self-compacting concrete: 2005" is referred to for the design and evaluation of SCC mix. Preliminary trial mixes are planned to study the behavior of NS and UFBFS, as well as their interactions. A slurry-based trial is planned to assess the effect of NS on the cement slurry. A concrete trial mix is initially conducted to determine the water-reduction efficiency and dosage of the selected superplasticizer.

Table 3. Mix design criteria

Description	Nominal mix	SCC mix
Grade mix	M60	M60
Target mean strength	$60 + 1.65 \times 5 = 71.25 \text{ MPa}$	$60 + 1.65 \times 5 = 71.25 \text{ MPa}$
Maximum Water to Binder Ratio	0.31	0.31
Nominal Maximum Size of coarse aggregates	20mm	12.5 mm
Assumed Air content	1%	1.5%
Workability Test	Slump Test (200+/-25mm)	Flow (600-700mm), T500, V-Flow & J-Ring Flow

Table 4. Fresh concrete property requirements of SCC

Fresh Concrete Properties	Test Method	Requirement
Flowing Ability	Slump Flow	600 +/- 50mm
Viscosity	T500	< 6 seconds
Viscosity	V-Flow	< 26 seconds
Passing Ability / Segregation resistance	J-Ring Flow	Difference between Slump Flow and J-Ring Flow < 50mm

A set of trial mixes for the M60 grade is planned to assess the effect of UFBFS on concrete compressive strength and to evaluate its potential to reduce OPC content in the mix. Trial mixes for the main objective of proportioning M60 grade SCC with UFBFS and NS are planned to be conducted subsequently. Details of the mix design criteria are provided in Table 3, and the fresh concrete

properties of the SCC considered in the experiments are provided in Table 4. After verifying fresh concrete properties, samples are taken, and specimens are cast to measure the hardened concrete properties of the mixes for comparison and evaluation.

4. Experimental Works on NS and UFBFS

4.1. Marsh Cone Test

To understand the influence of nano silica on workability (flowability), a Marsh cone test was conducted on a cement slurry. The Marsh cone test is used to measure the fluidity/rheology of one or more combinations of cementitious materials and superplasticizer [37]. It helps to establish cement-admixture compatibility to achieve an optimal superplasticizer dosage. The same method was used to determine the rheology of slurry with OPC and NS, measured as the time taken for a half-liter slurry to flow through the Marsh Cone. In the Marsh cone test, the flow duration of slurry mixes having varying NS percentage was determined. To evaluate the influence of NS in cement slurry, slurry mixes containing 0%, 0.25%, 0.5%, 0.75%, and 1.00% NS with respect to OPC content were prepared, with OPC content, water content, and SP dosage held constant. It is evident from Figs. 3 & 4, with an increase in the percentage of NS in the slurry, the slurry flow duration increased as the slurry became increasingly stiffer.

Many researchers have found that NS increases water demand in cement paste or concrete mixes owing to its very high surface area [38,39]. In other words, it alters the viscosity of the mix. The Marsh cone test of the cement slurry indicates that increasing the NS dosage reduces the slurry's flowability. Hence, it can be concluded that the addition of NS to the slurry medium of a concrete mix would increase the yield stress, thereby preventing segregation in the flowing concrete [40].

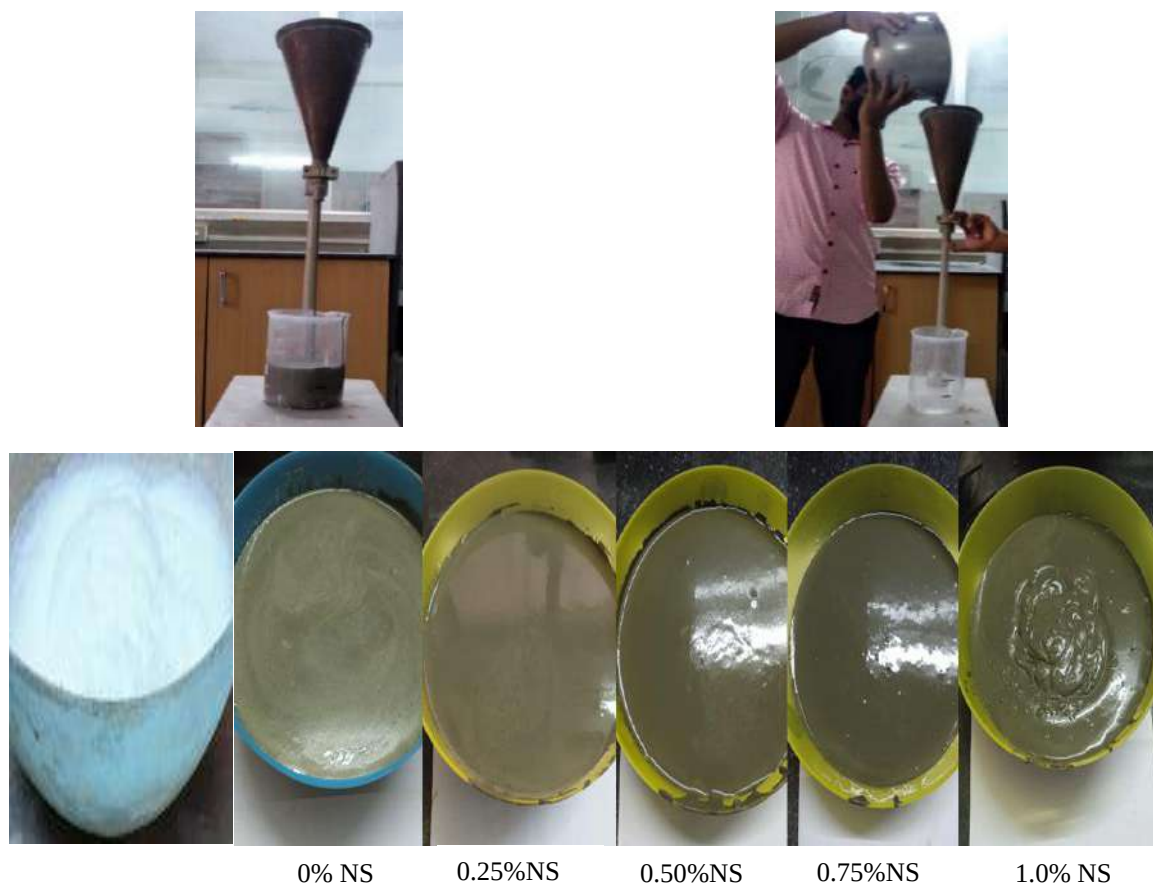


Fig. 3. Marsh Cone test, image of NS and images of slurry with varying % of NS

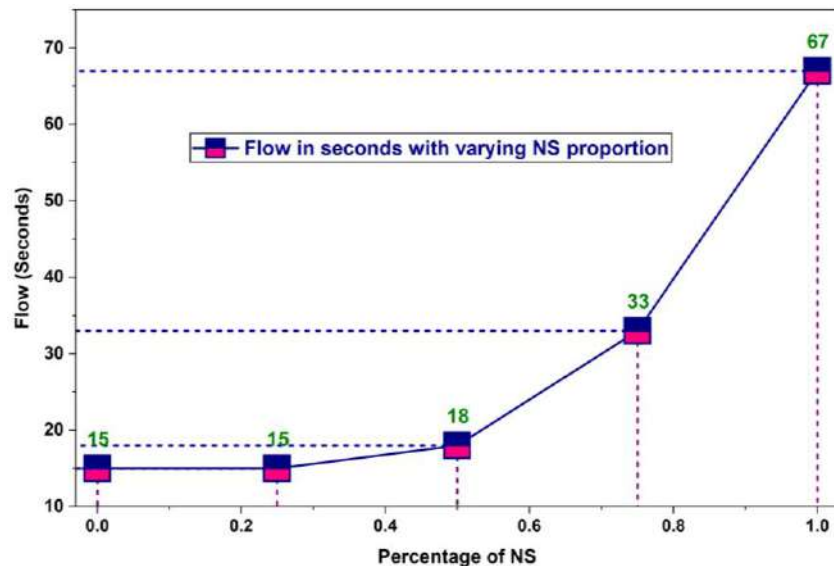


Fig. 4. Increase in flow duration with increase in NS proportion

4.2. SEM-EDX and XRF Analysis

Scanning electron microscopy (SEM) coupled with energy-dispersive X-ray spectroscopy (EDX) was performed on two paste samples: one containing pure OPC and the other containing a combination of OPC and NS. These paste specimens were cured for 28 days, and images were taken from their thin sections. SEM images in Figs. 5 and 6, when compared, show dispersion of NS Particles in the paste containing NS.

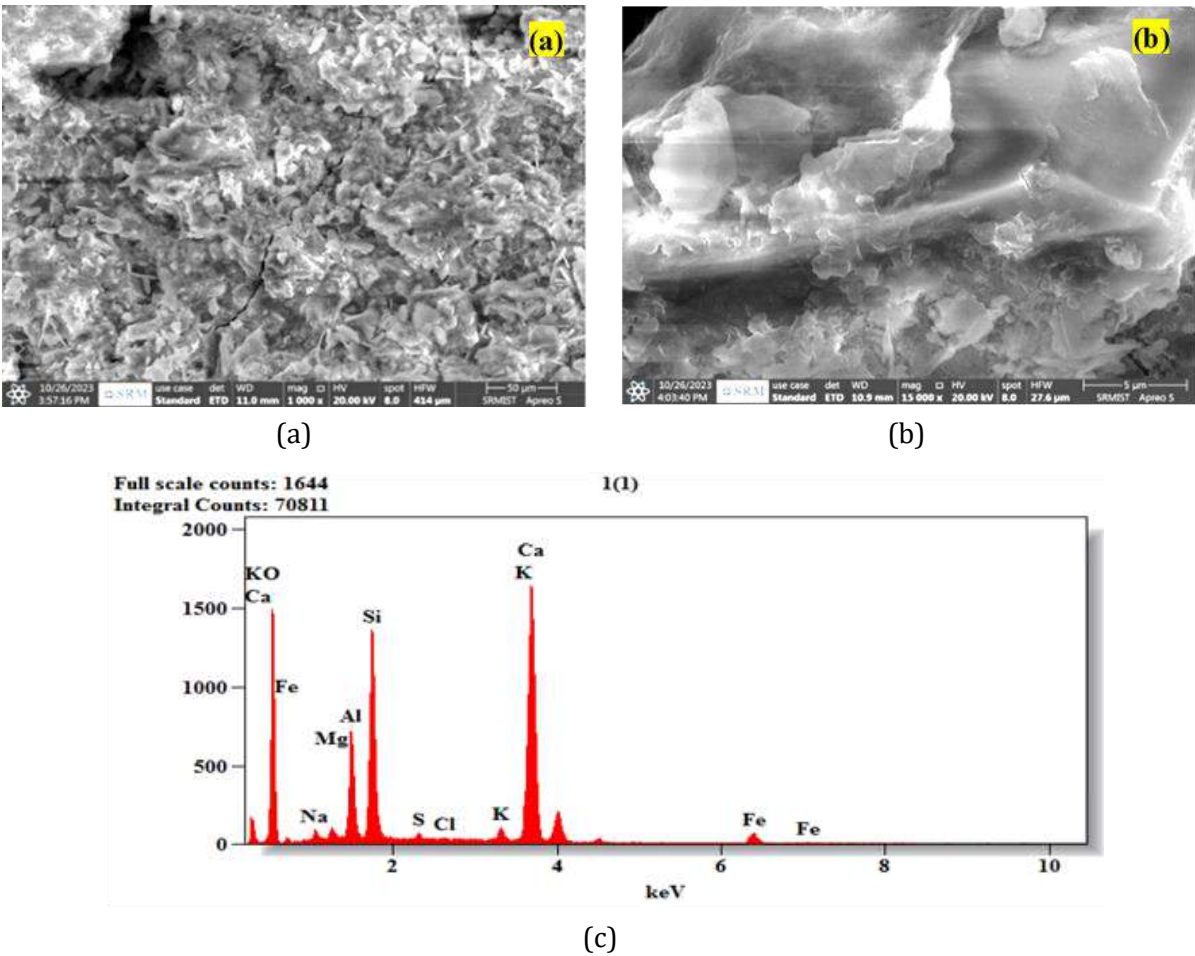


Fig. 5. Hydrated paste with OPC a) image 50 µm scale, b) image at 5 µm and c) EDX image

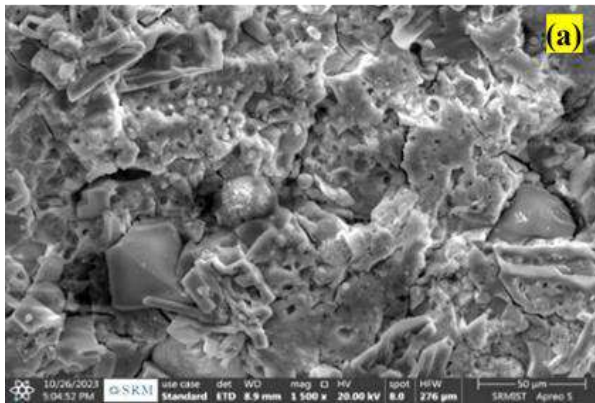
EDX data of the paste combining OPC and NS, as reported in Table 6, also indicate a relatively higher Si content (15.15%) and a lower Ca content (4.93%) in the vicinity of NS. In contrast, paste with pure OPC contains a higher Ca content (24.7%), as shown in Table 5. Particle size (5.1 μm) of NS in Fig. 6 (b) indicates that there is agglomeration of NS particles. The possible reason is explained by Lavergne F et al., who reported that flocculation of NS occurs due to the presence of Ca ions in the pore solution, which adsorb to the NS surface [39]. However, it is observed that, despite flocculation of NS, the slurry becomes more viscous, which is of particular importance to the current research.

Table 5. EDX data of the slurry with OPC

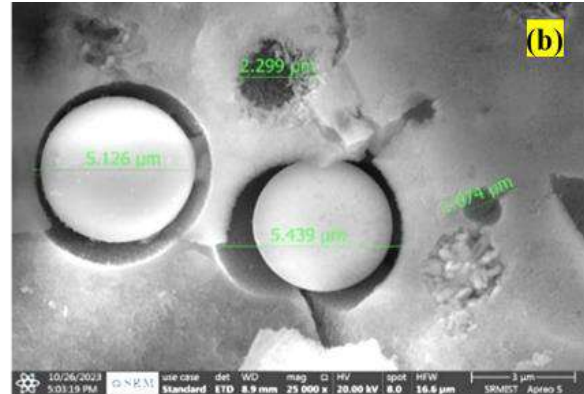
Element	Net counts	Weight (%)	Atom (%)	Atom error (%)	Formula
O	8125	52.66	70.29	± 0.80	O
Na	445	0.94	0.88	± 0.07	Na
Mg	463	0.65	0.57	± 0.05	Mg
Al	5369	5.68	4.50	± 0.11	Al
Si	11903	11.88	9.04	± 0.11	Si
S	329	0.29	0.20	± 0.03	S
Cl	32	0.03	0.02	± 0.02	Cl
K	830	0.89	0.48	± 0.03	K
Ca	20563	24.70	13.16	± 0.14	Ca
Fe	784	2.27	0.87	± 0.10	Fe
Total		100.00	100.00		

Table 6. EDX data of slurry with OPC and NS

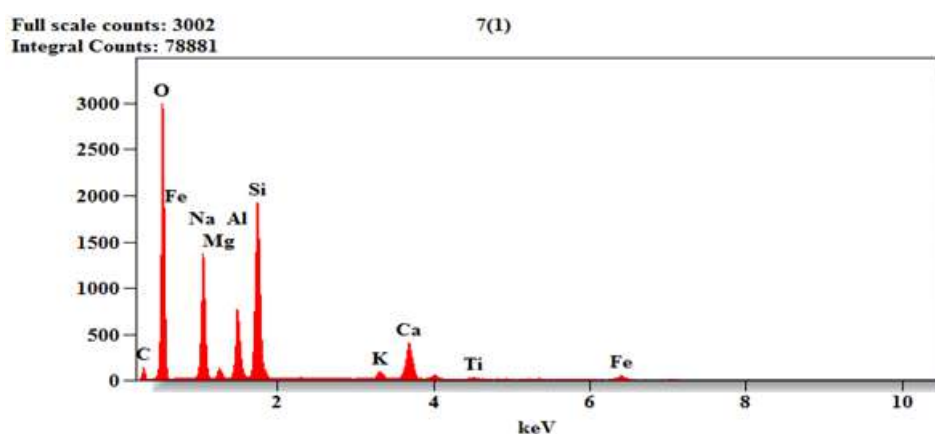
Element	Net counts	Weight (%)	Atom (%)	Atom error (%)	Formula
C	748	3.96	6.36	± 0.29	C
O	16841	52.54	63.31	± 0.48	O
Na	9638	14.30	11.99	± 0.15	Na
Mg	579	0.74	0.59	± 0.09	Mg
Al	5627	5.32	3.80	± 0.10	Al
Si	17382	15.15	10.40	± 0.10	Si
K	854	0.79	0.39	± 0.02	K
Ca	4920	4.93	2.37	± 0.06	Ca
Ti	190	0.27	0.11	± 0.02	Ti
Fe	862	2.00	0.69	± 0.04	Fe
Total		100.00	100.00		



(a)



(b)



(c)

Fig. 6. Hydrated paste with OPC & NS a) image 50 μm scale, b) image at 3 μm shows NS in agglomerated condition, and c) EDX image

4.2. Concrete Trial Mixes with UFBFS

Trial mixes were conducted to determine the dosage of superplasticizer (CONXL PCE 2635) to achieve an initial slump value of about 230 mm. From the trial mix M60-ADMX, as shown in Table 7, the chosen SP at a dosage of 3.2 kg/cum imparts the required workability, resulting in a slump value of 225 mm. In the next step, comparative trial mixes were conducted to determine the optimal UFBFS dosage based on concrete compressive strength. Five M60 Grade mix designs with varying content of UFBFS, i.e., @ 0 kg (M60-UF0), 15 kg (M60-UF15), 20 kg (M60-UF20), 30 kg (M60-UF30) and 40 kg (M60-UF40) were proportioned. The total binder content was kept constant at 570 kg/cum, and with an increase in UFBFS content, the amount of OPC in the mixes was reduced by the same amount, as shown in Table 7. Trial mixes were conducted to measure the workability of the mixes, as shown in Fig. 7, and the corresponding slump values are presented in Table 8. Compressive strength values for the mixes were recorded and presented in Table 8, and the results were analyzed as shown in Fig. 8. UFBFS was used in its “as produced” form along with other binders, namely OPC and fly ash.



Fig. 7. Slump test conducted on varying UFBFS %, and its influence on workability

Cube compressive strength test results show that increase in values are significant for mixes with UFBFS quantities ranging from 20 kg/cum to 30 kg/cum, with 20 kg/cum corresponding to a 5.4% UFBFS-to-OPC ratio and 30 kg/cum corresponding to an 8.3% UFBFS-to-OPC ratio. From 8.30% to 11.40%, the strength increase is marginal. Hence, the optimal replacement efficiency of UFBFS can be considered to fall between 5% and 8%. Another set of trial mixes was conducted for confirmatory purposes. Since optimal strength gain was found to fall between the mixes containing 20 kg and 30 kg UFBFS, a mix containing 25 kg UFBFS per cum was selected. Confirmatory trial mixes were conducted with a nil UFBFS mix (M60-UF0C) and the mix with 25 kg UFBFS (M60-UF25) as given in Table 7. Slump test values and compressive strength test values for these mixes are also shown in Table 8.

As confirmed by Ahmad S et al. [41] in their experimental work on UFBFS, the current study found that slump values increase marginally with increasing UFBFS percentage. An increase in compressive strength is also observed with increasing UFBFS content up to an 8% UFBFS-to-OPC ratio. The contribution of UFBFS to strength gain is highly associated with earlier experimental

findings [41,42]. Due to their extreme fineness, UFBFS particles disperse well and become so effective that their replacement ratio to OPC exceeds 1.0 at an optimal UFBFS/OPC ratio. Based on the trial mixes conducted, it can be concluded that incorporating UFBFS positively influences both the fresh concrete properties and the compressive strength.

Table 7. Mix proportion used for initial experimental works on UFBFS efficiency

Material	Initial	Trial Mixes with varying UFBFS %					Confirmatory Trial Mixes	
	M60-ADMX	M60-UF0	M60-UF15	M60-UF20	M60-UF30	M60-UF40	M60-UF0C	M60-UF25
OPC	390	390	375	370	360	350	390	350
Fly Ash	180	180	180	180	180	180	180	180
UFBFS	0	0	15	20	30	40	0	25
20mm	455	455	455	454	454	454	455	461
12.5mm	453	453	453	453	453	452	453	459
CS Sand	725	725	725	724	724	724	725	735
Water	174	174	174	174	174	174	174	170
Total	2377	2377	2377	2375	2375	2374	2377	2380
PCE 2635	3.2	3.25	3.15	3.00	2.80	2.70	3.2	2.9
W/B	0.31	0.31	0.31	0.31	0.31	0.31	0.31	0.31
Binder Content	570 kg	570 kg	570 kg	570 kg	570 kg	570 kg	570 kg	555 kg
% UFBFS/OPC	0.0%	0.0%	4.0%	5.4%	8.3%	11.4%	0.0%	7.1%

Table 8. Slump values and compressive strength values improve with an increase in UFBFS

Properties	Initial	Trial Mixes with varying UFBFS%					Confirmatory Trial Mixes	
	M60-ADMX	M60-UF0	M60-UF15	M60-UF20	M60-UF30	M60-UF40	M60-UF0C	M60-UF25
% UFBFS/OPC	0.0%	0.0%	4.0%	5.4%	8.3%	11.4%	0.0%	7.1%
Workability test, slump measurement in mm								
Initial slump	225	230	230	225	245	240	230	225
Cube compressive strength test results in MPa								
7 days	54.2	57.2	54.2	55.7	53.3	55.2	52.7	55.6
28 days	69.6	69.0	70.6	72.1	72.8	73.1	68.4	71.5
Increase in 28-day strength over reference mix, %	---	Ref. mix	2.3	4.5	5.5	5.9	Ref. mix	4.5

A second set of trial mixes was conducted for confirmatory purposes to optimize OPC content and measure the influence of UFBFS on compressive strength; the results also show similar findings. In the comparison between a mix without UFBFS (M60-UF0C) and a mix with 25kg/cum UFBFS (M60-UF25), the mix with UFBFS (M60-UF25) showed enhanced workability even when the superplasticizer dosage was reduced by 10%. Strength gain of mix M60-UF25 with 25 kg/cum of UFBFS is calculated to be 4.5% over the reference mix M60-UF0C, even with the reduction of 40kg/cum of OPC. With increased compressive strength in the M60-UF25 mix, it is shown that optimizing OPC and total binder content is possible with the incorporation of UFBFS. This approach serves as the basis for reducing OPC content and increasing aggregate content (to compensate for the mix volume) in the modified SCC mix design that is being experimentally investigated.

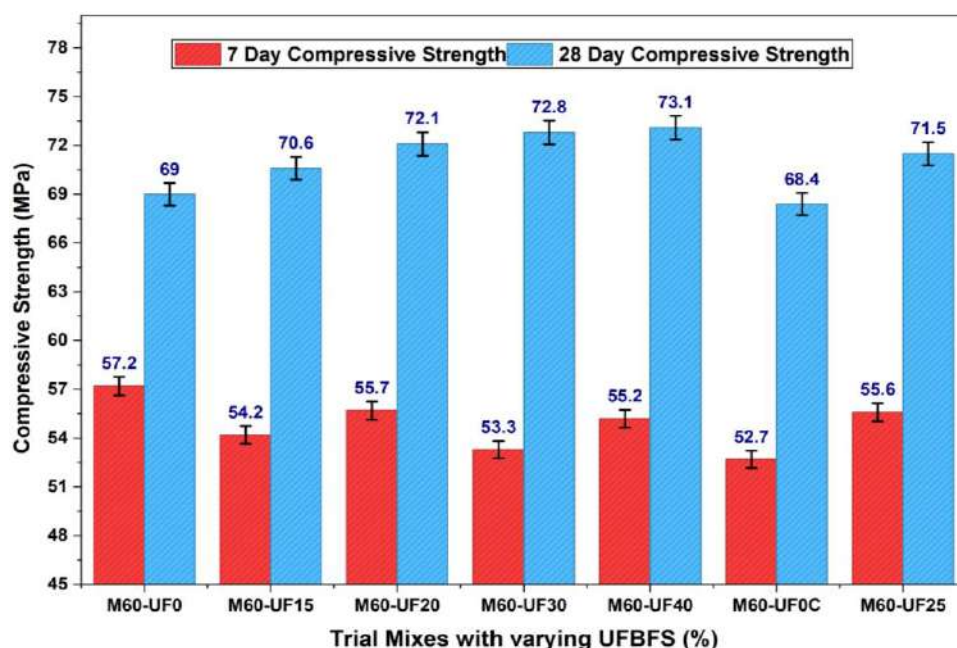


Fig. 8. Influence of UFBFS on 7 & 28 day compressive strength results

5. Experimental Investigations on the Application of UFBFS and NS in M60 SCC

Initial experimental studies conducted so far in this research work, using NS and UFBFS, have provided insight into the influence of these two materials on the behavior of cement slurry and concrete mix, particularly on fresh slurry/concrete properties. While NS increases the viscosity of the cement slurry, UFBFS enhances the workability of the concrete mix. NS, thus, is expected to play a role in making SCC robust in the fresh stage by controlling bleeding and preventing segregation. UFBFS, by enhancing workability, helps reduce water content and superplasticizer dosage. The ability of UFBFS to replace OPC at a ratio greater than 1:1 in terms of compressive strength performance is considered another important factor in reducing binder content and, consequently, meeting the objective of mitigating the probability of shrinkage in SCC mixes. Hence, the incorporation of NS & UFBFS into SCC mixes was found to be efficient, as it supports the aim to optimize binder and aggregate contents by reducing the former and increasing the latter, thereby reducing shrinkage risk. In this experimental work, the conventional self-compacting concrete (SCC-Conv) mix is designed with a combination of OPC and fly ash as binders, whereas the modified self-compacting concrete (SCC-Mod) is designed with a combination of OPC, fly ash, and UFBFS as binders, with NS as the rheology-modifying additive. A PCE-based HRWR superplasticizer was used to achieve the required flow.

5.1. Trial Mixes Comparison on Conventional M60 SCC and Modified M60 SCC with UFBFS And NS

A conventional M60 SCC mix with an OPC and fly ash combination of 390+180 kg/cum (SCC-Conv1) was evaluated first. In the initial trial mix, slight segregation, in the form of aggregate settling, was observed (Fig. 10(a)). Hence, this mix was discarded, and no further testing was performed. To enhance uniformity, the binder content was increased to 400+200 kg/cum, as conventionally done, and a trial mix (SCC-Conv2) was conducted. An increase in total binder content improved the uniformity of the SCC-Conv2 mix, as shown in Fig. 10 (b). The SCC-Conv2 mix met the required flow properties, as shown in Figs. 10(c), 10(d), and 10(e). After finalizing the conventional SCC mix design, a modified SCC (SCC-Mod) mix was developed to reduce the total binder content by combining OPC, fly ash, UFBFS and NS. While designing Modified M60 SCC, the quantities of 30 kg of OPC and 30 kg of fly ash were reduced. OPC was reduced based on the initial trial mixes, indicating that UFBFS offers greater OPC replacement potential, whereas 30 fly ash was reduced, as the dependency on higher binder content is expected to be reduced with the addition of NS. Fig.

9 shows the reduction in binder content and paste content in the SCC-Mod mix. M60 grade mix designs of SCC (Conv1), SCC (Conv2) and SCC (Mod) are given in Table 9.

Table 9. SCC mix designs, conventional and modified

Material	SCC-Conv 1	SCC-Conv 2	SCC-Mod
OPC	390	400	370
Fly ash	180	200	170
UFBFS	0	0	25
12.5mm	821	795	827
CS Sand	803	777	809
Water	167	174	165
Total density	2361	2346	2366
PCE 2635	5.1	5.1	4.8
Nano silica	0	0	1.2
UFBFS / OPC ratio	0	0	6.8%
W/B ratio	0.29	0.29	0.29
Total binder content by weight, kg	570	600	565
Paste content by volume, liters	376.5	396.2	372.1

Table 10. Fresh concrete properties test results

Properties Tested	SCC-Conv2	SCC-Mod
Fresh Concrete Properties		
Slump Flow Initial	660 mm	650 mm
T500, initial	4.5 sec	4.8 sec
V-Flow initial	12 sec	11 sec
J-Ring Flow, initial	620 mm	620 mm
Slump Flow, 1 hour	650 mm	650 mm
T500, 1 hour	4.8 sec	5.0 sec
V-Flow, 1 hour	13 sec	13 sec
J-Ring Flow, 1 hour	620 mm	620 mm

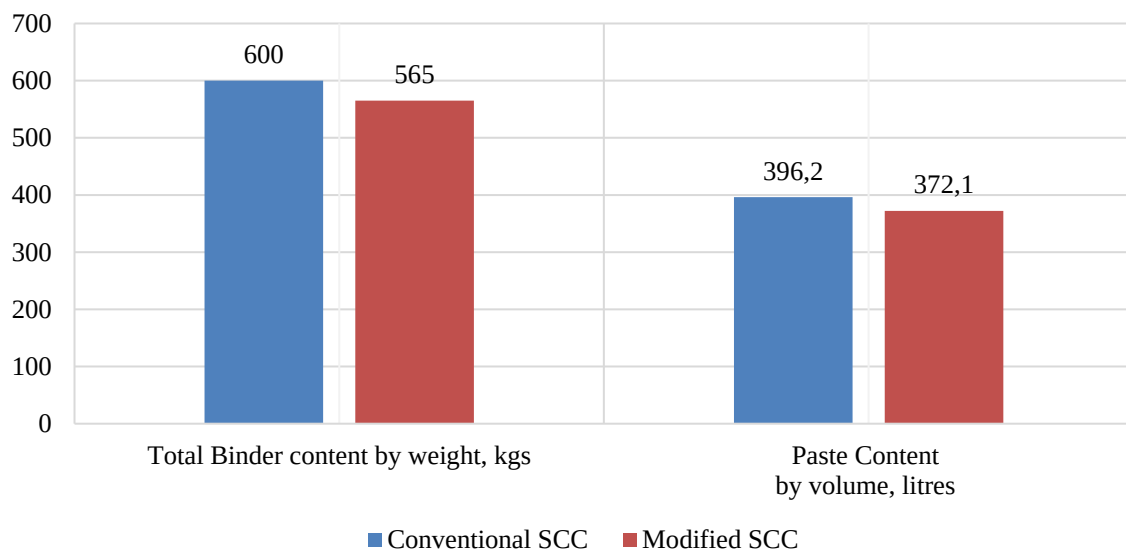


Fig. 9. Reduction in binder and paste contents in Modified SCC mix

The properties of the Modified SCC mix were measured and recorded, as shown in Fig. 11 and Table 10. Samples for testing hardened concrete properties, such as compressive strength, flexural strength, water penetration resistance, rapid chloride penetration resistance, and water

absorption, were cast and tested as shown in Fig. 12. Test results for hardened concrete properties are shown in Table 11. As conventionally practiced, a total binder content of 600 kg/cum is required to achieve uniformity in the SCC M60 mix and to meet the required SCC flow properties. With the incorporation of UFBFS and NS, the modified M60 SCC mix with reduced binder content of 565 kg/cum, is able to meet SCC flow requirements. The ability of NS to impart more viscosity in the flowable SCC mix and to make it more uniform is evident from the trial mix.

SCC-Conv1Mix (390+180)



(a) 12.5 mm settling

SCC-Conv2 Mix (400+200)

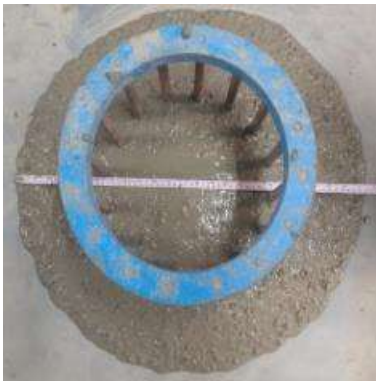


(b) Improved uniformity

Flow Tests on SCC-Conv2



(c) Slum Flow



(d) J-Ring Flow



(e) V-Flow

Fig. 10. Behaviour of SCC mixes: CC-Conv1 (390+180 kg/m³ binder content) showing segregation and SCC-Conv2 (400+200 kg/m³ binder content) exhibiting uniformity

Mix in the mixer machine



(a) SCC-Mod before the addition of NS



(b) SCC-Mod after the addition of NS

Flow Tests – SCC-Mod with NS

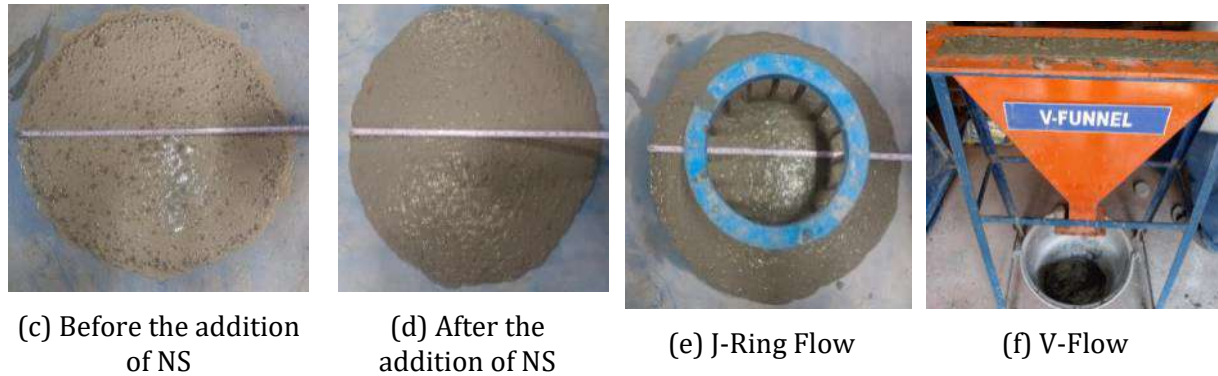


Fig. 11. SCC modified mix proportion comprising 370 kg/m³ OPC + 170 kg/m³ fly ash + 25 kg/m³ UFBFS + 1.2 kg/m³ NS exhibiting improved uniformity and fresh flow properties

5.2. Test Results of Hardened Concrete Properties

Compressive strength and Flexural strength properties of M60 SCC-Mod mix are comparable to those of M60 SCC-Conv2. However, a significant improvement in the durability properties of the SCC-Mod mix has been observed compared to the SCC-Conv2 mix. Water penetration resistance increased by 34%, chloride penetration resistance increased by 40% at 28-day age and by 67% at 56-day age, and water absorption reduced by 12%. As reported in other studies, the addition of UFBFS has enhanced concrete durability.

Table 11. Hardened concrete properties test results

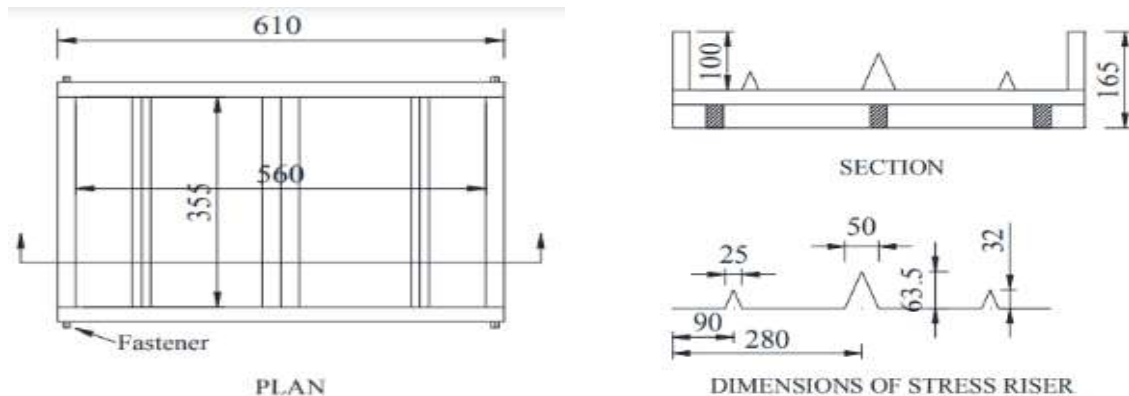
	M60 SCC-Conv2	M60 SCC-Mod
Cube compressive strength		
7 days	52.7 MPa	56.3 MPa
28 days	69.2 MPa	70.3 MPa
Beam flexural strength		
7 days	4.7 MPa	4.9 MPa
28 days	6.9 MPa	6.8 MPa
Water Penetration test results		
28 days	5.0 mm	3.3 mm
RCPT evaluation		
28 days	1341 coulombs	796 coulombs
56 days	986 coulombs	329 coulombs
Water absorption		
28 days	5.0 %	4.4 %



Fig. 12. Tests conducted on the strength and durability properties of SCC mixes

5.3. Plastic Shrinkage Investigation on Conventional SCC and Modified SCC Mixes

A plastic shrinkage study was conducted by casting both M60 SCC-Conv2 and M60 SCC-Mod mixes in a mold with stress risers to induce cracking. The mold set-up was as per ASTM C1579, as shown in Fig. 13. To maintain temperature and humidity, a humidity chamber was used. Temperature was maintained at 40 ± 2 °C, and Humidity at $40 \pm 5\%$ for both test samples. Both SCC-Conv2 and SCC-Mod mixes, as per the designs given in Table 9, were mixed in a laboratory pan mixer and were placed into the mold. Compaction was done with a stainless-steel tamping bar. The surface was finished with a trowel, and the test specimens were placed inside the humidity chamber immediately after finishing. Observations were made every 30 minutes.



(a) Mould with Stress Raisers



(b) Mould used for the study

Fig. 13. Plastic shrinkage mould used in the study

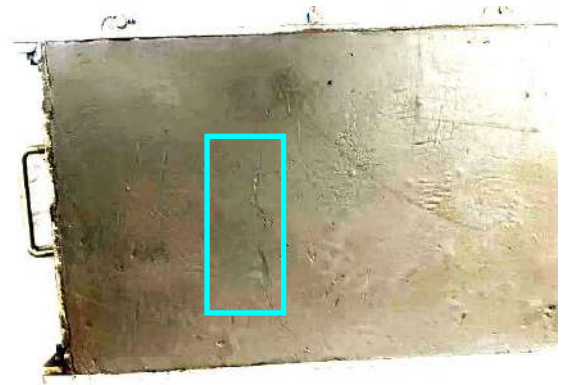
In the specimen cast with SCC-Conv2, a plastic shrinkage crack was observed at 2 hours after casting, as shown in Fig. 14(a). It was observed that the crack propagated slowly. The final appearance of the crack was observed at 12 hours, by which time the concrete had become hard enough to resist the plastic shrinkage crack, as shown in Fig. 14 (d). SCC-Mod observed for 12 hours until the mix hardened, as seen in Fig. 15, did not exhibit any early-age cracking, as the mix was able to withstand the stress induced by the stress risers.

It has been well documented by many researchers, including Jamali A et al. [43], that capillary pressure, with the formation of a meniscus between finer particles, is one of the main reasons for Plastic Shrinkage and subsequent cracking, as concrete is weak in the plastic state. As found in studies on SCC mixes by Bhirud Y et al. [44] and Rozière E et al. [45], mixes with a higher binder content tend to exhibit greater shrinkage. The reason is that a higher binder content makes capillary pores smaller, thereby reducing the meniscus radius and increasing capillary pressure, leading to greater shrinkage. An increase in aggregate content in Modified SCC (SCC-Mod) enhances resistance to shrinkage. A current experimental study on plastic shrinkage also shows that a Modified SCC (SCC-Mod) mix with reduced binder content and increased aggregate content resists

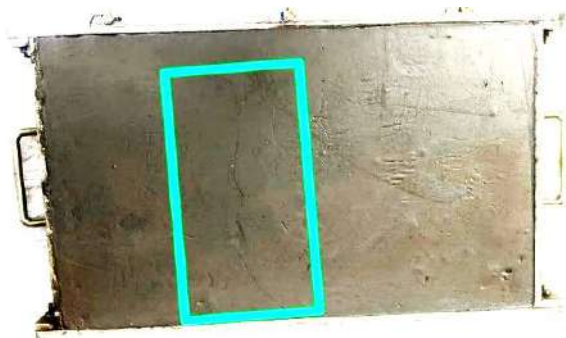
cracking at early ages. In contrast, a Conventional SCC (SCC-Con) mix cracks at early ages. The addition of nanosilica is found to modify the viscosity and contribute to early strength gain in the paste medium. These two properties are considered important factors in mitigating plastic shrinkage behaviour [46,47]. The addition of UFBFS and NS to the SCC-Mod mix has resulted in improved resistance to early-age shrinkage.



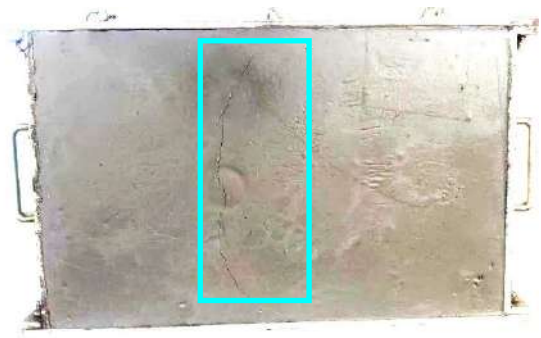
(a) Immediately after casting



(b) Initiation of cracks at 2 hours



(c) After 6 hours



(d) Appearance of Full Crack after 12 hours

Fig. 14. Initiation and propagation of PSC till concrete hardens



(a) Immediately after casting



(b) After 2 hours



(c) At 6th Hour



(d) No Crack after 2 hours

Fig. 15. SCC-Mod mix resisting PSC

5.4. Drying Shrinkage Investigations on Conventional and Modified SCC Mixes

A Drying Shrinkage study was conducted to compare SCC-Conv2 Mix and SCC-Mod Mix as per ASTM C157. Three prism specimens with dimensions of 75 mm x 75 mm x 285 mm were cast as shown in Fig. 16 (a) for each mix and cured for 14 days before being dried in a humidity chamber. A length comparator was used to measure changes in length over a 28-day drying period. The test setup is shown in Fig. 16 (b). Periodic length change values recorded through the 28-day test period are presented in Table 12 and Fig. 17.



(a) Specimens cast in the mould



(b) Length comparator

Fig. 16. Drying shrinkage assessment by measuring length change

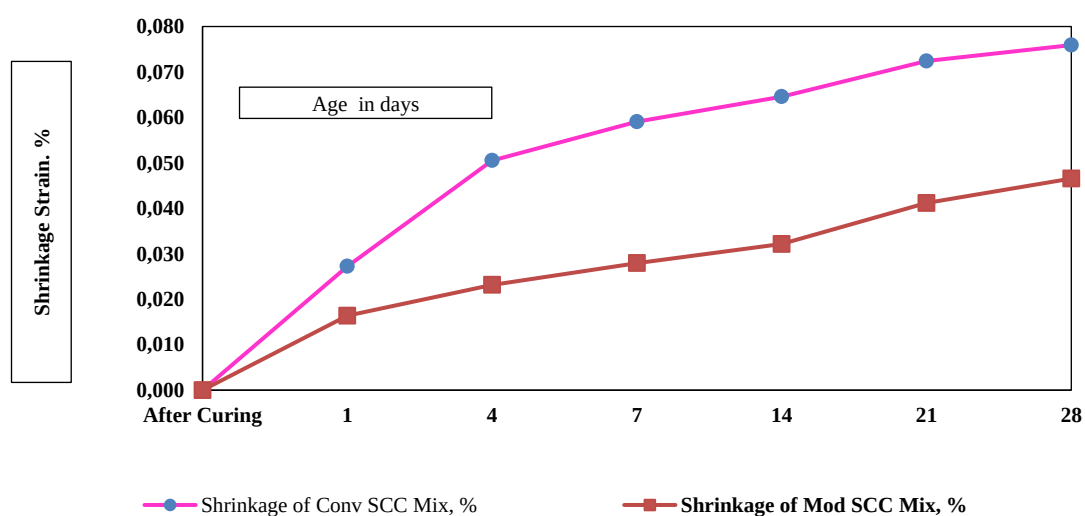


Fig. 17. Length change values over 28 days drying

Table 12. Length change values over 28 days of drying

Age of Testing, day	After Curing	Period of Drying					
		1day	4day	7day	14day	21day	28day
Shrinkage of SCC-Conv2 Mix (%)	0.000	0.027	0.051	0.059	0.065	0.072	0.076
Shrinkage of SCC-Mod Mix (%)	0.000	0.016	0.023	0.028	0.032	0.041	0.047

Drying shrinkage of modified M60 SCC with reduced paste content is lower than that of conventional M60 SCC. It must be noted that a reduction in free water associated with a reduction in binder content results in a reduction in gel-pore water. Drying shrinkage, a result of the loss of gel-pore water as discussed by many researchers [48–50], is thus reduced in SCC-Mod. An increase

in aggregate content in the SCC-Mod mix, compensating for the reduction in paste content, also contributes to reduced drying shrinkage.

5. Results and Discussions

- Conventional M60 SCC (SCC-Conv) mix requires a higher binder content to prevent segregation in the fresh state and to achieve uniformity.
- A modified approach is required to optimize binder content and reduce SCC's shrinkage probability. Incorporating UFBFS and nano silica is considered in this research work following an analysis of their potential through initial experimental work.
- It is evident that the addition of UFBFS increases workability, helping reduce the superplasticizer dosage by 10% and the water content by 4kg/cum, despite an increase in total aggregate content in the modified M60 SCC mix (SCC-Mod).
- The ability of nano silica to enhance the viscosity of the paste medium when added as the last ingredient is evident from experiments conducted on both the slurry and the M60 SCC mix. Similar findings have been reported by many researchers.
- It has been experimentally found that the addition of nano silica imparts greater uniformity in the SCC-Mod mix, demonstrating flowability similar to that of SCC-Conv, even when the total binder content is reduced by 35kg/cum compared with SCC-Conv.
- The SCC-Mod mix that incorporates UFBFS and nano silica shows compressive and flexural strengths similar to those of SCC-Conv, despite reduced binder and paste content.
- In durability tests, a 41% reduction in RCPT value at 28 days of age is observed in the SCC-Mod mix, which increases to 67% at 56 days of age. A 12% reduction in porosity at 28 days of age is also observed in the SCC-Mod mix. Water penetration resistance of SCC-Mod has increased by 34%.
- Plastic shrinkage behavior of the conventional M60 mix is more evident, with a crack emanating from the stress-inducing central section where a raiser is located. The crack initiated as early as 2 hours and propagated into a longer, wider crack within 12 hours after casting. The modified M60 SCC mix resisted cracking, and the specimen remained uncracked until the concrete hardened.
- Drying shrinkage tests show that the Modified M60 SCC mix, with an increased aggregate-to-paste ratio, exhibited 38% less shrinkage than the conventional M60 SCC mix, a significant difference.

6. Conclusions

The primary objective of developing a self-compacting concrete (SCC) mix with reduced shrinkage probability was achieved in this study. Results from the experiments demonstrated that incorporating UFBFS and nano silica (NS) reduced the total binder from 600 kg/m³ to 565 kg/m³, meeting the flow requirements and fresh properties of M60 SCC. The modified M60 SCC mix achieved a similar 28-day compressive strength of around 70 MPa and an equivalent flexural strength compared with conventional M60 SCC. Durability performance increased significantly, with a 41% decrease in RCPT value at 28 days, followed by a 67% decrease at 56 days. An increase of 12% in water penetration resistance and a decrease of 34% in water absorption were also observed at 28 days of age. The modified M60 SCC mix also did not exhibit the plastic shrinkage cracks observed with the conventional M60 SCC mix, and its drying shrinkage was reduced by about 38% compared with the conventional M60 SCC mix. The outcome shows that while UFBFS contributes to increased strength, NS primarily serves as a rheology modifier to achieve a uniform, stable mix, and the addition of these two finer materials enhances the durability and shrinkage resistance of SCC. From these experimental findings, it can be inferred that the combined use of UFBFS and NS could be an effective approach for producing a durable SCC mix with an optimized binder quantity and enhanced shrinkage resistance.

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References

- [1] Rudnicki T. Design of Self-Compacting Concrete with Reduced Cement Content by Aggregate Packing Method. *Materials* 2024;18:4. <https://doi.org/10.3390/ma18010004>.
- [2] Domone PL. Self-compacting concrete: An analysis of 11 years of case studies. *Cem Concr Compos* 2006;28:197–208. <https://doi.org/10.1016/j.cemconcomp.2005.10.003>.
- [3] Das D, Saravanan TJ, Bisht K, Kabeer KISA. A review of fresh properties of self-compacting concrete incorporating sugarcane bagasse ash. *Mater Today Proc* 2022;65:852–9. <https://doi.org/10.1016/j.matpr.2022.03.451>.
- [4] Kavyateja BV, Reddy PN, Kumar CA. Properties of self-compacting concrete modified with ultrafine slag. *Research on Engineering Structures and Materials* 2021. <https://doi.org/10.17515/resm2021.351st1008>.
- [5] Rich D, Glass J, Gibb AGF, Goodier CI, Sander G. Optimising construction with self-compacting concrete. *Proceedings of the Institution of Civil Engineers - Construction Materials* 2017;170:104–14. <https://doi.org/10.1680/coma.14.00025>.
- [6] Schmidt W, Msinjili NS. SCC: a solution to the challenges of concrete construction in Africa concrete technology. *Concrete Trends-Journal of the African Cement and Concrete Industry* 2014:16–20.
- [7] Akinpelu MA, Amao AO, Salman A-SM, Gabriel DS. Sustainable self-compacting concrete: A study on the combined effects of waste glass powder and metakaolin as cement replacements. *Research on Engineering Structures and Materials* 2025. <https://doi.org/10.17515/resm2025-610ma0105rs>.
- [8] Ghaboussi D, Yeh Lee K. Performance Analysis of Self-Compacting Concrete in Earthquake-Resistant Buildings. vol. 16. 2024.
- [9] Okamura H, Ouchi M. Self-Compacting Concrete. *Journal of Advanced Concrete Technology* 2003;1:5–15. <https://doi.org/10.3151/jact.1.5>.
- [10] The European Guidelines for Self-Compacting Concrete Specification, Production and Use “The European Guidelines for Self Compacting Concrete.” 2005.
- [11] Khoman RK, Owaid HM. Development of sustainable high performance self-compacting concrete incorporating natural and waste pozzolanic materials. *Research on Engineering Structures and Materials* 2025. <https://doi.org/10.17515/resm2025-912ma0520rs>.
- [12] Saleh Ahari R, Erdem TK, Ramyar K. Permeability properties of self-consolidating concrete containing various supplementary cementitious materials. *Constr Build Mater* 2015;79:326–36. <https://doi.org/10.1016/j.conbuildmat.2015.01.053>.
- [13] Issa M, Alhassan M, Shabila H, Krozel J. Laboratory performance evaluation of self-consolidating concrete. *Proceedings of the Second North American Conference on the Design and Use of Self Consolidating Concrete and the Fourth International RILEM Symposium on Self Consolidating Concrete*, Chicago: 2005, p. 857–62.
- [14] Aslani F, Nejadi S. Shrinkage behavior of self-compacting concrete. *Journal of Zhejiang University SCIENCE A* 2012;13:407–19. <https://doi.org/10.1631/jzus.A1100340>.
- [15] Şahin HG, Temel M, Mardani A. Determination of optimum VMA utilization dosage in cementitious systems: In terms of rheological and flowability properties. *Mater Today Proc* 2023. <https://doi.org/10.1016/j.matpr.2023.03.550>.
- [16] Khayat KH. Effects of Antiwashout Admixtures on Fresh Concrete Properties. *ACI Struct J* 1995;92:164–71.
- [17] Heniegal AM, Abd El salam Maaty A, Sagwa I. Influence of anti washout admixtures and coarse aggregate types on self-flowing underwater concrete properties. *International Journal of Civil and Structural Engineering* 2016;6:245–62. <https://doi.org/10.6088/ijcser.6022>.
- [18] Gettu R, Shareef NS, Ernest JDK. Evaluation of the robustness of SCC. *The Indian Concrete Journal* 2009;83:13–9.
- [19] García L, Valcuende M, Balasch S, Fernández-LLebrez J. Study of Robustness of Self-Compacting Concretes Made with Low Fines Content. *Journal of Materials in Civil Engineering* 2013;25:497–503. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000609](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000609).
- [20] Łązniewska-Piekarczyk B. Effect of viscosity type modifying admixture on porosity, compressive strength and water penetration of high performance self-compacting concrete. *Constr Build Mater* 2013;48:1035–44. <https://doi.org/10.1016/j.conbuildmat.2013.07.076>.

- [21] Zhu W, Gibbs JC. Use of different limestone and chalk powders in self-compacting concrete. *Cem Concr Res* 2005;35:1457–62. <https://doi.org/10.1016/j.cemconres.2004.07.001>.
- [22] Felekoglu B. Utilisation of high volumes of limestone quarry wastes in concrete industry (self-compacting concrete case). *Resour Conserv Recycl* 2007;51:770–91. <https://doi.org/10.1016/j.resconrec.2006.12.004>.
- [23] Rajendran D, Christopher CG, Muthu MS. Self - compacting concrete made with partial replacement of lime stone and quarry dust powder. *Research on Engineering Structures and Materials* 2022. <https://doi.org/10.17515/resm2022.346ma1001tn>.
- [24] Reiterman P, Jaskulski R, Kubissa W, Holčapek O, Keppert M. Assessment of Rational Design of Self-Compacting Concrete Incorporating Fly Ash and Limestone Powder in Terms of Long-Term Durability. *Materials* 2020;13:2863. <https://doi.org/10.3390/ma13122863>.
- [25] Sharmila P, Dhinakaran G. Compressive strength, porosity and sorptivity of ultra fine slag based high strength concrete. *Constr Build Mater* 2016;120:48–53. <https://doi.org/10.1016/j.conbuildmat.2016.05.090>.
- [26] Roncero J, Gettu R, Agulló L, Vazquez E. Flow behaviour of superplasticised cement pastes: Influence of silica fume. *The Indian Concrete Journal* 2002;31–5.
- [27] Smarzewski P. Influence of silica fume on mechanical and fracture properties of high performance concrete. *Procedia Structural Integrity* 2019;17:5–12. <https://doi.org/10.1016/j.prostr.2019.08.002>.
- [28] Yun K-K, Panov V, Kim S, Han S. Plastic shrinkage cracking properties of high-performance shotcrete with supplementary cementitious materials. *European Journal of Environmental and Civil Engineering* 2023;27:159–73. <https://doi.org/10.1080/19648189.2022.2032834>.
- [29] Yajun J, Cahyadi JH. Effects of densified silica fume on microstructure and compressive strength of blended cement pastes. *Cem Concr Res* 2003;33:1543–8. [https://doi.org/10.1016/S0008-8846\(03\)00100-5](https://doi.org/10.1016/S0008-8846(03)00100-5).
- [30] Diamond S, Sahu S. Densified silica fume: particle sizes and dispersion in concrete. *Mater Struct* 2006;39:849–59. <https://doi.org/10.1617/s11527-006-9087-y>.
- [31] Teng S, Lim TYD, Sabet Divsholi B. Durability and mechanical properties of high strength concrete incorporating ultra fine Ground Granulated Blast-furnace Slag. *Constr Build Mater* 2013;40:875–81. <https://doi.org/10.1016/j.conbuildmat.2012.11.052>.
- [32] Niewiadomski P, Stefaniuk D, Hoła J. Microstructural Analysis of Self-compacting Concrete Modified with the Addition of Nanoparticles. *Procedia Eng* 2017;172:776–83. <https://doi.org/10.1016/j.proeng.2017.02.122>.
- [33] Faez A, Sayari A, Manie S. Mechanical and Rheological Properties of Self-Compacting Concrete Containing Al₂O₃ Nanoparticles and Silica Fume. *Iranian Journal of Science and Technology, Transactions of Civil Engineering* 2020;44:217–27. <https://doi.org/10.1007/s40996-019-00339-y>.
- [34] Quercia G, Hüskén G, Brouwers HJH. Water demand of amorphous nano silica and its impact on the workability of cement paste. *Cem Concr Res* 2012;42:344–57. <https://doi.org/10.1016/j.cemconres.2011.10.008>.
- [35] Du S, Wu J, AlShareedah O, Shi X. Nanotechnology in Cement-Based Materials: A Review of Durability, Modeling, and Advanced Characterization. *Nanomaterials* 2019;9:1213. <https://doi.org/10.3390/nano9091213>.
- [36] Khan K, Ahmad W, Amin MN, Nazar S. Nano-Silica-Modified Concrete: A Bibliographic Analysis and Comprehensive Review of Material Properties. *Nanomaterials* 2022;12:1989. <https://doi.org/10.3390/nano12121989>.
- [37] Agulló L, Toralles-Carbonari B, Gettu R, Aguado A. Fluidity of cement pastes with mineral admixtures and superplasticizer—A study based on the Marsh cone test. *Mater Struct* 1999;32:479–85. <https://doi.org/10.1007/BF02481631>.
- [38] Ghafari E, Costa H, Júlio E, Portugal A, Durães L. The effect of nanosilica addition on flowability, strength and transport properties of ultra high performance concrete. *Mater Des* 2014;59:1–9. <https://doi.org/10.1016/j.matdes.2014.02.051>.
- [39] Lavergne F, Belhadi R, Carriat J, Ben Fraj A. Effect of nano-silica particles on the hydration, the rheology and the strength development of a blended cement paste. *Cem Concr Compos* 2019;95:42–55. <https://doi.org/10.1016/j.cemconcomp.2018.10.007>.
- [40] Zhang X, Hou J, Sun H, Zhang C, Huang J, Wang J, et al. Rheological property and stability of nano-silica modified self-compacting concrete with manufactured sand. *Constr Build Mater* 2023;401:132935. <https://doi.org/10.1016/j.conbuildmat.2023.132935>.
- [41] Ahmad S, Shakeel M, Ahmed S. Use of Ultrafine Slag in High Strength Concrete. vol. 5. 2013.
- [42] Reddy GKG, Ramadoss P. Influence of alccofine incorporation on the mechanical behavior of ultra-high performance concrete (UHPC). *Mater Today Proc* 2020;33:789–97. <https://doi.org/10.1016/j.matpr.2020.06.180>.

- [43] Jamali A, Mendes J, Nagaratnam B, Lim M. A new four stage model of capillary pressure in early age concrete: Insights from high capacity tensiometers. *Cem Concr Res* 2022;161:106955. <https://doi.org/10.1016/j.cemconres.2022.106955>.
- [44] Bhirud Y, Vaidya O, More S, Shaikh A, Pawar A, Pawar L, et al. Shrinkage behaviour of self-compacting concrete with higher binder content: experimental results and predictive equation. *Discov Mater* 2025;5:3. <https://doi.org/10.1007/s43939-025-00184-z>.
- [45] Rozière E, Granger S, Turcry Ph, Loukili A. Influence of paste volume on shrinkage cracking and fracture properties of self-compacting concrete. *Cem Concr Compos* 2007;29:626–36. <https://doi.org/10.1016/j.cemconcomp.2007.03.010>.
- [46] Sun H, Zhang X, Zhao P, Liu D. Effects of Nano-Silica Particle Size on Fresh State Properties of Cement Paste. *KSCE Journal of Civil Engineering* 2021;25:2555–66. <https://doi.org/10.1007/s12205-021-0902-3>.
- [47] Kolawole JT, Combrinck R, Boshoff WP. Plastic cracking behaviour of concrete and its interdependence on rheo-physical properties. *Compos B Eng* 2022;230:109527. <https://doi.org/10.1016/j.compositesb.2021.109527>.
- [48] Mastali M, Kinnunen P, Dalvand A, Mohammadi Firouz R, Illikainen M. Drying shrinkage in alkali-activated binders – A critical review. *Constr Build Mater* 2018;190:533–50. <https://doi.org/10.1016/j.conbuildmat.2018.09.125>.
- [49] Nawa T. Mesoscopic model of drying shrinkage and its application into control of shrinkage cracking. Sixth International Conference on Durability of Concrete Structures, 2019, p. 95–104. <https://doi.org/https://docs.lib.purdue.edu/icdcs/2018/keynote/7>.
- [50] Kanda Y, Kurihara R, Maruyama I. Reduction of drying shrinkage via C-S-H densification: The role of early-age thermal history. *Cem Concr Res* 2026;199:108037. <https://doi.org/10.1016/j.cemconres.2025.108037>.

Sustainable seismic strengthening technique for urban infill walls: An integrated analytic hierarchy process and multi-criteria decision analysis framework for earthquake-prone regions

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Abstract

This study presents an integrated Analytic Hierarchy Process and Multi-Criteria Decision Analysis (AHP-MCDA) framework for selecting the most sustainable seismic strengthening technique for masonry infill walls in earthquake-prone regions, with Aceh, Indonesia as a case study. The AHP-MCDA framework was selected for its ability to manage complex multi-criteria decisions by combining quantitative data, qualitative assessments, and expert judgment, while incorporating consistency checks essential when quantitative data is incomplete. Five criteria (structural performance, economic aspects, durability, ease of application, and availability of analytical models) were weighted through pairwise comparisons by ten local construction experts. Eight techniques were initially identified from the literature; however, two (Bamboo Reinforcement and Polypropylene Fiber Reinforcement) were excluded due to insufficient valid experimental data for standalone applications in reinforced concrete frames with masonry infill walls. The remaining six techniques (shotcrete, ferrocement, TRM, steel strips, FRP, and post-tensioning) were evaluated via structured literature review and cost simulations. Ferrocement achieved the highest MCDA score (0.77), driven by balanced performance: 161% shear capacity increase, low material cost (Rp120,243.73/m² ≈ USD 7.20), superior durability (heat, moisture, and corrosion resistance with service life >5 years), outstanding ease of application using locally available materials, and well-established analytical models. FRP ranked second (0.63) despite superior shear capacity (249% average increase), constrained by limited heat resistance, moisture degradation susceptibility, and low ease of application. Sensitivity analysis across five weighting scenarios confirmed framework robustness, with Ferrocement consistently ranked first or second. This fully re-calibratable framework provides an evidence-based tool for sustainable post-earthquake urban restoration, reducing demolition waste, conserving embodied energy, and supporting energy-saving construction through locally available materials that minimize transportation-related carbon emissions.

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1. Introduction

The inclusion of a dedicated section on Sustainability and Resilience in ACI 318-2025 (Appendix C) signals a paradigm shift in structural engineering: that in addition to a building's load-bearing capacity, the design of new buildings as well as the strengthening process of existing structures should also consider the service life to reduce the environmental impact of building component

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materials, without compromising structural integrity [1]. This shift simultaneously addresses the challenge highlighted by ISO 21930:2017, which underscores the significant environmental impact resulting from construction processes and activities in general [2]. Appendix C specifically emphasizes that material selection and construction methods should be evaluated for their lifecycle environmental footprint, a principle directly applicable to the selection of seismic strengthening techniques for masonry infill walls. However, neither ACI 318-2025 nor ISO 21930:2017 provides a practical framework for selecting the most sustainable strengthening technique for an earthquake-prone region with limited resources. It is this gap that the present research seeks to fill.

Aceh Province experienced a devastating earthquake of magnitude 9.3 on December 26, 2004, the strongest ever recorded in Asia [3], resulting in 227,898 casualties across 14 countries [4]. Another earthquake of magnitude 6.5 struck Pidie Jaya district on December 7, 2016, causing 104 deaths. These events have led the Ministry of Public Works and Housing of the Republic of Indonesia to revise the earthquake zoning map, affecting buildings designed before the current seismic code [5], rendering previously safe buildings unable to meet the updated safety requirements. Consequently, designers must utilize both structural and non-structural building components to increase building capacity and minimize strengthening costs. Among these components, masonry infill walls are often overlooked in structural analysis, despite their significant contribution to lateral load resistance as demonstrated through macro-modelling as an equivalent strut [6]. The significant vulnerability of these infill walls to seismic actions has been further evidenced by recent destructive earthquakes globally, where failure modes such as out-of-plane overturning and diagonal shear cracking were predominantly observed even under moderate ground motions [7]. Strengthening techniques modify this equivalent strut behavior by increasing the effective width, stiffness, and load-carrying capacity of the infill panel, thereby enhancing the overall seismic response of the frame-infill system. Preserving existing buildings through targeted strengthening techniques—rather than complete demolition and reconstruction—reduces construction waste, conserves embodied energy, and supports post-earthquake urban environment restoration.

Various strengthening techniques for infill walls have been explored in the literature, including shotcrete, ferrocement, textile reinforced mortar (TRM), polypropylene fiber reinforcement, bamboo reinforcement, steel strip reinforcement, fiber reinforced polymer (FRP), and post-tensioning. These eight techniques were selected based on two primary considerations: (1) their prevalence in the international literature on infill wall strengthening, as documented in comprehensive review studies [8], and (2) their potential for applicability in the Aceh region, considering local material availability and construction practices. The techniques represent a diverse range of approaches, from conventional cement-based methods to advanced composite materials, enabling a comprehensive comparison of sustainability versus performance trade-offs. However, their effectiveness in the specific conditions of Aceh remains unknown. No previous research has integrated local expert judgment with comprehensive criteria—especially the availability of analytical models—within an AHP-MCDA framework tailored to Aceh.

The criterion of "availability of analytical models" is particularly significant from a sustainability perspective. When engineers lack validated analytical models for a strengthening technique, they often resort to overly conservative designs that use excessive material quantities to compensate for uncertainty. This practice leads to material waste, increased embodied energy, and higher construction costs—all of which contradict sustainability principles. Conversely, techniques with well-established analytical models enable efficient, optimized designs that minimize material usage while ensuring safety, thereby supporting sustainable construction. Therefore, the inclusion of this criterion in the decision framework addresses both engineering reliability and environmental sustainability. Therefore, this study develops an AHP-MCDA framework to determine the optimal strengthening technique for masonry infill walls in earthquake-prone regions. The framework integrates five criteria: structural performance, economic aspects, durability, ease of application, and availability of analytical models, weighted through local expert judgment. Data were obtained from a structured literature review and cost simulations, followed by sensitivity analysis to assess robustness.

This research article provides a scientific, robust, and structured reference framework for determining infill wall strengthening techniques that can be used by stakeholders, academics, and the general public, enabling them to select the optimal strengthening technique. Ultimately, this can reduce major rehabilitation works that have environmental impacts, thereby supporting sustainability, while still maintaining building resilience.

The remainder of this paper is organized as follows: Section 2 provides a detailed description of the eight strengthening techniques evaluated in this study, including their material compositions, application methods, and key performance characteristics. Section 3 presents the comprehensive methodology, covering the literature review protocol, the AHP implementation for criteria weighting, the scoring procedures for each criterion, the MCDA integration, and the sensitivity analysis design. Section 4 reports the findings, including the AHP weights, the evaluation of each technique against the five criteria, the integrated MCDA rankings, and the sensitivity analysis results. Section 5 discusses the main findings, compares them with previous studies, and acknowledges the limitations of the research. Section 6 concludes the paper with key recommendations and directions for future research.

2. Strengthening Techniques for Infill Frames

Over time, researchers around the world began developing various methods to strengthen building elements, increasing the capacity of building structures to carry loads. [8] summarizes the effect of strengthening techniques on buildings, as presented in Table 1, which categorizes retrofitting measures into three main groups: individual element repair and strengthening (Group A), integrity enhancement (Group B), and seismic load reduction (Group C).

This research focuses on wall element strengthening techniques in the individual element repair and strengthening category (Group A). Based on Table 1, four main techniques improve at least three aspects of structural performance (S1: Strength, S2: Stiffness, and D: Ductility): Traditional reinforced plastering, bar addition, external steel reinforcement, and fiber-based reinforcement. Therefore, this study evaluated eight derived techniques. It is worth noting that some of these materials, such as bamboo, are rapidly renewable, while others rely on high-energy industrial products. The selection process must therefore balance structural efficacy with environmental sustainability. Table 1 uses "+" to indicate an increase in the respective property, "-" to indicate a decrease, and blank cells to indicate that the technique has no significant effect or that data is not available for that specific property.

Table 1. The effect of retrofit measures on building properties

	Technique	S1	S2	D	I	Fd	Dd
A	Grout and epoxy injection		+				
	Traditional reinforced plastering	+	+	+		-	+
	Rebar addition	+	+	+		-	+
	External steel reinforcement	+	+	+			
	Leaves connection				+	+	
	Fiber-based reinforcement	+	+			-	+
B	Construction of confining reinforced concrete elements				+		
	Diaphragm stiffening and connections				+		
	Stabilization of curved masonry structures and planar systems				+		
C	Base isolation					+	
	Dampers					+	

*A: Individual element repair and strengthening; B: Integrity enhancement; C: Seismic load reduction; S1: Strength; S2: Stiffness; D: Ductility; I: Irregularity; Fd: Force demand; Dd: Deformation demand

2.1 Traditional Reinforced Plastering

This type of strengthening technique uses conventional concrete mortar reinforced with either reinforcing steel or wire mesh. Traditional reinforced plastering is one of the most widely applicable techniques in the Aceh region due to the local availability of cement, sand, and

reinforcing materials, as well as the familiarity of local construction workers with plastering methods [9].

2.1.1 Shotcrete with Reinforcement

Shotcrete is mortar or concrete that is sprayed pneumatically onto a surface at high velocity [10]. The method of reinforcing walls with shotcrete is generally combined with reinforcing mesh. The material used for this technique is shotcrete concrete with a thickness of 50 mm, wire mesh reinforcement with a diameter of 4.5 mm and spacing of approximately 150 × 150 mm, and an anchor bar diameter Ø10 mm, length 35 cm [11]. When shotcrete with steel reinforcement is applied to unreinforced masonry walls, a significant increase in lateral load capacity and improved energy dissipation response can be observed. Based on experimental studies, the increase in lateral resistance ranges from 144% [12] to 210% [11], with an arithmetic mean of 177% across the reviewed studies (Table 11). However, it should be noted that the behavior of load-bearing unreinforced masonry walls (URM) differs from that of infill walls within reinforced concrete frames; the findings from URM studies [13] provide general performance indications but require careful interpretation when applied to infill wall systems.

The durability of shotcrete is significantly governed by the prevailing environmental conditions and the characteristics of the reinforcement used. With respect to thermal performance, shotcrete incorporating steel fibers exhibits behavior comparable to that of conventional concrete, with a tendency to provide improved insulation against heat transfer. Its compressive strength tends to increase at moderate temperatures, approximately in the range of 60–80 °C, but declines markedly when the temperature exceeds 150 °C [14]. Although steel fibers contribute to restraining the propagation of thermally induced cracks, they do not substantially extend the maximum temperature threshold at which the material can maintain its structural integrity.

Concerning resistance to moisture and corrosion, shotcrete generally demonstrates adequate robustness against extreme climatic variations and certain chemical exposures. However, field observations indicate that the use of steel fibers in persistently humid environments poses a notable risk of corrosion. The volumetric expansion associated with fiber corrosion can lead to cracking within the shotcrete layer, thereby representing a critical limitation for long-term service in wet conditions. Notwithstanding these constraints, shotcrete can achieve a service life exceeding five years when appropriately designed and maintained (Table 14). Regular inspection is advisable to monitor surface cracking in humid environments; however, these measures involve only simple, low-cost remedial actions.

Unlike mechanical retrofitting systems such as external post-tensioning or steel bracing—which require routine bolt tightening, periodic re-tensioning, or replacement of fatigued components—shotcrete is applied as a monolithic layer that integrates with the existing frame. It contains no mechanical joints, moving parts, or detachable sheets that necessitate regular replacement or adjustment. Consequently, the operational maintenance burden associated with this technique is negligible. Furthermore, the technique requires no specialized materials and utilizes only locally available basic constituents (cement, sand, and aggregate), making it particularly suitable for application in regions such as Aceh."

2.1.2 Ferrocement

Reinforced plastering can use a material called ferrocement. Ferrocement is a form of reinforced concrete using closely spaced multiple layers of mesh, small-diameter rods completely infiltrated with mortar, or encapsulated in mortar, or both [15]. The definition of "closely spaced" in ferrocement standards, typically refers to mesh spacing of 10–50 mm. In this study, the ferrocement application evaluated uses a mesh spacing of 25 mm, which falls within this range and is consistent with the reference study [16].

There is extensive research on the use of ferrocement to strengthen walls. The technique has contributed significantly to increasing the shear capacity of masonry walls. Based on the arithmetic mean of available experimental studies—including [17] with a 163% increase, [18] with 125%, [19] with 113%, [20] with 140%, [21] with 187%, [22] with 190%, [23] with 166%, and [24] with 200%—the average shear capacity increase is 161% (Table 11). Even when used for the repair of

portal frames with infill walls, it can improve the capacity of the frame-infill system to levels higher than the original frame [25].

Ferrocement requires 25 mm-thick 1:3 ferrocement mortar, 25 mm-spaced wire mesh with 1 mm wire diameter, threaded bolts, and steel plates for connections. A detailed illustration of the application on a masonry wall can be found in [16]. Ferrocement exhibits thermal conductivity of 0.3-1.5 W/m-K, with increased wire mesh volume fraction decreasing conductivity at low temperatures, making it suitable as a fire protection material [26]. However, compressive and flexural strength decrease dramatically above 400 °C [27]. A decrease in strength of more than 400 °C indicates that this material is heat-resistant. Temperature measurements taken during ferrocement construction depends primarily on conduction resistance. Ferrocement shows exceptional moisture resistance. Ultimate strength and stiffness are not affected by 1000 wet-dry cycles (freshwater and seawater), and first-crack strength actually increases due to continued hydration [28]. Durability studies in marine environments through field exposure tests and accelerated laboratory tests confirm the material's resilience [29]. The permeability of ferrocement is lower than conventional concrete due to fine aggregates, with a 3-4 mm cover sufficient, given that permeability is 10 times lower than that of normal concrete [30].

Corrosion studies demonstrate that silica fume (10%) provides the best protection, while fly ash addition, surface coating, and thicker cover are also effective [31]. Geopolymer-based ferrocement with nano fly ash shows very low chloride ion penetration (542.70 Coulomb) and crack time three times longer than conventional cement mortar [32]. Ferrocement structures in Cuba (Baconao Park) remain in good condition after decades, a 1984 ferrocement swimming pool still functions, and the Nervi ship "La Giuseppa" remains in good condition [33]. Ferrocement possesses a degree of toughness, ductility, durability, strength, and crack resistance that ensures long-term performance [34]. Ferrocement is supported by mature analytical models that enable reliable structural analysis. [18] developed a modelling equation as an equivalent strut for ferrocement-strengthened infill walls. This analytical framework provides engineers with the capability to predict the contribution of ferrocement strengthening in structural analysis, facilitating efficient design. The presence of an equivalent strut model (score 4, Table 16) ensures that ferrocement can be reliably incorporated into seismic design calculations. A key advantage of ferrocement is that it does not require periodic or routine maintenance. Ferrocement is characterized as a "durable waterproof no repairs crack resistance material" that eliminates the need for periodic waterproofing and repairs typically required by reinforced cement concrete structures [35] [36]. This characteristic confirms its top ranking in the 'Application Ease of Use' criterion (Table 15), where it received a perfect score for not requiring regular maintenance.

2.2 Fiber-Based Reinforcement

This type of strengthening technique, using fiber as the reinforcing material, generally requires special cement or epoxy to support the performance of the primary material used. From a sustainability perspective. Notably, epoxy-based systems have a higher carbon footprint compared to cement-based alternatives and present recycling challenges at the end of their service life [37].

2.2.1 Textile Reinforced Mortar (TRM)

A comprehensive state-of-the-art review confirms that Textile Reinforced Mortar (TRM) has emerged as a promising and extensively validated strengthening technique, offering superior fire resistance and substrate compatibility compared to epoxy-based alternatives [38]. Textile Reinforced Mortar has been extensively researched as a wall strengthening technique to increase the structural capacity of buildings [39]. This strengthening technique can improve the shear performance of reinforced elements. The technique requires materials in the form of rendering mortar, textile reinforcement (such as carbon-fiber, glass-fiber, or basalt-fiber mesh), and L-shaped connectors [40]. For the purposes of this study, glass-fiber textile mesh was selected as the representative TRM material for cost analysis and performance evaluation, as it is the most commonly documented type in the literature for infill wall applications and offers a balance between performance and cost.

Based on the arithmetic mean of available experimental studies—[39] with a 156% increase, [41] with 136%, and [40] with 158%—the average shear capacity increase is 150% (Table 11).

TRM can maintain structural properties up to 200 °C, but undergoes significant degradation above 300 °C and total loss of strength at 600 °C. High temperatures can reduce the TRM-to-masonry bond capacity by 35-50%, necessitating thermal protection for glass fibers in fire-prone applications [42]. TRM performs well in humid environments; in fact, outdoor exposure can enhance initial hydration and strength development [43]. Corrosion resistance depends on the fiber type used: PBO and brass-coated steel show excellent resistance in aggressive environments, while basalt and galvanized steel perform poorly; carbon fibers are also highly corrosion-resistant. TRM can achieve a service life exceeding 5 years, supported by aging data up to 2.5 years that demonstrate positive trends when appropriate environmental reduction factors are applied in design calculations [44]. TRM is supported by well-developed analytical models that facilitate reliable structural design. [45] developed a modelling equation as an equivalent strut for TRM-retrofitted infill walls. This analytical framework, validated through experimental studies, enables engineers to accurately predict the structural contribution of TRM strengthening in seismic design. The availability of an equivalent strut model (score 4, Table 16) ensures that TRM can be reliably incorporated into structural analysis.

2.2.2 Polypropylene Fiber Reinforcement

Polypropylene fibres are used as strengthening materials in concrete and mortar matrices. This material effectively improves performance, prevents separation at brick joints, and maintains wall quality, especially flexibility. However, based on available experimental studies [46], this material does not significantly improve the shear capacity of tested specimens when used as a standalone reinforcement technique. It is important to note that while the referenced study [47] reported a 187% increase in shear capacity for specimens using polypropylene fibers, this was achieved in combination with shotcrete and additional steel reinforcement (Ø6 mm bars at 150 mm spacing), representing a hybrid system rather than a standalone technique. For standalone polypropylene fiber reinforcement, the available evidence does not demonstrate significant shear capacity enhancement. Therefore, this technique is excluded from the MCDA analysis due to the absence of valid data for standalone applications in reinforced concrete frames with masonry infill walls.

2.2.3 Fiber reinforced polymer (FRP)

FRP is among the most researched materials for wall-strengthening techniques. The most common types used in structural reinforcement include Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP), Aramid Fibre Reinforced Polymer (AFRP), and Basalt Fibre Reinforced Polymer (BFRP). GFRP and CFRP are the most frequently applied for infill wall strengthening due to their well-documented performance and availability [37]. Based on the arithmetic mean of available experimental studies [48] with 300%, [49] with 135%, [50] with 247%, [51] with 350%, and [52] with 213%—FRP provides an average shear capacity increase of 249% (Table 11). For the purpose of this study, CFRP was selected as the representative FRP material for cost analysis and performance evaluation. CFRP represents the highest performance option among FRP types and is most extensively documented in the literature for infill wall applications, allowing for consistent comparison with other techniques. Existing analytical models still show differences between the analysis of GFRP and CFRP materials, requiring further study [37]. These differences create uncertainty in design calculations, highlighting the importance of the "Availability of Analytical Models" criterion in the AHP-MCDA framework. FRP exhibits complex durability characteristics that vary significantly across environmental conditions. CFRP experiences significant tensile strength reduction (up to 33%) at 200 °C, with bond strength losses of 80-90% at 250 °C [42]. At 250-350 °C, FRP loses approximately 50% of its strength, with total degradation occurring above 300-400 °C [53]. In fire scenarios, FRP loses up to 90% effectiveness at 150 °C, while TRM maintains 55% under similar conditions. Bond strength deterioration reaches 83% at 150 °C for FRP, compared to TRM which retains 85% up to 400 °C [53].

Moisture absorption in GFRP can reach up to 5.56%, with adhesive modulus decreasing linearly with water content [54]. Cyclic wet-dry exposure causes interfacial cracking, with fracture

toughness dropping 70% after 8 weeks of exposure [55]. Bond strength reductions of up to 32% have been observed in high-strength concrete interfaces under moisture conditions [56]. FRP demonstrates excellent performance, particularly CFRP which remains resistant to aggressive environments. FRP wrapping can increase corrosion resistance of steel reinforcement by 300% [57]. CFRP strips have shown tensile strength reductions of 26-38% and shear bond strength reductions of 32-62% over 5 years, with environmental reduction factors (CE) of 0.56-0.69 recommended for 50-year design life [58]. The application of this strengthening technique requires materials such as FRP sheets and epoxy resin [51]. A detailed illustration of the installation can be found in [59]. FRP is supported by comprehensive analytical models that enable reliable structural design. [60] developed a modelling equation as an equivalent strut for FRP-strengthened infill walls. This analytical framework provides engineers with validated equations for predicting the shear capacity contribution of FRP strengthening, enabling efficient and optimized designs. The availability of an equivalent strut model (score 4, Table 16) ensures that FRP can be reliably incorporated into seismic design calculations.

2.3 Bar Addition

This type of strengthening technique uses reinforcing bars to increase the shear and ductile capacity of infill walls and frame-infill systems; the reinforcing elements can be made from conventional steel or natural materials, such as bamboo. The term "Bar Addition" more accurately reflects the range of materials used, as "rebar" typically implies steel reinforcement in engineering terminology. For bamboo reinforcement, the reinforcing elements consist of bamboo strips or rods rather than conventional steel bars.

2.3.1 Bamboo Reinforcement

There are a few references discussing bamboo as a strengthening material, and this material is widely available in the Aceh region, making it worth considering. Studies [61,62] show that bamboo-reinforced brick/masonry walls demonstrate improved performance compared to unreinforced walls. However, it is important to clarify that these studies primarily focused on standalone masonry walls or non-engineered housing, not specifically on reinforced concrete frames with masonry infill walls.

Although effective in independent masonry walls, there is still a lack of quantitative data in the literature regarding its use specifically for strengthening infill walls within reinforced concrete frames. This data gap is a significant limitation, as the interaction between the frame and infill wall, as well as the behavior of bamboo as a reinforcing material under cyclic seismic loading, remains insufficiently documented. Issues regarding the bond (adhesion) of bamboo within concrete or mortar, durability (susceptibility to decay, insect infestation, and moisture absorption), and long-term performance in tropical climates are critical considerations that require further investigation before widespread structural application can be recommended. Consequently, bamboo reinforcement was excluded from the MCDA analysis due to the absence of valid experimental data for standalone applications in RC frames with masonry infill walls. Bamboo reinforcement has limited analytical model availability for infill wall applications. Research [63] developed a material shear equation for bamboo-reinforced concrete panels, but this does not extend to a comprehensive equivalent strut model for infill walls

2.4 External Steel Reinforcement

This type of strengthening technique uses external steel elements to enhance structural performance. The application methods vary depending on the specific technique.

2.4.1 Steel Strip Reinforcement

This technique uses a steel plate or strip attached to the wall surface using mechanical fasteners (screws and anchors) rather than adhesive bonding. Studies by [64] and [65] have demonstrated that the use of steel strip reinforcement effectively improves structural performance. Specifically, [66] reported a shear capacity increase of 214% (Table 11), which serves as the basis for the performance evaluation in this study. The material used for this strengthening technique is a 3-mm-thick steel strip plate, which is fastened with screws at 100 mm spacing on the beam and

column, and an L-shaped profile to link the plate to the foundation. The application is primarily mechanical, utilizing screws and anchors rather than adhesives, which facilitates installation but requires careful detailing to ensure proper load transfer.

The durability of steel strip reinforcement is primarily dependent on the material composition. Stainless steel strips (SAE 304) demonstrate excellent resistance to moisture and corrosion, with proven performance in outdoor applications without additional protection [65]). The stainless-steel material provides high corrosion resistance and has been shown to withstand long-term exposure (>5 years) in outdoor environments. However, the technique shows limitations in heat resistance, which is critical in the tropical climate of Aceh. The absence of comprehensive heat resistance data in the reviewed literature, combined with the inherent thermal conductivity of steel, suggests potential vulnerability to high-temperature exposure. Additionally, regular maintenance may be required to ensure connection integrity and prevent localized corrosion at fastener points.

Steel strip reinforcement has limited analytical model availability for infill wall applications. [67] contains the basic concept of the shear capacity of the material, but has not yet reached the configuration stage as reinforcement for infill walls. The absence of a validated equivalent strut model (score 2 out of 4, Table 16) means that engineers cannot reliably predict the contribution of steel strip strengthening in structural analysis.

2.4.2 Post Tensioning

This strengthening technique works by applying additional stress to increase the capacity of the strengthened element. To improve performance, 12-mm-diameter high-strength post-tensioning steel is required, comprising eight bars distributed on both sides of the wall element, and four 0.4×0.3 m steel support plates totaling four pieces are necessary. These dimensions represent the specific configuration used in the reference study [68] for infill wall strengthening and were adopted for the cost analysis and performance evaluation in this study. The post-tensioning system applies vertical pressure to the wall through the anchor plates and high-strength steel bars, which increases the compressive stress in the masonry infill, enhances the frame-wall interaction, and improves the overall lateral load resistance of the system.

The durability of post-tensioning systems is critically dependent on the quality of corrosion protection. Heat resistance of post-tensioning systems is not extensively discussed in the reviewed literature, representing a potential vulnerability in fire scenarios. Moisture resistance depends on grout quality and duct integrity [69]. Stainless steel strands show excellent corrosion resistance, nearly free from corrosion, while galvanized steel undergoes severe pitting (Evaluation of corrosion resistance of new and upcoming post-tensioning materials). The technique demonstrates resistance to stress-corrosion cracking in alkaline environments (pH 12.5), with failures occurring only in acidic conditions (pH < 3) [70].

Many post-tensioned bridges have operated successfully for over 60 years, and specimens tested under aggressive exposure conditions for up to 8 years have demonstrated durability when proper protection measures are implemented [71]. Post-tensioning technologies with proper grouting and duct systems can achieve long-term durability exceeding 5 years [72]. Post-tensioning has limited analytical model availability for infill wall strengthening applications. [68] established a straightforward concept regarding the performance of materials on infill walls, but this does not provide a comprehensive equivalent strut model. The technique has a score of 3 out of 4 (Table 16), indicating that while there is a clear conceptual understanding of performance, a validated equivalent strut modelling equation is not yet available.

3. Methodology

The AHP-MCDA framework was chosen because it handles complex multi-criteria decisions by combining quantitative, qualitative, and expert-judgment data. Unlike other methods, AHP breaks down the problem hierarchically and checks the consistency of expert judgment—essential when quantitative data is incomplete. Using AHP to get priority weights from local experts, then MCDA to rank alternatives, creates a decision-support model well-suited to Aceh. This structure also

includes sustainability factors like durability and ease of application, which affect the resource use and environmental footprint over the lifecycle of the building intervention

3.1 Research Framework

Based on Figure 1, this research is divided into 4 phases: Literature Review, Analytic Hierarchy Process (AHP), MCDA Framework, and Results.

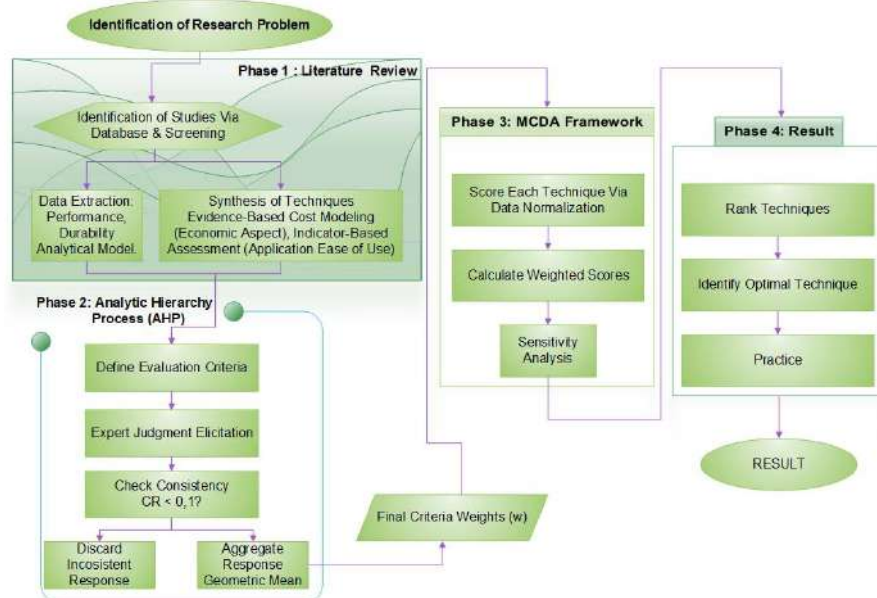


Fig. 1. Flowchart AHP-MCDA framework

3.2 Literature Review Protocol

To ensure a comprehensive and holistic evaluation of the available strengthening techniques, the literature review was not constrained by publication year. This decision was made to address the limited volume of experimental studies available for certain retrofitting materials, particularly when restricted to a narrow recent timeframe. By extending the search horizon to include foundational studies from past decades, the review encompasses the complete historical development, long-term durability assessments, and proven performance of each technique, thereby providing a sufficiently large and robust dataset for the Multi-criteria Decision Analysis. The systematic literature search was primarily conducted using the Scopus and Google Scholar databases, employing specific keywords (detailed in Supplementary Material Tables S1–S6) combined with Boolean operators to maximize relevant article retrieval. Through this unrestricted protocol, a total of 21 studies were identified for the Performance criterion (shear capacity increase), 75 studies for the Durability criterion, and 27 studies for the Availability of Analytical Models criterion, as summarized in Table 2. The detailed inclusion and exclusion criteria, alongside the complete screening process, are provided in the Supplementary Material.

Table 2. Number of studies included from the literature review for each criterion

Criterion	Number of studies
Performance (shear capacity increase)	21
Durability	75
Availability of analytical models	27

3.3 AHP Implementation

The AHP hierarchical structure was developed with four levels based on [73]. The aim was to weight the selection of the best infill strengthening technique for Aceh, which will be used in Phase 3: Multi-Criteria Decision Analysis (MCDA). The hierarchy consists of four levels: Goal (selecting the optimal technique), Criteria (the five main criteria), Sub-criteria (detailed attributes under each criterion), and Alternatives (the eight strengthening techniques).

3.3.1 Determining Selection Criteria and Sub-Criteria

In this phase, the selection criteria for strengthening techniques are developed. From various existing literature, there are several criteria that can be used, including Performance, Economic Aspect, Durability, Application Ease of Use, and Availability of Analytical Model as described in Table 3. The standard [74] implicitly and explicitly requires examination of the shear capacity of these elements, indicating that shear strength is an essential variable in assessing the performance of a building. In the book [75], the contents discuss in depth the mechanism of shear collapse and the importance of capacity design to prevent brittle shear collapse. They emphasize that shear design is a crucial part of the overall seismic-resistance process, indicating that the performance that must be considered in assessing a structure is its shear capacity. Cost is one of the economic aspects that must be reviewed in the assessment of a construction [76]. In [77], cost is a carefully calculated aspect, including materials, tool use, work methods, and worker ability. However, in this case, the review of strengthening technique selection focuses on the cost of materials and equipment, as these are the main factors in determining the unit price of the work.

Most of the damage in concrete structures is not caused by load failure, but by material degradation, thus confirming that durability is not an additional property but a fundamental need [78], even in the standard [79], there is a special chapter on durability criteria. Therefore, in decision-making, durability must be considered, particularly in the Aceh region. Resistance to heat, moisture, and corrosion, a service life of more than 5 years, and the absence of periodic maintenance are crucial factors. Durability directly contributes to sustainable construction by extending the service life of retrofitted buildings and reducing the frequency of repair interventions, thereby lowering long-term material and energy consumption.

Table 3. Sub-Criteria of Criteria Selecting Strengthening Technique

SC \ C	P	EA	D	AEOU	AOAM
1			Resistant to heat	Can be done by one person	Never been researched before
2			Resistant to moisture	No special equipment required	There has never been an equation at all
3	Increase Shear Capacity	Requires the lowest material cost	Resistant to corrosion	No need for special materials	There is a material shear equation
4			Can last more than 5 years	Does not require regular maintenance	There is a straight forward concept regarding the performance of materials on infill walls.
5				Materials available in Aceh	There is a modelling equation as an equivalent strut

*C: Criteria, SC: Sub-Criteria, P: Performance, EA: Economic Aspect, D: Durability, AEOU: Application Ease of Use, AOAM: Availability of Analytical Model

The buildability principle in the book [9] requires that designs priorities materials and methods that are easy to apply, locally available, and do not require complex maintenance to ensure cost and time efficiency. For this reason, in conducting a literature review of articles, it is necessary to consider that the work can be done individually, does not require special equipment or materials, does not require periodic or special maintenance, and that materials are available in the Aceh region. Emphasizing locally sourced materials not only simplifies logistics but also minimizes the carbon footprint associated with transportation, aligning with energy-saving technological principles. In [76] assessment of the existing structure requires detailed analysis, which in turn

necessitates structural analysis. The assessment results necessitate conceptual design, intervention, and risk control. This is not possible if the selected strengthening technique cannot be modelled for structural analysis, which in turn cannot produce a justified conceptual design.

3.3.2 Expert Data Collection

Data collection was conducted with ten construction experts residing in Aceh. Respondents were selected from state officials in charge of conducting building feasibility testing and recommending improvements to state buildings (Aceh Public Housing and Settlement Areas Office), academics/researchers, practitioners from contractors or consultants, and must be domiciled in Aceh. Respondents must have at least 5 years of work experience, as summarized in Table 4.

Table 4. List of experts

No.	Expert Category	Number of Experts
1	Government Officials	4
2	Academics / Lecturers	4
3	Practicing Structural Engineers / Consultants	2
	Total	10

This balanced representation ensures that the AHP weights reflect perspectives from regulation (government), academic theory, and practical field implementation. Each expert was asked to provide pairwise comparisons using a structured elicitation sheet Table 5, and the results were entered into the pairwise comparison matrix as shown in equation (1).

$$A = \begin{bmatrix} 1 & a_{12} & a_{13} & a_{14} & a_{15} \\ 1/a_{12} & 1 & a_{23} & a_{24} & a_{25} \\ 1/a_{13} & 1/a_{23} & 1 & a_{34} & a_{35} \\ 1/a_{14} & 1/a_{24} & 1/a_{34} & 1 & a_{45} \\ 1/a_{15} & 1/a_{25} & 1/a_{35} & 1/a_{45} & 1 \end{bmatrix} \quad (1)$$

Table 5. Definition of pairwise score matrix

Score	Intensity of Interest	Definition
1	(Equal Importance)	Both elements are equally important.
3	(Moderate Importance)	Experience and judgment slightly favor one element over another.
5	(Strong Importance)	Experience and judgment strongly favor one element over another.
7	(Very Strong Importance)	One element is very dominant, and its importance is evident in practice.
9	(Extreme Importance)	Evidence that supports one element over another has the highest level of corroboration possible.

*The even numbers 2, 4, 6, and 8 represent intermediate values between adjacent scale values.

3.3.3 AHP Calculation Procedure

After the experts' data are obtained and compiled into a Pairwise Comparison Matrix, the AHP calculation is performed. All AHP calculations were performed using Microsoft Excel.

- Expert Rating Aggregation (AIJ)

To aggregate the individual judgments of ten experts, the geometric mean of the K expert opinions associated with each cell (AIJ) is calculated using the Aggregating Individual Judgments (AIJ) method using equation (2) [80].

$$a_{ij}^{group} = \left(\prod_{k=1}^K a_{ij}^{(k)} \right)^{1/K} \quad (2)$$

Where a_{ij}^{group} = aggregated pairwise comparison value between criterion i and criterion j $a_{ij}^{(k)}$ = pairwise comparison value provided by expert k; K = number of experts (K = 10).

- Priority Weight Calculation

The priority weight calculation was performed using the Normalized Matrix Method (Row Average);

- Step 1: Normalize each column of the pairwise comparison matrix using equation (3);
- Step 2: Calculate the priority weight using the normalized matrix method (4) ;
- Step 3: Calculate the Maximum Eigenvalue using equation (5) and (6).

$$n_{ij} = \frac{a_{ij}}{\sum_{k=1}^n a_{kj}} \quad (3)$$

$$w_i = \frac{1}{n} \sum_{j=1}^n n_{ij} \quad (4)$$

$$\lambda_i = \frac{(A \cdot w)_i}{w_i} ; i = 1, 2, \dots, n \quad (5)$$

$$\lambda_{max} = \frac{1}{n} \sum_{i=1}^n \lambda_i \quad (6)$$

w_i is priority weight for criterion I; n_{ij} is normalised value in row i, column j, A is original pairwise comparison matrix; w is priority vector (weights); (A·w)I is i-th component of the product of matrix A with vector w; n is number of criteria (n = 5)

- Consistency Test

The Consistency Index (CI) is calculated using equation (7). The Consistency Ratio (CR) is calculated using the following equation (8). The value of RI, obtained from the random index table by [73], which is shown in Table 6.

$$CI = \frac{\lambda_{max} - n}{n - 1} \quad (7)$$

$$CR = \frac{CI}{RI} \quad (8)$$

Table 6. Random Index Value (RI)

n	1	2	3	4	5	6	7	8	9	10
RI	0.00	0.00	0.58	0.90	1.12	1.24	1.32	1.41	1.45	1.49

If $CR \leq 0.10$, the data are consistent and can be used; if $CR > 0.10$, the experts must re-evaluate their pairwise comparisons or replace other experts. In this study, all 10 experts' pairwise comparison matrices met the $CR \leq 0.10$ threshold ($CR = 0.0246$) in the first round; therefore, no data were excluded or re-evaluated. The aggregated pairwise comparison matrix also passed the consistency test

3.4 Scoring the Criteria

Before integrating with the MCDA framework, the sub-criteria for each assessed criterion must first be defined, and scores must be assigned to the criteria to enable proper quantification of the assessment.

3.4.1 Criteria Based on Literature Review

The performance of each strengthening technique was evaluated based on its shear strength improvement. For each technique, the arithmetic mean of shear capacity increases was calculated from all available experimental studies identified in the structured literature review, rather than relying solely on the maximum reported value, to avoid optimistic bias. These mean values were then directly used in the initial MCDA decision matrix. Durability was assessed using evidence from the sub-criteria listed in the durability column of Table 3 (resistant to heat, resistant to moisture, resistant to corrosion, can last more than 5 years). Scoring was based on the number of sub criteria satisfied by each strengthening technique, as defined in Table 7.

Table 7. Scoring of durability criteria

Sub-Criteria	Scoring
Resistant to heat	$\sum [\text{Sub} - \text{Criteria}]$
Resistant to moisture	
Resistant to corrosion	
Can last more than 5 years	

The availability of analytical models' criterion was evaluated by checking whether the reviewed articles for each strengthening technique satisfied any of the sub-criteria listed in Table 8. The highest level fulfilled determined the score.

Table 8. Scoring of availability of analytical models

Score	Sub-Criteria
1	Limited literature and lack of equations (combined from previously separate categories)
2	There is a material shear equation
3	There is a straightforward concept regarding the performance of materials on infill walls
4	There is a modelling equation as an equivalent strut

3.4.2 Criteria based on Market Survey and Practical Assessment

Criteria for which quantitative data were unavailable from the literature review were evaluated through cost simulations and practical applicability assessments. The assessed aspects are described below. The economic aspect was assessed by estimating the material cost of applying each strengthening technique per 1 m² of wall area. The unit price was calculated using the material components specified in the same article selected for the performance evaluation. Because some materials are not locally available, all prices were surveyed from online marketplaces, including shipping costs to the Construction and Building Materials Laboratory (LKBB) in Aceh Province, to ensure consistent treatment. The raw cost data (in Rp/m²) were entered directly into the initial MCDA decision matrix. The Normalization of this cost criterion is handled exclusively by the Linear Min-Max Normalization formula for cost criteria using equation (11), which automatically assigns the highest score (1.00) to the cheapest alternative and the lowest score (0.00) to the most expensive one. The application ease of Use criterion was assessed based on the sub-criteria listed in Table 9. The score for each strengthening technique equaled the number of sub criteria it satisfied.

Table 9. Scoring of application ease of use

Sub-Criteria	Scoring
Requires a specialised crew	$\sum [\text{Sub} - \text{Criteria}]$
No special equipment required	
No need for special materials	
Does not require regular maintenance	
Materials available in Aceh	

These five criteria — structural performance, economic aspect, durability, application ease of use, and availability of analytical models — were identified as the most critical determinants for the successful selection and implementation of infill wall strengthening techniques in Aceh, based on an in-depth review of international literature and civil engineering standards, and were comprehensively assessed by local Acehese experts.

3.5 MCDA Integration

After completing the Analytic Hierarchy Process (AHP), the weighted scores will be used to calibrate the results in the multi-criteria decision analysis. The Linear Max-Min Normalization method was selected because it preserves the relative distances between alternative values and is straightforward for mixed criteria (including both benefit and cost types), making it highly suitable for our framework

3.5.1 Initial Decision Matrix (X)

Results of scoring criteria, based on sub-criteria, are plotted in the initial decision matrix as follows.

$$X = \begin{bmatrix} x_{11} & x_{12} & x_{13} & x_{14} & x_{15} \\ x_{12} & x_{22} & x_{23} & x_{24} & x_{25} \\ x_{13} & x_{32} & x_{33} & x_{34} & x_{35} \\ x_{14} & x_{42} & x_{43} & x_{44} & x_{45} \\ x_{15} & x_{52} & x_{53} & x_{54} & x_{55} \\ x_{16} & x_{62} & x_{63} & x_{64} & x_{65} \end{bmatrix} \quad (9)$$

where: Rows ($i = 1, 2, \dots, 6$) represent the six strengthening techniques evaluated in this study; Columns ($j = 1, 2, \dots, 5$) represent the five evaluation criteria.

3.5.2 Normalization of Decision Matrix (R)

To normalize the initial decision matrix, the Linear Max-Min Normalization is used to normalize the value r_{ij} by using this equation [81]. For benefit criteria (where higher values are better, such as Performance, Durability, AEOU, and AOAM), equation (10) is used. For the cost criterion (Economic Aspect, where lower values are better), equation (11) is used. Linear Max-Min Normalization preserves the relative distances between alternatives and is straightforward for mixed criteria (benefit and cost), making it suitable for our MCDA framework.

$$r_{ij} = \frac{x_{ij} - \min_i(x_{ij})}{\max_i(x_{ij}) - \min_i(x_{ij})} \quad (10)$$

$$r_{ij} = \frac{\max_i(x_{ij}) - x_{ij}}{\max_i(x_{ij}) - \min_i(x_{ij})} \quad (11)$$

3.5.3 Weighted Matrix (V)

At this stage, the normalization matrix is multiplied by the weights obtained from the AHP process using equation (12) where w_j are the AHP weights and must satisfy equation (13). The final score for each alternative is then calculated using equation (14).

$$V = v_{ij} = w_j \cdot r_{ij} \quad (12)$$

$$\sum_{j=1}^n w_j = 1 \quad (13)$$

$$S_i = \sum_{j=1}^n v_{ij} \quad (14)$$

3.5.4 Aggregation Final Result

Since the normalized matrix has been obtained, the aggregation is performed by summing the weighted normalized values for each alternative. Techniques are then ranked according to these final scores.

3.5.5 Sensitivity Analysis

To ensure the generated data is robust, a sensitivity analysis is conducted by simulating five scenarios in Table 10. The scenarios represent different decision-making priorities: BASE reflects the original expert weights; AS.1 emphasizes technical aspects (Performance and Durability); AS.2 emphasizes economics and ease of application; AS.3 emphasizes the availability of analytical models; AS.4 is a control scenario with equal weights (independent of expert opinion) to test model stability; and AS.5 heavily prioritizes ease of application (40%) to represent an extreme case where practical constructability is the overriding concern.

Table 10. Scenario of sensitivity analysis

Scenario	P	EA	D	AEOU	AOAM
BASE	25.51%	24.81%	19.19%	12.89%	17.60%
AS.1	30.00%	21.50%	24.00%	11.20%	13.30%
AS.2	20.00%	30.00%	16.50%	18.00%	15.50%
AS.3	22.00%	22.00%	17.00%	11.00%	28.00%
AS.4	20.00%	20.00%	20.00%	20.00%	20.00%
AS.5	15.00%	20.00%	15.00%	40.00%	10.00%

*P: Performance, EA: Economic Aspect, D: Durability, AEOU: Application Ease of Use, AOAM: Availability of Analytical Model.

4. Result

The findings are organized sequentially, beginning with the criteria weights obtained through the analytical hierarchy process. The individual performance of each strengthening technique is then assessed against the five evaluation criteria, leading to an integrated MCDA ranking. A sensitivity analysis confirms the stability of the final recommendation under varied weighting scenarios.

4.1 AHP Results: Criteria Weighting

The Analytic Hierarchy Process (AHP) successfully produced a prioritized set of criteria weights based on assessments by 10 local experts in Aceh as mentioned in Table 4. These results provide a quantitative foundation for determining the relative importance of each criterion in selecting strengthening techniques.

4.2 Pairwise Comparison and Consistency Ratio (CR)

Based on the results of expert data collection, the average pairwise comparison matrix is obtained for all respondents using equation (2) as shown in matrix (15).

$$A = \begin{bmatrix} 1 & 1 & 1.92 & 1.73 & 1.25 \\ 1 & 1 & 1.58 & 2.18 & 1.07 \\ 0.52 & 0.63 & 1 & 2.19 & 1.25 \\ 0.58 & 0.46 & 0.46 & 1 & 1 \\ 0.80 & 0.93 & 0.8 & 1 & 1 \end{bmatrix} \quad (15)$$

Matrix Normalization was performed according to equation (3). The resulting λ_{max} value is 5.11. The CI value obtained is 0.0275. Because the number of criteria is 5, the RI value from Table 6 is 1.12. Consequently, the CR is 0.0246, far below the threshold of 0.10, indicating that the pairwise comparison assessment of experts has a very high level of consistency. This shows that the responses are not random but logical and consistent, hence reliable. Hence, the criteria weights generated through the AHP process are valid and reliable for use in further decision-making analysis.

4.2.1 Criteria Prioritization

Based on the assessments of ten local experts in Aceh, the Analytic Hierarchy Process (AHP) produced the following priority weights (Figure 2): Performance (25.51%), Economic Aspect (24.81%), Durability (19.19%), Availability of Analytical Models (17.60%), and Application Ease of Use (12.89%). The aggregated pairwise comparison matrix showed high consistency ($CR = 0.0246 < 0.10$). This weight distribution reflects the local priorities where structural performance and cost considerations are the most critical factors; the significant weight given to analytical model availability also underscores the importance of a robust modelling foundation in the design and evaluation of strengthening techniques.

The close weighting between Performance (25.51%) and Economic Aspect (24.81%)—a difference of only 0.7%—demonstrates that experts in Aceh attach as much importance to cost constraints as to seismic safety. This near-equal weighting justifies the study's preference for "low-cost yet effective" solutions such as Ferrocement, which balances technical performance with economic feasibility. The substantial weight attributed to the availability of analytical models (17.60%) reflects the importance experts place on "calculable and reliable" design. This high weight also relates to professional engineering accountability, as engineers require validated analytical models to justify their designs to authorities and insurers.

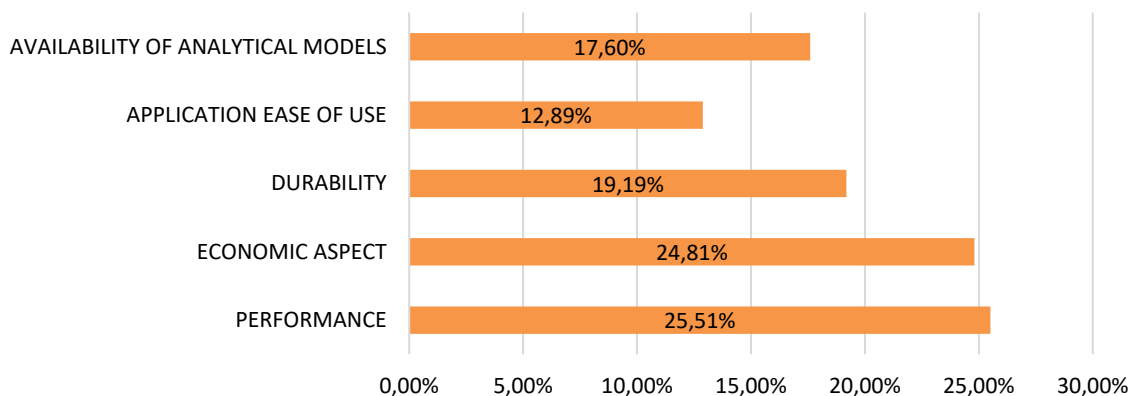


Fig. 2. Result of analytical hierarchy process

4.3 Assessment of Strengthening Techniques Based on Criteria

The assessment of the criteria was carried out according to the procedure outlined in the methodology section. The results obtained are described as follows.

4.3.1 Performance

Performance data were obtained from a structured literature review using arithmetic mean values of shear capacity increases from all available experimental studies, rather than relying solely on maximum reported values, to avoid optimistic bias. The results are summarized in Table 11.

Table 11. Performance of strengthening techniques based on literature review and MCDA evaluation

Strengthening Technique	Avg. Shear Capacity Increase (%)
Shotcrete with Reinforcement	177%
Ferrocement	161%
Textile Reinforced Mortar (TRM)	150%
Steel Strip Reinforcement	214%
Fiber Reinforced Polymer (FRP)	249%
Post-Tensioning	160%

**Polypropylene Fiber Reinforcement and Bamboo Reinforcement are excluded from the MCDA analysis due to insufficient valid data for standalone applications in reinforced concrete infill walls (see Section 5.3 for discussion). The shear capacity values represent the arithmetic mean of all available experimental studies identified in the structured literature review.*

The FRP technique shows the most significant increase in shear capacity, followed by steel strip reinforcement, shotcrete with reinforcement, ferrocement, post-tensioning, and TRM. Data on the improvement of bamboo and polypropylene as standalone techniques could not be obtained because their use in reinforced concrete frames with infill walls was not found using the keywords described in the methodology section. Figure 3 shows the radar diagrams of the performance capacities of the strengthening techniques reviewed.

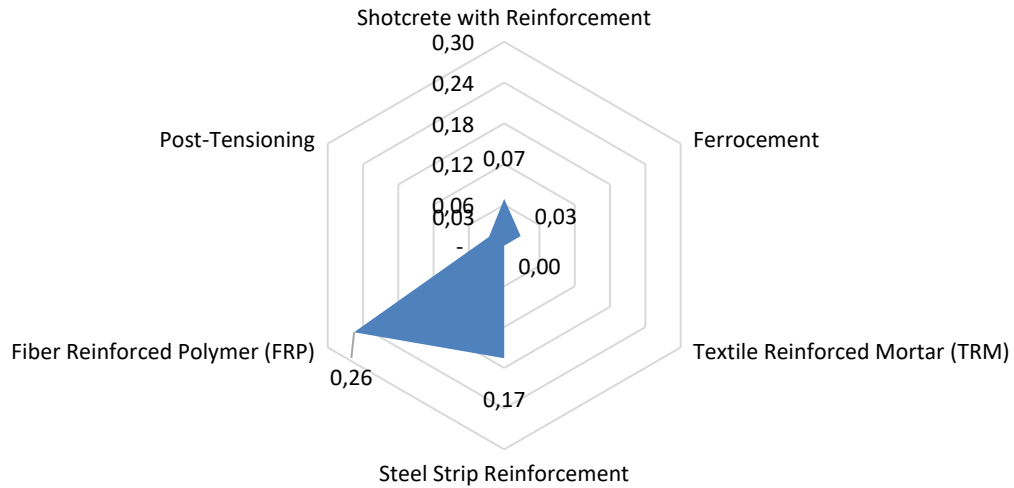


Fig. 3. Diagram of strengthening technique's performance

4.3.2 Economic Aspect

The calculation of material costs per meter was based on an online price survey, including shipping costs to the LKBB at Syiah Kuala University. This was done to ensure equal treatment of all reinforced engineering material components under review. Table 12 includes equivalent values in US Dollars (\$) at an exchange rate of Rp16,691.13 per USD (as of the survey date). Table 12 indicates that Textile Reinforced Mortar (TRM) has the highest cost (Rp3,650,718.41 per m²) and therefore receives the poorest score, resulting in a final score of 0.00. Conversely, Ferrocement is among the most economical techniques (Rp120,243.73 per m²) with the best score and a final score of 0.25. The other techniques have relatively lower costs than TRM and receive favorable economic scores. This indicates that, from an economic perspective, conventional techniques such as Ferrocement are more economically competitive, while TRM and FRP tend to be less affordable within construction budgets in Aceh. The radar diagram for the economic aspect is shown in Figure 4.

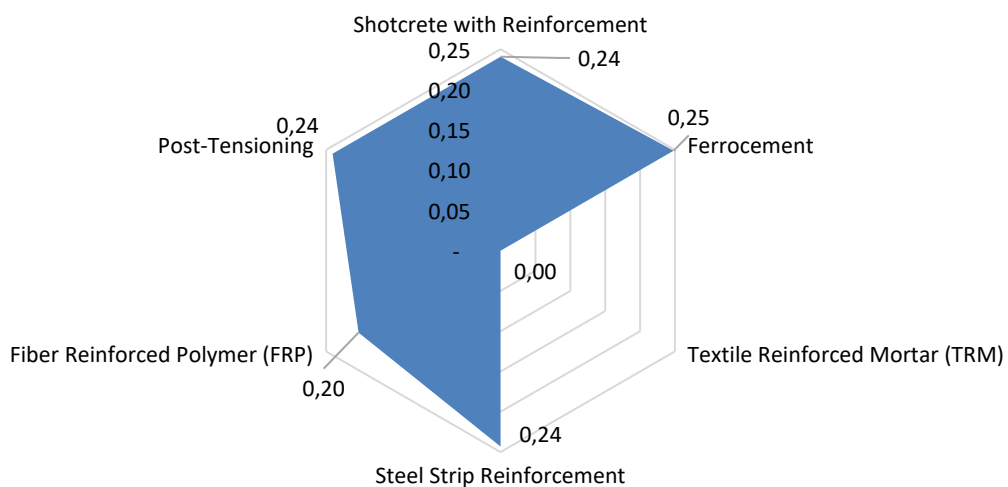


Fig. 4. Diagram of economic aspects of strengthening techniques

Table 12. Economic aspect assessment results

Strengthening Technique	Cost Estimate (Rp/M ²)	Cost Estimate (USD/m ²)
Shotcrete with Reinforcement	Rp239,129.09	USD 14.33
Ferrocement	Rp120,243.73	USD 7.20
Textile Reinforced Mortar (TRM)	Rp3,650,718.41	USD 218.72
Steel Strip Reinforcement	Rp204,941.47	USD 12.28
Fiber Reinforced Polymer (FRP)	Rp760,960.40	USD 45.59
Post-Tensioning	Rp225,935.00	USD 13.54

4.3.3 Durability

The durability assessment was carried out as part of a literature review; the results are shown in Table 13. The following results were obtained from the literature review to assess durability. The results of the durability criteria assessment, summarized in Table 13 and detailed in Table 14, showed significant variations in durability among the various strengthening techniques. Ferrocement and TRM exhibited the highest durability scores (4 out of 4), demonstrating resistance to heat, moisture, and corrosion, as well as a service life exceeding 5 years. In contrast, Shotcrete with Reinforcement was assigned a score of 2 out of 4, reflecting its limited heat resistance (compressive strength declines markedly above 150 °C) and its notable corrosion risk in persistently humid environments due to steel fiber degradation. FRP also scored 2 out of 4, as its heat vulnerability (up to 90% effectiveness loss at 150 °C) and significant moisture absorption (5.56% in GFRP, with 70% fracture toughness reduction) preclude it from being classified as fully durable. Steel Strip Reinforcement and Post-Tensioning both scored 3 out of 4, lacking comprehensive heat resistance data despite excellent performance in moisture and corrosion resistance. These findings highlight the importance of considering local environmental conditions—particularly Aceh's tropical humid climate—when selecting sustainable strengthening techniques. Scoring was based on the number of sub-criteria satisfied (Table 7); the radar diagram for this criterion is shown in Figure 5.

Table 13. Summary of durability from Literature Review

Strengthening Technique	Σ[Sub – Criteria]
Shotcrete with Reinforcement	2
Ferrocement	4
Textile Reinforced Mortar (TRM)	4
Steel Strip Reinforcement	3
Fiber Reinforced Polymer (FRP)	2
Post-Tensioning	3

Table 14. Assessment results of the durability criterion through literature review

Strengthening Technique	Resistant to heat	Resistant to moisture	Resistant to corrosion	Can last more than 5 years
Shotcrete with Reinforcement	-	✓	-	✓
Ferrocement	✓	✓	✓	✓
Textile Reinforced Mortar (TRM)	✓	✓	✓	✓
Steel Strip Reinforcement	-	✓	✓	✓
Fiber Reinforced Polymer (FRP)	-	-	✓	✓
Post-Tensioning	-	✓	✓	✓

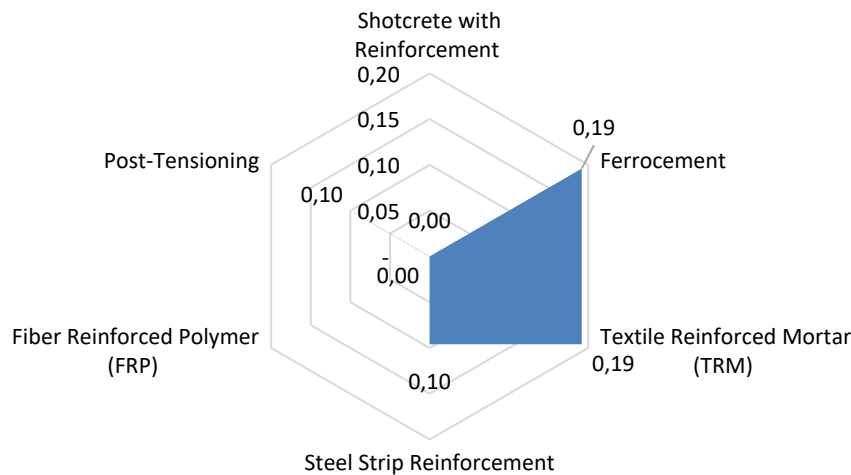


Fig. 5. Strengthening technique durability diagram

The durability assessment, as summarized in Table 14, indicates that while Ferrocement, TRM, Steel Strip Reinforcement, and Post-Tensioning demonstrate resistance to both moisture and corrosion, Shotcrete with Reinforcement exhibits notable corrosion vulnerability in humid environments, and FRP shows significant susceptibility to moisture degradation. While all six techniques demonstrate the ability to last more than five years, heat resistance emerges as the primary distinguishing factor. Ferrocement and Textile Reinforced Mortar (TRM) exhibit robust performance across all four durability sub-criteria. In contrast, Shotcrete with Reinforcement shows limitations in both heat resistance and corrosion resistance. Steel Strip Reinforcement and Post-Tensioning show limitations only in heat resistance, while fulfilling the other durability requirements.

4.3.4 Application Ease of Use

Based on the assessment of the Ease of Application criterion, as seen in Figure 6, Ferrocement achieved the highest score as it met almost all sub-criteria: it requires only a specialized crew rather than a single person, does not require specialized equipment, uses materials available in Aceh, and does not require complex routine maintenance. A complete overview of how the criteria is met can be seen in Table 15. In contrast, techniques such as Post-Tensioning and Fiber Reinforced Polymer (FRP) scored lower due to their reliance on specialized equipment, limited availability of locally sourced materials, and higher technical skill requirements. These results confirm that suitability to the local context, in terms of material resources, labor, and facilities, is a determining factor in the successful implementation of strengthening techniques in Aceh.

Table 15. Assessment results for the application ease of use criterion

Strengthening Technique	Requires a specialized crew	No special equipment required	No need for special materials	Does not require regular maintenance	Materials available in Aceh
Shotcrete with Reinforcement	-	-	✓	✓	✓
Ferrocement	✓	✓	✓	✓	✓
Textile Reinforced Mortar (TRM)	✓	✓	-	✓	-
Steel Strip Reinforcement	✓	✓	-	✓	✓
Fiber Reinforced Polymer (FRP)	✓	✓	-	✓	-
Post-Tensioning	✓	✓	✓	-	✓

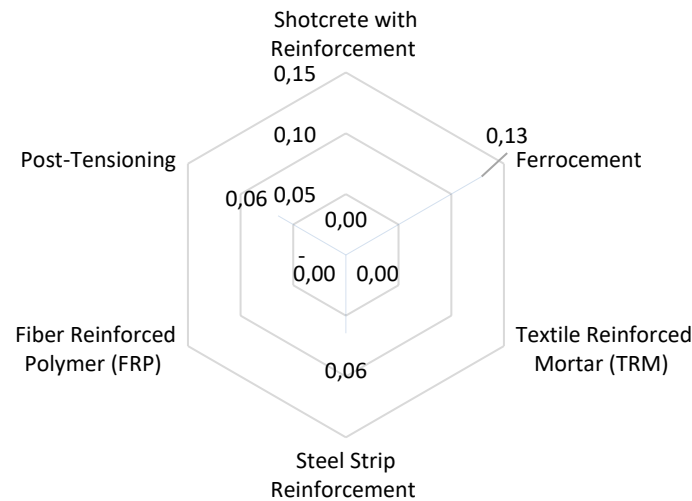


Fig. 6. Diagram of ease of application of strengthening techniques

4.3.5 Availability of Analytical Models

To assess the availability of analytical models for each strengthening technique, a dedicated literature review was performed, and the resulting scores for this criterion are presented in Table 16.

Table 16. Recapitulation of analytical model availability

Strengthening Technique	$\Sigma[\text{Sub} - \text{Criteria}]$
Shotcrete with Reinforcement	4
Ferrocement	4
Textile Reinforced Mortar (TRM)	4
Steel Strip Reinforcement	2
Fiber Reinforced Polymer (FRP)	4
Post-Tensioning	3

* Score definitions: 1 = Limited literature and lack of equations; 2 = There is a material shear equation; 3 = There is a straightforward concept regarding the performance of materials on infill walls; 4 = There is a modelling equation as an equivalent strut.

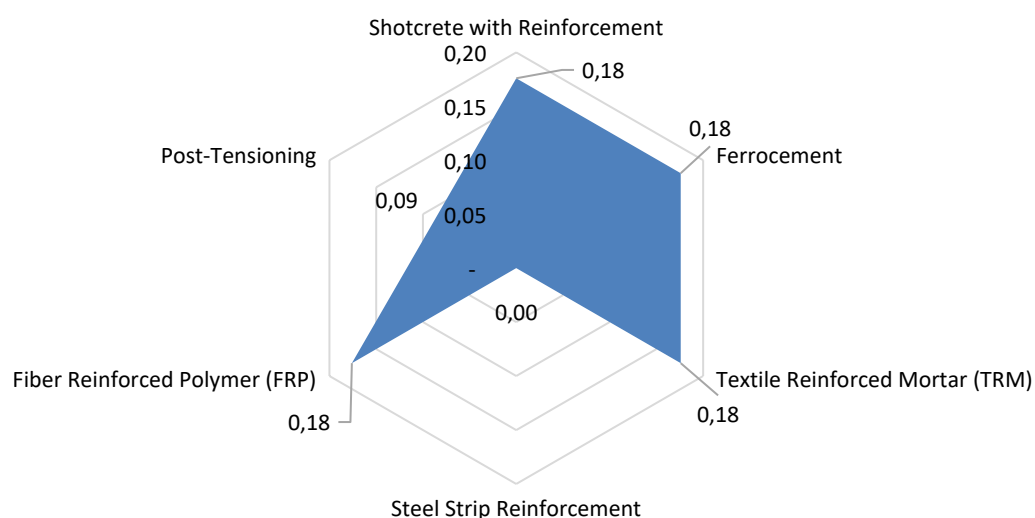


Fig. 7. Availability diagram of analytical model of strengthening technique

Based on the assessment of analytical model availability, the majority of strengthening techniques—such as Shotcrete with Reinforcement, Ferrocement, Textile Reinforced Mortar (TRM), and Fiber Reinforced Polymer (FRP)—show high model maturity, with a maximum score

of 4 (MCDA value of 0.18). This indicates that these techniques are supported by modelling equations as equivalent struts, enabling reliable structural analysis for earthquake resistance planning. Steel Strip Reinforcement (score 2) lacks a comprehensive analytical model, so its application requires further experimental studies. The inability to perform reliable design calculations due to the absence of validated models presents a significant engineering risk. These results confirm that the availability of comprehensive analytical models is a critical factor in assessing the feasibility of strengthening techniques for implementation in seismic regions such as Aceh. The diagram for this criterion is shown in Figure 7. The availability of analytical models serves as a guide for engineers in designing structures that use strengthening techniques; this is certainly a critical aspect that must be considered in studies.

4.4 AHP-MCDA Integration Results

The assessment was carried out, followed by the integration of the weights obtained from the AHP process using Equation (12). The subjective expert opinions (weights) from Phase 2 were combined with the objective literature and survey data (performance, cost, durability, etc.) from Phase 3 through the weighted matrix multiplication.

4.4.1 Decision Matrix and Normalization

Based on the assessment for each criterion, the scores are entered into the decision matrix, as shown below:

$$x = \begin{bmatrix} 177\% & \text{Rp. 239,129.09} & 2 & 3 & 4 \\ 161\% & \text{Rp. 120,243.73} & 4 & 5 & 4 \\ 150\% & \text{Rp. 3,650,718.00} & 4 & 3 & 4 \\ 214\% & \text{Rp. 204,941.47} & 3 & 4 & 2 \\ 249\% & \text{Rp. 760,960.00} & 2 & 3 & 4 \\ 160\% & \text{Rp. 225,935.00} & 3 & 4 & 3 \end{bmatrix} \quad (15)$$

The initial decision matrix was normalized by applying equations (10) and (11) to each criterion according to its optimization direction (maximization or minimization), and the resulting normalized matrix is presented in Table 17. By integrating the MCDA process with AHP, the normalized scores in Table 17 are multiplied by the AHP weights, yielding the integrated scores presented in Table 18.

Table 17. The normalized decision matrix

Strengthening Technique	Performance (Maximize)	Economic Aspect (Minimise)	Durability (Maximize)	Application Ease Of Use (Maximize)	Availability Of Analytical Models (Maximize)
Shotcrete with Reinforcement	0.27	0.97	0.00	0.00	1.00
Ferrocement	0.11	1.00	1.00	1.00	1.00
Textile reinforced mortar (TRM)	0.00	0.00	1.00	0.00	1.00
Steel strip reinforcement	0.65	0.98	0.50	0.50	0.00
Fiber reinforced polymer (FRP)	1.00	0.82	0.00	0.00	1.00
Post-tensioning	0.10	0.97	0.50	0.50	0.50

Table 18. AHP-MCDA integration

Strengthening Technique	P	EA	D	AEOU	AOAM	Final Score
	25.51 %	24.81 %	19.19 %	12.89 %	17.60 %	100.00 %
Shotcrete with Reinforcement	0.07	0.24	0.00	0.00	0.18	0.49
Ferrocement	0.03	0.25	0.19	0.13	0.18	0.77
Textile Reinforced Mortar (TRM)	0.00	0.00	0.19	0.00	0.18	0.37
Steel Strip Reinforcement	0.17	0.24	0.10	0.06	0.00	0.57
Fiber Reinforced Polymer (FRP)	0.26	0.20	0.00	0.00	0.18	0.63
Post-Tensioning	0.03	0.24	0.10	0.06	0.09	0.51

*C: Criteria, SC: Sub-Criteria, P: Performance, EA: Economic Aspect, D: Durability, AEOU: Application Ease of Use, AOAM: Availability of Analytical Model.

4.5 Final Ranking of Strengthening Techniques

Based on Table 19 and Figure 8, Ferrocement achieved the highest MCDA score of 0.77, securing the first-place ranking. Its success stems from an exceptional balance across all five criteria: a 161% shear capacity increase (Table 11), the lowest material cost among all techniques (Rp120,243.73/m², normalized score 1.00), superior durability (resistant to heat, moisture, and corrosion with a service life exceeding 5 years, normalized score 1.00), outstanding ease of application using locally available materials (normalized score 1.00), and mature analytical models (score 4, equivalent strut model available, normalized score 1.00).

Ferrocement's comprehensive strengths are further reinforced by its minimal maintenance requirements. Unlike techniques that demand periodic re-tensioning (post-tensioning), reapplication of protective coatings (FRP), or specialized fastener inspections (steel strip reinforcement), ferrocement is applied as a monolithic layer that integrates with the existing structure. It contains no mechanical joints, moving parts, or detachable components that necessitate regular replacement or adjustment. The material's inherent crack-arrest mechanism, dense mortar matrix, and uniform reinforcement dispersion provide long-term structural integrity without demanding specialized ongoing interventions. Routine visual inspection, which is a standard good practice for any durable infrastructure, does not constitute the 'regular maintenance' that would penalize a technique in the MCDA assessment. This characteristic confirms its perfect score in the 'Application Ease of Use' criterion (Table 15). While no technique is entirely maintenance-free, ferrocement's maintenance burden is limited to simple, low-cost visual checks—far less demanding than the specialized interventions required by competing techniques—making it particularly suitable for resource-limited settings such as Aceh.

Fiber Reinforced Polymer (FRP) ranked second with an MCDA score of 0.63. FRP's superior shear capacity (249% average increase, normalized score 1.00) and mature analytical models (score 4, normalized score 1.00) were insufficient to overcome its significant limitations; specifically, the corrected durability assessment revealed that FRP is not resistant to heat (losing up to 90% effectiveness at 150 °C) and is susceptible to moisture degradation (up to 5.56% absorption and 70% fracture toughness reduction), resulting in a durability normalized score of 0.00. Additionally, its high cost (Rp760,960.40/m², normalized score 0.82) and low ease of application (requires specialized equipment and materials not available in Aceh, normalized score 0.00) further penalized its overall performance. Steel Strip Reinforcement ranked third with an MCDA score of 0.57, improving its rank due to the recalibrated scoring. Its high shear capacity (214%, normalized score 0.65) and good economic performance (Rp204,941.47/m², normalized score 0.98) are offset by limited heat resistance and the absence of a validated equivalent strut model (score 2 out of 4, normalized score 0.00). This lack of analytical model availability presents a significant engineering risk for design applications.

Post-Tensioning ranked fourth with an MCDA score of 0.51, benefiting from moderate performance (160% shear capacity increase), competitive cost (Rp225,935.00/m², normalized score 0.97), and moderate analytical model availability (score 3 out of 4, normalized score 0.50). However, it was

constrained by its limited heat resistance (normalized score 0.00) and very low ease of application (requires specialized hydraulic equipment, normalized score 0.00). Shotcrete with Reinforcement dropped to fifth place with an MCDA score of 0.49, a significant decline from its previous ranking. Despite its moderate shear capacity (177%, normalized score 0.27) and mature analytical models (score 4, normalized score 1.00), the corrected durability assessment—acknowledging its limited heat resistance (compressive strength declines above 150 °C) and notable corrosion risk in humid environments—resulted in a durability normalized score of 0.00. Its low ease of application (normalized score 0.00) further constrained its ranking.

Table 19. Final Ranking of Strengthening Techniques

Strengthening Technique	MCDA Score
Shotcrete with Reinforcement	0.49
Ferrocement	0.77
Textile Reinforced Mortar (TRM)	0.37
Steel Strip Reinforcement	0.57
Fiber Reinforced Polymer (FRP)	0.63
Post-Tensioning	0.51

Textile Reinforced Mortar (TRM) consistently occupied the lowest rank with an MCDA score of 0.37. Despite excellent durability (score 4, normalized score 1.00) and mature analytical models (score 4, normalized score 1.00), TRM was severely penalized by its prohibitive cost (Rp3,650,718.41/m², the most expensive among all techniques, normalized score 0.00) and low ease of application (requires special materials not available in Aceh, normalized score 0.00). These economic and practical barriers render it unsuitable for widespread application in resource-limited settings such as Aceh. These results demonstrate that techniques offering an optimal balance among structural performance, economic factors, durability, ease of application, and analytical model availability dominate the top rankings. Ferrocement's first-place ranking confirms that balanced performance across all criteria—rather than excellence in a single attribute provides the most sustainable and resilient solution for Aceh's specific conditions.

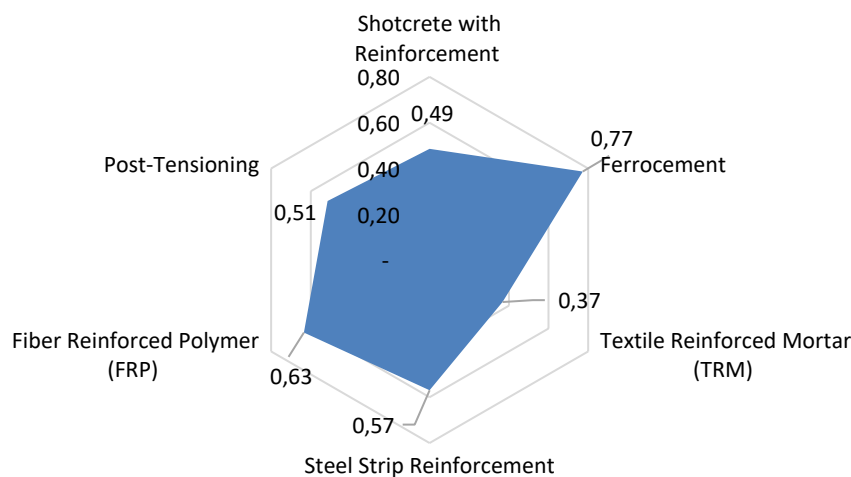


Fig. 8. MCDA diagram of strengthening technique

4.6 Sensitivity Analysis

To ensure the robustness of the generated data, a sensitivity analysis was conducted by simulating five alternative weighting scenarios (AS.1 to AS.5) against the original BASE scenario. The numerical weights and descriptions for each scenario are consolidated in Figure 9, allowing the reader to directly observe how weight changes affect the rankings. The results demonstrate that Ferrocement consistently ranked first or second across all scenarios, confirming its status as the most robust and superior option under various prioritization criteria. Notably, Ferrocement

remained competitive even when the weight of seismic performance was substantially increased to 30% in AS.1, securing the first position (0.73), outperforming FRP (0.61), and achieved the first position in the economically oriented scenario (AS.2) and the equal-weight control scenario (AS.4). In the extreme ease-of-application scenario (AS.5), Ferrocement reinforced its leading position with a score of 0.87, while FRP dropped to fourth place (0.41), confirming that poor applicability severely penalizes techniques requiring specialized equipment when practical feasibility is heavily prioritized. This consistency stems from its balanced excellence across all five criteria—rather than reliance on a single attribute—and should be explicitly characterized as a methodological success: the AHP-MCDA framework reliably identifies Ferrocement as the most sustainable and resilient option regardless of how stakeholders reprioritize the criteria.

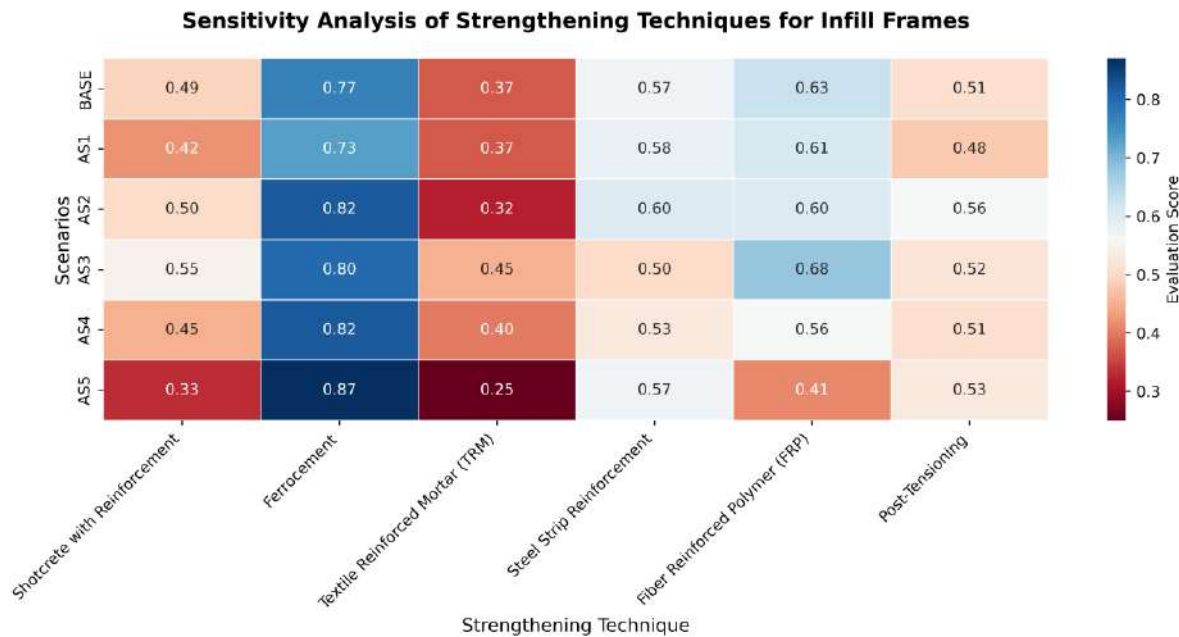


Fig. 9. Sensitivity analysis result

Fiber Reinforced Polymer (FRP) performed competitively in scenarios emphasizing performance and analytical models, securing the second position in both AS.1 and AS.3, with scores of 0.61 and 0.68, respectively. This reflects its superior shear capacity (normalized score 1.00) and mature analytical models. However, FRP dropped to third place in AS.2—the economically oriented scenario—where its high cost (Rp760,960.40/m², normalized score 0.82) and poor ease of application (normalized score 0.00) became significant disadvantages. In the base scenario, FRP ranked second with a score of 0.63. Shotcrete with Reinforcement exhibited notable sensitivity to the durability criterion. Its ranking varied between third and fifth positions depending on the weight assigned to durability. In scenarios where durability was de-emphasized (AS.2 and AS.4), Shotcrete performed relatively better, but it consistently remained below Ferrocement and FRP. Its limited heat resistance and corrosion vulnerability in humid environments constrained its overall competitiveness.

Steel Strip Reinforcement exhibited a marked sensitivity to the 'availability of analytical models' criterion. Despite its excellent shear capacity (214%, normalized score 0.65), it dropped to fifth place (0.50) in AS.3 because it entirely lacks a validated equivalent strut model (score 2 out of 4, normalized score 0.00). This inability to perform reliable design calculations presents a significant engineering risk and explains its intermediate ranking (third place) in other scenarios. Textile Reinforced Mortar (TRM) and Post-Tensioning consistently occupied the bottom ranks. TRM's primary disadvantage is its prohibitive cost (Rp3,650,718.41/m², the most expensive among all techniques), resulting in a normalized score of 0.00 for the economic criterion. TRM, despite its well-developed analytical models (normalized score 1.00), consistently remained at the lowest rank across all scenarios due to its prohibitive cost and poor ease of application. Post-Tensioning, while economically competitive (Rp225,935.00/m², normalized score 0.97), was significantly constrained by its limited ease of application (requires specialized hydraulic equipment,

normalized score 0.00) and moderate durability (score 3 out of 4, normalized score 0.50). The sensitivity analysis confirms that the proposed framework is both robust and recalibratable for different stakeholder priorities and regional contexts. The stable performance of Ferrocement across all scenarios provides high confidence in its recommendation as the optimal strengthening technique for Aceh.

5. Discussion

This section interprets the findings in relation to the study's objectives. It begins by discussing the main findings and their practical implications, followed by a comparison with previous research. The limitations of the study are then acknowledged to provide a balanced perspective on the conclusions drawn.

5.2 Main Findings and Implications

Before discussing the MCDA rankings, it is essential to interpret the AHP findings that underpin the entire decision framework. The pairwise comparison matrix (Matrix 15) revealed a ratio of 1.00 between the Performance and Economic Aspect criteria, indicating that local experts in Aceh consider these two criteria to be equally important. This is not a coincidence or a methodological artifact; rather, it reflects the unique socio-economic and seismic context of Aceh Province. On one hand, Aceh is one of the most earthquake-prone regions in Indonesia, having experienced the devastating 2004 Sumatra-Andaman earthquake (Mw 9.3) [3] and the 2016 Pidie Jaya earthquake (Mw 6.5) [4]. These events have caused massive casualties, destroyed thousands of buildings, and forced the revision of Indonesia's seismic design code (SNI 1726:2019). Consequently, local engineers and policymakers are acutely aware that structural performance—particularly shear capacity—is non-negotiable for ensuring life safety during future earthquakes. This awareness is reflected in the 25.51% weight assigned to the Performance criterion, the highest among all criteria.

On the other hand, Aceh faces significant economic constraints. According to the Indonesian Central Statistics Agency (BPS), Aceh's poverty rate in 2024 was approximately 14-15%, which is above the national average. The province's Gross Regional Domestic Product (GRDP) per capita remains below the national average, and reconstruction budgets are heavily reliant on central government funding. In this context, the Economic Aspect criterion received a weight of 24.81%, which is statistically comparable to that of Performance. This near-equal weighting demonstrates that local experts cannot afford to prioritize seismic safety at any cost. Strengthening techniques must be both effective and economically feasible for widespread adoption in Aceh's resource-limited settings. The AHP results ($CR = 0.0246$, well below the 0.10 threshold) confirm that this balanced prioritization is not based on random responses but reflects a consistent and logically coherent judgment among the ten experts from government, academia, and practice. The small difference of only 0.7% between Performance and Economic weights underscores a pragmatic 'safety-budget balance'—a trade-off that is particularly acute in developing regions where financial resources allocated to retrofitting must be balanced against other pressing social and economic needs.

This AHP finding has a direct and profound implication for the MCDA results that follow. Because local experts value performance and economy almost equally, the optimal strengthening technique cannot be the one with the highest shear capacity (FRP, 249%) or lowest cost alone (e.g. Polypropylene, although excluded from the final analysis). Instead, the ideal technique must offer the best compromise between these two dominant criteria, while also performing well in the other three criteria (durability, ease of application, and analytical model availability). As the subsequent MCDA ranking demonstrates, Ferrocement achieves this balance: it provides a 161% shear capacity increase (moderate but effective) at the lowest material cost (Rp120,243.73/m² $\approx$ USD 7.20), while excelling in durability and ease of application. In essence, Ferrocement is the optimal solution precisely because it aligns with Aceh's priority of achieving seismic safety within tight economic constraints—a finding that is not merely technical but deeply rooted in the socio-economic realities of the region.

The sensitivity analysis robustly supports the study's core sustainability claim. Selecting the most applicable technique, as opposed to solely the strongest one, represents a strategic prioritization rather than a performance compromise. The framework successfully filters out high-performance but contextually unsuitable materials (e.g., FRP's high cost and application difficulty; Steel Strip's lack of validated models) while elevating Ferrocement, which offers an optimal balance of structural safety, economic feasibility, local material availability, and engineering reliability.

The stability of Ferrocement as the best option and the consistent positioning of TRM among the lowest ranks provide high confidence in the final recommendation for implementation in Aceh. The adaptability of the framework, evidenced by the stable rankings despite substantial weight variations, confirms that the proposed AHP-MCDA framework is both robust and recalibratable for different stakeholder priorities and regional contexts. In summary, the sensitivity analysis confirms that sustainability—defined as selecting the most applicable technique rather than simply the strongest—is best achieved by balancing technical performance with local-context criteria such as economic feasibility, durability, ease of application, and analytical model availability. Ferrocement's consistent performance across all scenarios demonstrates that it represents the optimal balance for Aceh's specific conditions.

5.2 Comparison with Previous Studies

This study confirms the effectiveness of Ferrocement reported in earlier experimental work, while simultaneously extending it through a decision framework that incorporates local expert judgment and context-specific criteria. The findings highlight distinct trade-offs: FRP, despite its superior shear capacity, was penalized by moderate durability and high cost, while TRM, despite excellent durability and mature analytical models, was constrained by its prohibitive material cost (Rp3,650,718.41/m²). This demonstrates that high-performance materials are not necessarily ecologically appropriate, particularly in resource-limited settings like Aceh.

5.3 Limitations

While this study provides a robust and recalibratable AHP-MCDA framework for selecting sustainable seismic strengthening techniques, several limitations should be acknowledged to ensure a balanced interpretation of the findings and to guide future research directions. The exclusion of Bamboo Reinforcement and Polypropylene Fiber Reinforcement from the MCDA analysis, though justified by existing absence of valid experimental data for standalone applications in reinforced concrete frames with masonry infill walls, represents a notable gap. Bamboo, in particular, is a rapidly renewable material widely available in Aceh and other tropical regions, offering significant potential for low-carbon, cost-effective seismic retrofitting; however, the absence of quantitative data on its bond-slip behavior, durability under quasi-static or cyclic seismic loading in humid tropical climates precluded its inclusion. Similarly, while polypropylene fibers have demonstrated benefits in hybrid systems (e.g., combined with shotcrete), their standalone effectiveness for infill wall strengthening remains insufficiently documented, and future experimental campaigns should prioritize these materials to facilitate their integration into the framework as new evidence becomes available.

The performance evaluation relied on arithmetic means of shear capacity increases derived from a limited number of experimental studies (21 studies identified for the Performance criterion), which, while necessary to avoid optimistic bias, inherently masks the statistical variability and methodological heterogeneity across studies. The selected experimental studies employed varying test setups, including different loading protocols (cyclic versus monotonic), specimen scales (full-scale vs. reduced-scale), masonry types, and frame configurations, all of which may influence the reported shear capacity increases and complicate direct comparisons between techniques. Furthermore, for certain techniques such as Steel Strip Reinforcement and Post-Tensioning, the performance data were drawn from only one or two studies, reducing the statistical robustness of their mean values, and future research should quantify performance variability more accurately and establish more reliable effect sizes for each technique.

The durability assessment employed a binary scoring system based on the presence of evidence in the literature rather than the degree or rate of degradation, which, while aligning with the MCDA

framework's requirement for discrete scores, simplifies complex, continuous degradation phenomena. For instance, FRP's heat vulnerability is described in Section 2 as a loss of up to 90% effectiveness at 150 °C, yet it receives a binary "not resistant" score alongside Shotcrete, whose compressive strength declines markedly above 150 °C, failing to capture the rate of degradation, temperature thresholds, or time-dependent performance losses critical for service life predictions. Moreover, the literature surveyed for durability (75 studies) may not fully encapsulate Aceh's specific micro-climatic conditions characterized by persistent high humidity, elevated ambient temperatures, and potential exposure to saline coastal environments; consequently, the corrosion resistance of steel fibers in shotcrete, for instance, may be more severely compromised under Aceh's specific conditions than reported in temperate-climate studies, and localized durability testing replicating Aceh's tropical humid climate would serve to validate and refine the scoring system.

The economic assessment was limited to direct material costs, including shipping costs to ensure consistent treatment across all techniques, but it excluded several critical cost components such as labor costs (since specialized techniques like TRM, FRP, and Post-Tensioning require trained crews commanding higher wages), equipment rental (e.g., pneumatic sprayers for shotcrete and hydraulic jacks for Post-Tensioning, which may not be readily available in Aceh), scaffolding and site preparation, and business interruption costs for occupied buildings. Consequently, the economic ranking presented in this study represents a material-cost-centric perspective, and a full life-cycle cost analysis incorporating labor, equipment, maintenance, and downtime costs would provide a more comprehensive economic evaluation and may alter the rankings, particularly for labor-intensive or specialized techniques.

The AHP weights were derived from pairwise comparisons by ten local experts representing government officials, academics, and practicing structural engineers in Aceh, and while the high consistency ratio validates the logical coherence of their judgments, the resulting weights are context-specific to Aceh's regulatory environment, economic constraints, and construction practices, and may not be directly transferable to other earthquake-prone regions with different socioeconomic conditions, seismic hazard levels, or material supply chains. However, this limitation also represents the study's key strength, as the framework is explicitly designed to be fully recalibratable by adjusting criteria weights through local expert elicitation, and the sensitivity analysis confirms that Ferrocement consistently ranked first or second across all weighting scenarios, demonstrating the robustness of the final recommendation even under varying stakeholder priorities. However, the application of this framework to other regions must involve re-elicitation of local expert weights to ensure contextual relevance.

The performance criterion focused exclusively on in-plane shear capacity improvement, as measured by the increase in lateral load resistance of infill walls, which, while a critical indicator of seismic performance, does not capture other important structural attributes such as out-of-plane stability, ductility, and energy dissipation, all of which may be affected differently by various strengthening techniques and are essential for a holistic seismic assessment. These aspects were not explicitly included as sub-criteria in the MCDA framework, primarily due to the limited availability of quantitative data in the reviewed literature, and future studies should expand the performance criterion to include multiple seismic response indicators to enable a more comprehensive structural evaluation. Finally, while Aceh serves as a representative case study for earthquake-prone regions in developing countries with limited resources, the findings—particularly the highest ranking of Ferrocement—may not be universally applicable, as other regions with different material supply chains, construction traditions, or regulatory frameworks may find alternative techniques more suitable; nevertheless, the methodological framework itself is readily transferable and provides a structured approach for evidence-based decision-making in any seismic region. Despite these limitations, the consistency of Ferrocement's first-place ranking across all sensitivity analysis scenarios provides strong confidence that it is the most sustainable and resilient choice for Aceh's specific conditions, and the limitations identified above do not undermine the robustness of this conclusion but rather highlight opportunities for refinement and validation in future research.

6. Conclusions

This study successfully developed an integrated Analytic Hierarchy Process (AHP) and Multi-Criteria Decision Analysis (MCDA) framework to select the most sustainable seismic strengthening technique for masonry infill walls in earthquake-prone regions, using Aceh, Indonesia as a case study. The framework incorporated five criteria—structural performance, economic aspect, durability, application ease of use, and availability of analytical models—weighted by ten local construction experts through pairwise comparisons.

A key finding of this research is that strengthening techniques with exceptionally high shear capacity improvement are not necessarily the optimal choices when other sustainability-related criteria are considered. Fiber Reinforced Polymer (FRP), despite its superior structural performance and mature analytical models, was constrained by limited durability and low ease of application, and its high material cost further penalized its economic performance. These limitations resulted in FRP securing second place overall. Similarly, Shotcrete with Reinforcement, despite demonstrating moderate structural performance and mature analytical models, was penalized by limited heat resistance and notable corrosion risk in humid environments, leading to a lower ranking. Steel Strip Reinforcement, despite its excellent structural performance and good economic characteristics, was limited by poor heat resistance and limited analytical model availability, which presents a significant engineering risk for design applications. These results confirm that a balanced assessment across multiple criteria is essential for sustainable decision-making.

Ferrocement was identified as the most optimal technique overall. Its advantages, based on the evaluation, include: effective structural performance; the lowest material cost among all techniques, making it economically competitive; superior durability, being resistant to heat, moisture, and corrosion, with a long service life; outstanding ease of application, as it can be performed with locally available materials and does not require specialized equipment; and mature analytical models that enable reliable structural design. Its minimal maintenance requirements further enhance its practical feasibility, making it particularly suitable for resource-limited settings. Its balanced performance across all criteria makes ferrocement the most sustainable and resilient choice for Aceh, particularly when considering the near-equal importance placed on performance and economic aspects by local experts.

The AHP results revealed that local experts prioritize structural performance and economic aspects as the most decisive criteria, followed by durability, availability of analytical models, and application ease of use. The MCDA procedure proved robust, with sensitivity analysis across five alternative weighting scenarios confirming that ferrocement consistently ranked first or second in all scenarios, while Textile Reinforced Mortar (TRM) consistently ranked last due to its prohibitive cost and low ease of application. This demonstrates that the proposed framework is stable and adaptable to different stakeholder priorities.

The main contribution of this research is a scientific, robust, and structured decision framework that enables stakeholders, academics, and practitioners to select the most appropriate infill wall strengthening technique based on local conditions. The methodology is fully recalibratable and transferable to other seismic regions through adjustment of criteria weights using local expert judgment. By prioritizing techniques that extend building lifespan, minimize demolition waste, and conserve embodied energy, the framework directly supports post-earthquake urban restoration and energy-saving construction. The explicit inclusion of application ease of use and local material availability ensures practical feasibility in resource-limited settings such as Aceh.

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References

- [1] ACI Committee 318. Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary. Farmington Hills, MI: American Concrete Institute; 2025.
- [2] International Organization for Standardization. ISO 21930:2017 – Sustainability in buildings and civil engineering works — Core rules for environmental product declarations of construction products and services. Geneva, Switzerland: 2017.
- [3] West M, Sánchez JJ, McNutt SR. Periodically triggered seismicity at Mount Wrangell, Alaska, after the Sumatra earthquake. *Science* (1979) 2005;308:1144–6. <https://doi.org/10.1126/science.1112462>.
- [4] Jayasuriya S, McCawley P. The Asian tsunami: aid and reconstruction after a disaster. The Asian Tsunami, Edward Elgar Publishing; 2010.
- [5] Dona MT, Indra S, Priskasari E. Beban Gempa dan Base Shear menurut SNI 1726-2012 dan SNI 1726-2019 pada Gedung Fakultas Ilmu Keolahragaan (FIK) Universitas Negeri Malang: [Earthquake Loads and Base Shear According to SNI 1726-2012 and SNI 1726-2019 for the Faculty of Sports Science (FIK) Building at Malang State University]. *Prosiding SEMSINA* 2022;3:160–4.
- [6] Fikri R, Derakhshan H, Ingham J. Numerical evaluation of a non-ductile RCFMI building subjected to the Canterbury, New Zealand Earthquakes: A case study of the St Elmo Courts building. *Structures* 2020;28:991–1008. <https://doi.org/10.1016/j.istruc.2020.05.002>.
- [7] Deneko E, Bilgin H. Observed failure modes in existing URM buildings after November 26, 2019 earthquake in Albania. *Research on Engineering Structures and Materials* 2024;10:1085–107. <https://doi.org/10.17515/resm2024.106ea1202rs>.
- [8] Gkournelos PD, Triantafillou TC, Bournas DA. Seismic upgrading of existing reinforced concrete buildings: A state-of-the-art review. *Eng Struct* 2021;240:112273.
- [9] R. Chudley, R. Greeno. Building Construction Handbook. Seventh edition. Oxford, UK: Elsevier; 2008.
- [10] ACI Committee 506. Specification for Shotcrete (ACI 506.2-95) 1995.
- [11] Teymur P, Pala S, Yuksel E. Retrofitting of vulnerable reinforced concrete frames with wet-mixed shotcrete panels. *Advances in Structural Engineering* 2012;15:1–14.
- [12] Altin S, Anil Ö, Koprman Y, Belgin Ç. Strengthening masonry infill walls with reinforced plaster. *Proceedings of the Institution of Civil Engineers: Structures and Buildings* 2010;163:331–42. <https://doi.org/10.1680/stbu.2010.163.5.331>.
- [13] ElGawady M, Lestuzzi P, Badoux M. A review of conventional seismic retrofitting techniques for URM. 13th international brick and block masonry conference, vol. 4, Amsterdam; 2004.
- [14] Wang J, Niu D, Zhang Y. Mechanical properties, permeability and durability of accelerated shotcrete. *Constr Build Mater* 2015.
- [15] ACI Committee 549. 549.1R-18: Guide for the Design, Construction, and Repair of Ferrocement. Farmington Hills, MI, USA: 2018.
- [16] Sen D, Alwashali H, Shafiul Islam M, Maeda M. Investigation of the lateral capacity of ferro-cement retrofitted infilled masonry in RC frame and simplified prediction approach. *AIJ Journal of Technology and Design* 2020;26:159–63. <https://doi.org/10.3130/aijt.26.159>.
- [17] Vincent Sam Jebadurai S, Tensing D, Freeda Christy C. Enhancing Performance of Infill Masonry with Skin reinforcement Subjected to Cyclic Load. *International Journal of Engineering, Transactions B: Applications* 2019;32:223–8. <https://doi.org/10.5829/ije.2019.32.02b.06>.
- [18] Leeanansaksiri A, Panyakapo P, Ruangrassamee A. Seismic capacity of masonry infilled RC frame strengthening with expanded metal ferrocement. *Eng Struct* 2018;159:110–27. <https://doi.org/https://doi.org/10.1016/j.engstruct.2017.12.034>.
- [19] Xin R, Ma P. Experimental investigation on the in-plane seismic performance of damaged masonry walls repaired with grout-injected ferrocement overlay. *Constr Build Mater* 2021;282:122565. <https://doi.org/https://doi.org/10.1016/j.conbuildmat.2021.122565>.
- [20] Hasnat A, Ahsan R, Yashin SM. Quasi-static in-plane behavior of full-scale unreinforced masonry walls retrofitted using ferro-cement overlay. *Asian Journal of Civil Engineering* 2022;23:649–64. <https://doi.org/10.1007/s42107-022-00447-7>.
- [21] Longthong S, Panyakapo P, Ruangrassamee A. Seismic strengthening of RC frame and brick infill panel using ferrocement and expanded metal. *Engineering Journal* 2020.
- [22] Sen D, Zahura FT, Das A, Alwashali H, Islam MS, Maeda M, et al. A Comparative Investigation On Experimental Lateral Behaviour Of Bare Rc Frame, Non-Strengthened And Ferrocement Strengthened Masonry Infilled Rc Frame. *Bulletin of the New Zealand Society for Earthquake Engineering* 2024;57:85–96. <https://doi.org/10.5459/bnzsee.1656>.
- [23] Abidin ZU, Shahzada K, Gul A, Alqaryouti Y, Azab M, Rizwan M, et al. Strengthening of masonry infill wall panels in reinforced concrete structures using dual strengthening strategy of ferrocement and ECC overlays. *Cogent Eng* 2024;11. <https://doi.org/10.1080/23311916.2024.2388794>.

- [24] Sen D, Alwashali H, Tafheem Z, Sen D, Torihata Y, Alwashali H, et al. Investigation of the Ferro-cement laminated infilled masonry wall under cyclic lateral load. Pacific Conference on Earthquake Engineering and Annual NZSEE Conference, 2019.
- [25] Mahmud Amanat K, Alam M, Shahria Alam M. Experimental investigation of the use of ferrocement laminates for repairing masonry in filled RC frames. 2007.
- [26] Greepala V, Nimityongskul P, Tanchaisawat T, Parichartpreecha R. Thermal conductivity of ferrocement. Eleventh East Asia-Pacific Conference on Structural Engineering & Construction (EASEC-11), Taipei: 2008.
- [27] Al-Fahdawi FA, Al-Hadithi AI, Al-Asafi JA. The Mechanical Properties of Ferrocement Mortar with Waste Plastic Fibers at Elevated Temperatures. vol. 12. 2022. <https://doi.org/10.48084/etasr.5209>.
- [28] R. Sri Ravindrarajah, P. Paramasivam. Influence of weathering on ferrocement properties. Journal of Ferrocement 1986;16.
- [29] Mathews MS, Sudhakumar J, Jayasree P. Durability studies on ferrocement. Journal of Ferrocement 1993;23:15-23.
- [30] Alexander D. The durability of ferrocement and fibrous ferrocement in aggressive environments. Journal of Ferrocement 1992;22:373.
- [31] Mansur MA, Maalej M, Ismail M. Study on corrosion durability of ferrocement. ACI Mater J 2008.
- [32] Rajendran M. Corrosion assessment of ferrocement element with nano geopolymer for marine application. Structural Concrete 2021. <https://doi.org/10.1002/suco.202000783>.
- [33] Nadasan L, Onet T. Durability of ferrocement. Annals of the Faculty of Engineering Hunedoara 2013;11:89.
- [34] Ahmed HI, Robles-Austriaco L. State-of-the-art report on rehabilitation and restrengthening of structures using ferrocement. Journal of Ferrocement 1991;21:243-58.
- [35] Nadasan L, Onet T. Durability of ferrocement. Annals of the Faculty of Engineering Hunedoara 2013;11:89.
- [36] Burakale AS, Attarde PM, Patil MD. Ferrocement Construction Technology and its Applications-A Review. International Research Journal of Engineering and Technology 2020;07.
- [37] Kařuza M. FRP strengthening of AAC masonry walls - comparative analysis and discussion selected calculation methods. Archives of Civil Engineering 2023;69:97-110. <https://doi.org/10.24425/ace.2023.145255>.
- [38] Sagare SS, Kirthiga R, Elavenil S. A state of art of review on strengthening of concrete structures using fabric reinforced cementitious matrix. Research on Engineering Structures and Materials 2024;10:1231-60. <https://doi.org/10.17515/resm2024.133ma1226rv>.
- [39] Koutas L, Triantafillou TC, Bousias SN. Seismic retrofitting of a three-storey masonry-infilled RC frame with textile-reinforced mortar (TRM). Proceedings of the FRPRCS12, Sheffield; 2016.
- [40] Akhoundi F, Vasconcelos G, Lourenço P. In-Plane Behavior of Infills using Glass Fiber Shear Connectors in Textile Reinforced Mortar (TRM) Technique. International Journal of Structural Glass and Advanced Materials Research 2018;2:1-14. <https://doi.org/10.3844/sgamrsp.2018.1.14>.
- [41] Ferrara G, Caggegi C, Martinelli E, Gabor A. Shear capacity of masonry walls externally strengthened using Flax-TRM composite systems: experimental tests and comparative assessment. Constr Build Mater 2020;261:120490.
- [42] Cao S, Wu Z, Wang X. Tensile properties of CFRP and hybrid FRP composites at elevated temperatures. J Compos Mater 2009;43:315-30.
- [43] Dalalbashi A, Ghiassi B, Oliveira D V. Aging of lime-based TRM composites under natural environmental conditions. Constr Build Mater 2021.
- [44] Signorini C, Nobili A. Comparing durability of steel reinforced grout (SRG) and textile reinforced mortar (TRM) for structural retrofitting. Mater Struct 2021. <https://doi.org/10.1617/s11527-021-01729-3>.
- [45] Koutas L, Bousias SN, Triantafillou TC. Seismic Strengthening of Masonry-Infilled RC Frames with TRM: Experimental Study. Journal of Composites for Construction 2015;19. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000507](https://doi.org/10.1061/(asce)cc.1943-5614.0000507).
- [46] Hung TQ, Mizoguchi M, Takase Y. Strengthening Effect of the Fixing Method of Polypropylene Band on Unreinforced Brick Masonry in Flexural, Shear, and Torsion Behaviors. Buildings 2023;13. <https://doi.org/10.3390/buildings13112863>.
- [47] Dardaei S, Shakib H, Khalaf Rezaei M, Mousavi M. Analytical and experimental seismic evaluation of confined masonry walls retrofitted by steel-fiber and polypropylene shotcrete. Journal of Seismology and Earthquake Engineering 2014;16:271-80.
- [48] Almusallam TH, Al-Salloum YA. Behavior of FRP strengthened infill walls under in-plane seismic loading. Journal of Composites for Construction 2007;11:308-18.
- [49] Mosallam A, Banerjee S. Enhancement in in-plane shear capacity of unreinforced masonry (URM) walls strengthened with fiber reinforced polymer composites. Compos B Eng 2011;42:1657-70.

- [50] Erol G, Karadogan HF. Seismic strengthening of infilled reinforced concrete frames by CFRP. *Compos B Eng* 2016;91:473–91.
- [51] Rahman A, Ueda T. In-Plane Shear Performance of Masonry Walls after Strengthening by Two Different FRPs. *Journal of Composites for Construction* 2016;20. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000661](https://doi.org/10.1061/(asce)cc.1943-5614.0000661).
- [52] Ersoy U, Ozcebe G, Tankut T, Akyuz U, Erduran E, Erdem I. Strengthening of infilled walls with CFRP sheets. *Seismic Assessment and Rehabilitation of Existing Buildings*, Springer; 2003, p. 305–34.
- [53] Kodur VKR, Bhatt PP, Naser MZ. High temperature properties of fiber reinforced polymers and fire insulation for fire resistance modeling of strengthened concrete structures. *Compos B Eng* 2019;175:107104.
- [54] Heshmati M, Haghani R, Al-Emrani M. Effects of moisture on the long-term performance of adhesively bonded FRP/steel joints used in bridges. *Compos B Eng* 2016;92:447–62.
- [55] Tuakta C, Büyüköztürk O. Deterioration of FRP/concrete bond system under variable moisture conditions quantified by fracture mechanics. *Compos B Eng* 2011;42:145–54.
- [56] Benmokrane B, Ali AH. Durability and long-term performance of fiber-reinforced polymer as a new civil engineering material. *International Congress on Polymers in Concrete*, Springer; 2018, p. 49–59.
- [57] Gadve S, Mukherjee A, Malhotra SN. Corrosion of steel reinforcements embedded in FRP wrapped concrete. *Constr Build Mater* 2009.
- [58] Kanagavel K, Karbhari VM. Long-term durability of CFRP strips used in infrastructure rehabilitation. *Polymers (Basel)* 2025;17:1886.
- [59] El-Diasity M, Okail H, Kamal O, Said M. Structural performance of confined masonry walls retrofitted using ferrocement and GFRP under in-plane cyclic loading. *Eng Struct* 2015;94:54–69. <https://doi.org/10.1016/j.engstruct.2015.03.035>.
- [60] Binici B, Ozcebe G. Analysis of infilled reinforced concrete frames strengthened with FRPS. In: Wasti ST, Ozcebe G, editors. *Advances in Earthquake Engineering for Urban Risk Reduction*, Dordrecht: Springer Netherlands; 2006, p. 455–70.
- [61] Meguro K, Soti R, Navaratnaraj S, Numada M. Dynamic testing of masonry houses retrofitted by bamboo band meshes. *Journal of Japan Society of Civil Engineers, Ser A1 (Structural Engineering & Earthquake Engineering (SE/EE))* 2012;68:I_760-I_765.
- [62] Suhelmidawati E, Zamzarena R, Adibroto F, Syofiardi S. Experimental Study of Bamboo As A House Retrofitting Material For Developing Countries. *Jurnal Ilmiah Rekayasa Sipil* 2017;14:20–7. <https://doi.org/10.30630/jirs.14.2.103>.
- [63] Karthika S, Lakshimikandhan KN, Sivakumar P, Dhinakaran G. Load Deflection Characteristics of Sustainable Infilled Concrete Wall Panels. *Arab J Sci Eng* 2016;41:451–60. <https://doi.org/10.1007/s13369-015-1674-1>.
- [64] Dolce M, Nigro D, Ponzio FC, Marnetto R. The CAM system for the retrofit of masonry structures. *Proceedings of the 7th International Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control of Vibrations of Structures*, Assisi, Italy, 2001, p. 2–5.
- [65] Borri A, Corradi M, Castori G, Molinari A. Stainless steel strip – A proposed shear reinforcement for masonry wall panels. *Constr Build Mater* 2019;211:594–604. <https://doi.org/10.1016/j.conbuildmat.2019.03.197>.
- [66] Akin E, Korkmaz SZ, Korkmaz HH, Diri E. Rehabilitation of infilled reinforced concrete frames with thin steel plate shear walls. *Journal of Performance of Constructed Facilities* 2016;30:04015098.
- [67] Choi IR, Park HG. Steel plate shear walls with various infill plate designs. *Journal of Structural Engineering* 2009. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2009\)135:7\(785\)](https://doi.org/10.1061/(ASCE)0733-9445(2009)135:7(785)).
- [68] Soltanzadeh G, Osman H Bin, Vafaei M, Vahed YK. Seismic retrofit of masonry wall infilled RC frames through external post-tensioning. *Bulletin of Earthquake Engineering* 2018;16:1487–510. <https://doi.org/10.1007/s10518-017-0241-4>.
- [69] McCool GE. Evaluation of corrosion resistance of new and upcoming post-tensioning materials after long-term exposure testing. *repositories.lib.utexas.edu*; 2010.
- [70] Hahin C, Gambill J, Scott JK. An Investigation of the Susceptibility of Post-Tensioning Cables to Stress-Corrosion Cracking. *apps.dtic.mil*; 1977.
- [71] Tilly GP. Performance and management of post-tensioned structures. *Proceedings of the Institution of Civil Engineers-Structures and Buildings* 2002;152:3–16.
- [72] Jenkins RW, Kreger ME, Frosch RJ. Post-tensioning technologies. *rosap.nrl.bts.gov*; 2013.
- [73] Saaty TL. Decision making with the Analytic Hierarchy Process. *Scientia Iranica* 2002;9:215–29. <https://doi.org/10.1504/ijssci.2008.017590>.
- [74] Badan Standardisasi Nasional Republik Indonesia. SNI 1726:2019 – Tata cara perencanaan ketahanan gempa untuk struktur bangunan gedung dan non gedung: [Guidelines for earthquake-resistant design of building and non-building structures]. Jakarta: Badan Standardisasi Nasional; 2019.
- [75] Paulay T, Priestley MJN. *Seismic design of reinforced concrete and masonry buildings*. Wiley; 1992.

- [76] ISO C. Bases for Design of Structures-Assessment of Existing Structures. Report ISO 2010;13822.
- [77] International Organization for Standardization. ISO 21505:2017-Project, programme and portfolio management-Guidance on governance. Switzerland: International Organization for Standardization; 2017.
- [78] Monteiro PJM, Mehta PK. Concrete: Microstructure, Properties and Materials. Mc Graw Hill; 2006. <https://doi.org/10.1036/0071462899>.
- [79] ACI Committee 318. 318-19 Building Code Requirements for Structural Concrete and Commentary. American Concrete Institute; 2019. <https://doi.org/10.14359/51716937>.
- [80] Forman E, Peniwati K. Aggregating individual judgments and priorities with the analytic hierarchy process. Eur J Oper Res 1998;108:165–9.
- [81] Jahan A, Edwards KL. A state-of-the-art survey on the influence of normalization techniques in ranking: Improving the materials selection process in engineering design. Mater Des 2015;65:335–42. <https://doi.org/10.1016/j.matdes.2014.09.022>.

Investigation of asphalt mixtures with river aggregate, waste glass and PET modification for improved strength and moisture resistance in Iraq

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Abstract

Large quantities of waste plastic and glass are generated annually in Iraq, while the continuous extraction of natural aggregates for road construction increases pressure on natural resources. This study investigates the combined use of locally sourced Iraqi river aggregate, waste glass and waste polyethylene terephthalate (WPET) in asphalt mixtures, an approach that has received limited attention in previous studies. River sand was used as the fine aggregate, while waste glass was used as a partial replacement of fine aggregate at replacement levels of 10%, 20% and 30% by weight. WPET was added 8% WPET to the asphalt binder using the wet process at contents of 4%, 8% and 12% by binder weight. Marshall stability, indirect tensile strength and tensile strength ratio tests were conducted to evaluate the mechanical performance and moisture susceptibility of the mix. Results showed that waste glass improved the Marshall stability up to an optimum replacement level of 20%, before declining at higher replacement levels. Similarly, 8% WPET produced the optimum performance by increasing stability, indirect tensile strength and moisture resistance, whereas increasing the content to 12% reduced the mechanical performance of the mixture. Overall, the combined use of 20% waste glass and 8% WPET produced the optimum asphalt mixture, indicating that the proposed mixtures can enhance asphalt mixture performance. This approach offers a practical solution for constructing asphalt pavement in Iraq.

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1. Introduction

The construction industry has been a primary driver for urbanization and economic growth in Iraq but this rapid development has come at the cost of natural resource depletion, it has led to environmental degradation [1]. The ancient Mesopotamian cities of (Ur, Nippur and Uruk) developed along the Tigris and Euphrates rivers, where abundant natural resources supported monumental construction and urban expansion [2]. The conventional extraction of natural aggregates from quarries is no longer sustainable, as it causes significant habitat destruction and high carbon emissions, it poses significant environmental concerns [3, 4]. Consequently, adopting a circular economy in pavement engineering has become essential to promote the use of industrial by-products and waste materials in Iraq road construction [5, 6]. Waste glass and river sand are promising sustainable alternatives to conventional crushed fine aggregates [7]. Furthermore, recycled plastics and glass in asphalt pavements reduce carbon emissions, landfill waste and the consumption of virgin aggregates and bitumen [8-10]. Recent studies indicate that the use of aggregates accounts for approximately 1.8% of global greenhouse gas emissions [11]. Approximately 180 million tons of glass are produced annually worldwide, however, less than 20%

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is recycled, indicating that the present recycling rate remains insufficient and environmentally unsustainable [12]. Their use as polymer additives to enhance the engineering performance of asphalt mixtures has attracted considerable attention in pavement engineering [13, 14]. The use of PET as a binder modifier improves viscosity, stiffness and resistance to rutting, cracking and overall performance, supporting PET recycling in asphalt pavements [15, 16]. Ferreira et al. investigated recycled micro-PET in hot mix asphalt (HMA) and reported enhanced moisture resistance, particularly for regions with heavy rainfall without significantly affecting optimum asphalt content [17]. In Iraq, however, studies on waste glass in HMA remain limited with previous research primarily focusing on its use as a filler [18, 19]. Almuhamdi et al. evaluated the utilization of waste glass (WG) without additives in asphalt mixtures to improve mechanical performance as fillers in asphalt pavements [20]. Reducing waste generation through recycling is essential for long-term environmental protection, supporting resource recovery and **conserving** natural resources within sustainable construction strategies [21-24]. The transport of cores aggregates and fine aggregates from quarries, often located far from city centres, contributes to greenhouse gas emissions. Therefore, it is essential to rely on local resources to lower fossil fuel consumption associated with aggregate production and hauling. Despite increasing interest in sustainable asphalt materials, the combined use of river aggregate, WG and modified asphalt binders remains insufficiently and unexplored, particularly in Iraq. Previous studies have largely investigated individual recycled materials or low replacement levels, with limited attention to their combined application under local conditions. This study addresses this gap by evaluating for the first time in Iraq, the moisture resistance and mechanical performance of asphalt mixtures incorporating high volume river aggregates, WG and PET modified binder. The findings provide a sustainable pavement solution that maximizes local waste utilization while meeting modern pavement performance requirements.

2. Materials and Methods

In this experimental work, asphalt binder and aggregates (crushed and river) were verified per Iraqi standard specification for roads and bridge (SCRB) R/9. In terms of waste materials, utilizing waste plastic bottle and glass debris as additives. A comprehensive laboratory testing program was conducted by using Marshall and indirect tensile strength tests on hot mix asphalt specimens.

2.1. Aggregates and Filler

Coarse aggregate retained on the 4.75 mm sieve was crushed aggregate sourced from the Kerbala quarry, widely used in central and southern regions of Iraq. Fine aggregate that passing the (4.75mm) sieve consisted of stone screenings, crushed fine aggregate from Kerbala quarry, and river fine aggregate sourced from the Euphrates River. Both aggregate types were sieved and recombined to comply with the gradation limits of the SCRB- standard surface course [25]. Routine physical property tests were conducted, with results and SCRB limits summarized in Table 1.

Table 1. Physical properties of aggregates (crushed and river) used in this study.

Property	ASTM Designation	Coarse Aggregate	Fine Aggregate Crushed	River
Bulk (Specific Gravity)	C-(127) C-(128)	2.52	2.66	2.63
Apparent (Specific Gravity)	C-(127) C-(128)	2.66	2.69	2.64
Water Absorption (%)	C-(127) C-(128)	0.85	0.71	0.61
Resistance to Degradation of Coarse Aggregate (Los Angeles Test) (Max 30%)	C-(131)	25 %	...	...
Particle Angularity (Min 90%)	D-(5821)	95%	...	...
Content of Clay Lumps and Friable Particles % (Max 3%)	C-(142)	...	0.22	0.33



Fig. 1. Illustrative types of fine aggregates used in this study (a)Crushed aggregate, (b) River aggregate

The gradation of the aggregate was selected with specification limits are shown in Table 2. Ordinary Portland Cement (Tasluja) Type(I) was used as the mineral filler, presented in Table 3. The types of fine aggregates are used in present study are illustrated in Figure 1.

Table 2. Grading of the aggregates selected with specification SCRB (type IIIA) [25] and glass materials used in this study

Sieve Size		(SCRB) Specification Limits (%)	Passed Percent (%) Aggregates and Filler	Passed Percent (%) - Glass Material
(19 mm)	(3/4 ")	(100)	(100)	-
(12.5 mm)	(1/2 ")	(90)-(100)	(95)	-
(9.5 mm)	(3/8 ")	(76)-(90)	(83)	-
(4.75 mm)	(No.4)	(44)-(74)	(59)	(100)
(2.36 mm)	(No.8)	(25)-(58)	(43)	(62)
(300 μm)	(No.50)	(5)-(21)	(13)	(18)
(75 μm)	(No.200)	(4)-(10)	(7)	(3)

Table 3. Physical properties of filler (Ordinary Portland Cement (Tasluja) used in the present study

Property	Results
Specific Gravity	(3.12)
Percent Passing Sieve 0.075μm (No.200)	(95%)

2.2. Asphalt Binder

One type of binder was used in this laboratory investigation, it classified as a performance grade (PG70-16) equivalent to penetration grade of (40/50) [26], which had been supplied from AL-Daurah Oil Refinery located in the (south-west) of Baghdad the Capital of Iraq. The physical properties of binder were listed in Table 4. The actual properties of the asphalt binder were verified in accordance with ASTM specification.

Table 4. Physical characteristics of binder according to (ASTM and SCRB) standards limitations [25, 27]

Test	ASTM Standard	SCRB Limitations	Results
Penetration Test (100g, 5sec at 250°C), (0.1) mm	ASTMD5	(40/50)	45
Softening Point Test(°C)	ASTMD36-76	----	54
Kinematic Viscosity Test (1350°C)	ASTM2170	(>350)	384
Ductility Test (5cm/min at 250°C)	ASTMD113	(>100)	110
Flash Point Test (°C)	ASTMD92	(>232)	291
Specific Gravity Test at (250°C)	ASTMD70	----	1.04

2.3. Waste Glass and Polyethylene Terephthalate

Glass is a non-metallic inorganic material primarily composed of silicates and other minor oxides. Due to its brittle nature, waste glass can be easily crushed into small particles suitable for mixing. The specific gravity of waste crushed glasses (WG) about (2.5 g/cm³), low water absorption (0.3%–0.4%) by weight, high thermal stability up to (700°C) and high angularity (indices 12–13) that promotes interlocking despite its smooth surface [28, 29]. For asphalt mixture applications, particle sizes of (4.75mm) or smaller were suitable, the sieve analysis results for WG are presented in Table 2. Local waste plastic bottles, particularly polyethylene terephthalate (PET) are widely used as additive in asphalt mixture applications due to their favorable mechanical and physical properties. PET additive is a thermoplastic polymer characterized by (durability, high strength and resistance at high temperature). In this study, waste plastic bottle was collected, cleaned, and ground into small particles < (2.36mm) prior to binder incorporation [30]. Polyethylene terephthalate (PET) has a melting point of (260°C) and a specific gravity of (1.56), which are essential for interpreting its thermal behavior and compatibility within the asphalt mix, PET is recognized as one of the most widely recycled plastic wastes materials [31, 32]. The crushed waste glass (WG) and waste polyethylene terephthalate (WPET), were used in this study were illustrated in Figure 2.



Fig. 2. Illustrative types of waste are used in the study (a): Crushed waste glass (WG) and (b): Waste polyethylene terephthalate (WPET)

2.4. Sample and Mix Preparation

In Iraq, the conventional surface layer for asphalt pavement is commonly constructed using HMA. A systematic experimental program was conducted to prepare and evaluate asphalt mix. Initially, the conventional crushed fine aggregate (CFA) was completely replaced with river fine aggregate (RFA) at (100%), and the Marshall properties were determined to identify the optimum mixture (control mix). The optimum binder content (OBC) was determined as (4.7%) for the RFA mixture and (5%) for the CFA mixture by evaluating the mechanical and volumetric response of the RFA and CFA mixtures. As summarized in Table 5. These empirical results governed by SCRB criteria, provide the quantitative basis for establishing the design asphalt binder content for both fine aggregates types. Based on the selected mix design, WG was incorporated as a partial replacement of the RFA fraction passing the (4.75 mm) sieve at three different percentages (10%, 20% and 30%) by weight of total river fine aggregates. In addition, particles of waste polyethylene terephthalate (WPET) were introduced into the asphalt binder at three dosage levels to assess its modifying effect. Marshall specimens were then prepared using standard dimensions and compaction procedures, consistent with those used in the indirect tensile strength (ITS) test, to evaluate the influence of moisture susceptibility performance of the mixtures. In this research, the asphalt binder was heated to (160°C), after which WPET particles were added at the specified percentages by binder weight. The WPET-modified binder was prepared using the wet process and blended with a high-shear mixer at (1100rpm) for (120min) at (160°C) to ensure complete dispersion and homogeneity [30, 32]. Separately, the river aggregates and WG were oven dried to eliminate moisture before WG was incorporated as a partial replacement for the fine aggregate fraction using the dry process. The aggregate–WG blend was then mixed with the PET-modified binder at (160°C)

for at least (3min) or until all particles were uniformly coated and a homogeneous asphalt mixture was obtained [19, 28]. In the indirect tensile strength test and for Marshall test the dimension of samples with a diameter of (102mm), and with a height of (64mm).

Table 5. Marshall design results and OBC determination in the present study

Marshall Properties	CFA- Mixture	RFA- Mixture	SCRB Criteria
Stability- (KN)	10.3	7.2	8 min
Flow - (mm)	3.12	3.78	2 - 4
Air Voids - (%)	4.3	4.0	3 - 5
Binder Content - (%)	5.0	4.7	4-6

2.5. Laboratory Testing Program

The performance of the asphalt mix was evaluated by using standardized laboratory testing methods, included the Marshall test and moisture susceptibility test

2.5.1. Marshall Stability Test

To evaluate characteristics of hot mix asphalt incorporating waste glass, with and without waste Polyethylene terephthalate, Marshall stability test was conducted in accordance to ASTM (D1559). Cylindrical specimens were prepared using a standard compacted hammer and a cylindrical mold. Prior to testing, the samples were conditioned in a water bath at (60°C) for (30minutes) and tested for deformation resistance under a constant loading rate of (50mm/min). This standardized method was employed to determine key performance parameters, thereby providing a reliable assessment of the load-bearing capacity of the asphalt mix [33].

2.5.2. Moisture Susceptibility Test

The present study, moisture susceptibility of the hot asphalt mixtures was investigated by using the tensile strength ratio (TSR) in accordance with (AASHTO: T283) [34]. This method evaluates the resistance of asphalt mixtures to moisture damage by comparing the indirect tensile strength (ITS) before and after a water-conditioning (conditioned specimens (wet case) and unconditioned specimens (dry case)). Cylindrical samples were prepared and compacted to standard dimensions, then divided into two groups unconditioned tested at (25°C) and conditioned were immersed in water bath at (60°C) for (24hours) and subsequently tested at (25°C) to simulate field moisture effects (moisture susceptibility). In this test was performed by applying a compressive load along the specimen diametral plane at a constant deformation rate until its failure, inducing tensile stresses perpendicular to the loading direction by using a digital Marshall testing apparatus presented in Figure 3. The ITS was calculated from the highest stress calculated from the peak load at failure. TSR was determined as the ratio of the average ITS of conditioned samples to that of unconditioned samples. Higher TSR values indicate improved resistance to moisture damage and improved durability of the hot asphalt mix (HMA). The ITS was calculated by using equation1.

$$ITS = \frac{2 P}{\pi t D} \quad (1)$$

In this equation, [ITS] represents the indirect tensile strength (kPa), [P] is Peak Load at failure (KN), [t] is the sample thickness (cm) and [D] is the sample diameter (cm). Moisture susceptibility or the stripping potential of HMA, is determined by using equation2 as the ratio of the average indirect tensile strength of conditioned (wet case) specimen to unconditioned (dry case) specimen.

$$TSR = \frac{ITS_{wet}}{ITS_{dry}} \times 100 \quad (2)$$

In this equation, [TSR] represents the tensile strength ratio (%), [ITS_{wet}] represents the mean indirect tensile strength of the wet-conditioned specimen (kPa) and [ITS_{dry}] represents the mean indirect tensile strength of the dry-unconditioned specimen (kPa) [35].



Fig. 3. Illustrative indirect tensile strength test

3. Results and Discussion

3.1 Marshall Stability

The Marshall stabilities result for both hot mix glass asphalt (HMGA) and WPET-modified mixtures are shown in Table 6, while the comparative trends are graphically presented in Figure 4. The experimental data reveal that the incorporation of waste glass (WG) as a partial replacement for river fine aggregates yields a discernible improvement in Marshall stability compared to the conventional asphalt containing zero WG. Stability values of (HMGA) to comply with Iraqi standards specification limits (SCR/B/R9) for the surface layer, which determined the minimum allowable limit by (8kN) [25]. This improvement is further augmented by the introduction of WPET through the wet process method, where Marshall stability reached a peak value of (10.6kN) at an optimum WPET content of (8%). Further increases in the WPET and WG contents, the hot mix glass asphalt modified led to a slight reduction stability under compressive stresses. The observed increase in stability is due to both rheological and mechanical improvements. The use of angular WG increases the internal friction angle and enhances aggregate interlocking [36, 37]. On the other hand, WPET modification increased stiffness, viscosity and cohesion of the asphalt binder, resulting in improved resistance to rutting and compressive stresses [30, 38]. Furthermore, polymer-modified binders enhanced elastic recovery and reduced temperature susceptibility, which ensures to better structural integrity of the HMGA [39].

Table 6. Results for hot asphalt mix modified by WG and WPET

Sample	Waste Glass Content (%)	(Stability-KN)	(ITS-Kpa) Dry Condition	(ITS-Kpa) Wet Condition	TSR (%)
Control (Unmodified)	0	7.2	1180	770	65
	10	8.0	1250	800	64
	20	8.3	1320	830	63
	30	8.8	1100	650	59
Modified with (WPET 4%)	0	7.4	1260	880	70
	10	8.1	1300	940	72
	20	8.9	1360	1040	76
	30	9.4	1210	850	70
Modified with (WPET 8%)	0	7.8	1280	950	75
	10	8.4	1320	1060	80
	20	9.5	1390	1140	82
	30	10.6	1260	920	73
Modified with (WPET 12%)	0	7.3	1070	790	74
	10	7.9	1150	890	77
	20	8.7	1190	820	70
	30	9.6	1070	690	64

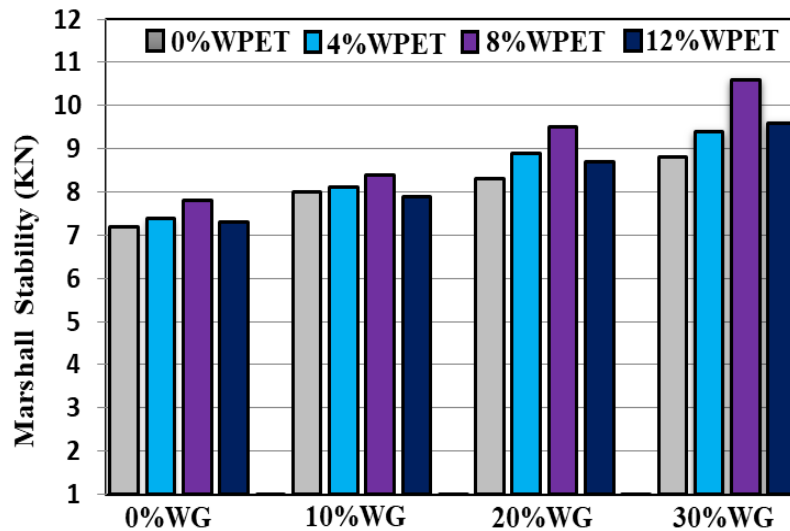


Fig. 4. Effect of WG and WPET on Marshall stability

3.2 Indirect Tensile Strength (ITS)

The results obtained from the indirect tensile strength test in wet and dry conditions are shown in Figures 5 and Figures 6, while the detailed values are summarized in Table 6. Both WG and WPET modification have a significant influence on the moisture resistance and mechanical performance of the asphalt mix. The incorporation of WPET leads to a noticeable improvement in indirect tensile strength (ITS) values compared to the control mixtures (unmodified mixtures) under both dry and wet condition. The experimental results showed that incorporation WPET in HMGA mixtures, caused ITS values to increase about (7%) up to (19%) in dry conditions and about (14%) up to (48%) in wet conditions comparison with non-WPET asphalt mixture samples. As illustrated in the Figures 5 and 6 that the ITS values increase with increasing WG content up to (20%) after which a decrease is observed with (30%) replacement of aggregate. This trend indicates that adding glass content enhances aggregate interlocking, while excessive WG content may weaken the internal structure of mixture due to increased brittleness or reduced cohesion [40].

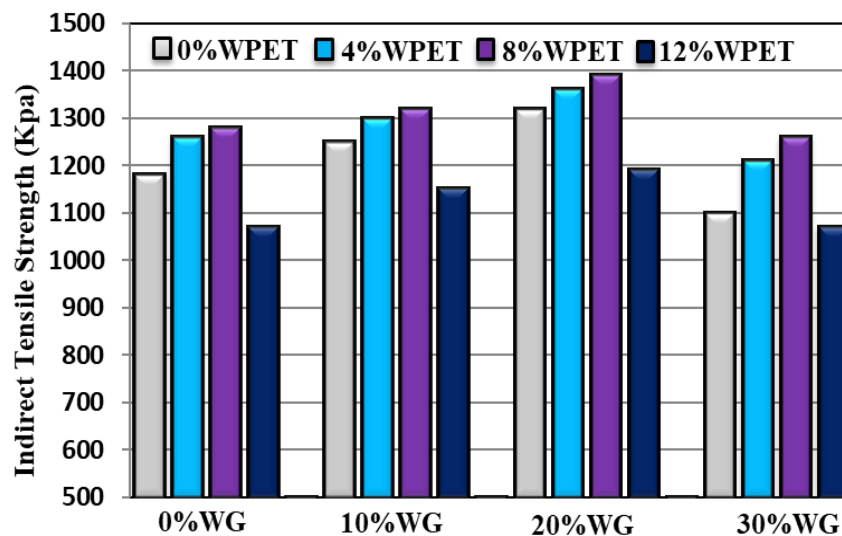


Fig. 5. Effect of WG and WPET on indirect tensile strength (ITS) in the dry condition

A similar behavior is evident under wet conditions, where the resistance to the moisture damage improvement up to (20%) WG and then reduce at higher replacement. Moreover, WPET modification enhances the resistance of mixture to moisture susceptibility (stripping damage), the adding of (8%) WPET showing the best performance of asphalt mixture, particularly at (20%) WG. This indicates that the combination of (8%) WPET and (20%) WG, where the highest ITS values are achieved under both wet and dry conditions and provides the optimum resistance to moisture

susceptibility. The improvement can be attributed to enhanced adhesion and cohesion within the mixture; this suggests an optimal interaction between asphalt binder and surface of the aggregates, which limits asphalt binder stripping and improves durability compared to unmodified mixtures. In contrast, mixtures with (12%) WPET show a reduction in performance, likely due to increased stiffness of the asphalt binder [41].

The tensile strength ratio (TSR) results were presented in Figure 7 and summarized in Table 6. It can be observed that the control mixtures exhibit lower TSR values, ranging from (59–65%), indicating limited resistance to moisture damage. In contrast, the incorporation of WPET-modified mixes exhibit improvement in TSR, confirming the positive role of WPET modification on moisture resistance (durability). As shown in Figure 7 and Table 6, TSR increases with addition WG up to (20%), after which a decline at (30%). This behavior is attributed to improved internal structure at moderate glass content, while higher glass content may reduce adhesion due to the smooth surface and impermeable properties of glass particles making glass-containing mixes particularly susceptible to moisture damage and stripping, as the hydrophilic nature of glass surfaces promotes moisture induced debonding at the asphalt binder aggregate interface [42].

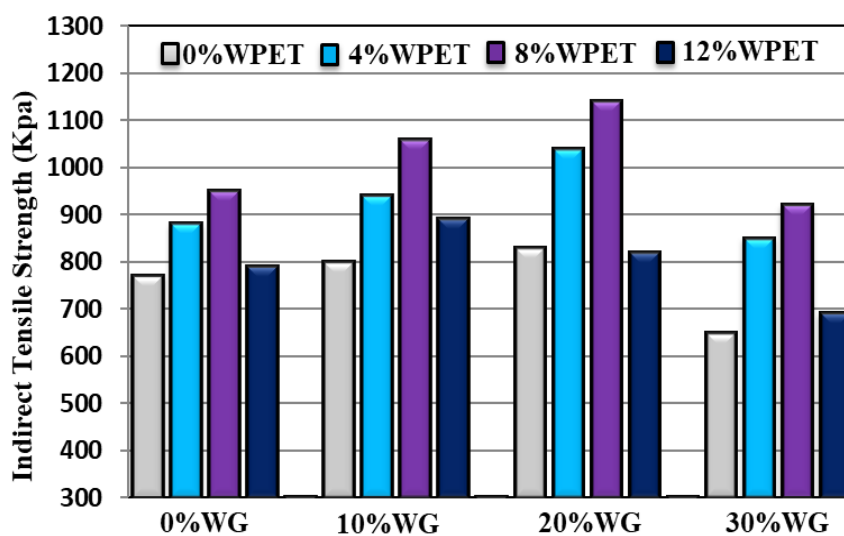


Fig. 6. Effect of WG and WPET on indirect tensile strength (ITS) in the wet condition

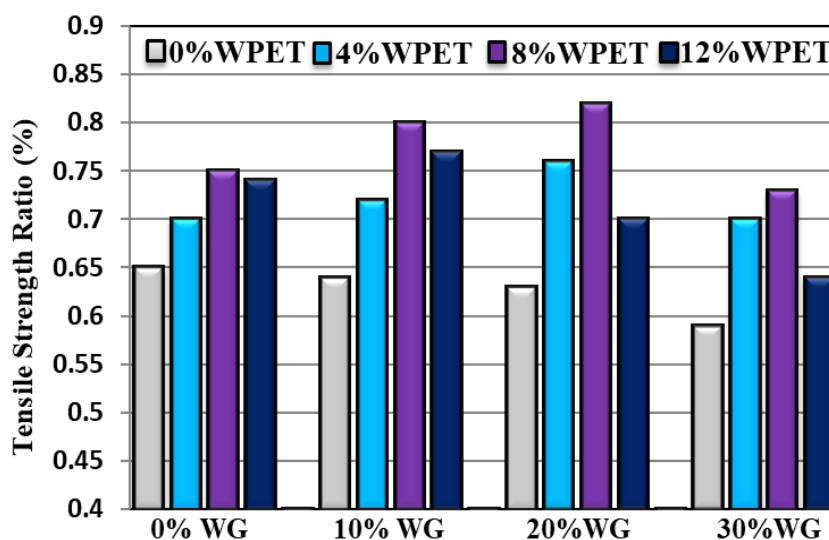


Fig. 7. The tensile strength ratio(TSR) result

Furthermore, the modified mixes, (8%) of WPET exhibit the highest TSR values up to (82%) at (20%) WG, indicating optimal resistance to moisture damage. However, at increasing WPET to (12%) there was reduction in TSR values, which may be to increased asphalt binder stiffness and reduced workability. Overall, the results indicate that combining (20%) WG with (8%) WPET

provides the optimal performance, with most modified mixes approaching or exceeding the minimum TSR (80%). Conversely, a TSR value of (60%) or below is generally considered unacceptable, signifying an asphalt mixture that is susceptible to moisture damage according requirement specified by the asphalt institute, while the (80%) threshold is standard, some agencies accept a minimum TSR values of (70%) based on their experience [43]. The ternary mixture achieved a TSR of (82%) exceeding the (80%) specification limit. This improvement demonstrates that the WPET-modified binder effectively enhanced adhesion and moisture resistance despite the use of river aggregates and WG [20, 32]. No general rule for moisture sensitivity performance with the incorporation of the new additives or modification technology. This demonstrates that when a new asphalt mixture is designed it is important engineering consideration to examine its resistance to moisture damage [44].

4. Conclusions

Based on the results obtained from the stability, indirect tensile strength and tensile strength ratio tests, and as illustrated in Figures (4–7) and Table 6, the following conclusions were drawn:

- The use of crushed waste glass (WG) as a partial replacement for fine aggregate is technically feasible, as the resulting mixtures exhibit mechanical properties comparable to conventional hot mix asphalt. The optimum performance was achieved at a (20%) WG replacement level, where both Marshall stability and indirect tensile strength (ITS) increased. However, increasing the WG content to 30% caused both properties to decline because excessive glass reduces adhesion due to its smooth and nonabsorbent surface, leading to a weaker asphalt binder and aggregate interface.
- The asphalt binder modification by using waste polyethylene terephthalate (WPET) improved the mechanical performance and moisture resistance of the asphalt mixtures. The optimal WPET content was identified at (8%), at which the highest ITS values were recorded under both wet and dry conditions. Moreover, the combined incorporation of (8%) WPET and (20%) WG yielded the best overall performance, indicating that this combination is the optimum mixture composition among those evaluated.
- The tensile strength ratio (TSR) results indicate that the combined use of WPET and WG improves the resistance of asphalt mixtures to moisture damage. TSR values increased with WG content up to (20%) particularly in WPET modified mixtures, demonstrating enhanced adhesion and reduced stripping potential. The maximum TSR value (82%) was achieved at (20%) WG with (8%) WPET, indicating optimal moisture damage resistance. However, increasing the WPET content beyond the optimum level (12%) leads to a slight reduction in both ITS and TSR values. This decline is attributed to excessive asphalt binder stiffness and reduced workability.
- The integrated use of locally available river aggregate with waste glass (WG) and waste polyethylene terephthalate (WPET) is technically feasible, providing enhanced mechanical performance and promoting waste recycling. Future life cycle assessment (LCA) and life cycle cost analysis (LCCA) are recommended to quantify the full environmental and economic benefits of this sustainable pavement system.
- Future studies are recommended to investigate the rutting resistance, fatigue cracking, low-temperature cracking and aging performance of modified mixtures to complement the findings of this work.

References

- [1] Saradha P. Properties and development of industrial waste-based artificial aggregate for sustainable construction: a systematic review. *Frontiers in Built Environment*.12:1746167. <https://doi.org/10.3389/fbuil.2026.1746167>
- [2] Pittman H. Highlands and lowlands: the Interaction of communities. *Expedition: The magazine of the University of Pennsylvania*. 2018;60(1):69-75.
- [3] Huang Y, Bird RN, Heidrich O. A review of the use of recycled solid waste materials in asphalt pavements. *Resources, conservation and recycling*. 2007;52(1):58-73. <https://doi.org/10.1016/j.resconrec.2007.02.002>

- [4] Darshanala A, Kandasamy S, Avudaiaippan S. Recycled fine aggregate as a sustainable alternative to natural sand: challenges, enhancement strategies, and performance evaluation. *Res. Eng. Struct. Mater.*, 2026;12(1):703-717.
- [5] Ibraheem HI, Al-Mosawe H, editors. The use of sustainable plastic polymer in asphalt mixtures: a review. 2nd International Conference on Emerging Trends and Applications in Artificial Intelligence (ICETAI 2024); 2024: IET. <https://doi.org/10.1049/icp.2025.0141>
- [6] Albayati A, Al-Mosawe H, Sukhija M, Naidu ANP. Appraising the synergistic use of recycled asphalt pavement and recycled concrete aggregate for the production of sustainable asphalt concrete. *Case Studies in Construction Materials*. 2023;19:e02237. <https://doi.org/10.1016/j.cscm.2023.e02237>
- [7] Khan MNN, Saha AK, Sarker PK. Reuse of waste glass as a supplementary binder and aggregate for sustainable cement-based construction materials: A review. *Journal of Building Engineering*. 2020;28:101052. <https://doi.org/10.1016/j.jobe.2019.101052>
- [8] Behbahani H, Hamed GH, Moghaddam GVN, Nikookar M. Improving the moisture performance of hot mix glass asphalt by high-density polyethylene as an asphalt binder modifier. *International Journal of Sustainable Building Technology and Urban Development*. 2019;10(4):184-93.
- [9] Hameed MA. Evaluation asphalt concrete mixture with plastic waste and crumb rubber as fine aggregate replacement in Iraq. *Res. Eng. Struct. Mater.*, 2026;12(3):1863-1874.
- [10] Wu X, Jusli E, Anggraini V, Jaya RP, Zhang X. Performance and environmental impacts of waste plastic-modified asphalt pavement: a comprehensive review. *Cleaner Materials*. 2025:100357. <https://doi.org/10.1016/j.clema.2025.100357>
- [11] De Bortoli A. Understanding the environmental impacts of virgin aggregates: Critical literature review and primary comprehensive life cycle assessments. *Journal of Cleaner Production*. 2023;415:137629. <https://doi.org/10.1016/j.jclepro.2023.137629>
- [12] Gedik A. An exploration into the utilization of recycled waste glass as a surrogate powder to crushed stone dust in asphalt pavement construction. *Construction and Building Materials*. 2021;300:123980. <https://doi.org/10.1016/j.conbuildmat.2021.123980>
- [13] Hameed MA, Mustafa S, Dulaimi A, editors. The effect of styrene butadiene styrene on hot asphalt mixes in Iraq. AIP Conference Proceedings; 2023: AIP Publishing LLC. <https://doi.org/10.1063/5.0163550>
- [14] Cardoso J, Ferreira A, Almeida A, Santos J. Incorporation of plastic waste into road pavements: A systematic literature review on the fatigue and rutting performances. *Construction and Building Materials*. 2023;407:133441. <https://doi.org/10.1016/j.conbuildmat.2023.133441>
- [15] Mustafa S, Hameed MA, Dulaimi A. Evaluation of the properties of modified local asphalt binder by using styrene butadiene rubber (SBR) or low-density polyethylene (LDPE). *Scientific Review Engineering and Environmental Studies (SREES)*. 2022;31(3):190-202. <https://doi.org/10.22630/srees.3375>
- [16] Ghabchi R, Dharmarathna CP, Mihandoust M. Feasibility of using micronized recycled Polyethylene Terephthalate (PET) as an asphalt binder additive: A laboratory study. *Construction and Building Materials*. 2021;292:123377. <https://doi.org/10.1016/j.conbuildmat.2021.123377>
- [17] dos Santos Ferreira JW, Marroquin JFR, Felix JF, Farias MM, Casagrande MDT. The feasibility of recycled micro polyethylene terephthalate (PET) replacing natural sand in hot-mix asphalt. *Construction and Building Materials*. 2022;330:127276. <https://doi.org/10.1016/j.conbuildmat.2022.127276>
- [18] Al-Tameemi AF, Al-Allaf MH, Al-Fayyadh ZT, Daud SA, Wang Y. Performance evaluation of hot mix asphalt utilising waste and byproduct materials as reactive powders. *Innovative Infrastructure Solutions*. 2025;10(9):429. <https://doi.org/10.1007/s41062-025-02222-4>
- [19] Jony HH, Al-Rubaie MF, Jahad IY. The effect of using glass powder filler on hot asphalt concrete mixtures properties. *Eng Technol J*. 2011;29(1):44-57. <https://doi.org/10.30684/etj.29.1.4>
- [20] Almuhamdi AD, Muhmood AA, Salih AO. Effects of crushed glass waste as a fine aggregate on properties of hot asphalt mixture. *Tikrit Journal of Engineering Sciences*. 2021;28(3):129-45. <https://doi.org/10.25130/tjes.28.3.10>
- [21] Arya A, Shukla R. Trash talk: A review of evolving waste management paradigms. *International Journal of Computational and Experimental Science and Engineering*. 2025;11(1). <https://doi.org/10.22399/ijcesen.891>
- [22] Juenger M, Winnefeld F, Provis JL, Ideker J. Advances in alternative cementitious binders. *Cement and concrete research*. 2011;41(12):1232-43. <https://doi.org/10.1016/j.cemconres.2010.11.012>
- [23] Vignesh R, Rahim AA. Emerging eco-friendly fiber-reinforced concrete with shaped synthetic aggregates using Taguchi grey relational analysis and utility concept. *Construction and Building Materials*. 2024;447:138039. <https://doi.org/10.1016/j.conbuildmat.2024.138039>
- [24] Madhu K, Jb AR, Shibuc N, Gd V. Assessing recycled asphalt pavement: Impact of waste cooking oil and waste engine oil as rejuvenators on mechanical properties. *Res. Eng. Struct. Mater.*, 2025; 11(2): 663-678.
- [25] SCRB/R9. General specification for roads and bridges, Section R/9, hot-mix asphalt concrete pavement, Baghdad, Iraq2003.

- [26] Falih T, Abed AH, editors. Enhancing shear strength of bonding materials used for asphalt concrete and composite pavement layers. E3S Web of Conferences; 2023: EDP Sciences. <https://doi.org/10.1051/e3sconf/202342703002>
- [27] ASTMStandards. Roads and Paving Materialsin: Annual Book of the American Society for Testing and Materials. USA2004.
- [28] Su N, Chen J. Engineering properties of asphalt concrete made with recycled glass. Resources, conservation and recycling. 2002;35(4):259-74. [https://doi.org/10.1016/S0921-3449\(02\)00007-1](https://doi.org/10.1016/S0921-3449(02)00007-1)
- [29] Saradha P, Meena T. Properties and development of industrial waste-based artificial aggregate for sustainable construction: a systematic review. Frontiers in Built Environment. 2026;12:1746167. <https://doi.org/10.3389/fbuil.2026.1746167>
- [30] Mashaan NS, Chegenizadeh A, Nikraz H, Rezagholilou A. Investigating the engineering properties of asphalt binder modified with waste plastic polymer. Ain Shams Engineering Journal. 2021;12(2):1569-74. <https://doi.org/10.1016/j.asej.2020.08.035>
- [31] Cuadri A, Partal P, Ahmad N, Grenfell J, Airey G. Chemically modified bitumens with enhanced rheology and adhesion properties to siliceous aggregates. Construction and Building Materials. 2015;93:766-74. <https://doi.org/10.1016/j.conbuildmat.2015.05.098>
- [32] Agha N, Hussain A, Ali AS, Qiu Y. Performance evaluation of hot mix asphalt (HMA) containing polyethylene terephthalate (PET) using wet and dry mixing techniques. Polymers. 2023;15(5):1211. <https://doi.org/10.3390/polym15051211>
- [33] ASTM. ASTM D1559-89, Standard test method for resistance to plastic flow of bituminous mixtures using marshall apparatus, Annual Book of ASTM Standards USA1995.
- [34] AASHTO T. 283; Resistance of compacted asphalt mixtures to moisture-induced damage. American Association of State Highway and Transportation Officials (AASHTO): Washington, DC, USA. 2003;2:1-6.
- [35] Pasandín AR, Pérez I, Ramírez A, Cano MM. Moisture damage resistance of hot-mix asphalt made with paper industry wastes as filler. Journal of cleaner production. 2016;112:853-62. <https://doi.org/10.1016/j.jclepro.2015.06.016>
- [36] Salem ZTA, Khedawi TS, Baker MB, Abende R. Effect of waste glass on properties of asphalt concrete mixtures. Jordan Journal of Civil Engineering. 2017;11(1).
- [37] Lachance-Tremblay É, Vaillancourt M, Perraton D. Evaluation of the impact of recycled glass on asphalt mixture performances. Road Materials and Pavement Design. 2016;17(3):600-18. <https://doi.org/10.1080/14680629.2015.1103778>
- [38] Ameri M, Nasr D. Performance properties of devulcanized waste PET modified asphalt mixtures. Petroleum Science and Technology. 2017;35(1):99-104. <https://doi.org/10.1080/10916466.2016.1251457>
- [39] Leng Z, Sreeram A, Padhan RK, Tan Z. Value-added application of waste PET based additives in bituminous mixtures containing high percentage of reclaimed asphalt pavement (RAP). Journal of cleaner Production. 2018;196:615-25. <https://doi.org/10.1016/j.jclepro.2018.06.119>
- [40] Hamid A, Ahmad N, Zaidi B, Khalid RA, Hafeez I, Hussain J, et al. GlasSphalt: A borosilicate based Sustainable Engineering material for Asphalt pavements. Sustainability. 2023;15(5):4277. <https://doi.org/10.3390/su15054277>
- [41] Singh P, Tophel A, Swamy AK. Properties of asphalt binder and asphalt concrete containing waste polyethylene. Petroleum Science and Technology. 2017;35(5):495-500. <https://doi.org/10.1080/10916466.2016.1265559>
- [42] Arabani M, Mahdi Nazar M, Hamedi GH. Evaluating the effect of liquid antistrip additives on moisture sensitivity of glassphalt. AUT Journal of Civil Engineering. 2021;5(3):507-20.
- [43] AsphaltInstitute. MS-2 Asphalt mix design methods. 7th, editor: Asphalt Institute; 2014. 197 p.
- [44] Dr Robert N. Hunter ASaPJR. The Shell Bitumen Handbook Sixth edition2015. <https://doi.org/10.1680/tsbh.58378>

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Comparative performance-based analysis of sustainable soil stabilizers via secondary experimental data: RCA, RAC, fly ash, and nano MgO

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Abstract

In this work, the experimental data of four eco-friendly soil stabilizers—recycled concrete aggregate (RCA), recycled aggregate concrete (RAC), Class F fly ash, and nano-MgO— used for low plasticity clay (CL) are synthesized in a comparative manner. A well-organized secondary database was developed for OMC and MDD, UCS, soaked CBR, ultimate bearing capacity, and settlement. Differences between studies were controlled by limiting the data to similar test conditions and calculating changes from each source study's respective untreated control. OMC was raised by the fly ash from 17.2% to 20.2%, while MDD was decreased from 1.80 g/cm³ (3% dosage) to 1.69 g/cm³ (9% dosage). RCA and RAC yielded maximum dry density (MDD) of 1.86 and 1.89 g/cm³, respectively. At 28 days, nano-MgO exhibited the highest UCS (520 kPa) and soaked CBR (11.6%) at 1% dosage. Fly ash at 6% led to the highest ultimate bearing capacity for the non-nano cases (3,434 kPa), while the highest bearing capacity (4,120 kPa) and the lowest settlement (3.8 mm) were reached by nano-MgO. Based on UCS, CBR, bearing capacity, and settlement control, an equal-weight min-max index placed the tested conditions in this order: nano-MgO, RAC, fly ash at 9%, fly ash at 6%, and RCA. Pearson *r* for CBR versus UCS was 0.98 indicating very strong correlation, while *r* for MDD versus ultimate bearing capacity was -0.02 indicating no clear linear relation between ultimate bearing capacity and MDD. The conclusions are limited to short-term laboratory data for CL soils; cost, life-cycle impacts, durability, and field performance require separate evaluation.

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1. Introduction

A number of factors that control soil bearing capacity, and the amount of settlement soil can undergo without damaging the pavement layer are key factors defining which shallow foundations are safe and serviceable. Subgrade soils beneath pavements and slabs may be weak and fail to supply enough load carrying capacity and/or deformation resistance, leading to early pavement distress and higher maintenance needs [1]. Traditional stabilization methods (e.g., with cement or lime) can achieve higher strength and stiffness but may also increase embodied carbon and material consumption, and material consumption; furthermore, excessive cement content can lead to brittle behavior [2]. These constraints have led to considering other stabilizers that could enhance bearing capacity and deformation stability, and/or decrease the reliance on virgin binders [3]. Recycled construction materials are a common consideration for geotechnical enhancement. Construction and demolition waste derived recycled concrete aggregate (RCA) can be used to enhance the performance of clayey soils because of angular-particle interlocking and further contributions of frictional resistance [4]. Recycled Aggregate Concrete (RAC) in which the recycled

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aggregate still has adhered mortar on its surface may offer enhanced bonding and stiffness, geopolymer-stabilized recycled aggregates also show promise for layer applications in pavements/road bases [5]. These products enable circular applications of construction waste, though their performance is influenced by the quality of aggregates, the dosage, and test condition. Fly ash is also an industrial by-product extensively investigated as a soil stabilizer due to the fact that its pozzolanic reactions can provide increment in strength with time [6, 7]. It also has a dosage-dependent impact on compaction: the relatively low specific gravity and high surface area of Class F fly ash tends to increase water demand and decrease dry density for larger contents. Fly ash can also decrease cement requirements and enhance subgrade performance but an appropriate curing time should be taken into account as well as the variability of the chemical composition of the fly ash [8, 9].

Nano-MgO is intriguing as the interparticle contacts and pore structure at the nanoscale can be altered with a small dosage of this material. Studies available on nano-MgO-treated clay indicate enhancement in bearing capacity and settlement response [20]. In the studies of nano-clay, nano-silica and other nanoparticles, references are available to provide mechanistic insights [10–15, 21, 22, 26], however, these studies do not address nano-MgO within the present database context. Reported nanomaterial effects are also still sensitive to dosage, curing, and moisture conditions; thus, results from contaminated or gypseous soils or soil that exhibits a very high plasticity cannot be transferred to a low-plasticity clay unambiguously. Although the individual stabilizers have been widely studied, comparative assessments of recycled aggregates, industrial by-products and nano-MgO within the same analytical context are still scarce. Therefore, the aim of this study is to collate comparable previously published laboratory results for RCA, RAC, fly ash, and nano-MgO in a low-plasticity clay and to contrast their influences on compaction, UCS, soaked CBR, ultimate bearing capacity, and settlement. The review is explicitly a synthesis of previously collected data and does not involve any new laboratory investigations. This study answers the following research questions:

- To what extent do RCA, RAC, Class F fly ash and nano-MgO enhance the strength, bearing capacity and settlement behavior of low-plasticity clay?
- What is the performance ranking when UCS, soaked CBR, ultimate bearing capacity, and settlement controls are combined in an understandable equal-weight index?
- Within the confines of the existing secondary data set, can a relationship between stabilizer application rates and any of the selected engineering parameters be established?

2. Methodology

2.1. Data Collection and Synthesis Process

Instead of performing new experiments, this work adopted a rounded, secondary-data synthesis investigation approach within the boundaries of the-peer reviewed evidence base. The present source set of literature was retrospectively registered by a focused search strategy including the terms “low-plasticity clay” or “CL soil” with “fly ash,” “recycled concrete aggregate,” “recycled aggregate concrete,” “nano-MgO,” “UCS,” “CBR,” “bearing capacity,” and “settlement.” Potential studies were screened for relevance to material, soil type, test condition, and reported engineering parameter. As exact initial counts for retrieval and exclusion were not kept during the initial database build, this work is not presented as a PRISMA systematic review; the limitation is explicitly acknowledged. Quantitative values were extracted from studies of fly ash stabilization [6, 7, 23, 25, 30, 31], RCA and RAC stabilization [5, 18, 19, 24, 27–29], and nano-MgO-treated clay [20]. References concerning nano-clay, nano-silica, or broad nanomaterial reviews [10–15, 21, 22, 26] were used only to support mechanistic discussion and were not treated as nano-MgO observations. Extracted fields included soil classification, stabilizer type and dosage, test standard or procedure, curing period, OMC, MDD, UCS, soaked CBR, ultimate bearing capacity, settlement, and the corresponding untreated-control value.

Eligibility was restricted to low-plasticity clay (CL) or closely comparable cohesive fine-grained soils. Compaction data were limited to standard Proctor energy (ASTM D698 or the comparable IS

2720 Part 7 procedure), CBR values to soaked conditions, UCS values to 7, 14, and 28-days curing periods, and load–settlement values to controlled model-footing or plate-load tests. To reduce cross-study baseline differences, each stabilized value was also expressed relative to the untreated specimen reported in the same source: $\text{change (\%)} = [(\text{stabilized value} - \text{untreated value}) / \text{untreated value}] \times 100$. Raw values at comparable conditions were retained in the summary tables. Replicate-level observations were generally unavailable; consequently, pooled standard deviations and formal inferential tests were not calculated. Missing parameters were treated as data gaps rather than imputed.

2.2. Materials and Engineering Parameters

The reference material was natural low-plasticity clay (CL). The evaluated stabilizers were Class F fly ash, RCA, RAC, and nano-MgO. The control properties used for comparison were Atterberg limits, specific gravity, OMC, MDD, UCS, soaked CBR, ultimate bearing capacity, and settlement. Table 1 summarizes the representative properties of the untreated reference soil in the synthesized database.

Table 1. Representative physical and mechanical properties of the untreated low-plasticity clay (CL) in the synthesized database

Property	Value
Liquid Limit (%)	38–42
Plasticity Index (%)	12–16
Specific Gravity	2.65–2.70
OMC (%)	17.2
MDD (g/cm ³)	1.78
UCS (kPa)	132–145
Soaked CBR (%)	2.1

Fly ash observations represented mainly low-calcium Class F ash (ASTM C618) added at 3%, 6%, and 9% of dry soil mass. These conditions provide compaction, curing-dependent UCS, soaked CBR, bearing-capacity, and settlement values over the dosage range most frequently reported in the selected sources [6, 23, 25]. RCA and RAC observations were represented at approximately 10% dry-mass replacement. RCA consisted of processed construction and demolition concrete, whereas RAC retained adhered cementitious mortar that may contribute to bonding. Nano-MgO observations were represented at 1% dry-soil mass [20]. These dosage levels were not assumed to be universally optimal; they represent the comparable and commonly reported conditions available in the selected source datasets.

Table 2. Characterization of the representative source datasets used in the secondary synthesis

Parameter	Material condition	Representative datasets (n)	Range	Principal references
OMC (%)	Fly ash (3–9%)	3	18.4–20.2	[6,23,25]
MDD (g/cm ³)	Fly ash (3–9%)	3	1.69–1.80	[6,23,25]
OMC (%)	RCA/RAC (10%)	4	18.1–18.3	[5,18,19,24]
MDD (g/cm ³)	RCA/RAC (10%)	4	1.86–1.89	[5,18,19,24]
UCS, 28 d (kPa)	All stabilized conditions	5	330–520	[6,18–20,23]
Soaked CBR (%)	All stabilized conditions	5	8.4–11.6	[6,18–20,23]
Ultimate bearing capacity (kPa)	All stabilized conditions	5	2,550–4,120	[6,18–20,23]
Settlement (mm)	All stabilized conditions	5	3.8–6.8	[6,18–20,23]

Note: n is the number of representative source datasets/conditions available for the given comparison. Replicate level data were not available and pooled standard deviations were not calculated.

Table 2 represents the characterization of the representative source datasets used in the secondary synthesis performance was evaluated using the values reported in Tables 2-4. Each stabilized

condition was scored separately for four criteria: 28-days UCS, soaked CBR, ultimate bearing capacity, and settlement control. For the three benefit criteria, min-max score = $(x - x_{\min}) / (x_{\max} - x_{\min})$; for settlement, score = $(x_{\max} - x) / (x_{\max} - x_{\min})$. The equal-weight composite index was the arithmetic mean of the four dimensionless scores. 6% and 9% Fly ash were treated as two separate treatments, and the material-level hierarchy used the maximum of its two composite scores. The index is an open descriptive ranking, not a formal AHP/TOPSIS decision model, and does not consider dosage, cost, carbon footprint, availability, or long-term durability.

3. Results

3.1. Compaction Characteristics

Compaction behavior was compared using optimum moisture content (OMC) and maximum dry density (MDD). Table 3 summarizes the untreated and stabilized conditions. The maximum dry density (MDD) and optimum moisture content (OMC) of the untreated soil were 1.78 g/cm³ and 17.2%, respectively. Fly ash raised OMC incrementally up to 20.2% at dosage level of 9%. MDD showed a minor increase up to 1.80 g/cm³ at 3% fly ash, followed by a decrease to 1.74 g/cm³ at 6% and 1.69 g/cm³ at 9% fly ash, which is in line with the reduced specific gravity and elevated water demand of the fines. RCA and RAC led to increases in MDD to 1.86 and 1.89 g/cm³, which equates to relative improvements of 4.5% and 6.2%, respectively; this pattern concurs with granular packing and angular-particle interlocking. Up to a dosage of 1%, nano-MgO raised the MDD to 1.84 g/cm³ (3.4%) but increased OMC only from 17.2% to 17.5%. These statements are qualitative, since the secondary dataset does not include replicate-level variances that would allow for statistical testing of significance.

Table 3. Compaction characteristics of low-plasticity clay stabilized with RCA, RAC, Class F fly ash, and nano-MgO

Stabilizer Type	Stabilizer Content (%)	OMC (%)	MDD (g/cm ³)
Untreated soil	0	17.2	1.78
RCA	10	18.1	1.86
RAC	10	18.3	1.89
Fly ash	3	18.4	1.80
Fly ash	6	19.8	1.74
Fly ash	9	20.2	1.69
Nano-MgO	1.0	17.5	1.84

3.2. Mechanical Strength and Bearing Performance

3.2.1 Unconfined Compressive Strength (UCS)

The untreated soil had UCS values of 132, 138, and 145 kPa at 7, 14, and 28 days, respectively. Table 4 summarizes the curing-dependent strength development of all conditions. All stabilizers increased UCS relative to the untreated soil. With 6% fly ash, UCS increased from 285 kPa at 7 days to 410 kPa at 28 days; at 9% fly ash, the corresponding values were 300 and 430 kPa. The time-dependent gain is consistent with progressive pozzolanic reactions reported for Class F fly ash [23].

Table 4. Unconfined compressive strength of untreated and stabilized soil at different curing periods

Stabilizer Type	Stabilizer Content (%)	UCS at 7 days (kPa)	UCS at 14 days (kPa)	UCS at 28 days (kPa)
Untreated soil	0	132	138	145
RCA	10	240	290	330
RAC	10	310	390	460
Fly ash	6	285	350	410
Fly ash	9	300	380	430
Nano-MgO	1.0	420	480	520

RCA increased from 240 to 330 kPa between 7 and 28 days, whereas RAC increased from 310 to 460 kPa. The greater RAC values may reflect residual cementitious mortar, but this mechanism is inferred from the source literature [5, 29] rather than verified by microstructural testing in the present synthesis. Nano-MgO produced the highest values, increasing from 420 kPa at 7 days to 520 kPa at 28 days. Figure 1 compares the 28-days UCS values.

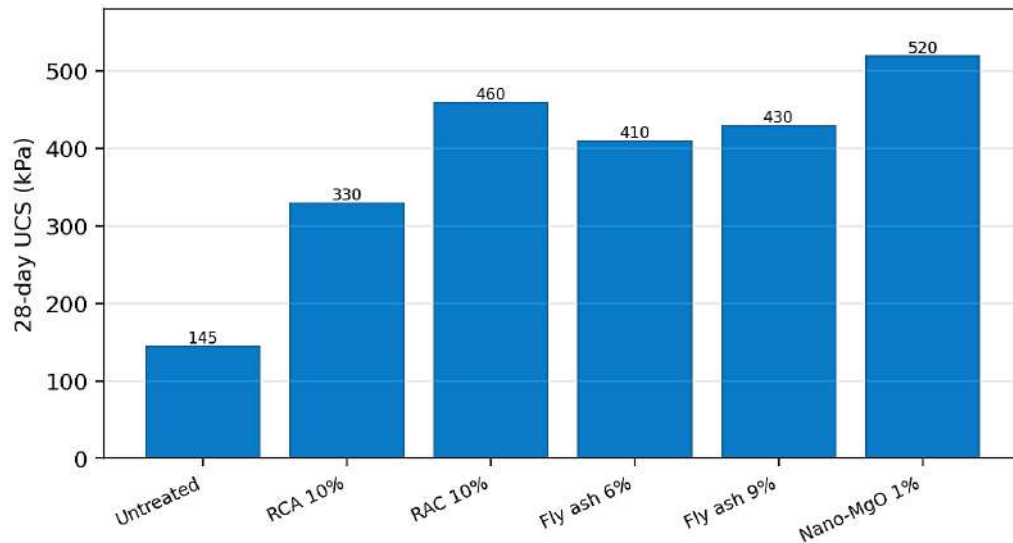


Fig. 1. Comparison of unconfined compressive strength at 28 days for untreated and stabilized soils

3.2.2 California Bearing Ratio (CBR)

The untreated soaked CBR was 2.1%. Fly ash produced values of 8.9% at 6% dosage and 10.6% at 9% dosage. RCA and RAC produced soaked CBR values of 8.4% and 10.5%, equivalent to 4.0 and 5.0 times the untreated value. Nano-MgO produced the highest soaked CBR, 11.6% at 1% dosage, or approximately 3.5 times the untreated value. Figure 2 presents all soaked CBR values. The mechanistic explanations are literature-based and should not be interpreted as direct microstructural observations from this secondary dataset.

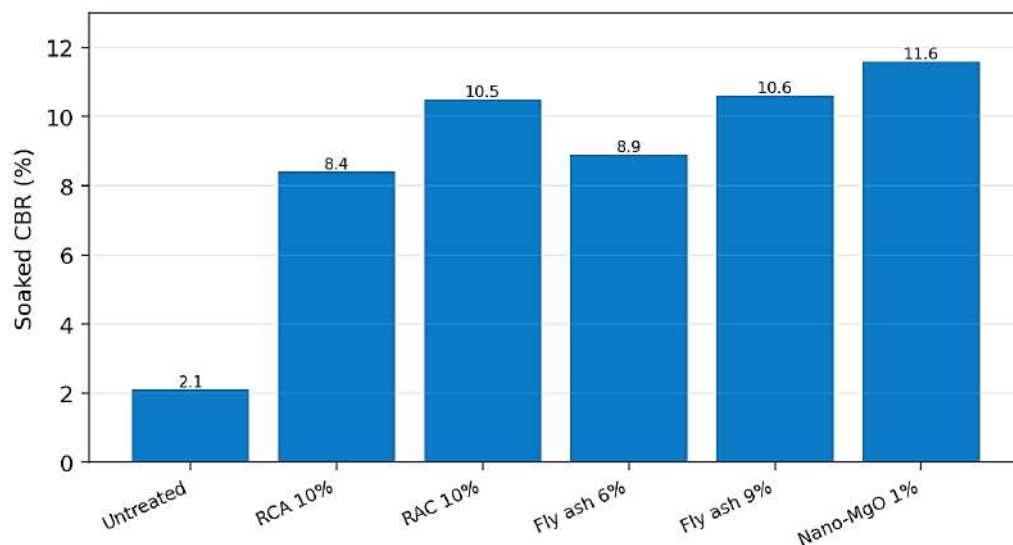


Fig. 2. Soaked California bearing ratio values for untreated and stabilized soils

3.3. Load-Settlement Behavior

Published model-footing and bearing-capacity data were used to compare the load-settlement response. Table 5 reports ultimate bearing capacity and settlement at a common applied stress of

5 kg/cm² (approximately 490 kPa). Standardizing settlement to this stress permits direct comparison of the reported single-point settlement values, but it does not reproduce complete load–settlement curves. The untreated soil had an ultimate bearing capacity of approximately 981 kPa and a settlement of 12.5 mm at the common applied stress. Figure 3 compares the ultimate bearing-capacity values.

Table 5. Ultimate bearing capacity and settlement of untreated and stabilized low-plasticity clay

Stabilizer Type	Stabilizer Content (%)	CBR (%)	Ultimate Bearing Capacity (kPa)	Settlement at Applied Stress of 5 kg/cm ² (mm)
Untreated soil	0	2.1	981	12.5
RCA	10	8.4	2,550	6.8
RAC	10	10.5	2,943	5.4
Fly ash	6	8.9	3,434	6.2
Fly ash	9	10.6	3,336	6.0
Nano-MgO	1.0	11.6	4,120	3.8

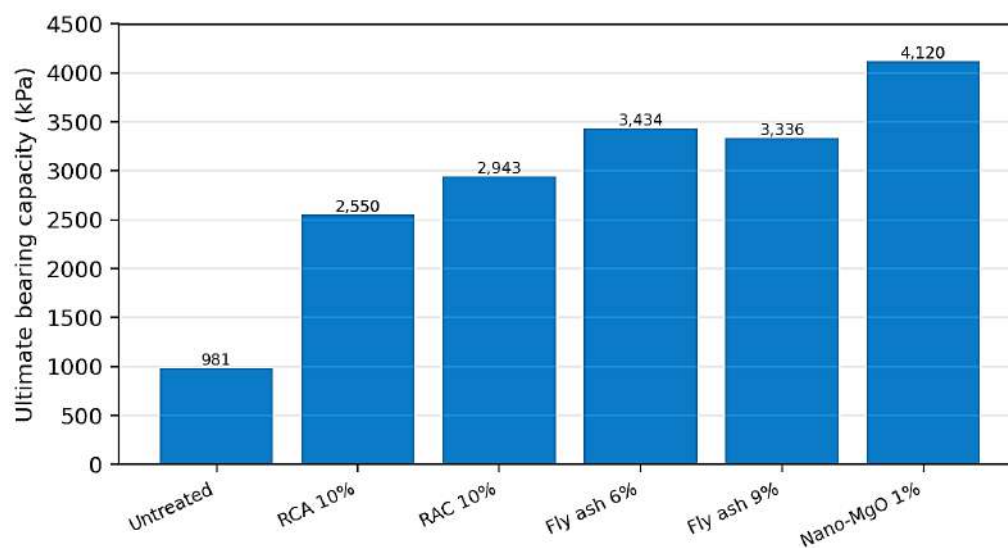


Fig. 3. Ultimate bearing capacity of untreated and stabilized soils obtained from model footing tests

Nano-MgO produced the highest ultimate bearing capacity (4,120 kPa) and the lowest settlement (3.8 mm). Fly ash at 6% produced 3,434 kPa, which exceeded the values for RAC (2,943 kPa) and RCA (2,550 kPa) in this specific parameter. RAC nevertheless provided lower settlement (5.4 mm) than fly ash at 6% (6.2 mm), fly ash at 9% (6.0 mm), and RCA (6.8 mm). Thus, the bearing-capacity result alone does not determine the overall performance hierarchy. Only one settlement value at a common stress was available for each condition. Consequently, curve stiffness, yield behavior, and ductile–brittle response was not inferred. Complete load–settlement curves should be reported in future source datasets and field validation studies.

3.4 Summary of Comparative Performance

The equal-weight min–max procedure described in Section 2.2 generated the scores listed in Table 6. Nano-MgO received the maximum score for all four criteria and therefore ranked first. RAC ranked second (composite index 0.514), narrowly ahead of fly ash at 9% (0.495). Fly ash at 6% ranked fourth (0.335), and RCA ranked fifth (0.000) within the tested set. At the material level, the resulting hierarchy was nano-MgO, RAC, fly ash, and RCA. This hierarchy reflects only the four measured short-term mechanical criteria. The normalized profiles are shown in Figure 4. Across the untreated and stabilized conditions, 28-days UCS and soaked CBR exhibited a strong positive descriptive association (Pearson $r = 0.98$; $n = 6$; Figure 5). By contrast, MDD and ultimate bearing capacity showed no clear linear association ($r = -0.02$; $n = 6$; Figure 6), indicating that density alone did not explain the bearing-capacity differences in this small dataset. These correlations are

exploratory and should not be interpreted as inferential evidence because of the limited sample size and cross-study heterogeneity.

Table 6. Equal-weight normalized performance scores for the stabilized conditions

Condition	UCS score	CBR score	Bearing-capacity score	Settlement-control score	Composite index	Rank
RCA (10%)	0.000	0.000	0.000	0.000	0.000	5
RAC (10%)	0.684	0.656	0.250	0.467	0.514	2
Fly ash (6%)	0.421	0.156	0.563	0.200	0.335	4
Fly ash (9%)	0.526	0.688	0.501	0.267	0.495	3
Nano-MgO (1%)	1.000	1.000	1.000	1.000	1.000	1

Note: Benefit criteria were normalized as $(x - x_{min})/(x_{max} - x_{min})$; settlement control was normalized as $(x_{max} - x)/(x_{max} - x_{min})$. All four criteria received equal weight.

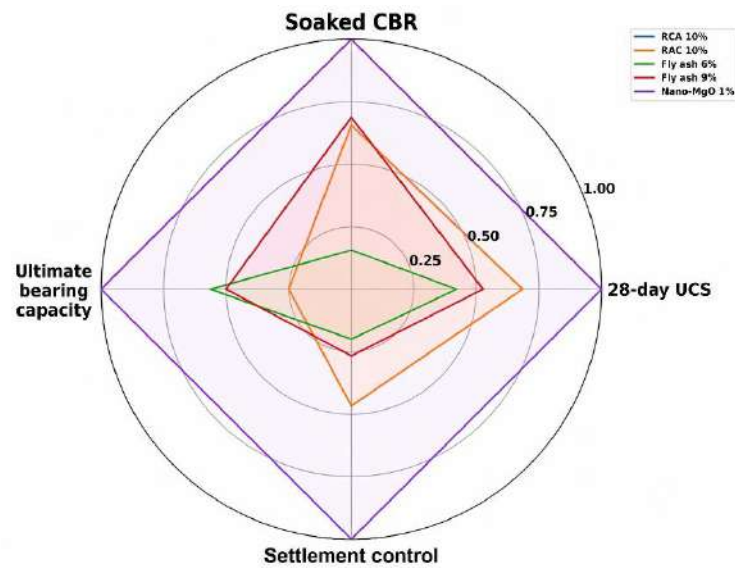


Fig. 4. Normalized performance profiles for the five stabilized conditions

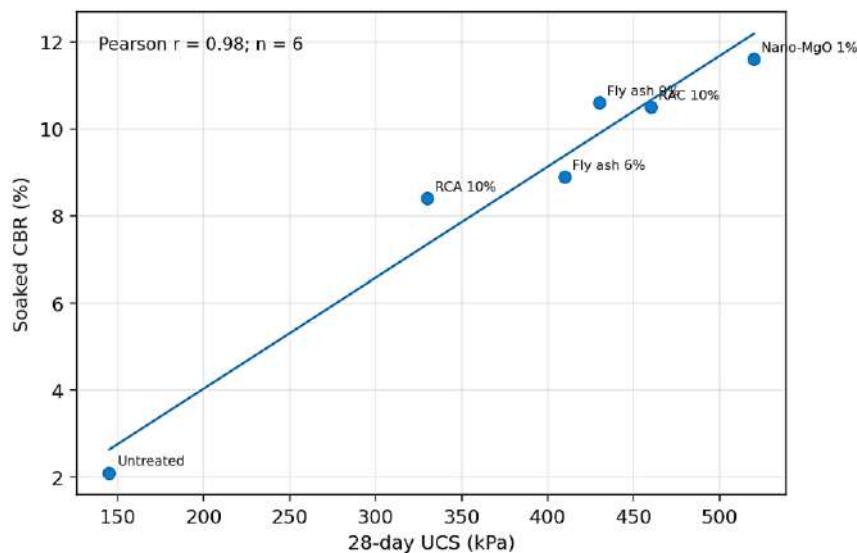


Fig. 5. Descriptive relationship between 28-days UCS and soaked CBR (Pearson $r = 0.98$; $n = 6$)

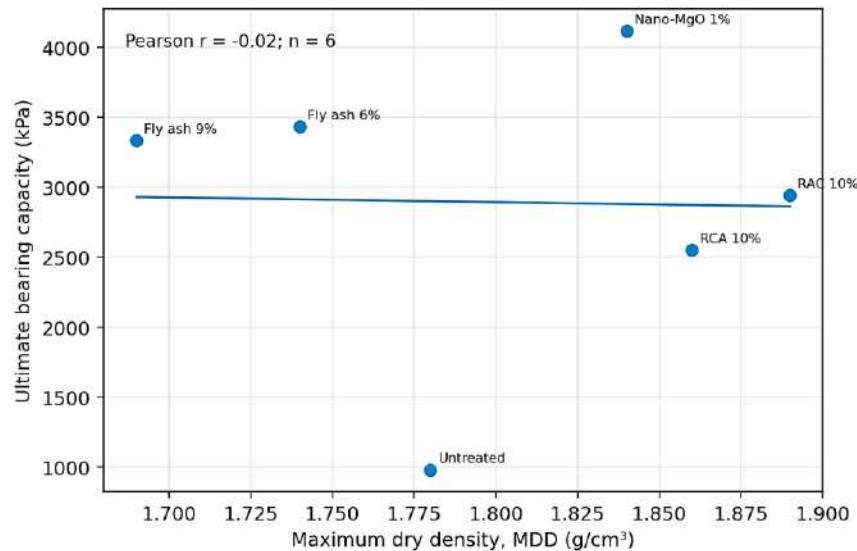


Fig. 6. Descriptive relationship between MDD and ultimate bearing capacity (Pearson $r = -0.02$; $n = 6$)

4. Discussion

The synthesized data show that the four stabilizer systems modify low-plasticity clay through different compaction and bonding pathways. Class F fly ash increased water demand and reduced MDD beyond 3% dosage, which is consistent with its low specific gravity and fine-particle dilution effect. RCA and RAC increased MDD, plausibly because angular particles improved granular packing. Nano-MgO increased MDD by 3.4% at 1% dosage with only a small OMC change, consistent with a micro-void-filling interpretation. These mechanisms are supported by cited literature but were not directly verified by chemical or microstructural tests in this secondary-data study. Strength development also differed by material. Fly ash showed gradual curing-dependent gains associated with pozzolanic reactions [23]. RCA primarily provided frictional interlocking, whereas the higher RAC values may additionally reflect bonding from adhered mortar [5, 29]. Nano-MgO produced the largest early and 28-days UCS values, which is consistent with nanoscale pore filling and interparticle bonding reported for nano-MgO-treated clay [20]. Evidence from nano-clay and other nanomaterials [10–15, 21, 22, 26] is used only as comparative context and not as direct evidence for the nano-MgO dataset.

The composite ranking explains why fly ash was not ranked based on its bearing capacity result alone. 6% FA performed better than both RA scenarios in ultimate bearing capacity, while RAC achieved significantly higher UCS, higher CBR and lower settlement. RAC was rated slightly higher than fly ash at 9% by the equal-weight index, whereas nano-MgO remained unambiguously the best. The strong UCS–CBR relationship confirms the internal consistency of the strength parameters, but the lack of a definite MDD–bearing-capacity relationship indicates that the density of compaction alone is inadequate to explain the results. Bonding, curing, particle interaction, and test configuration are likely operated as well. Using recycled aggregates and industrial by-products could be more environmentally friendly since this practice can contribute to waste management and raw material conservation [27–29, 31]. Yet, there was no assessment of the carbon footprint, of leaching, life-cycle, impacts or cost in this investigation. In the same manner, low dosage of nano-MgO alone cannot demonstrate cost-effectiveness. The environmental and economic benefits should be viewed as hypotheses that must be tested through formal life-cycle assessment and cost-benefit analysis.

5. Conclusions

A structured secondary database of published results for low-plasticity clay from laboratory tests was used in this study to evaluate RCA, RAC, Class F fly ash, and nano-MgO. The OMC, MDD, curing-dependent UCS, soaked CBR, ultimate bearing capacity, and settlement were included in the

comparison. Differences across studies were mitigated through explicit inclusion criteria and normalizing to each source study's untreated control; still, the analysis is a synthesis of published data rather than a priori experimental program.

- During compaction process, OMC was increased from 17.2% to 20.2% while MDD was decreased from 1.80 g/cm³ at 3% dosage to 1.69 g/cm³ at 9% dosage as a result of addition of fly ash. RCA and RAC, on the other hand, resulted in an increased MDD of 1.86 g/cm³ and 1.89 g/cm³, respectively, and 1% nano-MgO addition increased MDD to 1.84 g/cm³ with only a marginal rise in OMC. These findings suggest that the stabilizers may “not play the same game”: fly ash alters primarily water demand and curing-dependent bonding, whereas recycled aggregates and nano-MgO have distinct packing and particle-interaction properties.
- All stabilized states, both UCS and soaked CBR, were higher than that of the untreated soil. The maximum 28-days UCS (520 kPa) and soaked CBR (11.6%) were attained by nano-MgO at 1% dosage. RAC recorded 460 kPa and 10.5%, while RCA 330 kPa and 8.4%. Fly ash attained 410 kPa and 8.9% at 6% dosage, while it was 430 kPa and 10.6% at 9% dosage. The strong descriptive UCS-CBR relationship ($r = 0.98$) indicates that the two strength measures are consistent for this dataset, but it does not imply causality.
- The highest ultimate bearing capacity (4,120 kPa) and lowest settlement (3.8 mm) were obtained by Nano-MgO. With 6% fly ash, the bearing capacity was 3,434 kPa, the highest among the non-nano conditions, but RAC showed better settlement control and more robust UCS/CBR results. When using the equal-weight normalized index, the order of the test conditions is nano-MgO, RAC, fly ash at 9%, fly ash at 6%, and RCA. At the predicting material level, the order was nano-MgO, RAC, fly ash, and RCA. This ranking is only for the four short-term mechanical criteria, and does not take into account the efficiency of dosage, cost, availability, durability and environmental implications.
- The findings support the technical potential of recycled aggregates, fly ash, and nano-MgO for improving CL subgrades, but they do not justify universal substitution for cement or lime. RCA may be suitable where moderate improvement and waste reuse are priorities; RAC may be preferred when higher strength and settlement control are required; fly ash is relevant where extended curing is feasible; and nano-MgO may be considered where high mechanical performance at low dosage is valuable. These statements are performance interpretations, not project-specific design recommendations.
- Variability is a source of error for source soils, laboratories, stabilizer compositions, and test configurations; replicate-level data is not completely available and formal pooled uncertainty estimates are not available; the number of comparable conditions is small; and a descriptive equal-weight index rather than a validated decision model is employed. Only short-term laboratory properties were taken into account. Durability with cyclic and wet-dry and freeze-thaw exposure, chemical deterioration, leaching, field-scale constructability, life-cycle implications, and cost were not considered. This may be applied to low-plasticity clay and is not suggested to be applied to other soil classes without confirmation.
- In future work, the entire source-level dataset and the search log should be published and the search log should be published, replicate observations should be retained, full load-settlement curves should be reported, and larger datasets should be used for regression, uncertainty, and sensitivity analysis to alternative weights. Field experiments under traffic and environmental loads are necessary to validate long-term performance. Comparative life-cycle assessment, evaluation of leachability of materials and cost benefit analysis, should be included with mechanical tests. Systems comprising RCA or RAC with nano-MgO are promising candidates for systematic research, but also in this case synergy potential has to be proven experimentally before making any claims in practical applications.

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References

- [1] Amhadi TS, Assaf GJ. Improvement of Pavement Subgrade by Adding Cement and Fly Ash to Natural Desert Sand. *Infrastructures*. 2021;6(11):151. <https://doi.org/10.3390/infrastructures6110151>
- [2] Rai P, Qiu W, Pei H, Chen J, Ai X, Liu Y, et al. Effect of Fly Ash and Cement on the Engineering Characteristic of Stabilized Subgrade Soil: An Experimental Study. *Geofluids*. 2021;2021:1-11. <https://doi.org/10.1155/2021/1368194>
- [3] U EE, Onyekpe U. Data on one-dimensional vertical free swelling potential of soils and related soil properties. *Data in Brief*. 2021;39:107608. <https://doi.org/10.1016/j.dib.2021.107608>
- [4] Mohanty KN, Pradhan PK, Mohanty CR. Stabilization of expansive soil using industrial wastes. *Geomechanics and Engineering*. 2017;12(1):111-125. <https://doi.org/10.12989/gae.2017.12.1.111>
- [5] Tabyang W, Suksiripattanapong C, Phetchuay C, Laksanakit C, Chusilp N. Evaluation of municipal solid waste incineration fly ash based geopolymer for stabilised recycled concrete aggregate as road material. *Road Materials and Pavement Design*. 2021;23(9):2178-2189. <https://doi.org/10.1080/14680629.2021.1959385>
- [6] Karthick J, Rajesh S, Saravana Kumar M, Thinakaranraj M, Vijayaram M, Ramkumar T. A study on soil stabilization using fly ash. *Nanoscience and Nanotechnology*. 2018;1(1):1-7. <https://doi.org/10.18063/nn.v0i0.433>
- [7] Karthik S, Kumar EA, Gowtham P, Elango G, Gokul D, Thangaraj S. Soil stabilization by using fly ash. *IOSR Journal of Mechanical and Civil Engineering*. 2014;10(6):20-26. <https://doi.org/10.9790/1684-1062026>
- [8] Fendi YM, Saeed KA, Sachit DE. Comparison Between the Calcium-Based Stabilizer and Non-Organic Agents on the Stabilization of Contaminated Soil. *Civil Engineering Journal*. 2026;12(1):278-291. <https://doi.org/10.28991/CEJ-2026-012-01-016>
- [9] Dayaratne D, Lin Y, Maghool F, Arulrajah A, Senanayake M. Flexural fatigue behavior of low-carbon pavement materials using geopolymer stabilized recycled concrete and recycled glass blends. *Materials Today Sustainability*. 2025;32:101269. <https://doi.org/10.1016/j.mtsust.2025.101269>
- [10] Khalkhali AB, Safarzadeh I, Manbar HR. Investigating the effect of nanoclay additives on the geotechnical properties of clay and silt soil. *Journal of Civil Engineering and Materials Application*. 2019;3:63-74.
- [11] Karkush M, Salman AD, Abbood AN, Emad R. Evaluating the role of nano-clay and nano-metakaolin in mitigating the collapsibility of gypseous soils. *Research on Engineering Structures & Materials*. 2026;12(2):1139-1151. <https://doi.org/10.17515/resm2026-1351ic1120rs>
- [12] Abd Hacheem Z, Al-Khyat S, Saleh MS, Hasan HA. Numerical Analysis of Load-Bearing Capacity in Contaminated and Uncontaminated Soils Treated with Nanomaterials. *Civil Engineering Journal*. 2025;11(9):3592-3605. <https://doi.org/10.28991/CEJ-2025-011-09-03>
- [13] Karkush MO, Almurshedi AD, Karim HH. Investigation of the impacts of nanomaterials on the micromechanical properties of gypseous soils. *Arabian Journal for Science and Engineering*. 2023 Jan;48(1):665-75. <https://doi.org/10.1007/s13369-022-07058-z>
- [14] Bilal M, Raja MA, Samanta S, Sheob M. Innovative Approaches to Soil Stabilization Using Nanomaterials and Nanotechnology. *Applications of Nanomaterials in Civil and Environmental Engineering*. 2025:181-196. https://doi.org/10.1007/978-981-96-3953-3_11
- [15] Karkush M, Ali SD. Experimental investigation of the leaching characteristics of gypseous soil treated with nano-silica. *Journal of Umm Al-Qura University for Engineering and Architecture*. 2025 Dec 15:1-7. <https://doi.org/10.1007/s43995-025-00278-y>
- [16] Oyekan GL, Adeyemi GO. Dataset of CBR of soils stabilised using agro-based pozzolans and cementitious binders (Version 1) [Data set]. Mendeley Data. 2021.
- [17] Saputra NA, Putra R. The Correlation Between CBR (California Bearing Ratio) and UCS (Unconfined Compression Strength) Laterite Soils in Palangka Raya as Heap Material. *IOP Conference Series: Earth and Environmental Science*. 2020;469(1):012093. <https://doi.org/10.1088/1755-1315/469/1/012093>
- [18] Ahmed ST, Kabir MU, Zahid CZB, Tareque T, Mirmotalebi S. Improvement of subgrade California Bearing Ratio (CBR) using recycled concrete aggregate and fly ash. *Hybrid Advances*. 2024;5:100153. <https://doi.org/10.1016/j.hybadv.2024.100153>
- [19] Nanda SK, Giri JP, Priyadarshini M. Quantitative evaluation of soil stabilization using RCA and GGBS: a statistical perspective. *Journal of Building Pathology and Rehabilitation*. 2024;9(2). <https://doi.org/10.1007/s41024-024-00444-0>
- [20] Bagriacik B, Mahmutluoglu B. Improvements to Load-Bearing Capacity and Settlement of Clay Soil after Adding Nano-Mgo and Fibers. *Polymers*. 2022. <https://doi.org/10.2139/ssrn.4016028>
- [21] Soltani A, Moradi A. Novel approach to improve geotechnical properties of clay soil by nano-clay and OPC particles. *Environmental Earth Sciences*. 2025;84(14). <https://doi.org/10.1007/s12665-025-12397-9>
- [22] Abd KG, Abd AH. Nanomaterials for Soil Stabilization: A Review. *Journal of Advanced Sciences and Engineering Technologies*. 2025;8(1):1-27. <https://doi.org/10.32441/jaset.8.1.4>

- [23] Ahmad S, Shah Alam Ghazi M, Syed M, Al-Osta MA. Utilization of fly ash with and without secondary additives for stabilizing expansive soils: A review. *Results in Engineering*. 2024;22:102079. <https://doi.org/10.1016/j.rineng.2024.102079>
- [24] Keshavarz M, Shourijeh PT, Hekmatzadeh AA. The erosion resistance of clay soil treated with recycled concrete aggregates. *Construction and Building Materials*. 2024;443:137733. <https://doi.org/10.1016/j.conbuildmat.2024.137733>
- [25] Sambre T, Endait M, Patil S. Sustainable soil stabilization of expansive soil subgrades through lime-fly ash admixture. *Discover Civil Engineering*. 2024;1(1). <https://doi.org/10.1007/s44290-024-00063-1>
- [26] Chaudhary V, Yadav JS, Dutta RK. A critical appraisal on strength, mineralogy and morphology of clayey soil stabilised with nano-additives. *Geomechanics and Geoengineering*. 2024;19(5):840-867. <https://doi.org/10.1080/17486025.2024.2324869>
- [27] Salah Hashem A, Shaban AM. Sustainable Use of Recycled Concrete Aggregate for Soil Improvement. *IOP Conference Series: Earth and Environmental Science*. 2024;1374(1):12028. <https://doi.org/10.1088/1755-1315/1374/1/012028>
- [28] Tiyaangthong S, Yoosuk P, Krosoongnorn K, Sakdinakorn R, Tabyang W, Phojan W, et al. Stabilization of Recycled Concrete Aggregate Using High Calcium Fly Ash Geopolymer as Pavement Base Material. *Infrastructures*. 2022;7(9):117. <https://doi.org/10.3390/infrastructures7090117>
- [29] Lin Y, Maghool F, Arulrajah A, Horpibulsuk S. Feasibility of recycled concrete aggregate stabilized with one-part geopolymers as semi-rigid inclusion columns. *Construction and Building Materials*. 424, 135825. <https://doi.org/10.1016/j.conbuildmat.2024.135825>
- [30] Baba Sankar A, Viswanath D, Paluri Y. Performance Evaluation of Expansive Soil Stabilized with Fly Ash, Calcium Carbide Residue, and TerraZyme. *Indian Geotechnical Journal*. 2025. <https://doi.org/10.1007/s40098-025-01348-7>
- [31] Alqawasmeh HM. Enhancing the sustainability of geotechnical engineering with utilization of fly Ash. *Discover Materials*. 2025;5(1). <https://doi.org/10.1007/s43939-025-00183-0>

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Improved SA algorithm for masonry layout optimization of building infill walls

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Abstract

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This paper proposes a hybrid optimization algorithm that fuses multiple methods to address the weak global exploration ability, frequent local optima, and poor engineering adaptability in masonry layout optimization of building infill walls. The method builds a multilayer cooperative framework. It first uses the Genetic Algorithm to create a diverse population. It then applies Simulated Annealing to perform probabilistic jumping optimization. After that, it introduces sparse A search to verify topological feasibility. It finally relies on a cooperative mechanism of Adaptive Whale Optimization and iterative local search to explore the solution space in depth. Experiments on the simultaneous localization and mapping-building information modeling coupled dataset and the building information modeling component multimodal dataset show that the algorithm reaches a standard block utilization rate of 98.76 percent. It also keeps the cutting loss rate as low as 2.79% and achieves a peak stagger-joint compliance rate of 97.11%. In irregular wall scenarios, it reduces cost by up to 33.87%. The results show that this algorithm improves the optimization quality and engineering applicability of masonry layout and provides reliable technical support for precise construction and efficient material use of building infill walls.

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1. Introduction

The accuracy of masonry layout for building infill walls directly determines construction efficiency and material use. Intelligent optimization algorithms create coordinated multi objective optimization and offer new solutions for accurate layout [1]. However, traditional layout optimization algorithms perform poorly in material utilization, construction feasibility, and structural stability when the layout must satisfy complex structural constraints [2]. Accurate and intelligent masonry layout of building infill walls therefore plays an important role in improving construction quality and reducing engineering cost. The hybrid Genetic Algorithm and Simulated Annealing algorithm (GA-SA) combines population evolution strategies with probabilistic jumping mechanisms [3–4]. But existing GA-SA methods still rely on prior knowledge for parameter adjustment. They also show weak search efficiency in high-dimensional solution spaces and limited adaptability in dynamic construction scenarios [5]. This study introduces a topological verification mechanism based on Sparse A Search (SAS). It also combines the intelligent search ability of the Adaptive Whale Optimization Algorithm (AWOA) and the fine-grained exploration of Iterative Local Search (ILS). The method builds a multilayer cooperative layout optimization framework. It aims to improve the overall performance of masonry layout through coordinated global exploration, local optimization, and engineering feasibility verification. This framework provides

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technical support for green construction and industrialized building development. The innovation of this study lies in building a layout optimization model that integrates multiple intelligent algorithms through multilayer cooperation. The model applies the coordinated mechanism of GA and SA. It also merges the adaptive mechanism of AWOA with the search characteristics of ILS. The method improves engineering adaptability and solution efficiency while ensuring layout quality. It offers an innovative solution for precise construction and efficient material use of building infill walls.

1.1 Literature Review

As a classical global optimization algorithm, the SA algorithm has been widely applied in various fields, and its intelligent jump mechanism inspired by the thermodynamic annealing process has received continuous attention and research from scholars at home and abroad. For example, Gangadevi E et al. proposed a recognition method based on SA-optimized support vector machines to address the problem of crop diseases. This method combined SA with the fruit fly optimization algorithm to avoid global optimum, and then performed classification to accurately identify tomato plant diseases [6]. Kosanoglu F's team proposed a hybrid algorithm based on SA algorithm and deep reinforcement learning to prevent asset failures. This algorithm used deep reinforcement learning to find the optimal solution, and then used this solution as the initial solution of the SA algorithm to find the final optimal solution more quickly [7]. Pashaei E et al. proposed a gene selection strategy based on SA and binary coot optimization algorithm for better gene selection. This strategy used SA optimization to improve the coot algorithm to find stable information genes [8]. Lim K C W's team proposed a hyper-heuristic method based on SA algorithm for flexible job shop scheduling. This method used two variants of SA hyper-heuristics to conduct investigations based on machine assignment and job sequencing rules for job shop scheduling [9]. Zanazzo E et al. proposed a local search technique based on SA for medical student scheduling. This technique used SA to guide the combination of two different neighborhood relationships for medical student scheduling [10].

As a core component of building construction quality and efficiency, the masonry layout of building infill walls directly relates to wall structural safety, material utilization efficiency, and engineering progress control. Many domestic and foreign scholars have conducted in-depth studies on this topic. For example, Valentino J's team proposed a method based on discontinuity layout optimization and limit analysis for wall masonry layout optimization. This method used the smeared continuum approach to model the masonry, and then used limit analysis to determine the load-bearing capacity of the masonry for wall masonry layout optimization [11]. Vu Hong Son P et al. proposed a layout optimization method based on hybrid ant lion optimization algorithm to address the problem that optimization programs were only suitable for small-scale problems. This method used the ant lion optimization algorithm to avoid local optimum, and then found the optimal solution through this algorithm for effective layout optimization [12]. Tan Y's team proposed a courtyard division method based on concrete components to address the contradiction between concrete components and construction site area. This method used rectangular algorithms to calculate the number of stacks, and then used non-dominated sorting genetic algorithm for simulation to obtain better layout schemes [13]. Shang D et al. proposed an architectural structure layout optimization method based on image segmentation to address the problem of numerous constraints in building structures. This method used semantic simultaneous localization and mapping framework to achieve real-time integration, and then constructed a grid graph-based optimization model for architectural structure layout optimization [14]. Zheng Z et al. proposed a layout optimization method based on multi-population genetic algorithm for building module layout optimization. This method used multi-population genetic algorithm and typical module units to establish an integrated framework for modular construction building module layout and optimization for automated layout [15].

Based on the above content, although the existing research has made certain progress in the field of building infill wall masonry layout, it mostly focuses on macroscopic structural optimization or the application of specific algorithms, and there is still a lack of systematic research at the microscopic level on the coordinated optimization of details in masonry layout, the diversity of

masonry block specifications and the feasibility of construction. Specifically, the existing methods lack an optimization strategy that can comprehensively consider these microscopic factors when dealing with the flexibility of masonry block arrangement, the diversity of masonry block specifications, and the balance between construction efficiency and material utilization. Therefore, this study proposes a building infill wall masonry layout method based on multi-objective collaborative optimization, aiming to address the limitations of existing methods in the handling of details and the diversity of masonry block specifications. The study proposes a multi-layer collaborative optimization architecture that integrates the mechanisms of GA and SA, the rapid search ability of SAS, and the advantages of AWOA and ILS to construct an intelligent optimization algorithm for building infill wall masonry layout. Regarding the contribution of the research method to the current research gap, it is (1) proposing a multi-objective optimization model for masonry layout based on the integration of GA and SA, which can simultaneously consider the flexibility of masonry block arrangement, the diversity of masonry block specifications, and the balance between construction efficiency and material utilization. By introducing an adaptive weight adjustment mechanism, it achieves dynamic trade-offs among multiple objectives, thereby improving the comprehensive performance of the layout scheme. (2) Designing a hybrid optimization strategy based on the combination of SAS rapid search and AWOA-ILS local refinement, which uses the rapid search ability of SAS to quickly locate high-quality solution spaces and then uses the local refinement ability of AWOA-ILS to deeply explore the solution space, thereby improving the quality of the solution while maintaining search efficiency. Compared with existing optimization methods, the research method has significant advantages in the flexibility of masonry block arrangement and construction feasibility, and can effectively handle the masonry block adaptation issues at the detail nodes.

2. Masonry Layout Algorithm Based on Improved SA For Building Infill Walls

2.1 Masonry layout algorithm based on improved SA for building infill walls

In the intelligent optimization of masonry layout for building infill walls, accurate construction and green building require coordinated computation between layout pattern generation and intelligent optimization algorithms. This process builds a complete solution from data perception to decision output [16]. The SA algorithm uses its strong global exploration ability to search the optimal or near-optimal masonry layout [17]. The workflow of SA for masonry layout optimization is shown in Figure 1.

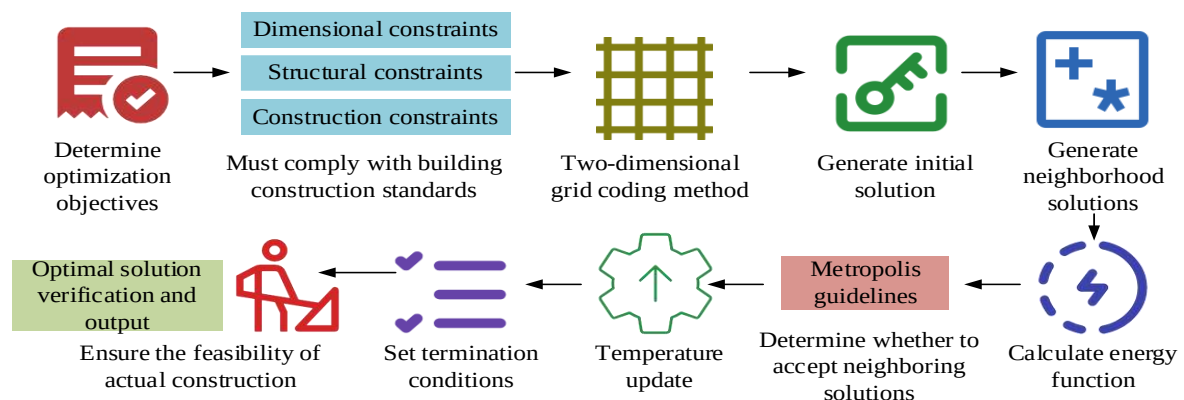


Fig. 1. Flowchart for SA to optimize the layout of masonry infill walls in buildings

According to Figure 1, SA first sets the optimization objectives. The objectives are minimum block loss and maximum construction efficiency under multiple constraints. It then generates an initial solution that satisfies the constraints based on standard construction logic. After that, the algorithm enters the core annealing iteration. It adjusts the current solution to create a neighborhood solution. It evaluates the solution with the energy function. It decides whether to update the solution based on the acceptance rule. It lowers the temperature based on the fast-to-slow schedule. The algorithm stops when the temperature reaches the threshold, the energy does not change across iterations, or the maximum iteration number is reached. It then verifies engineering

feasibility and outputs the optimal solution. The initial temperature in the SA parameter is set to 1000, the cooling rate is set to 0.95, and the termination temperature is set to 1×10^{-3} . At each temperature, the length of the Markov chain is set to 100, which means conducting 100 neighborhood searches at each temperature. The design of the energy function comprehensively considers three dimensions: block utilization rate, construction convenience, and structural safety. The weight of block utilization rate is set to 0.5, the weight of construction convenience is set to 0.3, and the weight of structural safety is set to 0.2. The parameter selection method is based on a combination of empirical trial and error and sensitivity analysis. Firstly, parameter pre tuning is carried out through small-scale test cases, and then the orthogonal experimental design method is used to systematically screen the key parameter combinations, ultimately determining the parameter configuration that balances the convergence speed and solution quality of the algorithm. The temperature update of SA is shown in Equation (1).

$$T_{k+1} = \alpha \cdot T_k \quad (1)$$

In Equation (1), T_k represents the temperature of the k -th iteration. T_{k+1} represents the temperature of the $k+1$ -th iteration. α represents the temperature coefficient. The state transition probability of SA is shown in Equation (2).

$$\begin{cases} P=1 & E(s') \leq E(s) \\ P=e^{-\frac{E(s')-E(s)}{T_k}} & E(s') > E(s) \end{cases} \quad (2)$$

In Equation (2), $E(s)$ represents the energy of the current state. $E(s')$ represents the energy of the new state. P represents the transition probability. s and s' represent the current state of the algorithm and the new state generated by the algorithm. But standard SA shows a single search direction in complex solution spaces and suffers from early convergence [18]. This study therefore introduces GA into SA to form a hybrid algorithm that enhances global optimization performance. GA conducts preliminary screening of population size, crossover probability, and mutation probability through pre-experiments to determine their reasonable range of values. On this basis, orthogonal experimental design was adopted to finely tune the above parameters, with the weighted sum of population diversity and convergence speed as the evaluation index, and the optimal parameter combination of GA was finally determined. The workflow of GA optimizing SA is shown in Figure 2.

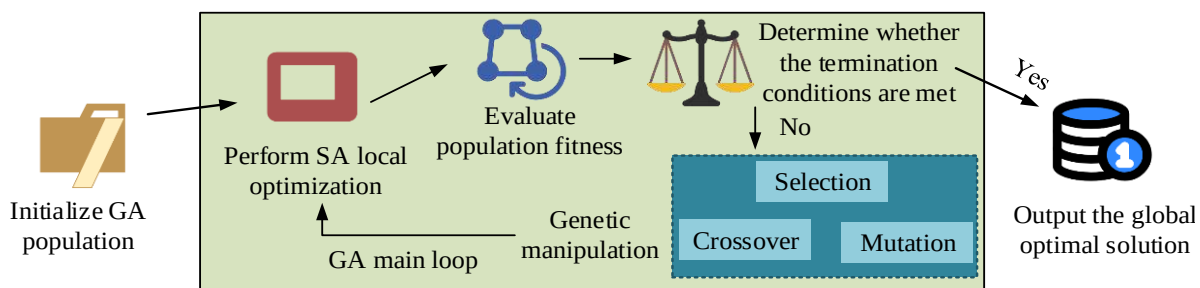


Fig. 2. Flowchart of GA optimizing SA

According to Figure 2, GA first creates an initial population that represents different masonry layout schemes. Each individual then enters the SA local optimization stage. The probabilistic jumping mechanism improves the quality of each individual. The optimized population then undergoes genetic operations. The new population enters another SA local optimization process. The cooperation between the global exploration of GA and the local search of SA outputs the optimal masonry layout for building infill walls. The layout of the masonry structure adopts a binary encoding method, mapping the arrangement scheme of the masonry blocks into chromosome individuals. Each gene position corresponds to the position status of a masonry block,

with 0 indicating that no masonry block is placed at that position and 1 indicating that a masonry block is placed. Assuming that the horizontal direction of the wall can accommodate 15 bricks and the vertical direction can accommodate 24 layers, the encoding length is 360. The selection function of GA is shown in Equation (3).

$$\begin{cases} p_i(t) = \frac{f(x_i(t))}{F(t)} \\ \sum_{j=1}^{k-1} p_j(t) \leq r < \sum_{j=1}^k p_j(t) \end{cases} \quad (3)$$

In Equation (3), $p_i(t)$ represents the selection probability of the i -th individual in the t -th generation. $f(x_i(t))$ represents the individual fitness. $F(t)$ represents the sum of all fitness values. $\sum_{j=1}^{k-1} p_j(t)$ and $\sum_{j=1}^k p_j(t)$ represent the cumulative selection probabilities of the first $k-1$ individuals and the first k individuals. The crossover function is shown in Equation (4).

$$x_a(t+1) = (a_1, \dots, a_c, b_{c+1}, \dots, b_L) x_b(t+1) = (b_1, \dots, b_c, a_{c+1}, \dots, a_L) \quad (4)$$

In Equation (4), $x_a(t+1)$ represents the gene sequence of the parent individual in the next generation. c represents the random crossover point. L represents the gene length. The mutation function is shown in Equation (5).

$$g'_j = \begin{cases} 1 - g_j & \text{if } r < p_m \\ g_j & \text{otherwise} \end{cases} \quad (5)$$

In Equation (5), g'_j is the value at the j -th gene position, and r is a random number. p_m represents the mutation probability. Although GA-SA enhances layout optimization by combining population-based search and probabilistic jumps, it still shows limited search efficiency in large solution spaces. The SAS algorithm improves efficiency through sparse state sampling and adaptive heuristic evaluation. It builds an effective layout path through multi-constraint topological spaces [19]. Compared to traditional A* algorithms, SAS exhibits stronger global optimization capabilities in masonry layout, especially suitable for complex layout problems under multiple constraint conditions. The core mechanism is to transform the masonry layout problem into a path search problem in sparse state space, dynamically evaluate the priority of each state node through heuristic functions, and use adaptive sampling strategies to balance exploration and utilization. Therefore, the study introduces the SAS algorithm into the GA-SA algorithm to construct an optimization algorithm (GA-SA-SAS, abbreviated as GSSA) to improve search efficiency. The workflow of GSSA for masonry layout optimization is shown in Figure 3.

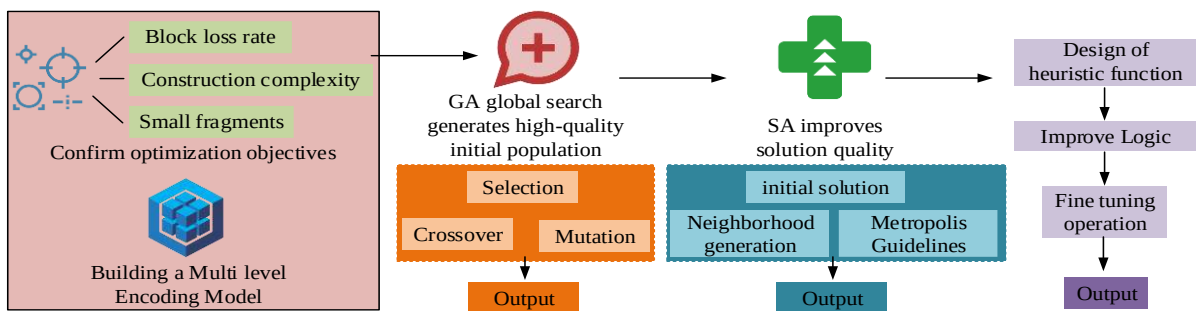


Fig. 3. Flowchart of GSSA for masonry layout optimization

According to Figure 3, the method first sets the objectives as minimum block loss and minimum construction complexity. It defines constraints such as mortar thickness and cutting blocks. It builds a two-dimensional grid and sequence-encoding hybrid model. GA then performs global

search. It creates an initial population, executes genetic operations, and outputs high-quality individuals. These individuals then enter SA as initial solutions. SA performs neighborhood search and probabilistic acceptance under the acceptance rule. It lowers the temperature based on the cooling coefficient and outputs the minimum-energy solution. SAS then uses its heuristic function to optimize the block placement path and sequence. It solves construction adaptation problems and stops when the termination condition is satisfied. The algorithm outputs the optimal layout. The heuristic function in SAS dynamically guides the search direction by evaluating the sum of the estimated cost from the current node to the target node and the consumed cost in a sparse state space, thereby prioritizing paths with lower construction complexity and higher material utilization in masonry layout. This function integrates block dimensions, mortar joint constraints, and cutting limitations, adaptively adjusting the weights of each factor to balance global optimality and local feasibility in the search process. The study sets adaptive weights based on block size deviations and mortar joint thickness fluctuations, dynamically optimizing the exploration-exploitation balance coefficient through real-time feedback, thereby refining the path selection strategy in masonry layout.

During the execution of construction constraints, the study transforms block dimensions, mortar joint thickness, and cutting limitations into constraint conditions through encoding rules, dynamically checking the feasibility of each state node during the search process. If a current node violates constraints, its priority is reduced via a penalty function, guiding the search toward feasible regions. Additionally, a backtracking mechanism records historical states, automatically reverting to the last feasible node and reselecting branch paths when the search repeatedly falls into local optima, avoiding invalid iterations. The study introduces a dynamic constraint relaxation strategy, relaxing mortar joint thickness or cutting block quantity limits when construction material utilization is low to expand the feasible solution space. When material utilization falls below 85%, the relaxation mechanism is triggered, allowing the permissible deviation of mortar joint thickness to increase from $\pm 2\text{mm}$ to $\pm 3\text{mm}$, while also permitting a 10% increase in the upper limit of cutting blocks. The total cost of SAS is shown in Equation (6).

$$f(n) = g(n) + h(n) \quad (6)$$

In Equation (6), $f(n)$ represents the total cost of the current placement state. $g(n)$ represents the actual cost from the initial unplaced state to state n . $h(n)$ represents the estimated minimum cost from state n to the fully placed state. To ensure the feasibility of the solution, the model incorporates structural safety redundancy coefficients and construction tolerance ranges in the constraints, enabling the optimization results to meet the specification requirements while also having the operational feasibility and fault-tolerant capability in actual engineering. Additionally, during the optimization solution process, each iteration strictly verifies the constraints. If a solution violates any constraint, it will be automatically eliminated and a new candidate solution will be generated to ensure that the final output solution fully complies with all constraints. Moreover, the model also introduces a penalty function mechanism, applying penalty terms to the candidate solutions that violate the constraints, guiding the search direction towards the feasible region.

2.2 Masonry Layout Algorithm Based on GSSA

GSSA coordinates global exploration and local optimization in masonry layout optimization for building infill walls. But the three-layer coupling of the algorithm still causes high computational cost. AWOA introduces dynamic adaptive weights and a nonlinear convergence factor. It balances the spiral encircling behavior and random hunting behavior. It maintains population diversity and accelerates convergence. It therefore improves the generation efficiency and stability of masonry layout in engineering environments [20]. The structure of AWOA for masonry layout optimization is shown in Figure 4. According to Figure 4, AWOA contains four core components. In the population initialization module, each whale individual uses a position vector to represent a complete masonry layout. The core improvement module uses an adaptive mechanism to balance global exploration and local exploitation. The position update module contains three search modes inspired by whale hunting behavior.

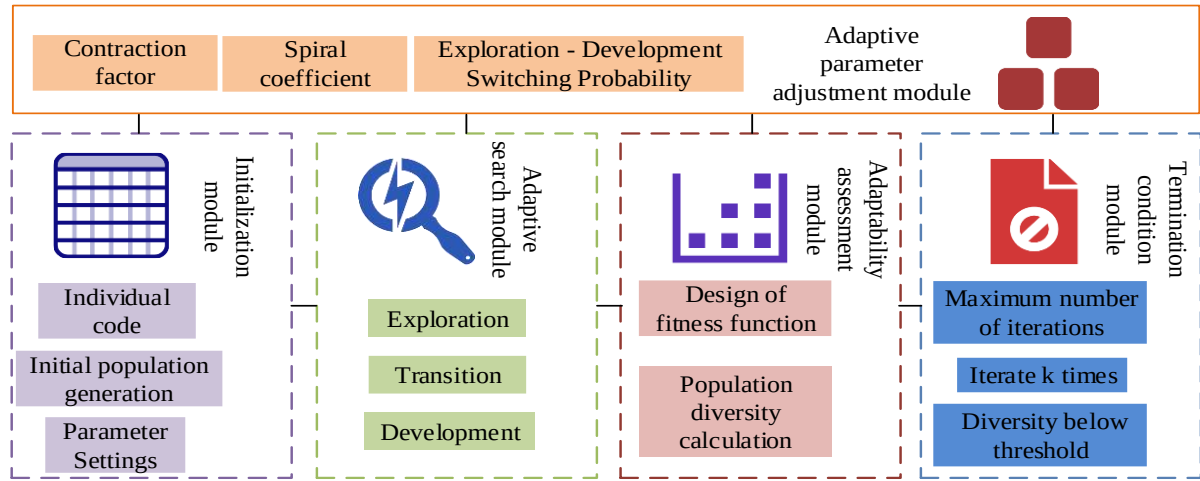


Fig. 4. Structure of AWOA

The evaluation and iteration module evaluates the quality of solutions with the fitness function and uses an elite retention strategy to keep the optimization direction correct. The encircling behavior of AWOA is shown in Equation (7).

$$X(t+1) = X(t) + r(X^* - X(t)) \quad (7)$$

In Equation (7), $X(t)$ and X^* represent the current position and the optimal position. r represents a random number in the range $[-1, 1]$. The hunting behavior of AWOA is shown in Equation (8).

$$\begin{cases} X(t+1) = X^* - A \cdot D_1 \cdot e^{bl} \cdot \cos(2\pi l) \\ D_1 = |C \cdot X^* - X(t)| \end{cases} \quad (8)$$

In Equation (8), D_1 represents the distance between the current data and the optimal data. A and C represent coefficient vectors. b is a constant that controls the spiral shape. l is a random number in the range $[-1, 1]$. The prey search behavior of AWOA is shown in Equation (9).

$$X(t+1) = X_{rand} - A \cdot D_2 \cdot e^{bl} \cdot \cos(2\pi l) \quad (9)$$

In Equation (9), D_2 represents the distance between the current data and a random data point. X_{rand} represents the position of a randomly selected data point. Because of uncertain behavior, the final update of the data position uses a random probability to determine which behavior is used. The behavior selection rule is shown in Equation (10).

$$X(t+1) = \begin{cases} X^* - A \cdot |C \cdot X^* - X(t)|, p < 0.5 \\ D_2 e^{bl} \cos(2\pi l) + X^*, p \geq 0.5 \end{cases} \quad (10)$$

In Equation (10), P represents the random probability. The algorithm performs the hunting behavior when the probability is less than 0.5. It performs the prey search behavior when the probability is greater than or equal to 0.5. The parameter update method of the AWOA algorithm adopts an adaptive adjustment strategy, dynamically adjusting the values of the coefficient vector and random probability based on the number of iterations, thus achieving a dynamic balance between global search and local development. In the early stages of iteration, larger coefficient vectors and lower random probability thresholds are advantageous for global search to extensively explore the solution space. The study set it as an initial value of 0.9 and linearly decreased it to 0.1 with the number of iterations. At the same time, the random probability threshold gradually increased from 0.5 to 0.9, making the algorithm more inclined towards local fine development in

the later stage. Traditional AWOA relies heavily on the guidance of the leader whale. This reliance may cause the algorithm to fall into a single search pattern because of population social behavior. ILS avoids this problem through multi-scale perturbation and elite restart. It prevents insufficient exploration and early convergence. This study therefore introduces ILS into AWOA to build the AWOA-ILS algorithm. The workflow of AWOA-ILS for masonry layout optimization is shown in Figure 5.

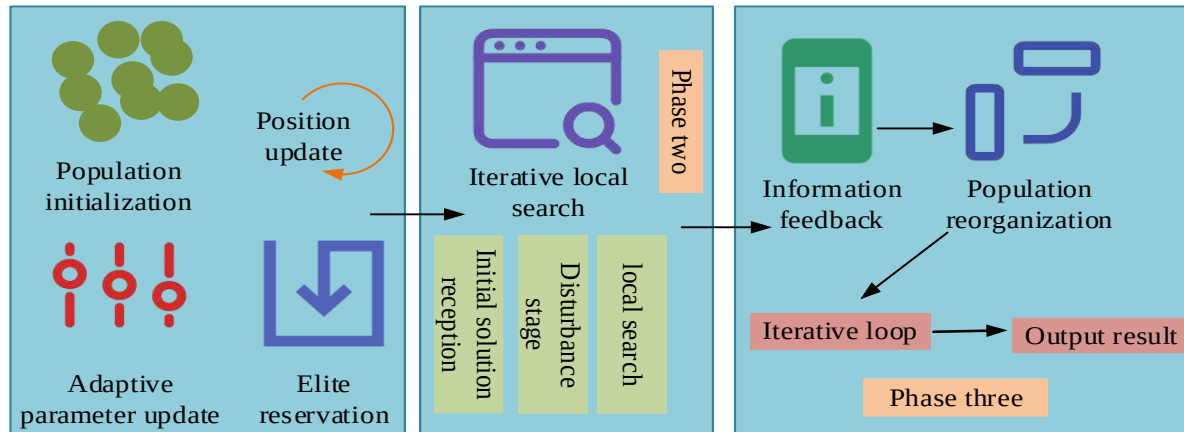


Fig. 5. AWOA-ILS's flowchart for masonry layout optimization

As shown in Figure 5, during the iterative process of the AWOA-ILS algorithm, AWOA is responsible for global exploration and population update, while the ILS module performs deep perturbation and restart of the elite solution in the local stage. In each generation iteration, AWOA first generates candidate solutions based on adaptive parameters and updates the positions of the whale population; subsequently, the ILS module performs multi-scale perturbation operations on the current optimal solution, including random exchange of block types, fine-tuning of block position coordinates, and recombination of construction sequences, to generate several variant solutions. After local search, if the fitness of these variant solutions is better than the original solution, the original optimal solution is replaced and re-injected into the population, thereby guiding the population to escape from the local optimal region. The local search expression of ILS is shown in Equation (11).

$$\text{cost}(s^*) = \sum_{k=1}^n d(v_k, v_{k+1}) \quad (11)$$

In Equation (11), $\text{cost}(s^*)$ represents the total cost of path s^* . n represents the total number of nodes to visit. $d(v_k, v_{k+1})$ represents the distance from node v_k to node v_{k+1} . v_k represents the k -th visited node. The perturbation and operation expression of ILS is shown in Equation (12).

$$s' = \text{swap}(s^*, \text{randselect}(s^*, k)) \quad (12)$$

In Equation (12), s' is the new solution produced after perturbation, and s^* is the current local optimal solution. k represents the perturbation strength parameter. Based on the advantages of AWOA-ILS, this study integrates AWOA-ILS with GSSA to form the AWOA-ILS-GSSA algorithm (AIGSSA). This algorithm provides a more effective optimization strategy for masonry layout of building infill walls. The workflow of AIGSSA is shown in Figure 6.

As shown in Figure 6, the AIGSSA algorithm consists of five core modules: GA, SA, SAS, AWOA, and ILS. These modules work together to complete the entire optimization process from generating the initial population to outputting the final solution. The algorithm first generates the initial masonry arrangement population through the GA module, then the SA module performs local optimization on the population individuals, the SAS module verifies the topological feasibility, the AWOA module uses the enclosure-bubble network mechanism to mine high-quality solutions, and the ILS module implements multi-scale perturbation on the elite solutions.

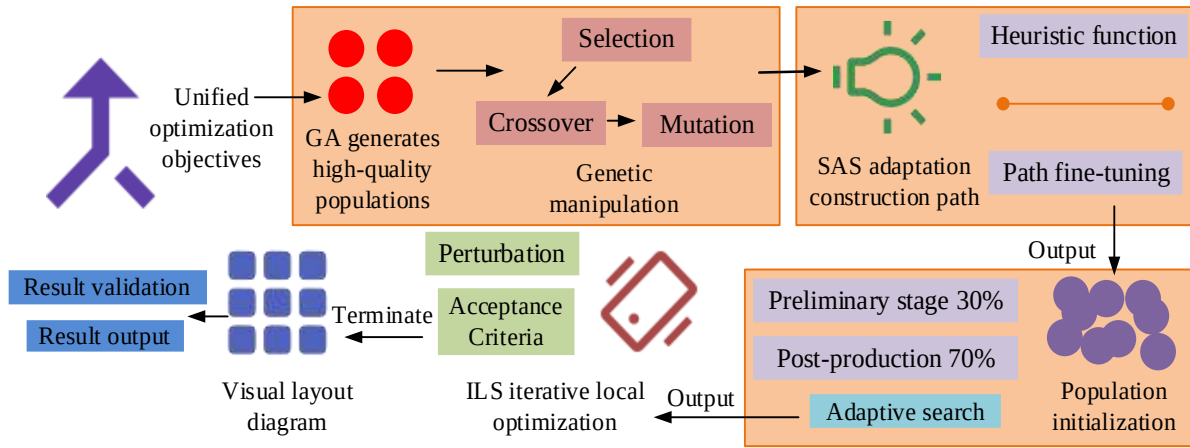


Fig. 6. AIGSSA's flowchart for masonry layout optimization

The modules form a closed-loop optimization system through the exchange of solution information and population recombination mechanisms. On the basis of ensuring the feasibility of the construction, it achieves the global optimization of the masonry arrangement scheme and the combination of engineering practicability. The optimization process maintains the diversity of the initial population through the crossover and mutation operations of the GA module, avoiding premature convergence. Subsequently, the SA module accepts inferior solutions with a certain probability through the Metropolis criterion, further enriching population diversity and preventing falling into local optima. In the SAS module, individuals with reasonable preservation structures are selected through topological constraints to ensure that the population does not deviate from the feasibility boundary while increasing diversity. The AWOA module conducts deep mining of the population through the enclosing bubble network mechanism, improving the quality of solutions while maintaining diversity. The ILS module dynamically balances local and global searches by applying multi-scale perturbations to elite solutions, avoiding population homogenization. The GA module stops when the number of iterations reaches 50 or the fitness changes by less than 0.1% for 5 consecutive generations. The SA module stops when the temperature drops to 0.01 °C. The SAS module automatically terminates after completing all individual verifications. The AWOA module stops when iterating for 30 generations or when the optimal solution has not been updated for 10 consecutive generations. The ILS module stops when iterating for 20 generations or when the elite solution has not improved for 5 consecutive generations. The optimization objective function of the AIGSSA wall masonry arrangement optimization algorithm is shown in Equation (13).

$$F(x) = \alpha \cdot f_{structural}(x) + \beta \cdot f_{efficiency}(x) + \gamma \cdot f_{cost}(x) \quad (13)$$

In Equation (13), $F(x)$ represents the optimization objective function, x is the decision variable, $f_{structural}(x)$ is the structural rationality index function, α is the weight coefficient of the structural rationality indicator, $f_{efficiency}(x)$ is the construction efficiency indicator function, β is the weight coefficient of the construction efficiency indicator, $f_{cost}(x)$ is the economic indicator function, and γ is the weight coefficient of the economic indicator. The three weight coefficients are determined through the Analytic Hierarchy Process (AHP), taking 0.45, 0.35, and 0.20 respectively, to balance the priority relationship among structural safety, construction efficiency, and economy. This objective function guides the algorithm during the optimization process to prioritize improving construction efficiency while also considering material cost control, thereby achieving the maximum comprehensive benefit in engineering practice. The structural rationality indicator function is shown in Equation (14).

$$f_{structural}(x) = \sum_{i=1}^n w_i \cdot s_i(x) \quad (14)$$

In Equation (14), n represents the total number of structural rationality sub-indicators, i is the sequence number of the sub-indicators, w_i is the weight of the i th indicator, which is determined by expert scoring and engineering experience, $s_i(x)$ is the scoring function of the i th indicator, and the function is linearly normalized to $[0,1]$. The construction efficiency indicator and economic efficiency indicator functions are both in the form of multi-indicator weighted summation. The weights of the sub-indicators are also determined through expert assessment and normalized to a unified dimension. Among the sub-indicators of structural rationality, the weight of the block joint length is the highest, set at 0.50. The uniformity of the mortar joint and the overall stability of the wall are set at 0.30 and 0.20 respectively, highlighting the dominant role of the joint quality on the structural safety. In the construction efficiency indicator, the weight of the number of block types is set at 0.40, the regularity of arrangement is set at 0.35, and the operation complexity is set at 0.25, reflecting the core role of reducing the types of components in improving construction efficiency. In the economic indicators, the weight of material utilization is set at 0.50, the cost of a single block is set at 0.30, and the transportation and storage costs are set at 0.20.

This optimization problem can be modeled as a Mixed Integer Linear Programming (MILP) model, where the selection of block types and the layout schemes of blocks are integer variables, while the structural rationality, construction efficiency and economic indicators are continuous variables. The constraints include structural safety limits, material cost upper limits, block size modulus matching, and construction operability requirements. The objective function is the maximization of the weighted sum of three indicators, and its mathematical expression is shown in Equation (13). In Equation (13), α , β , and γ are the weight coefficients of the structural rationality, construction efficiency and economic indicators, satisfying $\alpha + \beta + \gamma = 1$. Their values can be dynamically adjusted according to the actual engineering requirements. The optimization solution can be obtained by using heuristic algorithms such as branch and bound method or genetic algorithm, while ensuring the solution accuracy, and taking into account the computational efficiency and engineering practicability.

To conduct theoretical analysis on convergence, a convergence criterion is introduced and a threshold for the rate of change of the objective function value during the iteration process is defined. When the relative change of the objective function for three consecutive iterations is less than the preset tolerance ε , the algorithm is judged to have converged. At the same time, a multi starting point strategy is adopted to avoid getting stuck in local optimal solutions, and each starting point runs independently to select the global optimal result. When the values of α , β , and γ fluctuate within the range of $[0.2, 0.5]$, the variation range of the structural rationality index does not exceed 8%, the variation range of the construction efficiency index does not exceed 6%, and the variation range of the economic index does not exceed 5%. This indicates that the model has good robustness with respect to the weight parameters. When the number of block types is between 3 and 5, the optimization results tend to be stable. The structural rationality index reaches above 0.85, the construction efficiency index exceeds 0.80, and the economic index remains above 0.75. When the number of block types exceeds 5, the construction efficiency index shows a significant decrease, with a drop of 12%, indicating that too many block types will increase the construction complexity and weaken the efficiency advantage. In summary, when the number of block types is 4, the comprehensive score of the three indicators reaches the optimal value, achieving a good balance among structural rationality, construction efficiency, and economy.

3. Performance Verification of The AIGSSA Masonry Layout Optimization Algorithm

3.1 Optimization Quality Analysis of the AIGSSA Algorithm

To verify whether the optimization quality of the AIGSSA wall masonry layout optimization algorithm was excellent, this study introduced three comparison algorithms for comparative experiments with AIGSSA: the wall masonry layout optimization algorithm based on Particle Swarm Optimization and Ant Colony Optimization (PSO-ACO), the wall masonry layout optimization algorithm based on Artificial Bee Colony and Glowworm Swarm Optimization (ABC-GSO), and the wall masonry layout optimization algorithm based on Gravity Search Algorithm and

Differential Evolution (GSA-DE). The experimental system used Windows 11, with an Intel i9-14900KS central processing unit and an NVIDIA RTX 4060ti graphics card. The datasets used were the SLABIM-BIM Coupled Dataset (SLABIM) and the Building Information Modeling Component Multimodal Dataset (BIMCompNet). The SLABIM dataset was generated through deep integration of actual construction site point cloud data captured by simultaneous localization and mapping technology with design data from building information models, accurately reflecting the wall geometric forms, spatial relationships, and construction states in real construction site environments. The BIMCompNet dataset integrated BIM model components from various sources and types, covering walls, blocks, and associated components of different specifications, materials, and construction logic. The dataset SLABIM contains 12,000-point cloud-BIM paired samples, covering 6 common wall types and 4 construction stages; the BIMCompNet dataset contains 8,500 multi-modal component samples, covering 15 block specifications and 5 connection methods. In the experiments, the two datasets were respectively used to verify the adaptability and generalization ability of the algorithm in different scenarios. The SLABIM dataset was randomly divided into training set, validation set and test set, with a ratio of 7:1:2, to evaluate the wall layout optimization effect of the algorithm on real construction scenarios; the BIMCompNet dataset was stratified sampled by component type and divided into training set, validation set and test set in a ratio of 8:1:1 to test the robustness of the algorithm in multi-modal component layout. The original point cloud data of the SLABIM dataset was sourced from the construction site of a large commercial complex project, and was preprocessed through noise removal, registration and down sampling, and then spatially aligned and semantically labeled with the corresponding BIM model. The BIMCompNet dataset was constructed by combining the public BIM model library and self-built models, and underwent preprocessing steps such as format standardization, component classification and attribute annotation. All data were de-identified and did not involve specific project privacy information. To ensure the fairness and reproducibility of the experimental results, all experiments were conducted under the same hardware environment. Each group of experiments was repeated 5 times and the average value was taken as the final result.

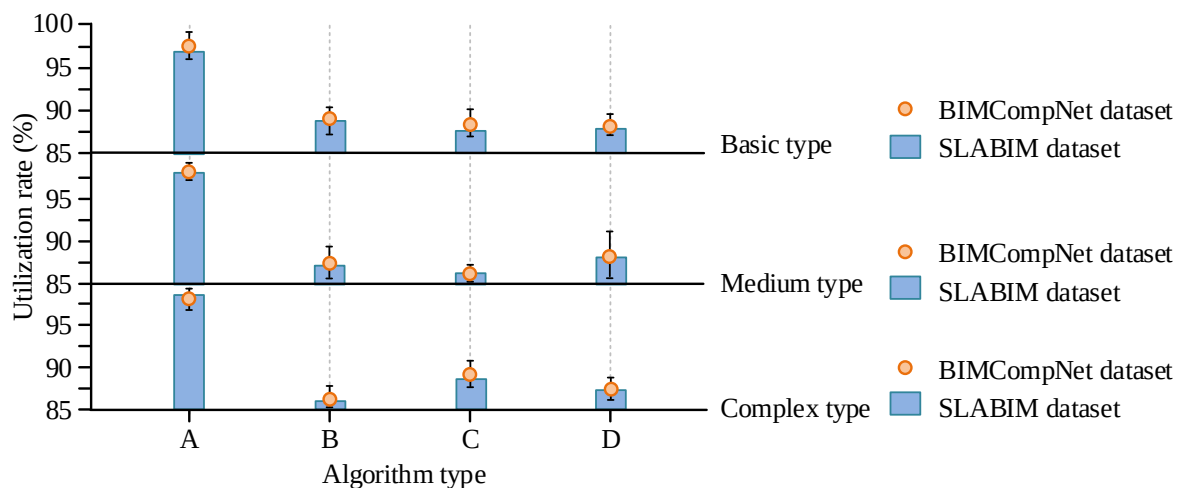


Fig. 7. Comparison of standard block utilization rate results

To verify the material control capability of the AIGSSA algorithm, this study tested the standard block utilization rate of the four algorithms for layout optimization under different wall conditions in the SLABIM and BIMCompNet datasets. The test results are shown in Figure 7. In Figure 7, A, B, C, and D represented the AIGSSA algorithm, PSO-ACO algorithm, ABC-GSO algorithm, and GSA-DE algorithm, respectively. As shown in Figure 7, in the SLABIM dataset, when the AIGSSA algorithm faced basic, medium, and complex wall conditions, its standard block utilization rates were 98.67%, 97.89%, and 98.24%, respectively. The standard block utilization rates of the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE were much lower than the AIGSSA algorithm. In the BIMCompNet dataset, the standard block utilization rates of the AIGSSA algorithm were 98.14%, 98.08%, and 97.83%, respectively, and remained higher than the other three comparison algorithms. In summary, the AIGSSA wall masonry layout optimization algorithm

maintained high standard block utilization rates when facing wall conditions of different complexity levels, verifying the excellent performance of the AIGSSA algorithm in optimization quality. Computational complexity refers to the time and space resources required by an algorithm during its execution, and it is an important metric for evaluating the efficiency of an algorithm. To demonstrate the superiority of the AIGSSA algorithm in material loss, this study tested the cutting loss rate of materials during layout optimization for the four algorithms in the two datasets. Cutting loss rate = $(1 - \text{Standard block usage area} / \text{Total wall area}) \times 100\%$. The test results are shown in Figure 8.

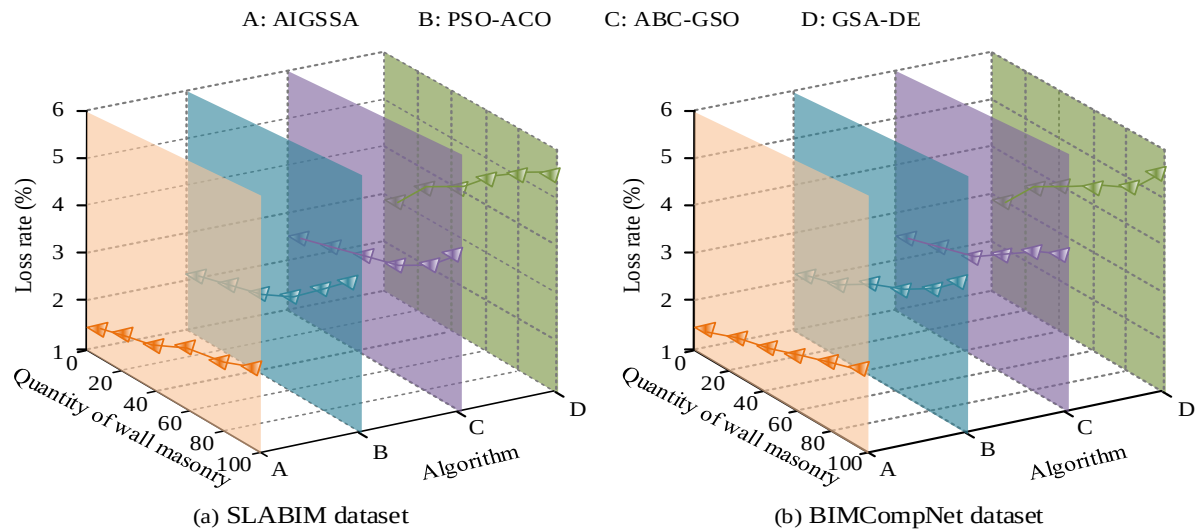


Fig. 8. Comparison of cutting loss rate test results

As shown in Figure 8(a), in the SLABIM dataset, when the number of wall masonries was 100, the cutting loss rate of the AIGSSA algorithm was 2.86%. For the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE, when the number of wall masonries was 100, the cutting loss rates were 4.21%, 4.33%, and 5.82%, respectively. In the BIMCompNet dataset in Figure 8(b), when the number of wall masonries reached 100, the cutting loss rate of the AIGSSA algorithm further decreased to 2.79%. In summary, the AIGSSA algorithm, with its unique optimization mechanism and efficient search strategy, could more accurately find the optimal solution during the wall masonry layout optimization process, thereby effectively reducing the cutting loss rate. Subsequently, to verify the rationality of the masonry sequence of the AIGSSA algorithm, this study tested the joint staggering compliance rate of the four algorithms during layout optimization in the above two datasets. The calculation formula for the staggered joint compliance rate is as follows: Staggered joint compliance rate = $(\text{Number of blocks meeting the staggered joint requirements} / \text{Total number of blocks}) \times 100\%$. The test results are shown in Figure 9.

According to Figure 9(a), in the SLABIM dataset, when the masonry layer data volume was 30 layers, the joint staggering compliance rate of the AIGSSA algorithm was 96.13%. The joint staggering compliance rates of the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE were 89.25%, 84.33%, and 82.14%, respectively. As shown in Figure 9(b), in the BIMCompNet dataset when the number of masonry layers was 30, the joint staggering compliance rate of the AIGSSA algorithm was 97.11%, and was much higher than the other three comparison algorithms. In summary, the AIGSSA algorithm dynamically enhanced the global search capability of layout schemes by constructing a multi-layer collaborative optimization architecture, demonstrating significant advantages in joint staggering compliance rate, thereby verifying the excellent performance of this algorithm in building infill wall masonry layout optimization.

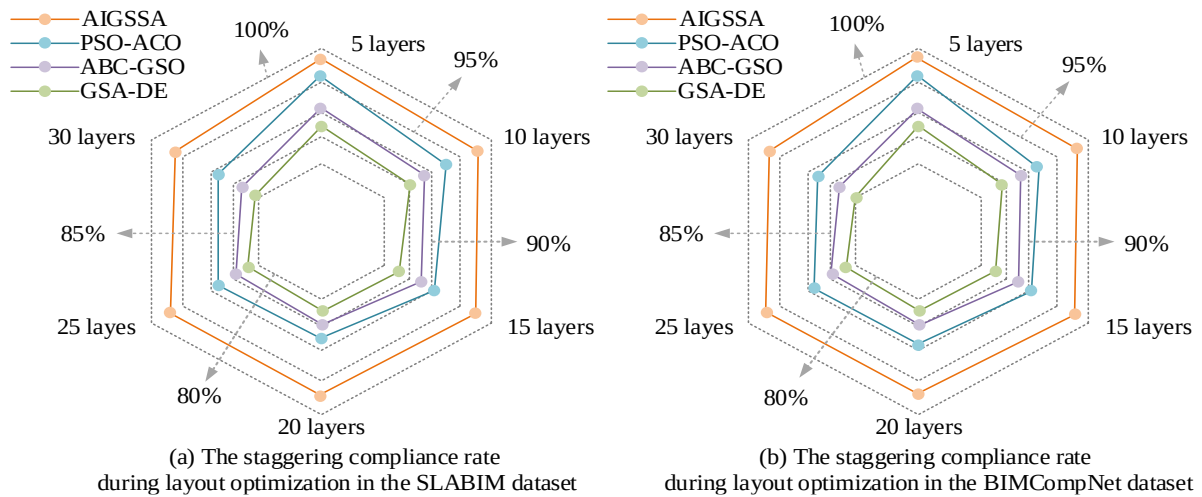


Fig. 9. Comparison of test results for joint staggering compliance rate

3.2 Verification of The Practical Layout Optimization Effect of AIGSSA

After verifying the optimization performance of the AIGSSA algorithm, the study proceeded to verify the engineering practicality of this algorithm. This study selected straight long walls with regular structural forms, straight axes, and no corners or abrupt cross-sections, and irregular walls including curved walls, polygonal corner walls, and obliquely cut walls as experimental scenarios. This study conducted comparative experiments with the three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE in the above two scenarios. To demonstrate the cost change advantage of the AIGSSA algorithm after layout optimization, this study tested the cost change rate of the four algorithms in the above two scenarios. The test results are shown in Figure 10.

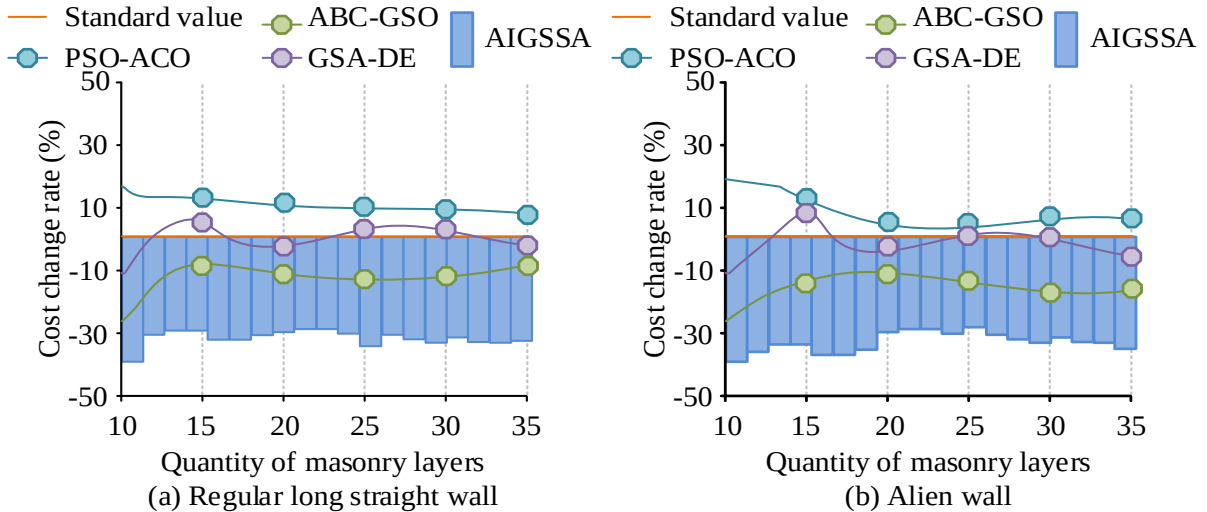


Fig. 10. Comparison of cost change rates in different scenarios

In Figure 10(a), in the regular straight long wall scenario when the number of masonries was 35, the cost change rate of the AIGSSA algorithm was -32.07%, which indicated that the AIGSSA algorithm reduced costs by 32.07% in practical application. The cost change rates of the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE were -9.68%, -3.56%, and 9.62%, respectively. In the irregular wall scenario in Figure 10(b), the cost change rate of the AIGSSA algorithm was -33.87%, which was also much better than the other algorithms. In summary, the AIGSSA algorithm could effectively reduce masonry layout costs, and the reduction magnitude was significantly better than other comparison algorithms, proving the practicality and efficiency of the AIGSSA algorithm in actual engineering. Next, to verify the capability of the layout optimization scheme obtained by the AIGSSA algorithm for long-term wall use, this study tested the masonry

cracking probability after simulated use for 3 years, 4 years, 5 years, and 6 years for the four algorithms. The study utilized finite element simulation to apply temperature cycling loads and humidity change loads to the wall model, simulating the thermal-humid coupling effect in the actual usage environment. At four time points of 3 years, 4 years, 5 years, and 6 years, the proportion of the wall elements with stress exceeding the tensile strength of the masonry was statistically calculated for each algorithm optimization scheme, which was used as the estimated value of the cracking probability. The test results are shown in Figure 11.

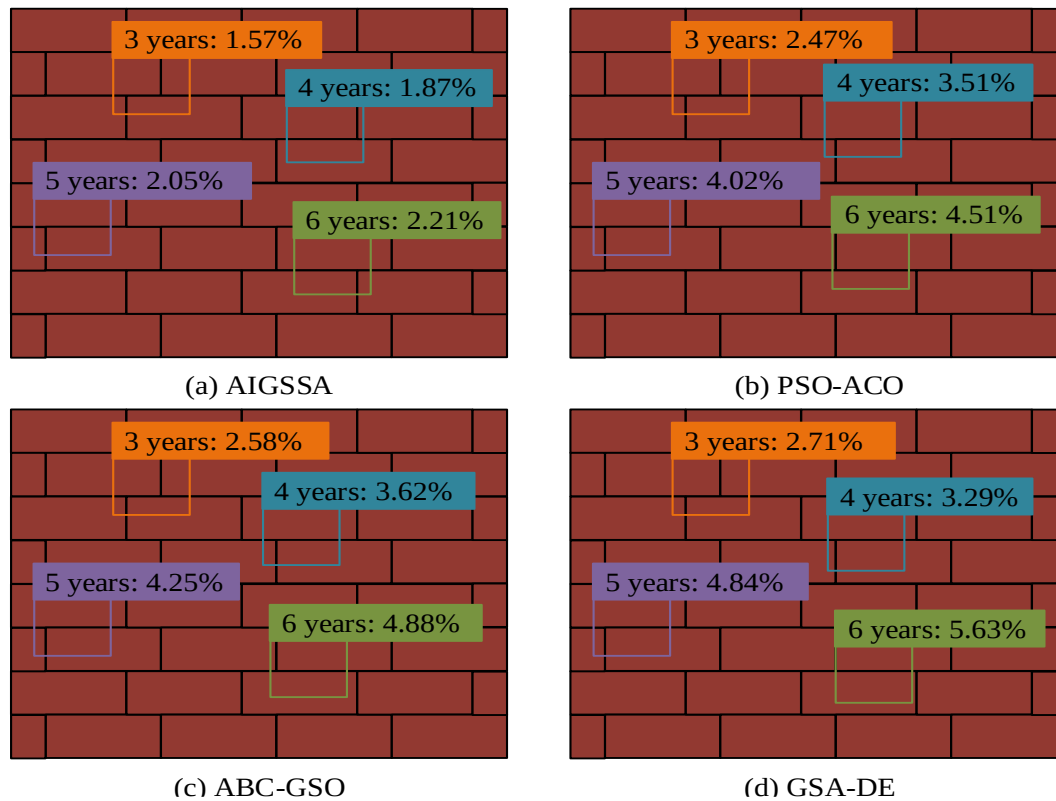


Fig. 11. Comparison of cracking probabilities in masonry of different ages

As shown in Figure 11(a), for the optimization scheme obtained by the AIGSSA algorithm, the masonry cracking probabilities after simulated use for 3 years, 4 years, 5 years, and 6 years were 1.57%, 1.87%, 2.05%, and 2.21%, respectively. In Figures 11(b), (c), and (d), the cracking probabilities after simulated use for 6 years for the optimization schemes obtained by the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE were 4.51%, 4.88%, and 5.63%, respectively. In summary, the AIGSSA algorithm demonstrated significant advantages in the cracking probability during simulated long-term wall use, verifying the effectiveness and engineering practicality of the AIGSSA algorithm in layout optimization. Finally, to verify the applicability of the AIGSSA algorithm in complex scenarios, this study tested the wall adaptation rate of the four algorithms when facing different wall types. Wall adaptability rate refers to the proportion of the arranged scheme optimized by the algorithm that can be applied in different types of walls, reflecting the algorithm's ability to adapt to complex engineering conditions. The calculation formula is: Wall adaptability rate = Number of applicable wall types / Total number of wall types $\times 100\%$. The test results are shown in Table 1.

According to the information in Table 1, in northern regions, the adaptation rates of the AIGSSA algorithm when facing four wall conditions of irregular walls, multi-opening walls, regular walls, and historic building restoration walls were 97.65%, 97.54%, 98.69%, and 96.62%, respectively. The wall adaptation rates of the other three comparison algorithms PSO-ACO, ABC-GSO, and GSA-DE were much lower than the AIGSSA algorithm. In southern regions, the adaptation rates of the AIGSSA algorithm when facing the same four wall conditions still maintained high adaptation levels and were much higher than the other three comparison algorithms. In summary, the AIGSSA wall

masonry layout optimization algorithm demonstrated extremely high adaptation rates when facing different types of walls in different regions.

Table 1. Comparison of compatibility results for different wall types (%)

Region	Algorithms	Irregular walls	Multi-opening walls	Regular walls	Historic building restoration walls
Northern region	AIGSSA	97.65±0.63*&#	97.54±0.58*&#	98.69±0.70*&#	96.62±0.55*&#
	PSO-ACO	92.23±0.81	91.64±0.80	89.65±0.84	88.27±0.79
	ABC-GSO	90.14±0.79	90.56±0.89	87.55±0.77	88.32±0.78
	GSA-DE	89.25±0.84	89.69±0.88	86.48±0.81	87.28±0.80
Southern region	AIGSSA	97.79±0.58*&#	97.46±0.60*&#	98.76±0.63*&#	96.75±0.65*&#
	PSO-ACO	92.36±0.60	93.52±0.61	90.12±0.59	88.34±0.62
	ABC-GSO	90.17±0.77	89.45±0.73	88.45±0.75	88.21±0.76
	GSA-DE	89.51±0.90	88.53±0.82	87.61±0.83	86.89±0.89

Note: In Table 1, * indicates that the results of AIGSSA are significantly different from those of PSO-ACO ($p < 0.05$); & indicates that the results of AIGSSA are significantly different from those of ABC-GSO ($p < 0.05$); # indicates that the results of AIGSSA are significantly different from those of GSA-DE ($p < 0.05$).

The research methods include GA, SA, SAS, AWOA, and ILS optimization algorithms. To evaluate the marginal performance improvement brought by each optimization layer, ablation experiments were designed. The comparison indicators include wall adaptability rate, arrangement efficiency, material utilization rate, and algorithm convergence speed. The experimental settings were consistent with the previous ones. Ablation comparisons were conducted for the five algorithms (GA, SA, SAS, AWOA, ILS). The ablation experiment results are shown in Table 2.

Table 2. Results of ablation experiments on the marginal performance improvement of each optimization layer

Ablation group	Wall fit rate (%)	Arrangement efficiency (%)	Material utilization rate (%)	Convergence speed (number of iterations)
GA	88.12	85.34	82.67	150
SA	90.45	87.21	84.33	130
SAS	91.78	90.56	87.89	110
AWOA	92.00	90.78	88.72	108
ILS	92.11	91.04	89.64	100
Complete model	95.77	96.42	93.15	80

As shown in Table 2, in the ablation experiments, as the optimization layers were gradually added, all performance indicators showed a steady improvement trend. From the GA model to the complete model, the wall adaptation rate increased from 88.12% to 95.77%, the arrangement efficiency increased from 85.34% to 96.42%, the material utilization rate increased from 82.67% to 93.15%, and the convergence speed was reduced from 150 iterations to 80 iterations. Among them, the leap from SAS to AWOA was the most significant, with the wall adaptation rate increasing by 0.22 percentage points, the arrangement efficiency increasing by 0.22 percentage points, the material utilization rate increasing by 0.83 percentage points, and the convergence speed accelerating by 2 iterations.

To evaluate the computational efficiency and scalability of the research method SIGSSA, the study conducted a comparative experiment by comparing it with modern reinforcement learning methods. The comparison methods included three deep reinforcement learning algorithms: DQN, PPO, and SAC. The comparison indicators were the average number of convergence iterations, the time taken for a single decision, and the final wall adaptation rate. The four algorithms were applied

to four different-sized wall arrangement scenarios in locations A, B, C, and D, and each scenario was run 20 times and averaged. The experimental results are shown in Table 3.

Table 3. Performance Comparison of Different Algorithms in Four Scenarios

Project		Average convergence iteration count	Single decision time (ms)	Final wall adaptation rate (%)
Scene A	DQN	120	45	92.34
	PPO	98	45	93.78
	SAC	105	38	93.12
	SIGSA	80	25	95.77
Scene B	DQN	121	45	92.11
	PPO	96	46	93.58
	SAC	104	40	93.36
	SIGSA	81	24	95.80
Scene C	DQN	119	48	92.00
	PPO	97	46	93.33
	SAC	106	39	93.24
	SIGSA	80	23	95.69
Scene D	DQN	122	49	92.06
	PPO	100	45	93.22
	SAC	102	40	93.36
	SIGSA	78	22	95.78

As shown in Table 2, SIGSSA demonstrated the best performance in all four scenarios. The average number of convergence iterations was 79.75, which was 33.9%, 18.6%, and 23.3% lower than DQN, PPO, and SAC respectively; the average time per single decision was 23.5 ms, which was 51.0%, 48.4%, and 40.5% lower than DQN, PPO, and SAC respectively; the final wall adaptation rate was 95.76%, which was 3.7%, 2.3%, and 2.5% higher than DQN, PPO, and SAC respectively. These results indicate that SIGSSA has significant advantages in terms of computational efficiency and wall adaptation quality. Moreover, it maintains stable performance in different scenarios, verifying its good generalization ability and robustness.

To evaluate the performance of SAS compared to the traditional A* algorithm and to prove the rationality of choosing SAS in the research, a set of comparative experiments was designed. In the same wall layout scenario, the path planning was conducted using both SAS and A* algorithms, and the average path length, average planning time, and path smoothness were recorded. The experimental results showed that SAS shortened the average path length by approximately 12% compared to A*, reduced the average planning time by approximately 35%, and improved the path smoothness by approximately 18%. This indicates that SAS outperforms the traditional A* algorithm in terms of path planning efficiency and path quality, further verifying its effectiveness and practicality in intelligent path guidance in complex wall environments.

4. Conclusions and Recommendations

Traditional building infill wall masonry layout optimization algorithms highly depended on the quality of initial schemes. When facing complex construction constraints, they had low layout efficiency, high material loss rates, and insufficient engineering adaptability. Therefore, this study proposed an intelligent masonry layout optimization algorithm for walls based on a multi-layer collaborative architecture. This algorithm generated diverse populations through GA, used SA for probabilistic jump optimization, introduced SAS to complete topology verification, and then combined AWOA with ILS to achieve deep exploration of the solution space. The experimental results showed that this algorithm performed excellently in the SLABIM and BIMCompNet dataset tests, with the highest standard block utilization rate reaching 98.76%, the lowest cutting loss rate at 2.79%, and the peak joint staggering compliance rate reaching 97.11%. In irregular wall and regular wall scenarios, this algorithm reduced costs by up to 23.87%, and the cracking probability

after simulated use for 6 years was 2.21%. In summary, the AIGSSA algorithm could accurately meet the multi-constraint requirements of masonry layout, effectively reduce construction complexity and material waste, and provide reliable technical support for the development of building industrialization. However, this algorithm still has shortcomings in collaborative layout across multiple building types. Future research will continue to optimize in engineering practice to improve the algorithm's adaptability to large-scale scenarios. To achieve scalability in industrial-scale masonry projects, the research will further explore the deep integration of this algorithm with the BIM system, develop an intelligent layout module that can provide real-time feedback, and introduce multi-objective optimization strategies to balance cost, schedule, and quality. At the same time, the research will attempt to combine reinforcement learning and transfer learning techniques to enable the algorithm to have the ability to transfer knowledge across projects, reducing the cost of re-training for new scenarios. Additionally, it will further explore the algorithm's layout capabilities in irregular walls, curved walls, and complex joints, and build a unified optimization framework for all types of building components.

Reference

- [1] Sberna P, Demartino C, Vanzi I. Cost-effective topology optimization of masonry structure reinforcements by a linear static analysis-based GA framework. *Bull Earthquake Eng.* 2024;22(8):4143-67. <https://doi.org/10.1007/s10518-024-01900-5>
- [2] Portioli FPA, Lourenço PB. Nonlinear static analysis of masonry structures with mortar joints and cracking units by optimization-based rigid block models. *Earthquake Eng Struct Dynam.* 2024;53(13):3963-82. <https://doi.org/10.1002/eqe.4206>
- [3] Keshavarzfard R, Naderi A. A sustainable HazMat logistic network design considering scale of economy and route sensitivity and solving with Hybrid GA-SA. *Int J Res Ind Eng.* 2024;13(3):257-73.
- [4] Atasi C, Kern J, Ramprasad R. Design of recyclable plastics with machine learning and genetic algorithm. *J Chem Inf Model.* 2024;64(24):9249-59. <https://doi.org/10.1021/acs.jcim.4c01530>
- [5] Du Q, Qi X, Zou PXW. Bi-objective optimization framework for prefabricated construction service combination selection using genetic simulated annealing algorithm. *Eng Constr Archit Manag.* 2024;31(10):3921-45. <https://doi.org/10.1108/ECAM-10-2022-1000>
- [6] Gangadevi E, Rani RS, Dhanaraj RK. Spot-out fruit fly algorithm with simulated annealing optimized SVM for detecting tomato plant diseases. *Neural Comput Appl.* 2024;36(8):4349-75. <https://doi.org/10.1007/s00521-023-09295-1>
- [7] Kosanoglu F, Atmis M, Turan HH. A deep reinforcement learning assisted simulated annealing algorithm for a maintenance planning problem. *Ann Oper Res.* 2024;339(1):79-110. <https://doi.org/10.1007/s10479-022-04612-8>
- [8] Pashaei E, Pashaei E. Hybrid binary COOT algorithm with simulated annealing for feature selection in high-dimensional microarray data. *Neural Comput Appl.* 2023;35(1):353-74. <https://doi.org/10.1007/s00521-022-07780-7>
- [9] Lim KCW, Wong LP, Chin JF. Simulated-annealing-based hyper-heuristic for flexible job-shop scheduling. *Eng Optim.* 2023;55(10):1635-51. <https://doi.org/10.1080/0305215X.2022.2106477>
- [10] Zanzazzo E, Ceschia S, Dovier A. Solving the medical student scheduling problem using simulated annealing. *J Sched.* 2025;28(2):233-46. <https://doi.org/10.1007/s10951-024-00806-z>
- [11] Valentino J, Gilbert M, Gueguin M. Limit analysis of masonry walls using discontinuity layout optimization and homogenization. *Int J Numer Method Eng.* 2023;124(2):358-81. <https://doi.org/10.1002/nme.7124>
- [12] Son PVH, Souliya FV. A hybrid ant lion optimizer (alo) algorithm for construction site layout optimization. *J Softw Civ Eng.* 2023;7(4):50-71.
- [13] Tan Y, Yu D, Yang S. Precast concrete component yard layout optimization considering safety and efficiency. *J Asian Archit Build Eng.* 2025;24(4):2800-16. <https://doi.org/10.1080/13467581.2024.2373837>
- [14] Shang D, Zou G. Optimizing Architectural Layouts and Spatial Configurations in Polar Environments Using Image Segmentation and Semantic Mapping Techniques. *Int J Mach Learn Comput.* 2025;42(4):1977. <https://doi.org/10.18280/ts.420412>
- [15] Zheng Z, Li Y, Torres J. An integrated method of automated layout design and optimization for modular construction. *Eng Constr Archit Manag.* 2024;31(3):1016-36. <https://doi.org/10.1108/ECAM-04-2022-0329>
- [16] Groumpos PP. A Critical Historic Overview of Artificial Intelligence: Issues, Challenges, Opportunities, and Threats. *Artif Intell Appl.* 2023;1(4):197-213. <https://doi.org/10.47852/bonviewAIA3202689>

- [17] Aydoğan E, Cetek C. Aircraft route optimization with simulated annealing for a mixed airspace composed of free and fixed route structures. *Aircraft Eng Aerospace Technol.* 2023;95(4):637-48. <https://doi.org/10.1108/AEAT-11-2021-0343>
- [18] Quankun M, Yanfei Z, Jinliang G. Traversal path planning of agricultural robot based on memory simulated annealing and A* algorithm. *J South China Agric Univ.* 2023;41(4):127-32.
- [19] Lin Q, Ma H. SACHA: Soft actor-critic with heuristic-based attention for partially observable multi-agent path finding. *IEEE Robot Autom Lett.* 2023;8(8):5100-7. <https://doi.org/10.1109/LRA.2023.3292004>
- [20] Zhang J, Li H, Parizi MK. HWMWOA: A Hybrid WMA-WOA algorithm with adaptive cauchy mutation for global optimization and data classification. *Int J Inf Technol Decis Mak.* 2023;22(4):1195-252. <https://doi.org/10.1142/S0219622022500675>

Synergistic stabilization of highly gypseous soils using nano-calcium carbonate and class C fly ash

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Abstract

To address the stability issues associated with highly gypseous soils under water exposure, the effects of nano-calcium carbonate (NCC), Class C fly ash (FA), and their combination on the shear strength and compaction characteristics of highly gypseous soils containing 56% gypsum were systematically investigated. In laboratory tests, three FA concentrations (5%, 10%, and 15%) and three NCC concentrations (0.5%, 1%, and 1.5%) were used as well as a blend of 0.5% NCC and 10% FA. Results indicate remarkable improvements in cohesion and internal friction angle of all treated samples, with the percentage of improvement being most pronounced under soaked conditions. The best combination (0.5% NCC + 10% FA) showed the highest after-soaking cohesion (39.2 kPa) and friction angle (41.8°) with the improvement ratios of 600% and 55.97%, respectively, compared to the untreated soil. Maximum dry density (MDD) increased with NCC content, reaching 1.782 g/cm³ at 1.5% NCC, while increasing FA content led to a decrease in MDD from 1.67 to 1.605 g/cm³. The blend had an MDD of 1.70 g/cm³, effectively optimizing the opposing effects of the two additives on compaction. The improvements are primarily attributed to the void-filling effect of FA, nanoscale densification and enhanced particle bonding by NCC, and the synergistic physicochemical interaction between the two additives. Early pozzolanic reactions may also have contributed. The improvements were further validated by microstructural analyses (XRD and SEM), which confirmed the formation of cementitious products. The mixture proved to be more resistant to the loss of strength due to soaking and more durable in different moisture conditions. The results of this study fill a critical gap in the literature on the combined stabilization of NCC and Class C FA and provide practical suggestions for improving the mechanical and compaction properties of gypseous soils in arid and semi-arid regions.

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1. Introduction

Gypseous soils are regarded as one of the most problematic soils in geotechnical and transportation engineering, particularly in the dry and semi-arid regions. These soils are found on some 1.5 million km² of Earth's surface, according to the United Nations' Food and Agriculture Organization [1]. Gypseous soils cover about 32% of the total geographical area of Iraq and are widely distributed in different parts of the country as shown in Fig. 1 [2]. For gypsum to form in soils, there must be an external supply of gypsum, arid climate conditions and temperatures above 20°C. Gypsum, accumulated in soil deposits under these environmental conditions, plays an important role in their engineering properties. In civil engineering, gypseous soils are recognized as such on the basis of their gypsum content irrespective of texture or visual characteristics. Other researchers have found some limitations to this categorization, stating quantities of gypsum between 3% and 6% [3].

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Gypsum is a naturally occurring mineral, mostly calcium sulphate dihydrate ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$). It is commonly found in sedimentary rocks of various geological ages, and it is also frequently found in crystalline deposits of various sizes in soil layers [4]. Gypsum is a temporary cement between soil particles in the absence of water, but the bond is weak and becomes greatly weakened when the soil is wet. The main engineering problem of gypseous soils is their metastable structure and very high susceptibility to collapse in the wet state. Water penetration into the soil causes the dissolution of the gypsum minerals and, as a consequence, the bonds between particles are broken down, thus leading to a fast change of the soil skeleton without any other external load [5]. This can thus result in large settlements and a considerable loss of strength of the soil in a very short time. The gypseous soils are unsuitable for many engineering applications unless proper stabilization measures are adopted for such soils. Moreover, the intensity of these problems increases with increasing gypsum content as the physical and mechanical properties of the soil become more sensitive to changes in moisture, temperature and loading conditions [6]. The collapse behavior of gypsiferous soils has led to serious engineering problems in many infrastructure projects. These challenges are manifested in excessive settlement under foundations, failure of pavement layer, cracking and collapse of residential structures and deterioration of hydraulic structures and irrigation systems [7]. Such failures may lead to significant economic losses and severe safety hazards, especially in areas constructed on highly gypsiferous deposits. This has made the enhancement of engineering properties of these soils a prime area of research in geotechnical engineering.

Many researchers have studied the application of various additives and stabilization agents for solving the problems of gypseous soils. The main aim of these studies has been to decrease soil collapsibility and to improve its mechanical properties in dry and wet conditions. Zedan et al. [8] conducted an extensive study on the gypseous soil with 34–36% of gypsum to investigate the effect of addition of concrete waste and asphalt waste on compaction characteristics, shear strength, collapsibility, and California Bearing Ratio (CBR). They observed that the dry density and the cohesion increased and the moisture content and the collapse potential decreased with the addition of 6 % concrete waste. Similarly, the addition of 2% asphalt waste increased the cohesion and helped in the reduction of soil collapse. The best overall performance was obtained when mixing both waste materials, which increased the dry density, cohesion and CBR values and decreased the tendency for collapse.

In 2020, Al-Obaidi et al. [9] studied the effect of nano-silica and silica fume on soils with 62% gypsum. The effect of each additive on different soil properties was studied. The results showed that the addition of silica fume (SF) and nano-silica fume (NSF) into the highly gypseous soils improved the engineering properties of the soils significantly in dry and wet conditions. Specifically, the addition of nano-silica enhanced the friction angle and reduced the potential for collapse of gypsum soils at nano-silica contents of 1% to 5% under all moisture conditions. Nano-silica at a concentration of 3% resulted in the highest performance. The use of alternative stabilizers for gypseous soils has also been investigated. For instance, Theyab et al. [10] studied effect of casein as a stabilizing agent in 2022. Results indicated that addition of casein decreased the maximum dry density and increased the optimum moisture content. However, this treatment reduced the collapse potential significantly by between 65 and 80 percent and improved the strength characteristics of the soil. Emad and Salman [11] studied the behavior of sandy gypseous soil containing 54% of gypsum and treated with nanomaterials at 1%, 2%, and 3%. They found that the collapse potential decreased and the shear strength increased with increasing concentration of nanomaterials. Moreover, the degree of improvement was higher with longer curing times, indicating that nanomaterials are effective in stabilizing gypseous soils. Bağrıaçık et al. [12] investigated the performance of clay stabilized with conventional CaCO_3 and nano- CaCO_3 . The results demonstrated that nano- CaCO_3 achieved superior improvement at a dosage of only 0.75%, which was attributed to its nanoscale particle size and high specific surface area. The nanoparticles effectively penetrated the soil micro-voids, promoted interparticle bonding during curing, and contributed to the development of a denser soil structure. Consequently, the treated soil exhibited higher bearing capacity and reduced settlement compared with soil treated with conventional CaCO_3 .

Mohammadi et al. [13] investigated the effect of nano-calcium carbonate (NCC) on the geotechnical properties of clayey sand (SC) soil by incorporating 0.3%, 0.7%, 1.1%, and 1.5% NCC. The results showed that NCC significantly increased the uniaxial compressive strength, stiffness, and particle recrystallization while reducing soil deformation. The optimum NCC content was 0.7% for soils containing 10% and 20% clay and 1.1% for soil containing 30% clay, demonstrating that the optimum dosage depends on soil composition and mineralogy. More recently, Muhauwiss et al. [14] studied the performance of gypseous soil stabilized with a biopolymer, Arabic gum, under repeated wetting and drying cycles. The soil tested had about 40 percent of the gypsum. From the results, it was observed that the shear strength increased with the addition of 2% Arabic gum after five cycles where internal friction angle and cohesion reached a maximum value. However, with further cycling, the unconfined compressive strength decreased. The results indicated that Arabic gum can improve the soil stability in the short term, but its effectiveness decreases with the increase in wetting and drying cycles. Jassam et al. [15] specifically reported that treatment of gypseous soil with 45% gypsum with the silane coupling agent KH-570 at dosages of 2%, 4% and 6% resulted in significant decrease of collapse potential and notable improvement of shear strength. Their study showed that cohesion and internal friction increased by 89% and 9.3% under semi-dry condition and 119% and 250% under soaked condition, respectively compared to untreated samples. KH-570 is more costly than traditional stabilizers like lime or cement. But KH-570 is an economical solution for stabilizing gypseous soils because of its low dosage and excellent waterproofing capability.

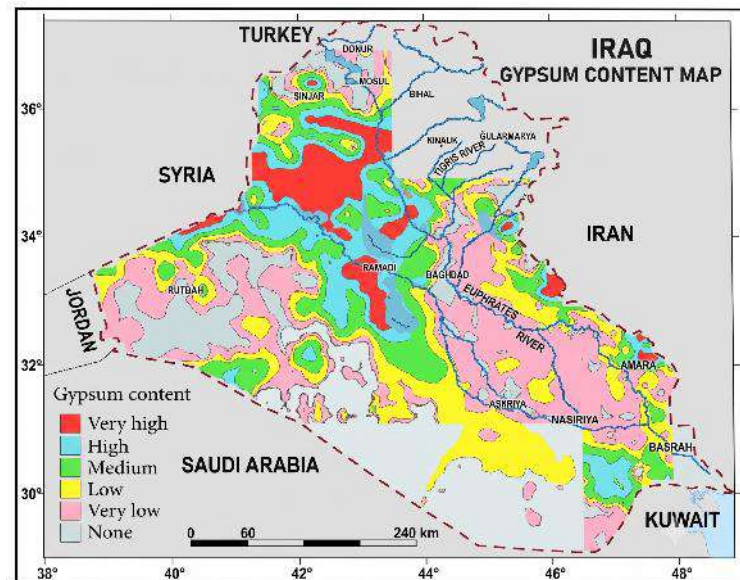


Fig. 1. Map of gypsiferous soil in Iraq [2]

On the other hand, fly ash is often used as soil improvement material in geotechnical engineering. Its application improves compressibility, shear strength and cohesion of soil, as well as reducing earth pressure. Fly ash is used in the base and sub-base layers of road construction as a binder or filler thus resulting in the increased strength of the road in the long run, reduced swelling of expansive soils and enhanced bearing capacity [16,17]. Edil et al. [18] investigated the effectiveness of self-cementing Class C fly ash for stabilizing soft fine-grained soils. Their results showed that the addition of 10% and 18% Class C fly ash significantly improved the California Bearing Ratio (CBR) and resilient modulus (M_r), with the 18% fly ash content producing the highest values among the investigated mixtures. These findings demonstrate the effectiveness of Class C fly ash as a stabilizing agent for improving soil engineering properties. Despite its well-documented benefits, the use of high-calcium Class C fly ash in sulfate- and gypsum-rich soils requires careful consideration. Previous studies have reported that calcium oxide and alumina released from Class C fly ash may react with sulfate ions in the presence of water, leading to the formation of ettringite. Although controlled ettringite formation may contribute to strength development, excessive formation can induce swelling, cracking, and volumetric instability in sulfate-bearing soils.

Therefore, understanding this mineralogical reaction is essential when evaluating the suitability of Class C fly ash for stabilizing highly gypseous soils [19]. Previous studies have demonstrated the effectiveness of various stabilizing agents for improving the engineering behavior of gypseous soils. Nevertheless, several limitations remain. Most investigations evaluated individual additives and focused on specific engineering properties, with limited attention given to the combined performance of different stabilizers. In addition, relatively few studies investigated very highly gypseous soils under both dry and soaked conditions or examined the underlying microstructural mechanisms.

Although significant advances have been made in understanding the stabilization of gypseous soils in the previous studies, there are still some critical research gaps. Most of the previous studies have focused on conventional additives, waste materials, biopolymers, silica-based stabilizers or general nanomaterials. However, the potential of nano calcium carbonate to improve particle bonding and increase soil strength has been investigated in few studies concerned with the strength behaviour of gypseous soils. Likewise, the use of Class C fly ash in stabilizing the highly gypseous soil has not been adequately explored. Also, literature relating to the combined application of nano calcium carbonate and Class C fly ash is scarce and the interaction between the two additives and their combined influence on the strength characteristics of soil is not well understood. The synergy between the high calcium content of Class C fly ash and the nucleation effect of nano-calcium carbonate is expected to enhance particle bonding and promote the formation of cementitious products, thereby improving the shear strength of very highly gypseous soil. The present study is conducted to examine the combined effects of nano calcium carbonate and Class C fly ash on shear strength parameters of gypseous soil by using direct shear tests.

2. Materials and Methodology

2.1. Gypseous Soil

The current study includes a soil sample taken from Tikrit University, in Salah ad-Din Governorate at a depth of 0.5 m. It contains 56 percent of gypsum. Fig. 2 shows the particle size distribution. The soil physical and chemical parameters are shown in Table 1. The soil is classified as poorly graded sand (SP) according to the Unified Soil Classification System (USCS) [20]. The coefficients of curvature (C_c) and uniformity (C_u) shown in Table 1 indicate that the soil is poorly graded. The chemical study indicates that the soil is highly gypseous. The pH values reported show slightly alkaline conditions. Barazanji [21] classified the soil as Very Highly Gypsiferous. These results highlight the need for geotechnical engineers to pay close attention to the hazards associated with such soils in engineering applications. The classification of gypsum soils according to Barazanji is shown in Table 2.

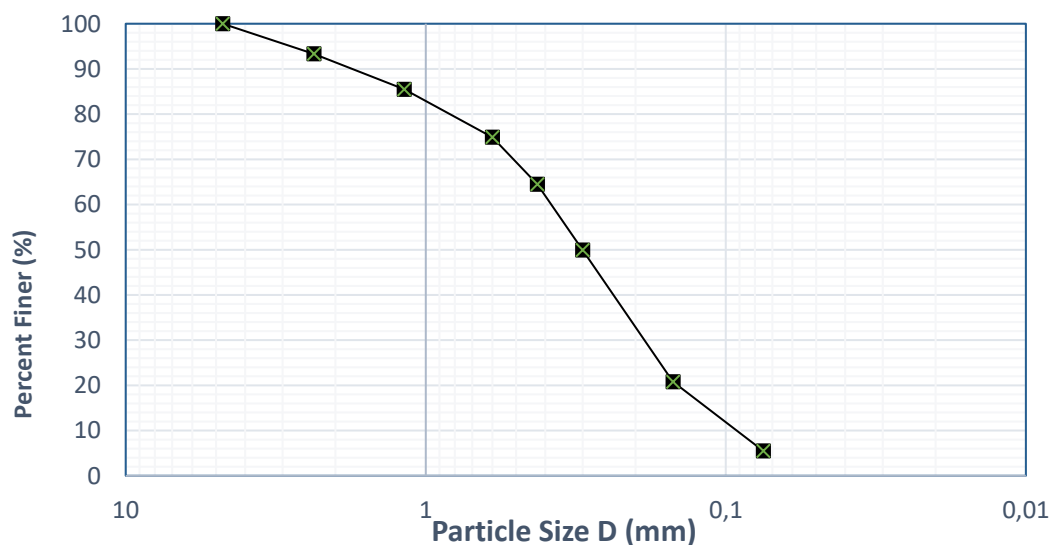


Fig. 2. Particle size distribution of the gypseous soil sample

Table 1. Physical and chemical characteristics of the gypseous soil

Properties/Description		value	Standard
Specific Gravity (Gs)		2.41	(ASTM D854)[22]
Atterberg limits	Liquid limit (L.L)%	25	(ASTM D4318) [23]
	Plastic limit (P.L)%	N.P	
	D10 (mm)	0.09	
	D30 (mm)	0.19	
	D60 (mm)	0.39	
Coefficient of Uniformity (Cu)		4.44	(ASTM D4318) [23]
Coefficient of Curvature (Cc)		1	
USCS (Unified Soil Classification system)		SP	
Unit Weight at Field (γ_f) g/cm ³		1.54	(ASTM D1556) [24]
Field moisture content, (ω)%		6.5	
Maximum Dry Unit Weight (γ_{dmax}) g/cm ³		1.71	(ASTM D1557) [25]
Optimum Moisture Content (O.M.C)%		11.5	
Gypsum Content %		56	
Total Soluble Salts (T.S.S) %		61	
pH		8.04	

Table 2. Classification of soil depends on gypsum content [21]

Gypsum Content %	Classification
0-0.3	Non Gypsiferous
0.3-3	Very Low Gypsiferous
3-10	Low Gypsiferous
10-25	Medium Gypsiferous
25-50	Highly Gypsiferous
>50	Very Highly Gypsiferous

2.2 Fly Ash (FA)

Fly ash is a fine powder, a by-product of burning pulverized coal in electrical power generating plants. The flue gases are treated with bag filters or electrostatic precipitators. Fly ash has pozzolanic and cementitious properties that make it important in civil engineering, building, and environmental applications [26]. The fly ash used in this study was manufactured by Euro build Construction Chemicals and Coatings LLC, located in the Industrial Area, Umm Al Quwain, Sharjah, United Arab Emirates, and produced in 2025. The material is shown in Fig. 3.



Fig. 3. Class C fly ash (FA)

The fly ash is of Class C and contains a high percentage of calcium ($\text{CaO} > 10\%$) and is pozzolanic and cementitious in nature. These characteristics make the gypsum soil more stable and less liable to collapse. Fly ash was used as an additive in the ratios of 5%, 10% and 15% in this investigation. Table 3 lists the manufacturing specifications, physical characteristics, and oxide composition of the Class C fly ash (FA).

Table 3. Physical properties and oxide composition of Class C FA

Property	Description/value	Oxide Composition	Content %
Color	White to Light Grey	SiO ₂	38.6
Specific Gravity	2.37 g/cm3	Al ₂ O ₃	22.16
Blaine Fineness	6500 cm2/g	Fe ₂ O ₃	5.9
Moisture Content	% 1.5-1	CaO	22.3
General Appearance	Fine Powder	MgO	6.86
Particle size	Predominantly < 45 μm	SO ₃	0.82
ASTM Classification	ASTM C618- Class C Fly Ash	Na ₂ O	0.22
		K ₂ O	0.29
Loss on Ignition (LOI)			≤ 2.5

2.3 Nano Calcium Carbonate (Nano- CaCO_3)

The word 'nano' comes from the Greek for one billionth of a meter. One nanometer is equal to 10^{-9} m [27]. Various chemical and physical techniques can be used for the synthesis of Nano calcium carbonate (Nano- CaCO_3). Common methods are sol-gel, wet carbonation and chemical precipitation.



Fig. 4. Nano- CaCO_3 (NCC)

Table 4. Specifications of Calcium Carbonate Nanoparticles (CaCO_3 , 97.5%, 15–40 nm).

Property	Typical Value
Form / Appearance	White Nano Powder
Average Particle Size	15 – 40 nm
Specific Surface Area	$\approx 40 \text{ m}^2/\text{g}$
Chemical Composition	$\text{CaCO}_3 > 97.5\%$
MgO (Magnesium Oxide)	$< 0.9\%$
Fe (Iron)	$< 0.1\%$
Moisture Content	$\approx 0.5\%$
Whiteness	$\approx 93\%$
Bulk Density	$\approx 0.68 \text{ g/cm}^3$
pH	8.0 – 9.0
Insoluble in HCl	$< 0.5\%$
Crystal Shape	Cubic
Manufacturer	Sky Spring Nanomaterials, Inc.

These methods result in very small particles with large surface area and controlled geometries. The synthesis method directly affects the particle size and morphology. This subsequently influences the effectiveness of particle filling in soil pores, improves binding and contributes to interactions with clay minerals [28]. The present study used Nano- CaCO_3 as an additive with fly ash to improve the engineering properties of gypseous soils. Fig. 4 shows a sample of Nano- CaCO_3 (NCC), and the main specifications are given by the manufacturer in Table 4. Nano- CaCO_3 mixing percentages of 0.5%, 1% and 1.5% were tested.

2.4. Sample Preparation

The maximum dry density and optimum moisture content, which were determined by the Modified Proctor Test, were used to prepare the samples after treatment with Class C fly ash, calcium carbonate nanoparticles, or a mixture of both in accordance with ASTM D1557 [25]. Specimens were prepared for testing with the parameters shown in Table 5. All additions were calculated as a percentage of the dry weight of the soil. The soil was oven-dried at 45°C (ASTM D2216-19) [29] and mixed thoroughly with the required amount of NCC and/or FA in dry condition to make sure the distribution was even. The dry mixing was performed manually for a minimum of 5 minutes. Gradually distilled water was added and continuously mixed until a homogeneous mix was obtained and the desired moisture content was achieved. The soil required for each specimen was divided into three equal portions based on mass and placed sequentially into the direct shear box. Each portion was compacted using a hand rammer with seven controlled blows. The number of blows was established through preliminary trials, which showed that seven blows per layer were sufficient to compact the entire soil mass and achieve the target maximum dry density obtained from the Modified Proctor compaction test. After compaction of the final layer, the flatness of the specimen surface was verified before conducting the direct shear test. The sample was tested directly in a semi-dry state. For soaked tests, the selected normal stress was applied before inundation, and the specimen was soaked under this load in distilled water for 24 h prior to shearing. The 24 h soaking period was selected in accordance with previous studies on gypseous soils for short-term shear strength evaluation [30]. The gypseous soil was stabilized by adding 5%, 10%, and 15% of fly ash and 0.5%, 1.0% and 1.5% of nano calcium carbonate (NCC). After establishing the optimal percentages, additional samples were prepared using the same mixing and compaction procedures as per ASTM. Subsequently, the specimens were subjected to testing.

Table 5. Compaction test results

Soil Type	NCC%	FA %	γ_{dmax} (g/cm ³)	O.M.C %
56% Gypsum	0	0	1.715	11.5
	0.5	0	1.76	11.1
	1	0	1.77	10.46
	1.5	0	1.782	10.1
	0	5	1.67	12.75
	0	10	1.62	13.6
	0	15	1.605	14.8
	10	0.5	1.7	12.2

2.5 Direct Shear Test

Direct shear test was performed using a shear testing apparatus according to ASTM D3080 (2011) [31]. A 5 kN load cell was used to measure the shearing force. The specimen was sheared at a displacement rate of 1 mm/min in a direction parallel to the moving shear box. Normal stresses of 50, 100, and 150 kPa were applied during the direct shear tests. Before shearing, each specimen was consolidated under the applied normal stress until the dial gauge reading stabilized. Drainage was maintained throughout both the consolidation and shearing stages in accordance with ASTM D3080. For soaked specimens, the water reservoir in the shear box was filled after applying the normal stress, and the specimens were soaked for 24 h before the shear test. The test was performed to determine shear strength parameters, a cohesion (c) and internal friction angle (ϕ) for untreated and treated gypseous soil stabilized with NCC and FA. The dimensions of the tested

specimens were 60 x 60 x 25 mm. Table 6 presents the shear strength parameters of untreated soil (equivalent field unit weight) and blend-treated soils (maximum dry unit weight) under dry and soaked conditions.

Table 6. Shear strength parameters of untreated and blend-treated gypseous soils

Parameter	Untreated soil	Blend-treated soil
Cohesion (c) (kPa)	15.5	42
Cohesion (c) after soaking (kPa)	5.6	39.2
Friction angle (ϕ) (°)	32.12	42.5
Friction angle (ϕ) after soaking (°)	26.8	41.8

3. Results and Discussions

The results showed that the dry density of gypseous soil treated with FA decreased slightly and the Optimum Moisture Content (O.M.C) increased. The soil's dry density increased slightly with NCC treatment, while its optimal water content decreased. The reason for this behavior is that the specific gravity of the soil is higher than that of fly ash, whereas the increase in density with NCC is attributed to enhanced packing and densification due to its nano-size effect. Fig. 5 and 6 show a correlation between FA, NCC concentrations, and maximum dry density of soil, respectively.

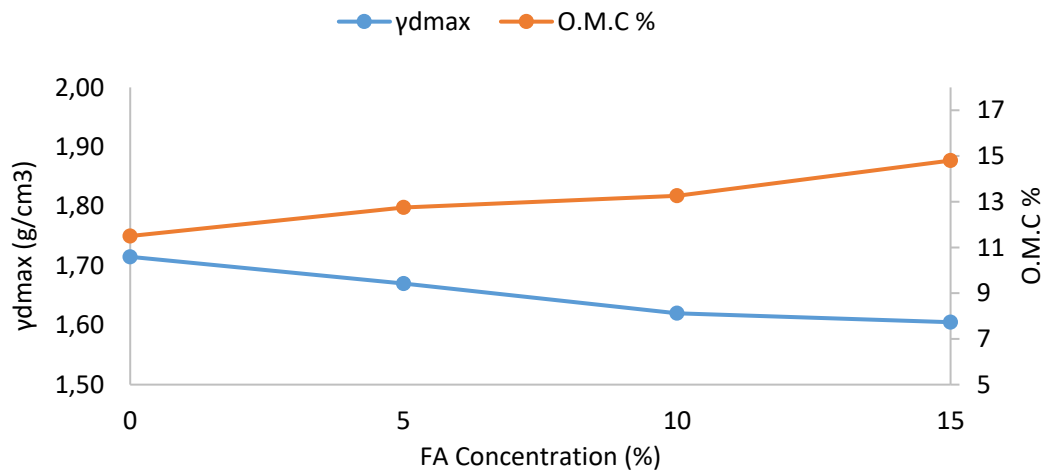


Fig. 5. Relationship between FA concentration, maximum dry density, and optimum moisture content of gypseous soil

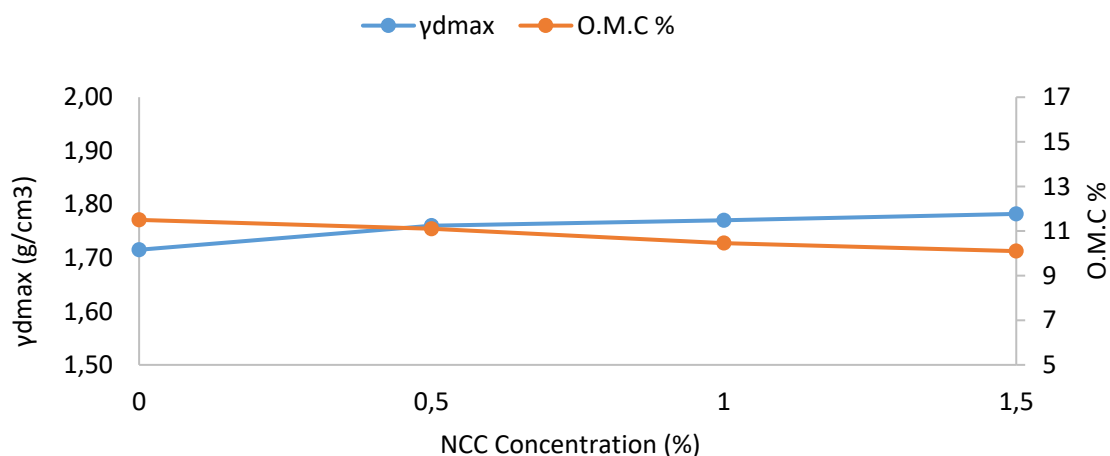


Fig. 6. Relationship between NCC concentration, maximum dry density, and optimum moisture content of gypseous soil

3.1. Direct Shear of Treated Soil

A series of laboratory tests were conducted to study the effect of fly ash (FA) and nano calcium carbonate (NCC) on the shear strength parameters (c , ϕ) of gypseous soil with various concentrations of additives. Three FA concentrations (5%, 10% and 15%) and NCC concentrations (0.5%, 1%, and 1.5%) were incorporated into the testing program. Additionally, the mixture of 0.5% NCC and 10% FA was applied to soil. As shown in Fig. 7, the overall increase in shear strength for all treated gypseous soil samples (56% gypsum content) is apparent compared with untreated soil in both dry and soaked conditions. The figure shows that the samples treated with the blend of 0.5% NCC and 10% FA have the highest shear strength envelopes for all normal stress levels. The trend directly indicates the better performance of the combined additives compared to FA or NCC alone. Representative photographs of the specimens treated with the blend of 0.5% NCC and 10% FA after direct shear testing under dry and soaked conditions are presented in Fig. 8, illustrating the physical deformation and failure patterns developed along the shear plane.

Figures 9 and 10 show the effect of FA content on the cohesion and internal friction angle of gypseous soil under semi-dry and soaked conditions. In general, both shear strength parameters increased with increasing FA content under both testing conditions. Under semi-dry conditions, the maximum cohesion and internal friction angle occurred at 15% FA. In contrast, under soaked conditions, both parameters increased up to 10% FA and then decreased slightly at 15% FA, indicating that 10% FA was the optimum content for improving the shear strength parameters after soaking. Overall, the observed improvement in shear strength is consistent with previous studies, which reported that fly ash enhances the engineering properties and strength characteristics of soils, although the optimum dosage depends on soil type, gypsum content, and testing conditions [16- 18].

Similarly, Figs. 11 and 12 show the effect of NCC content on the cohesion and internal friction angle of gypseous soil under semi-dry and soaked conditions. In general, both shear strength parameters increased progressively with increasing NCC content under both testing conditions. The maximum cohesion and internal friction angle were obtained at 1.5% NCC under both semi-dry and soaked conditions, indicating that 1.5% was the optimum NCC content within the investigated range. Overall, this trend agrees with previous studies, which reported that nano-calcium carbonate enhances the engineering properties and strength characteristics of soils through improved particle bonding, although the optimum NCC content varies depending on soil type and testing conditions [9, 11-13]. Focusing on the quantitative results, the cohesion (c) after soaking reached 27.6 kPa with 10% FA, 33 kPa with 1.5% NCC, and 39.2 kPa for the blend (0.5% NCC + 10% FA), compared to 5.6 kPa for the untreated soil. The improvement ratios of cohesion in the dry state were 116.19% for FA, 132.26% for NCC and 170.97% for the blend. After soaking, these enhancement ratios increased dramatically to 392.86% (FA), 489.29% (NCC), and 600% (blend), as shown in Fig. 13. With respect to the internal friction angle (ϕ), before soaking, values were 32.12° for untreated soil, 36.5° for 15% FA, 36° for 1.5% NCC, and 42.5° for the blend, with corresponding improvement ratios of 13.64% (FA), 12.08% (NCC), and 32.32% (blend). After soaking, ϕ increased from 26.8° (untreated) to 34.5° (10% FA), 34.6° (1.5% NCC), and 41.8° (blend), with improvement ratios of 28.73% (FA), 29.1% (NCC), and 55.97% (blend), as depicted in Fig. 14.

The observed enhancements are mechanistically attributed to the synergistic physical and chemical effects of the additions. The FA helps to fill the voids of the soil and promote pozzolanic reactions that lead to the formation of more cementitious compounds, as found by Zedan et al. [8] and other reviews concerning the use of fly ash for soil stabilization [15, 16]. The NCC because of its nano scale particle size and high surface area, densifies the soil microstructure and improves interparticle interaction in agreement with the findings of Al-Obaidi et al. [9] and Emad and Salman [11]. The highest values of cohesion and friction angle in this study indicate the synergistic effect of the FA and NCC mixture. This fills a significant gap in the existing research and highlights the superiority of combined additives over the single additive method. The blend ratio of 0.5% NCC and 10% FA was selected because it represents a practical and potentially cost-effective solution, suggesting potential improvements in field compaction workability. This approach ensured

enhanced shear strength without adverse effects on compaction. However, it is worth mentioning that the considerable increase in shear strength after soaking is a sign of the efficiency of the blend in compensating for the strength loss caused by water, which is a major problem for gypseous soils [9, 11]. Overall, these results indicate that the use of a combination of NCC and FA is a practical and effective way of stabilizing highly gypseous soils, particularly under varying moisture conditions.

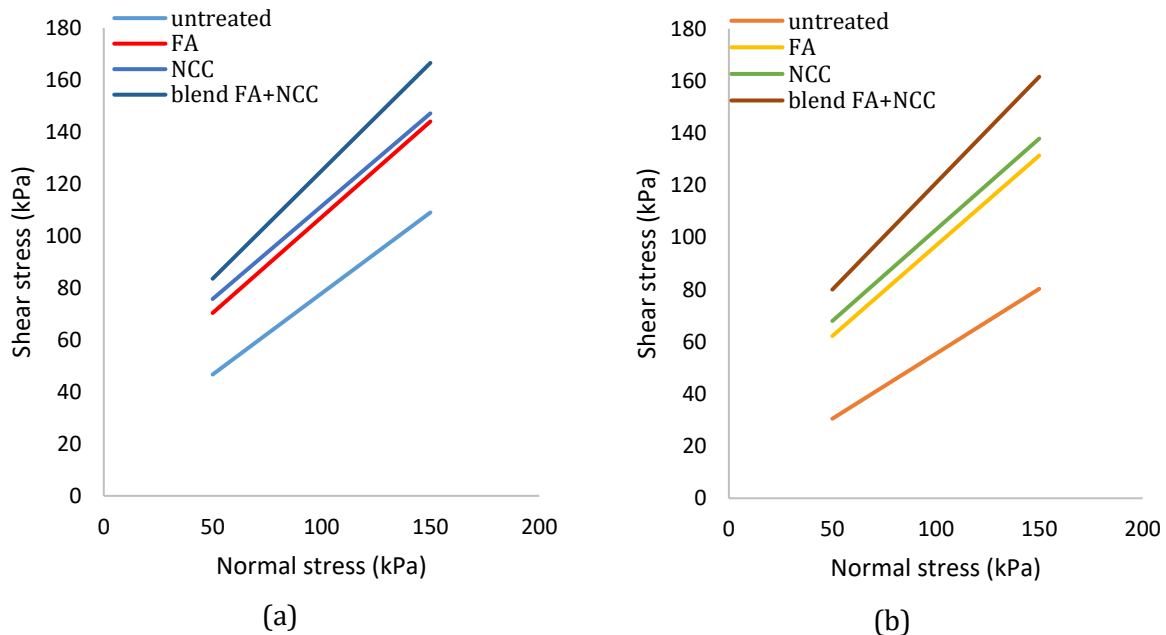


Fig. 7. Results of direct shear tests for untreated and treated soils under: (a) dry condition and (b) soaked condition

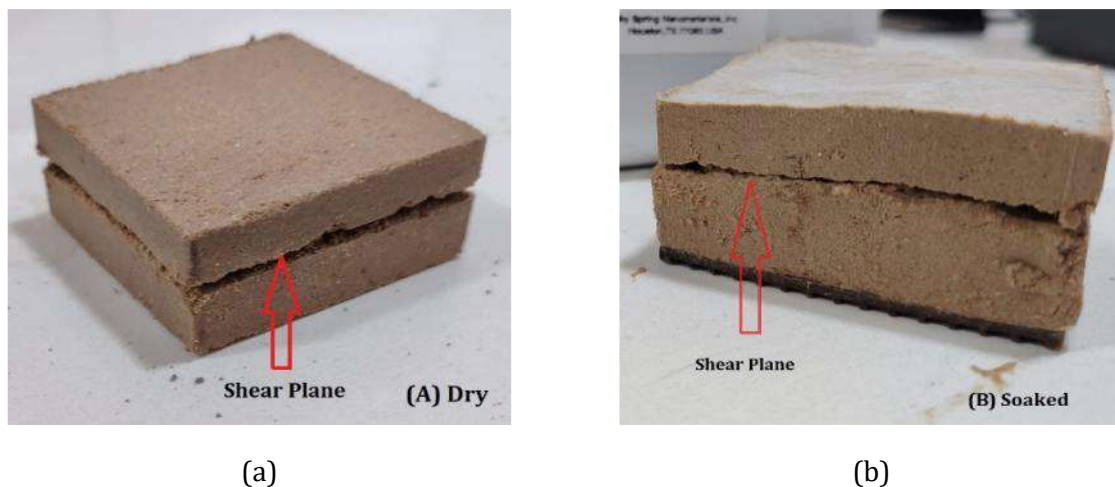


Fig. 8. Physical deformation and failure patterns of gypseous soil treated with a mixture of 10% Class C fly ash and 0.5% nano-calcium carbonate after direct shear testing under (A) dry and (B) soaked conditions

Microstructural and mineralogical studies were conducted using X-ray diffraction (XRD) and scanning electron microscopy (SEM) to evaluate the effect of stabilization of gypseous soil by a mixed additive containing 0.5% nano-calcium carbonate (NCC) and 10% Class C fly ash. The XRD analysis of the untreated soil (Fig. 15) shows pronounced crystalline peaks mainly corresponding to gypsum (G) at $2\theta \approx 11.7^\circ$, 20.7° , and 31.1° , confirming its dominance. Additional peaks for quartz (Q) and calcite (C) are also present, indicating their minor roles in the mineralogical composition. Upon treatment with nano calcium carbonate and fly ash (Fig. 16), the pattern undergoes significant transformation. The primary gypsum (G) peaks are notably reduced in intensity, while several new crystalline peaks emerge, especially those associated with ettringite (E) and more pronounced features for quartz (Q) and calcite (C).

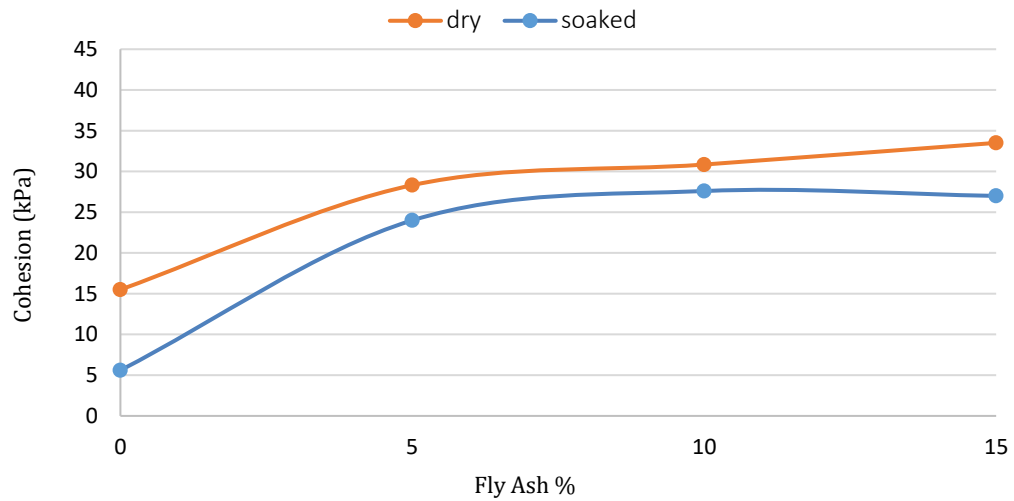


Fig. 9. Effect of fly ash content on cohesion of gypseous soil under dry and soaked conditions

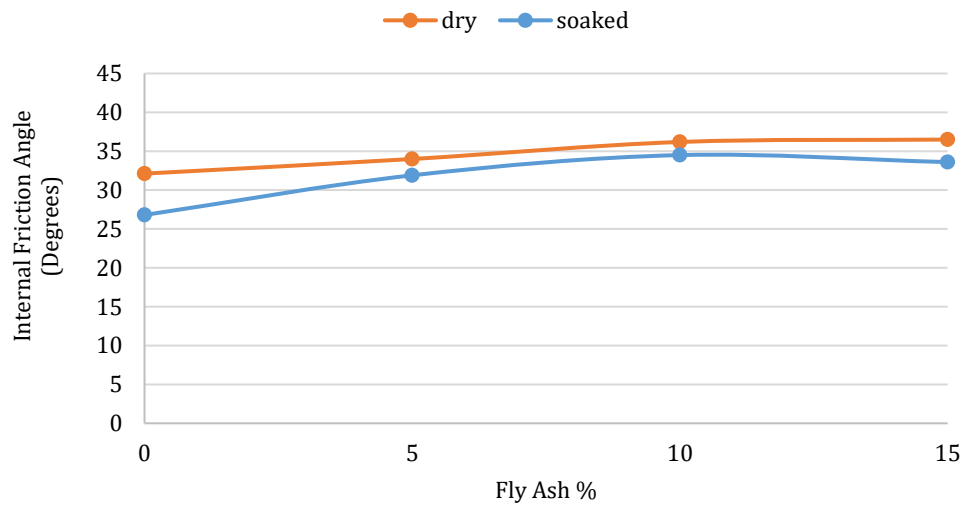


Fig. 10. Effect of fly ash content on internal friction angle of gypseous soil under dry and soaked conditions

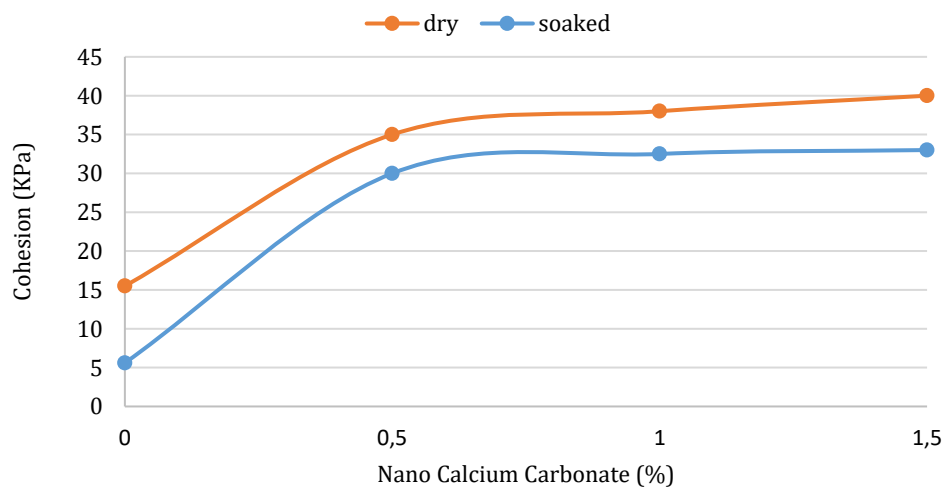


Fig. 11. Effect of nano calcium carbonate content on cohesion of gypseous soil under dry and soaked conditions

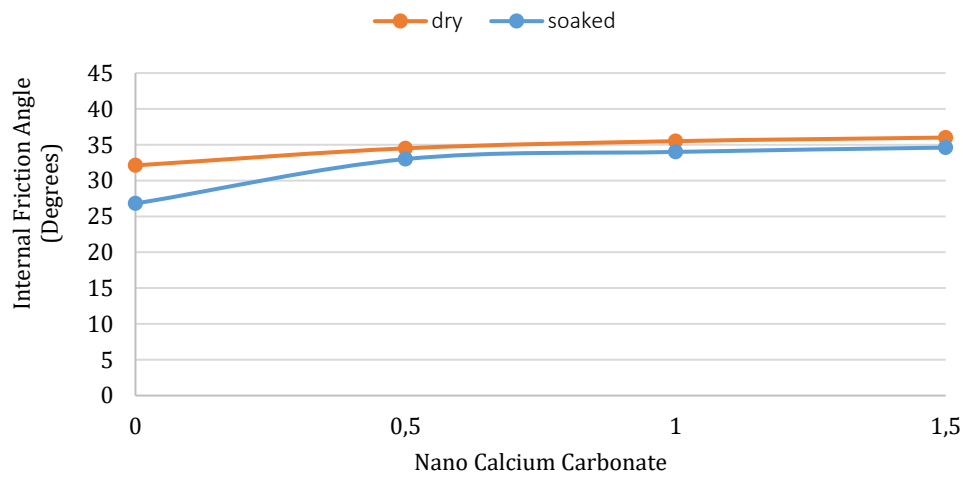


Fig. 12. Effect of nano calcium carbonate content on internal friction angle of gypseous soil under dry and soaked conditions

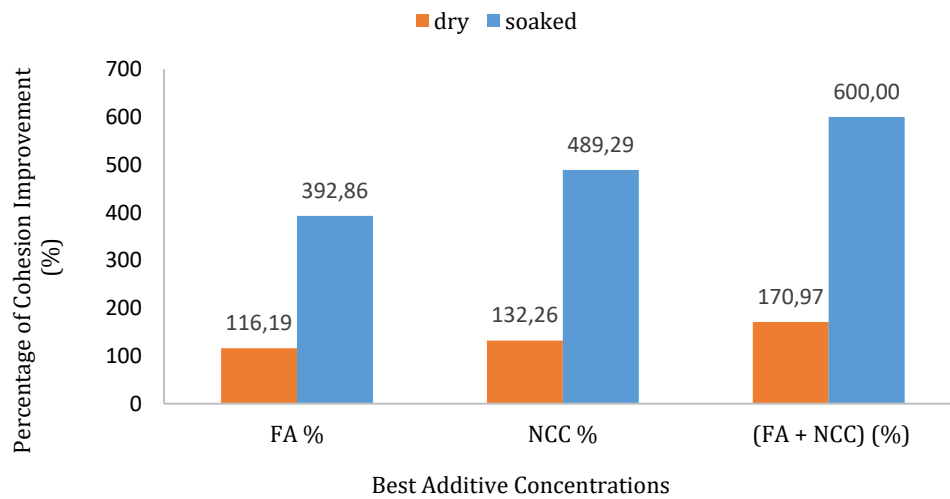


Fig. 13. Percentage improvement in cohesion for gypseous soil treated with best additive concentrations under dry and soaked conditions.

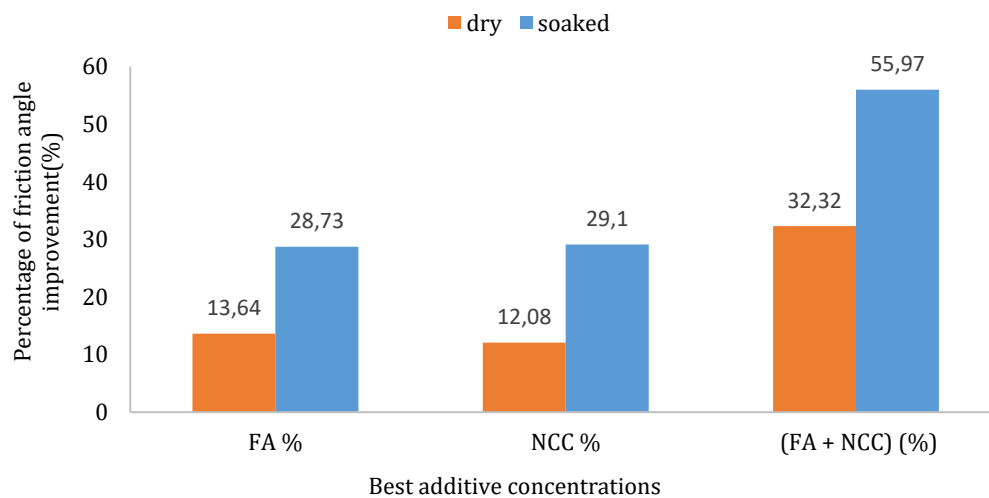


Fig. 14. Percentage improvement in internal friction angle for gypseous soil treated with best additive concentrations under dry and soaked conditions

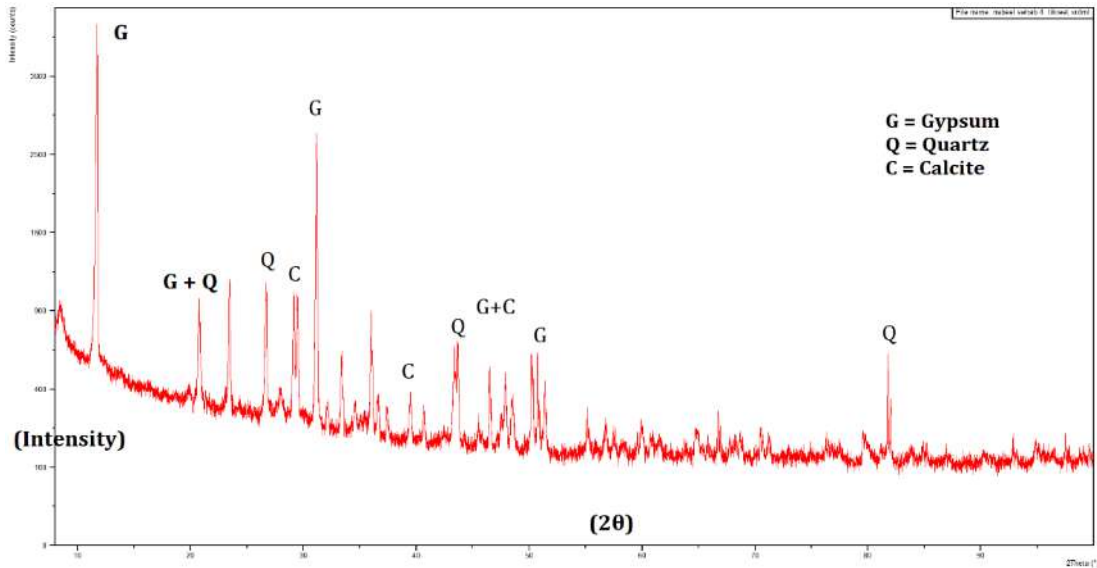


Fig. 15. XRD pattern for untreated gypseous soil

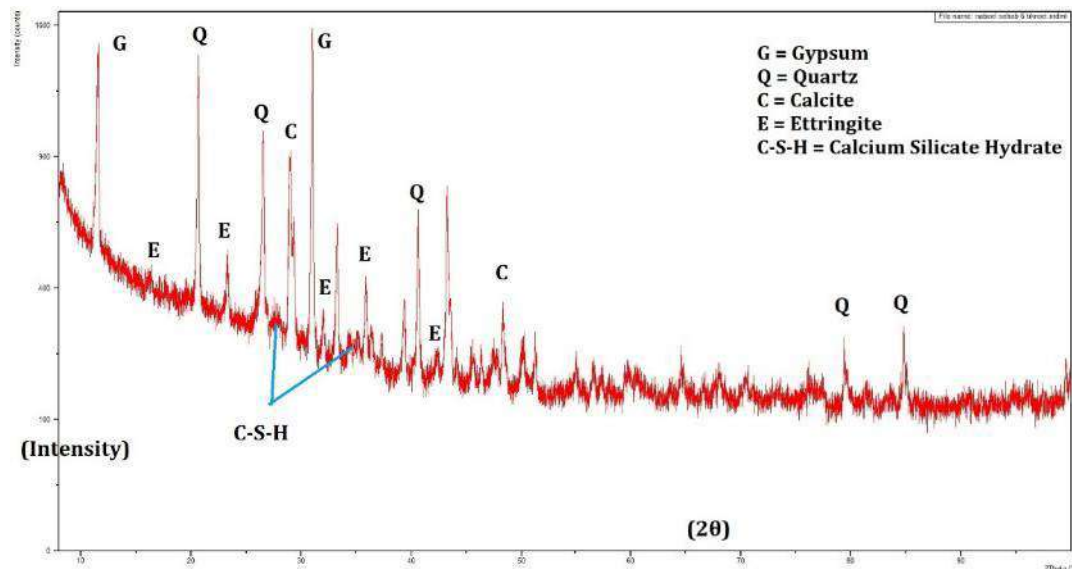


Fig. 16. XRD pattern for treated gypseous soil by 0.5% NCC + 10% FA

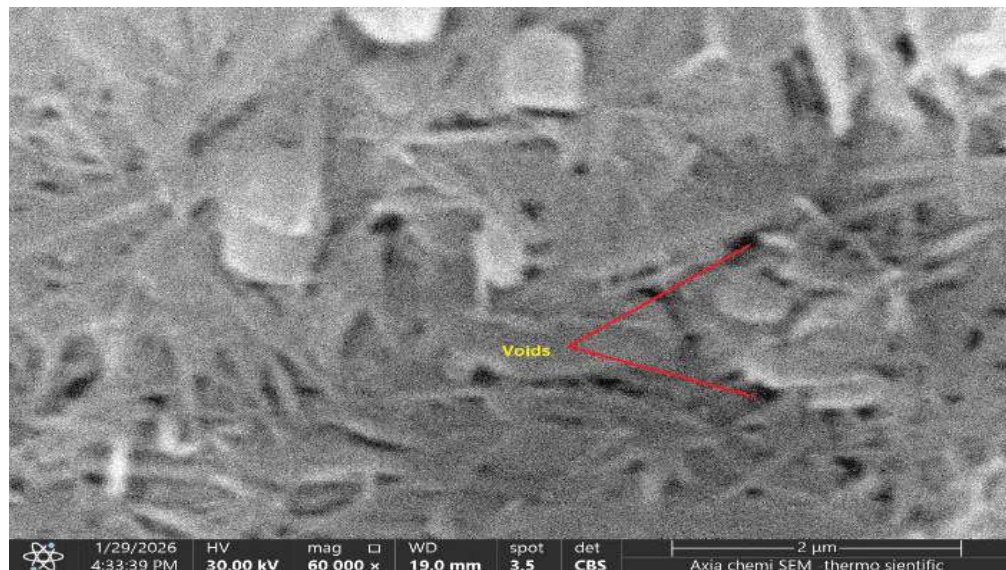


Fig. 17. SEM image of treated gypseous soil with 0.5% NCC and 10%FA

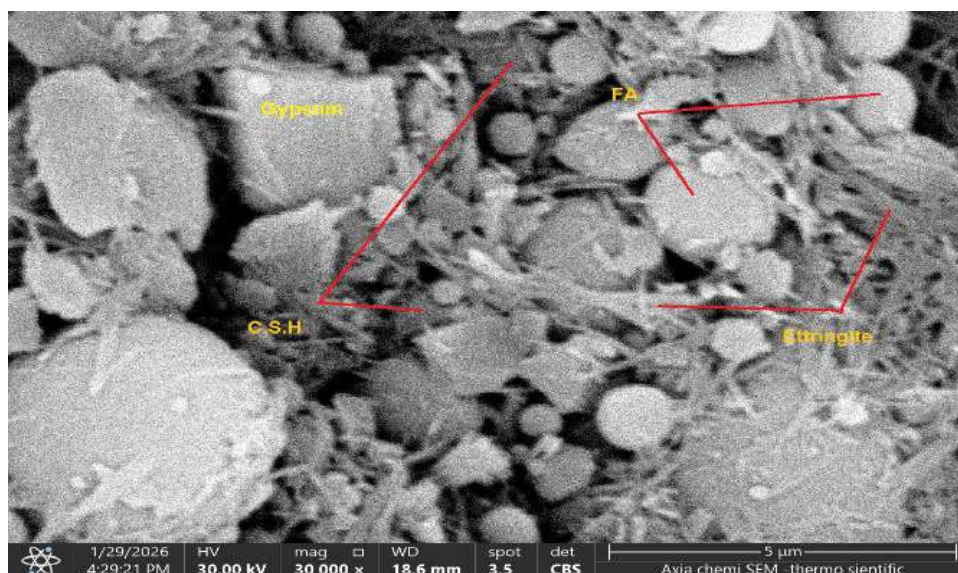


Fig. 18. SEM image of treated gypseous soil with 0.5% NCC and 10%FA

Importantly, instead of a sharp diffraction peak, the presence of calcium silicate hydrate (C-S-H) is identified by a broad amorphous hump or elevation in the baseline within the 2θ range typically associated with C-S-H. This characteristic broadening reflects its poorly crystalline or semi-amorphous structure, which is a common trait for C-S-H phases formed during pozzolanic reactions. The emergence of these features, coupled with the reduction of gypsum peaks, demonstrates extensive mineralogical transformation, with the development of a more complex and stable mixture of crystalline and amorphous phases in the treated soil. Although the formation of ettringite contributed to the observed improvement in soil strength through particle bonding and cementation, previous studies have reported that excessive or delayed ettringite formation in sulfate-rich soils may result in long-term expansion and deterioration under prolonged moisture exposure. The present study did not include long-term durability or expansion tests; therefore, the potential effects of ettringite-induced swelling on the long-term performance of the stabilized soil were beyond the scope of this work and warrant further investigation. The SEM observations confirmed the mineralogical results by showing major changes in the soil fabric. The microstructure of the untreated soil was relatively loose and discontinuous with many voids and little bonding between particles (Fig. 17). In contrast, the treated specimen (Fig. 18) showed a denser and more cohesive matrix with a marked reduction in pore spaces and improved particle interlocking. Within the treated matrix, spherical particles attributable to fly ash and acicular ettringite crystals could be distinguished, where they functioned as bridging and binding phases strengthening the contacts among soil particles. The combined evidence obtained from the XRD and SEM analyses (Figs. 15-18) provides a rational explanation for the improvement in the engineering performance of the stabilized soil, in particular the enhancement in shear strength and durability observed after treatment.

4. Conclusions

This study comprehensively evaluated the effects of nano calcium carbonate (NCC), Class C fly ash (FA), and their combination on the shear strength and compaction properties of highly gypsiferous soils (56% gypsum content). The following key conclusions are drawn:

- The addition of NCC, FA, and their blend resulted in significant improvements in both cohesion and internal friction angle, with the most pronounced percentage of relative improvement observed under soaked conditions compared to the untreated soaked state.
- The blend of 0.5% NCC and 10% FA, selected to balance mechanical gains and density effects, provided the highest observed values for cohesion (39.2 kPa) and internal friction angle (41.8°) after soaking, with improvement ratios of 600% and 55.97%, respectively. These results demonstrate the effectiveness of the blended additives in compensating for the

strength loss caused by soaking, which represents one of the most critical engineering challenges associated with highly gypseous soils.

- The treated soils exhibited marked increases in maximum dry density (MDD) with increasing NCC content, reaching a peak of 1.782 g/cm³ at 1.5% NCC (compared to 1.715 g/cm³ for untreated soil). This increase reflects enhanced packing and densification due to the nano-size effect of NCC.
- Conversely, increasing FA content led to a gradual decrease in maximum dry density, from 1.67 g/cm³ at 5% FA to 1.605 g/cm³ at 15% FA, likely due to the lower specific gravity and increased water demand for lubrication, which subsequently reduces compaction efficiency.
- The combined addition of 0.5% NCC and 10% FA achieved a maximum dry density of 1.7 g/cm³, indicating a balanced effect—higher than FA alone, but slightly less than pure NCC—while maintaining improved shear strength characteristics.
- These results confirm that the selected blend ratios not only enhance shear strength and water-resistance but also optimize compaction behavior, suggesting potential improvements in field compaction workability.
- These improvements, confirmed by XRD and SEM analyses, are attributed to the void-filling effect of fly ash, nanoscale densification, enhanced particle bonding by NCC, and the synergistic physicochemical interaction between the two additives. Early pozzolanic reactions may also have contributed, although their full development generally requires longer curing periods.
- Although the formation of ettringite may have contributed to the observed improvement in the engineering properties of the stabilized soil, previous studies have indicated that excessive or delayed ettringite formation in sulfate-rich soils may pose a risk of long-term expansion under prolonged moisture exposure. Therefore, long-term swelling and durability tests are recommended to assess the long-term performance of the proposed stabilization method.
- Overall, this approach addresses a critical gap in the literature regarding the combined use of NCC and Class C FA for gypseous soil stabilization, and provides practical recommendations for geotechnical applications in arid and semi-arid regions.

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References

- [1] Food and Agriculture Organization of the United Nations. Management of gypsiferous soils. Vol. 62. Rome, Italy: FAO; 1990.
- [2] Al-Kaaby FS. Distribution of the gypsiferous soil in Iraq. Baghdad, Iraq; 2007.
- [3] Al-Emami OH. Collapsibility of gypseous soil under fluctuation of ground water table [MSc thesis]. Baghdad, Iraq: Department of Building and Construction Engineering, University of Technology; 2007.
- [4] Al-Muftay A. Effect of gypsum dissolution on the mechanical behavior of gypseous soil [PhD thesis]. Baghdad, Iraq: University of Baghdad; 1997.
- [5] Mansour ZM, Chik Z, Taha MR. On the procedures of soil collapse potential evaluation. J Appl Sci. 2008;8(23):4434-4439. <https://doi.org/10.3923/jas.2008.4434.4439>
- [6] Solis R, Zhang J. Gypsiferous soils: an engineering problem. In: Sinkholes and the Engineering and Environmental Impacts of Karst. Reston, VA, USA: ASCE; 2008. p. 742-749. [https://doi.org/10.1061/41003\(327\)72](https://doi.org/10.1061/41003(327)72)
- [7] Al-Beiruty M. Collapse potential determination of gypseous soils [MSc thesis]. Baghdad, Iraq: University of Technology; 2003.
- [8] Zedan AJ, Hummadi RA, Hussein SA. Effect of Adding Mixture of (Concrete Waste and Asphalt Waste) on the Properties of Gypseous Soil. Tikrit Journal of Engineering Sciences. 2019;26(1):20-25. <https://doi.org/10.25130/tjes.26.1.03>
- [9] Al-Obaidi MT, Al-Mukhtar MT, Al-Dikhil OM, Hannona SQ. Comparative study between silica fume and nano silica fume in improving the shear strength and collapsibility of highly gypseous soil. Tikrit J Eng Sci. 2020;27(1):72-78. <https://doi.org/10.25130/tjes.27.1.10>
- [10] Theyab F, Muhauwiss FM, Alabdraba WM. Enhancing gypseous soil behavior using casein from milk wastes. J Mech Behav Mater. 2022;31(1):306-313. <https://doi.org/10.1515/jmbm-2022-0041>

- [11] Emad R, Salman AD. Shear strength and collapsibility of gypseous soil improved by nanomaterials. IOP Conf Ser Earth Environ Sci. 2024;1374(1):012004. <https://doi.org/10.1088/1755-1315/1374/1/012004>
- [12] Bağrıaçık B, Mahmutluoğlu B, Ürünveren A. A comprehensive study on the assessment of CaCO₃-, Nano-CaCO₃-, and glass fiber chopped strand (GFCS)-treated clay in terms of bearing capacity and settlement enhancements. Applied Sciences. 2025;15(14):7779. <https://doi.org/10.3390/app15147779>
- [13] Mohammadi M, Rajabi AM, Khodaparast M. Experimental and numerical evaluation of the effect of nano calcium carbonate on geotechnical properties of clayey sand soil. KSCE Journal of Civil Engineering. 2022;26(1):35-46. <https://doi.org/10.1007/s12205-021-1914-8>
- [14] Muhauwiss FM, Hussein AH, Al-Sumaiday HG. Behavior of gypseous soil treated by Arabic gum biopolymer under cycles of wetting and drying. Tikrit J Eng Sci. 2025;32(3):1-9. <https://doi.org/10.25130/tjes.32.3.10>
- [15] Jassam MG, Abd AH, Al-Sumaiday HG. Collapsibility and strength of gypseous soil waterproofed by silane coupling agent. Res Eng Struct Mater. 2026;12(3):1174-1190. <https://doi.org/10.17515/resm2026-1640ma0426rs>
- [16] Pandian NS. Fly ash characterization with reference to geotechnical applications. J Indian Inst Sci. 2004;84(6):189-216.
- [17] Kumar P, Sharma RS. Effect of fly ash on engineering properties of expansive soils. J Geotech Geoenviron Eng. 2004;130(7):764-767. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:7\(764\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:7(764))
- [18] Edil TB, Acosta HA, Benson CH. Stabilizing soft fine-grained soils with fly ash. Journal of Materials in Civil Engineering. 2006;18(2):283-294. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2006\)18:2\(283\)](https://doi.org/10.1061/(ASCE)0899-1561(2006)18:2(283))
- [19] Ehwailat KIA, Ismail MAM, Ezreig AMA. Ettringite formation and stabilization methods of sulfate-bearing soil: A state-of-the-art review. Indian Geotechnical Journal. 2022;52:927-941. <https://doi.org/10.1007/s40098-022-00619-x>
- [20] ASTM D6913/D6913M-17. Standard test methods for particle-size distribution (gradation) of soils using sieve analysis. West Conshohocken, PA, USA: ASTM International; 2017.
- [21] Barzanji F. Gypsiferous soils of Iraq [PhD thesis]. Ghent, Belgium: Ghent University; 1973.
- [22] ASTM D854-14. Standard test methods for specific gravity of soil solids by water pycnometer. West Conshohocken, PA, USA: ASTM International; 2014.
- [23] ASTM D4318-17e1. Standard test methods for liquid limit, plastic limit, and plasticity index of soils. West Conshohocken, PA, USA: ASTM International; 2017.
- [24] ASTM D1556/D1556M-15e1. Standard test method for density and unit weight of soil in place by sand-cone method. West Conshohocken, PA, USA: ASTM International; 2015.
- [25] ASTM D1557-12e1. Standard test methods for laboratory compaction characteristics of soil using modified effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³)). West Conshohocken, PA, USA: ASTM International; 2012.
- [26] Thomas MDA. Optimizing the use of fly ash in concrete. Skokie (IL): Portland Cement Association; 2007. 1-24.
- [27] Yu QL. Application of nanomaterials in alkali-activated materials. In: Nanotechnology in eco-efficient construction. Cambridge, UK: Woodhead Publishing; 2019. p. 97-121. <https://doi.org/10.1016/B978-0-08-102641-0.00005-0>
- [28] Changizi F, Haddad A. Application of nano materials in improving geotechnical properties of soils: a review study. J Rehabil Civ Eng. 2023;11(4):138-158.
- [29] ASTM D2216-19. Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass. West Conshohocken, PA, USA: ASTM International; 2019.
- [30] Karim, H.H., Schanz, T. and Ibrahim, A.N. Microstructural characterization of Iraqi gypseous soils. In: Proceedings of the 2nd International Conference of Buildings, Construction and Environmental Engineering (BCEE2), American University of Beirut, October 2015, vol. 17, pp. 17-18.
- [31] ASTM D3080/D3080M-11. Standard test method for direct shear test of soils under consolidated drained conditions. West Conshohocken, PA, USA: ASTM International; 2011.



Shear strength, CBR, and interface interaction of pumice-modified clay reinforced with Geostrip: A large-scale laboratory study

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Abstract

Clayey soils are challenging in geotechnical engineering because of their high compressibility, moisture sensitivity, and low shear strength. This study investigates lightweight and sustainable pumice, locally termed Ponza, for stabilizing clayey soil collected from Al-Amiriya, Baghdad, Iraq. Pumice was incorporated at replacement ratios of 10%, 20%, 30%, 40%, and 50% by dry mass. For each mixture, the dry constituents were blended, mixed with water at the corresponding optimum moisture content, and compacted before testing; CBR specimens were then soaked for 72 h. The experimental program included modified Proctor compaction, soaked California Bearing Ratio tests, and large-scale direct shear tests under normal stresses of 25, 50, and 75 kPa for unreinforced and Geostrip-reinforced specimens. A 200-mm-wide shear box was selected to accommodate the 4.75–9.5 mm pumice particles and minimize particle-size and boundary effects. Increasing pumice content reduced the maximum dry density and produced an overall increase in optimum moisture content because of the lightweight and porous nature of pumice. CBR generally improved at higher pumice contents owing to greater particle interlocking and the development of a frictional load-bearing skeleton. The shear response gradually changed from cohesion to friction-controlled behavior. For unreinforced samples, the friction angle increased from 16.3° for natural clay to 31.0° for 50% pumice, and cohesion decreased from 19.69 kPa to 10.50 kPa. Average interface interaction coefficients of 0.94–0.99 are shown to be efficient at stress transfer and compatible with the clay-pumice mixtures and Geostrip reinforcement.

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1. Introduction

Clayey soils are challenging geotechnically in roads, embankments, retaining structures, and shallow foundations because moisture-induced changes in soil fabric can cause settlement, deformation, and loss of bearing capacity and shear strength [1–6]. These problems have led to stabilization and reinforcement in weak fine-grained soils together. Pumice is a porous volcanic lightweight aggregate characterized by low specific gravity, high water absorption, and a rough, angular surface. These properties can lead to a lower weight of compacted fills while modifying particle arrangement, frictional resistance, and interparticle interlocking [7,8,26,29]. Compared to cement or lime-based treatment, coarse pumice is more of a physical change to the soil skeleton than chemical stabilization. However, its environmental and economic benefits are highly dependent on local availability, processing, and transport. Recently, pumice in fine-grained soils

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has been investigated using different forms, particle sizes, and replacement ratios. Aljboori et al. [26] evaluated clay mixed with pumice-based lightweight aggregate at 10%, 30%, 40%, 50%, and 60% dry mass using conventional compaction, consolidation, and strength tests. Their findings showed that the increase in the lightweight aggregate content reduced soil density and increased the frictional contribution to strength gradually, but too much replacement disturbed the clay matrix. Saltan et al. [29] demonstrated the potential of pumice waste to improve poor clayey pavement subgrades with this approach. Nasiri et al. [30] explored 5–15% fine pumice powder in contaminated clay and found 10% to be a good content for long-term strength and microstructure improvement. However, the fine pumice powder under curing behavior is very different from the coarse pumice particles acting as a lightweight granular modifier. Most of the available literature on compaction, CBR, unconfined compressive strength, consolidation, or drained shear parameters is limited in terms of the direct evidence for pore-water-pressure response and improved interface behavior in coarse clay–pumice mixtures [26,29,30]. The Mohr–Coulomb approach of cohesion and internal friction angle of the shear strength is widely used in the analysis and design of foundations, slopes, embankments, and earth-retaining structures [11–14]. The cohesion may be decreased because the clay-to-clay bond is broken (see section 2.2 below) and the mobilized friction angle is increased because of the angularity of the pumice particles, surface roughness, dilation, and confinement-dependent interlocking of the clay matrix [24–26]. At low pumice contents, isolated coarse particles can form local voids and weaken the clay matrix in the presence of low compactive effort. This transition must therefore be assessed both in penetration resistance and shear-strength tests rather than through compaction alone.

In reinforced-soil systems, overall performance also depends on shear transfer along the soil–geosynthetic interface. Previous pullout and interface studies have shown that reinforcement geometry, soil type, and the contact condition between clay and geosynthetic reinforcement strongly influence the mobilized interaction resistance [9,10]. ParaWeb 2D is a polymeric strip reinforcement composed of high-tenacity polyester yarn tendons encased in a protective polyethylene sheath [27]. Compared with continuous sheet-type geosynthetics, discrete strips can be arranged at selected elevations and lengths to mobilize tensile resistance while adding little self-weight; however, their effectiveness depends directly on the resistance developed along the soil–strip interface [15–18,27]. Interface behavior is governed by adhesion, surface texture, moisture condition, particle size and arrangement, normal stress, reinforcement geometry, and the mechanical characteristics of the polymeric strip [15–20,31,32]. Chai and Saito [31] showed that the interface shear response of clayey soils varies with geosynthetic type and contact condition. Choudhary and Krishna [20] demonstrated that particle size and geosynthetic structure influence interface friction and interaction coefficients, while Muluti et al. [32] confirmed that the configuration of the critical interface affects both peak and large-displacement strength. Consequently, the interaction of ParaWeb 2D with clay–pumice mixtures cannot be inferred reliably from soil-only strength and must be evaluated directly at the soil–strip contact surface.

Because the soil–strip response must be measured directly, large-scale direct shear testing is particularly important for the 4.75–9.5 mm pumice fraction used in the present study. A small shear box may exaggerate particle-size and boundary effects, and individual coarse particles can disproportionately influence rotation, dilation, crushing, and measured shear resistance when the specimen dimensions are insufficient relative to the maximum particle size. Deiminiat et al. [28] demonstrated that the specimen-width-to-maximum-particle-size ratio significantly influences direct shear results for coarse materials. The 200-mm-wide box adopted in this study provides a width-to-maximum-particle-size ratio of approximately 21, thereby reducing the scale and boundary effects expected in a conventional small shear box.

The previous investigation [26] characterized clay–lightweight aggregate mixtures using smaller pumice particles and conventional-scale laboratory testing, without evaluating the ParaWeb soil–reinforcement interface under the present large-scale configuration. The current study builds upon this previous work by using a coarser pumice fraction of 4.75–9.5 mm, soaked California Bearing Ratio tests at three compactive efforts, and investigating the soil–soil and soil–ParaWeb interface shear behavior in a 200-mm-wide direct shear box at 25, 50, and 75 kPa. Hence, this paper is not only a retelling of the previous work, but is focused on the effect of the particle medium on the

interface and the engineering ability of the lightweight clay-pumice mix. Therefore, the novelty of the present study is that it allows for the investigation of pumice modification, CBR performance, Geostrip reinforcement, and large-scale soil-geosynthetic interface in a single experimental setting. This paper fills the gap in the literature on the shear transfer between coarse pumice-modified clay and polymeric reinforcement strips by measuring the change of cohesion, friction angle, interface adhesion, and interaction coefficient with increasing pumice content.

This study aims to investigate the synergistic effects of pumice inclusion and ParaWeb 2D Geostrip reinforcement on compaction, bearing resistance, internal shear strength, and soil-strip interface behavior through large-scale testing, with particular emphasis on failure mechanisms and lightweight embankment and mechanically stabilized earth backfill applications. Because pore-water pressure was not directly measured in the present program, the reported interface interpretation is limited to the measured compaction, CBR, and direct-shear responses.

2. Materials and Methods

2.1. Temperature Gradient

The natural clay was collected from the Al-Amiriya area, western Baghdad, Iraq (33.299593° N, 44.272867° E), at a depth of 1.5–2.0 m. The soil was air-dried and pulverized before testing. Its physical and chemical properties are summarized in Table 1. The soil was classified as low-plasticity clay (CL), with a liquid limit of 30.5%, a plastic limit of 19.8%, and a plasticity index of 10.7%. The gradation comprised 85% fines passing the No. 200 sieve and 15% sand-sized material; the complete distribution is shown in Fig. 1. The specific gravity was 2.76, while the optimum moisture content and maximum dry density were 16% and 1.835 g/cm³, respectively.

Table 1. Physical and chemical properties of the natural soil

Property	Value
Soil classification	CL
Specific gravity	2.76
Liquid limit (%)	30.5
Plastic Limit (%)	19.8
Plasticity index (%)	10.7
Maximum dry density (g/cm ³)	1.835
Optimum moisture content (%)	16
Sand%	15
Silt%	19
Clay%	66
pH	7.4
Sulfate content, SO ₃ (%)	0.103
Gypsum content (%)	0.221
Chloride content, Cl ⁻ (%)	0.031

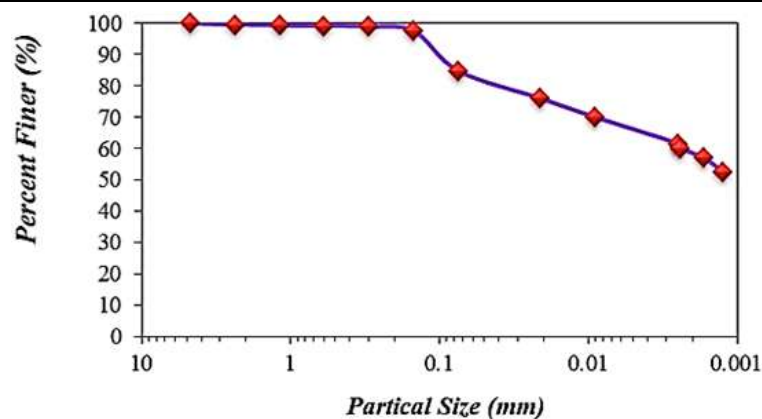


Fig. 1. Particle-size distribution curve of the natural clay

2.2 Pumice and Geostrip (ParaWeb 2D)

Pumice (locally termed Ponza) was imported from northern Iran and used as a lightweight granular additive. Pumice was incorporated at 10%, 20%, 30%, 40%, and 50% by dry mass of soil. The moist mixture was then homogenized before compaction and specimen preparation as shown in Fig. 2. Table 2 presents the measured physical and chemical properties of the material used in the experimental program. The aggregate was silica-rich, porous, and absorptive, characteristics that explain the reduction in mixture density and the non-monotonic water demand. ParaWeb 2D, as shown in Fig. 3, reinforcement consisted of high-tenacity polyester yarn tendons encased in a polyethylene sheath. The manufacturer reports an approximate strip width of 90 mm, thickness of 3 mm, and nominal ultimate tensile capacity of 75.4 kN per strip.



Fig. 2. Selected pumice fraction used in the study (passing 9.5 mm and retained on 4.75 mm sieve)

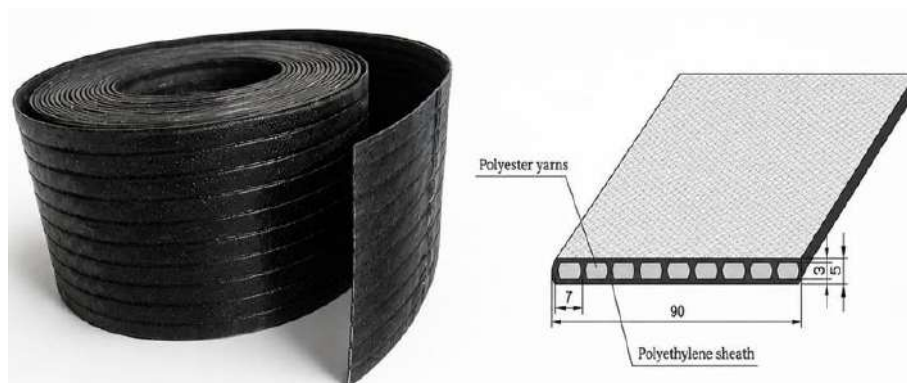


Fig. 3. ParaWeb 2D reinforcement strip and cross-sectional schematic (nominal strip width = 90 mm)

Table 2. Summary properties of the pumice used in this study.

Property	Value	Source/status
Selected particle-size range	4.75–9.5 mm	Present program
Specific gravity	1.931	Measured material property
Water absorption (%)	14.830	
Dry bulk density (kg/m ³)	995	
SiO ₂ (%)	61.126	Chemical analysis
Al ₂ O ₃ (%)	14.069	
Fe ₂ O ₃ (%)	8.342	
CaO (%)	8.422	
MgO (%)	3.300	

2.3 Experimental Work

Physical model and index testing were performed at the Burj Babel Company for Studies and Scientific Research laboratories, whereas the large-scale direct shear testing was conducted at the Roads Laboratory, Mustansiriyah University.

2.3.1 Test Sample Preparation

Pumice was used as a coarse, lightweight granular modifier at replacement ratios of 10%, 20%, 30%, 40%, and 50% by dry mass of the natural soil. The pumice fragments were crushed and sieved, and the fraction that passed the 9.5 mm sieve and was retained on the 4.75 mm sieve was chosen. This very coarse fraction was chosen so as to preserve the lightweight aggregate character of pumice and to make it possible for the particles to have roughness, angularity, and mechanical interlocking in the clay matrix rather than to be treated as fine powder or chemical binder. The required dry masses of clay and pumice were mixed manually until the coarse particles were evenly distributed. The water that was the optimum moisture content of each mixture was then added gradually, and a visually homogeneous mixture without wet or dry zones to be found in the mixture was made. The specimens were prepared immediately after mixing, and there was no time for the water to be balanced. The same mixing and placement procedure was maintained for all specimens to minimize segregation and preparation variability.

2.3.2 Compaction Test

Modified Proctor compaction tests according to ASTM D1557-12(2021) (method C) were carried out to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the natural clay and clay-pumice mixture. A 152.4 mm diameter mold with an approximate height of 116.4 mm was used for compacting the sample in five thick, roughly equal layers (in which each layer received 56 uniformly distributed blows from a 4.54-kg rammer falling through a height of 457 mm). The material was processed through the 19.0 mm (3/4-in.) sieve. While the standard Proctor test is often used for fine-grained soils with moderate compaction, the modified compactive effort is selected to take into account relatively high field compaction required for pavement subgrades, highway embankments, and reinforced backfill. The 152.4 mm mold was also chosen for the gravel-sized pumice to reduce particle size and boundary effects and to provide a good representative compaction response for clay-pumice mixtures. The modified compactive effort was applied to be about 2700 kNm/m³.

2.3.3 California Bearing Ratio (CBR)

Soaked California Bearing Ratio (CBR) tests were generally done in line with ASTM D1883-21 except for the modified soaking period of 72 h and penetration rate of 1 mm/min. Clay-pumice mixtures of 0%, 10%, 20%, 30%, 40%, and 50% pumice were mixed with water at the appropriate optimum moisture content (OMC). Three samples for each mixture were prepared and compacted in five approximately equal layers using 10, 25, and 56 blows per layer in order to represent three compactive efforts. A 4.54 kg rammer (with two 2.27 kg surcharge weights) was used to compress each sample and was applied during penetration testing. The specimens were soaked in water for 72 h, during which vertical expansion was monitored using a dial gauge. The percentage swell was measured at the end of the soaking time. After soaking, the specimens were allowed to drain for about 15 min while maintaining the surcharge. Penetration testing was done at a fixed rate of 1 mm/min, and the loads were measured at the specified penetration depths. The CBR value was measured from the loads at 2.5- and 5.0-mm penetration in comparison to the standard loads.

2.3.4 Large-Scale Direct Shear

A large-scale direct shear apparatus was used in this work (Fig. 4) to apply normal stresses of 25, 50, and 75 kPa. The apparatus was manufactured by Banyhussan and is described in [21]. It consists of a lower rectangular shear box (200 × 250 × 100 mm) and an upper square shear box (200 × 200 × 100 mm). The larger lower box was used to maintain approximately the same overlap and shearing area during horizontal displacement, and reduce the area correction uncertainty of the conventional equal-sized shear box halves. There are two restraining arms to keep the upper box horizontal and the lower one vertical. The steel loading plate was pressed very hard to provide

a normal stress level, and an electric jack was driven on the lower box to lift it up. The shear force was measured using the load cell with a 5-ton weight. The surface of the metal-to-metal contact and guiding surfaces were lubricated to minimize the friction and not to cause damage to the soil-soil or soil-Geostrip shear interface. For the reinforced specimens, one 90 mm wide ParaWeb 2D strip with a nominal tensile capacity of 75.4 kN was placed flat and centered to the horizontal shear plane. The longitudinal direction of the strip was aligned with the direction of shearing, and the strip was taut and free of folds in order to conform to the clay-pumice mixture. The soil mixtures were prepared at the optimum moisture contents. The dry mass of each mixture was calculated from the known internal volume of the shear box and the maximum dry density obtained from the Modified Proctor test according to Eq. 1.

$$Md = \rho_{d,target} Vb \quad (1)$$

Where Md is the required dry mass, $\rho_{d,target}$ is the target dry density, and Vb is the specimen volume. The corresponding moist mass was determined from Eq.2:

$$M_{moist} = Md(1 + w) \quad (2)$$

Where w is the optimum moisture content expressed as a decimal. The prepared moist mixture was divided into three approximately equal portions and placed in the shear box in three layers. Each layer was compacted by uniform manual tamping until the calculated total mass occupied the predetermined box volume, thereby achieving the target dry density. The same target density, layer thickness, and tamping procedure were maintained for all reinforced and unreinforced specimens to minimize variations in specimen density and pumice-particle distribution. After specimen preparation, the required normal stress was applied, and the displacement rate of 1 mm/min was adopted based on the operating procedure of the large-scale apparatus and the comparable testing program [21]. Shearing was continued to a horizontal displacement of 30 mm for each normal-stress level. Because the rate was not derived from specimen-specific consolidation measurements, the results are interpreted as rate-dependent shear parameters rather than strictly drained parameters.

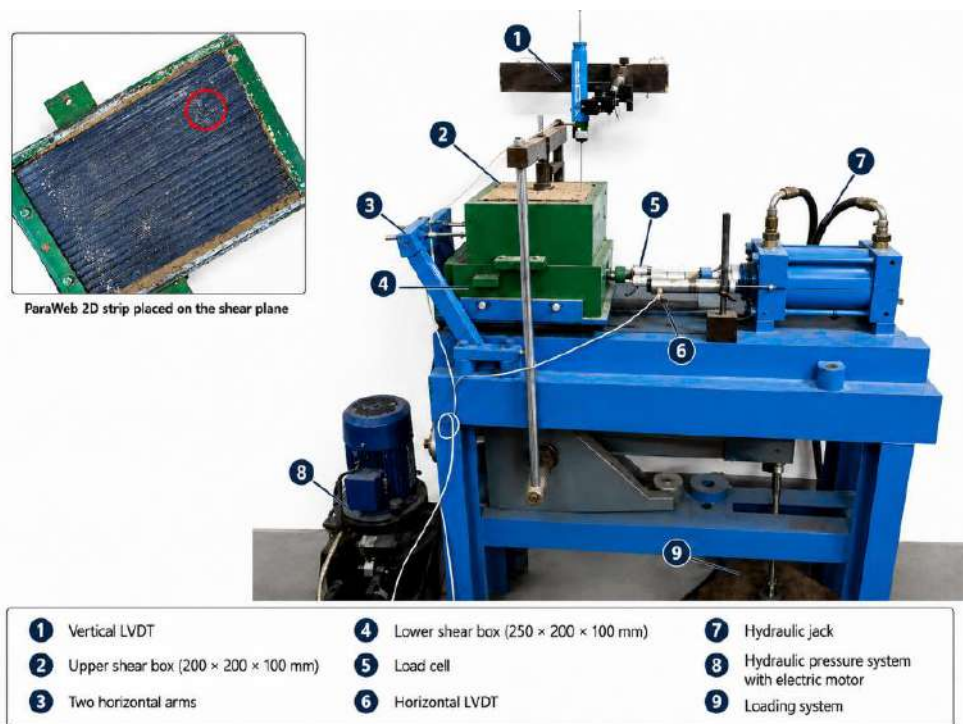


Fig. 4. Large-scale direct shear apparatus and numbered components

3. Results and Discussion

3.1. Compaction Results

The Modified Proctor results presented in Fig. 5 show that the natural clay achieved a maximum dry density (MDD) of approximately 1.84 g/cm^3 at an optimum moisture content (OMC) of 16%. Increasing the pumice content to 50% reduced the MDD to approximately 1.64 g/cm^3 , corresponding to a reduction of about 10.9% relative to the natural clay. At the same replacement level, the OMC increased to approximately 17%, representing an increase of about 6.3%. But the OMC response was non-monotonic: the highest value of approximately 18.5% was reached at 40% pumice, which corresponds to an increase of about 15.6% from the natural soil and a slight decrease at 50% pumice. The decrease in MDD is mostly because of the low specific gravity and porous structure of pumice, whereas the change of OMC is caused by the competing effects of replacing water-requiring clay fines and water absorption by the porous pumice particles. For MSE applications, horizontal earth pressure is influenced simultaneously by the soil-mixture unit weight and the friction-dependent active earth-pressure coefficient as expressed in Eq. 3.

$$\sigma_h = K_a \gamma z \quad (3)$$

Where σ_h is active horizontal earth pressure, K_a is Rankine's active earth-pressure coefficient, $\gamma = \rho \times g$ is dry unit weight, ρd is dry density of the soil mixture, g is gravitational acceleration, z is the depth below the ground surface and z is the depth below the ground surface. For level backfill and zero wall friction, Rankine theory gives $K_a = \tan^2(45^\circ - \phi/2)$. So increasing the friction angle from 16.3° for natural clay to 31.0° for 50% pumice mixture reduces the active horizontal pressure from 0.562 to 0.320, a decrease of about 43%. At the same time, the dry density is decreasing from approximately 18.05 kN/m^3 to 16.09 kN/m^3 and the active horizontal pressure is decreasing from about 10.14 kPa for natural clay to 5.15 kPa for 50% pumice mixture; the total reduction is around 49%. So, the decrease in lateral pressure is a product of the lower unit weight and the nonlinear reduction of K_a with the higher friction angle, and not due to a constant K_a .

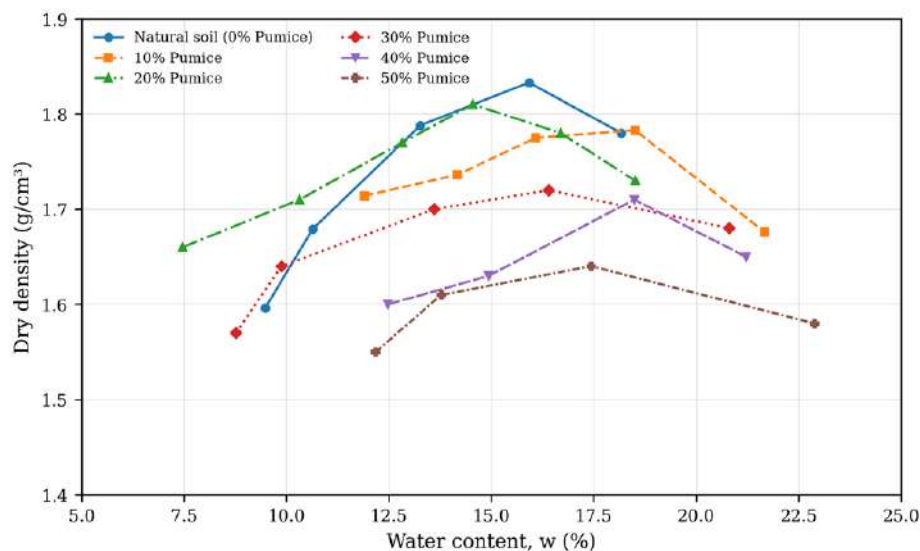


Fig. 5. Compaction curves for clay mixed with different pumice contents

3.2 CBR Test

Fig. 6 and Table 3 show the relationship between dry density and CBR for clay-pumice mixtures at three compactive efforts. For each mixture, the higher the compactive effort, the higher the dry density and the CBR, which proves that the compaction state is positively related to penetration resistance. The higher density makes for closer particle contact, less deformability and compressibility of specimens, and it also ensures that the internal load transfer path does not change as much as the pumice content. However, the response to the increase in pumice content is

not monotonic, because the contribution of a cohesive clay matrix, contact between pumice and particles, void, and so on changes as the replacement ratio increases and the compactive effort increases. At 10 blows per layer of 10% pumice, a decrease in CBR from 1.400% for the natural soil to 0.531%. At that low energy, the small amount of coarse porous pumice might have interrupted the clay matrix without a continuous granular skeleton, and the local voids and incomplete seating of the pumice made them less resistant to penetration. With more compactive effort, the particles were embedded better, and CBR improved to 4.780% and 6.073% at 25 and 56 blows, respectively.

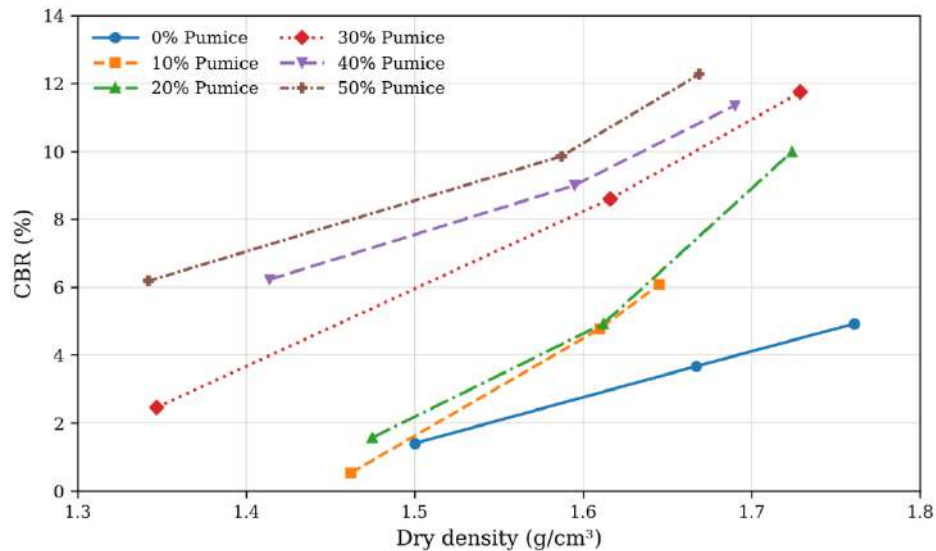


Fig. 6. Relationship between soaked CBR and dry density for the clay-pumice mixtures at three compactive efforts

Table 3. Soaked CBR results for the clay-pumice mixtures

Pumice content (%)	Number of blows per layer	Dry density (g/cm ³)	CBR (%)
0%	10	1.500	1.400
	25	1.667	3.670
	56	1.761	4.925
10%	10	1.462	0.531
	25	1.610	4.780
	56	1.645	6.073
20%	10	1.475	1.573
	25	1.612	4.924
	56	1.724	9.989
30%	10	1.347	2.463
	25	1.616	8.595
	56	1.729	11.739
40%	10	1.414	6.229
	25	1.595	9.000
	56	1.690	11.347
50%	10	1.342	6.181
	25	1.587	9.850
	56	1.669	12.265

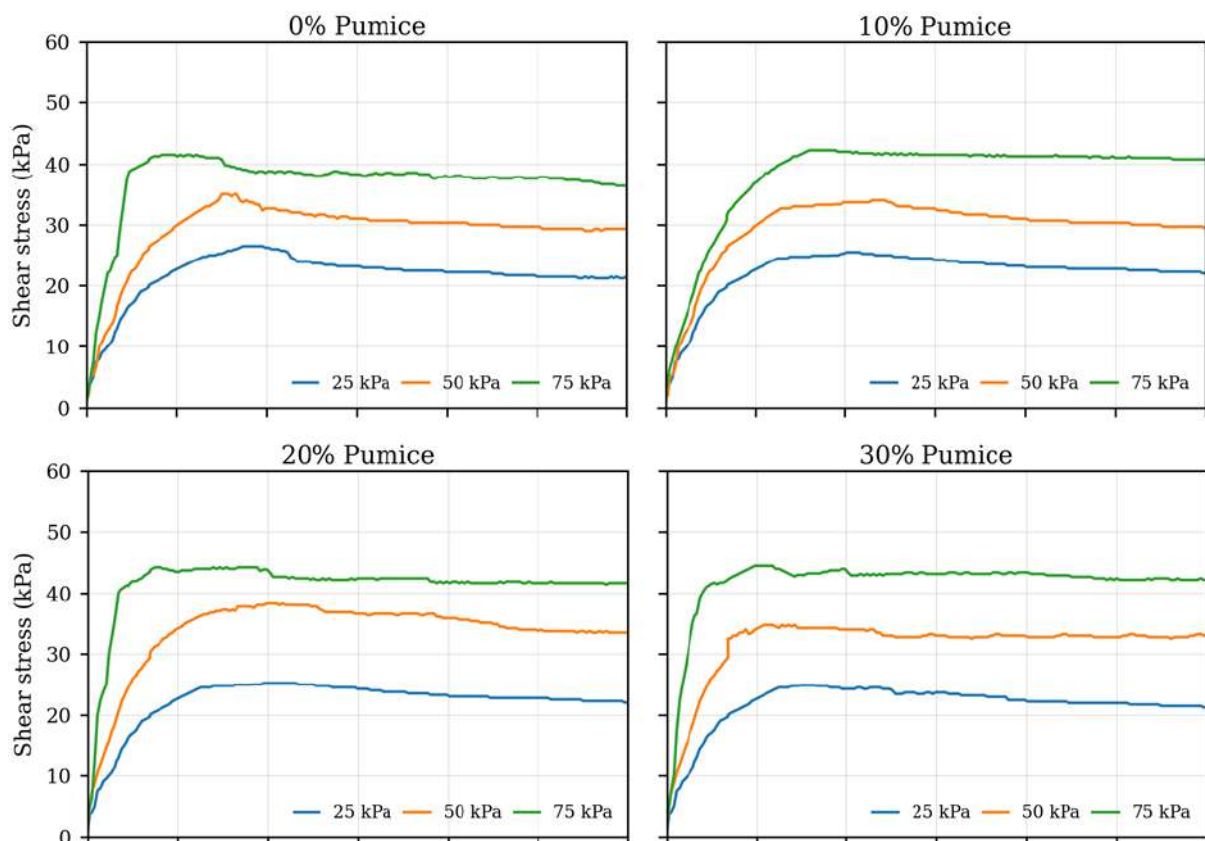
The difference between the 30% and 40% pumice mixture was related to compactive effort. At 10 blows (6.229% versus 2.463%) and 25 blows (9.000% vs 8.595%) the mixture of 40% pumice had a higher CBR, whereas at 56 blows (11.739% vs 11.347%), it had a difference. At low and intermediate compactive efforts, the higher portion of coarse particles in the 40% mixture increased granular contact and penetration resistance. At the highest compactive effort, the 30%

mix seemed to have a more favorable balance of clay-matrix continuity and pumice interlocking. At this high content of pumice, enough clay fines were available to coat and seat the coarse pumice particles, fill particle voids, and rearrange them during compaction. At a higher content of pumice (40%), the amount of clay available per unit surface of coarse particles decreased, the clay fraction filled and lubricated less and less, and the particle seating and filling had less uniformity and localized voids, and at 56 blows, the CBR was slightly lower. As the difference between the 30% and 40% mixtures at 56 blows was small (3.3%) it should be considered only as a mixture structure effect and not a universal optimum content. The 50% mixture produced the highest CBR at 25 and 56 blows (9.850% and 12.265%) while the 40% mixture was slightly higher at 10 blows, which confirms that the preferred pumice content depends on the compactive effort. At higher pumice content, the overall improvement in CBR is attributed to the development of a stiffer load-bearing skeleton through more granular contact and mechanical interlocking. However, the porous and possibly crushable nature of pumice is likely to increase the compressibility of pumice and therefore the mixture gradation in a higher field stress. These limitations, together with the sensitivity of the clay-coating and void-filling mechanisms to mixing quality, should be considered when extrapolating these results to full-scale pavement and reinforced-backfill applications.

3.3. Large-Scale Direct Shear Tests

3.3.1 Group 1 (Unreinforced)

Fig. 7 presents the shear stress–horizontal displacement responses of the unreinforced clay–pumice mixtures under normal stresses of 25, 50, and 75 kPa. The natural clay exhibited a pronounced peak shear stress followed by post-peak softening, consistent with rupture of the cohesive clay fabric and subsequent particle rearrangement [21]. At normal stress $[(\sigma)_n] = 25$ kPa, the natural clay produced the highest measured peak shear strength (26.4 kPa), whereas the 50% pumice mixture produced the highest values at $\sigma_n = 50$ and 75 kPa (40.0 and 55.75 kPa, respectively). The mixtures containing 10–30% pumice therefore represent a transition zone in which clay bonding, frictional sliding, and particle interlocking must be considered simultaneously, not a range of maximum shear strength. As the pumice content rose to 40–50% the response became more friction-controlled.



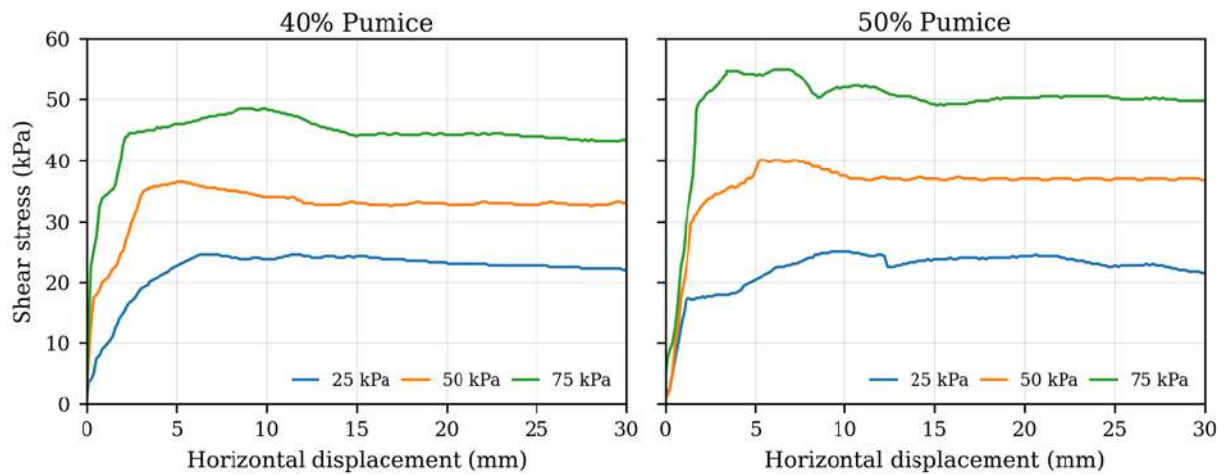


Fig. 7. Shear stress–horizontal displacement curves for unreinforced clay–pumice mixtures

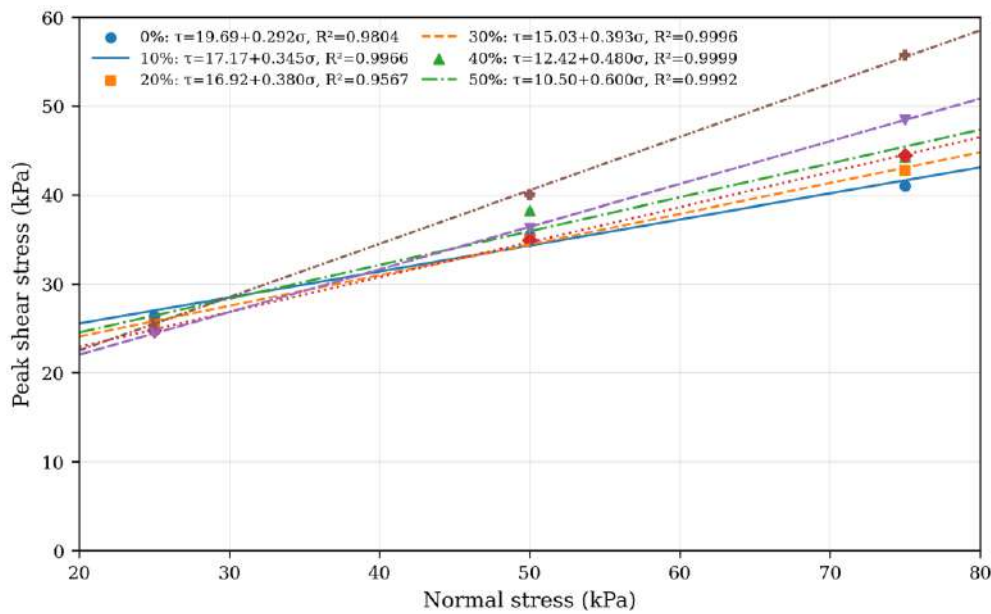


Fig. 8. Mohr–Coulomb failure envelopes for unreinforced clay–pumice mixtures

Table 4. Peak shear-strength parameters of unreinforced clay–pumice mixtures

Pumice content (%)	Peak shear strength, τ (kPa)			Cohesion, c (kPa)	Friction angle, ϕ (°)	R^2
	$\sigma = 25$ kPa	$\sigma = 50$ kPa	$\sigma = 75$ kPa			
0%	26.40	35.50	41.02	19.69	16.30	0.9804
10%	25.50	35.00	42.75	17.17	19.00	0.9966
20%	25.25	38.25	44.25	16.92	20.81	0.9567
30%	24.75	34.90	44.40	15.03	21.46	0.9996
40%	24.50	36.25	48.50	12.42	25.64	0.9999
50%	25.75	40.00	55.75	10.50	31.00	0.9992

The angular shape and rough vesicular surfaces of pumice made it less prone to particle sliding and rotation, promoted confinement-dependent mechanical interlocking, and encouraged the formation of a more continuous granular force transfer skeleton. These factors increased the mobilized frictional resistance and produced a relatively stable response after peak [24,25]. The Mohr–Coulomb failure envelopes in Fig. 8 and the parameters in Table 4 show strong linear relationships between normal stress and peak shear stress, with R^2 values ranging from 0.9567 to 0.9999. Further increasing the pumice content from 0% to 50% increased the friction angle from 16.3° to 31.0° (by approximately 90.2%), and the cohesion decreased from 19.69 to 10.50 kPa (by

approximately 46.7%). The reduction in cohesion is due to the failure of the continuous clay-to-clay bonding network as the pumice particles gradually replace the fine-grained matrix. The angularity of the surface and porous nature of the pumice, on the other hand, made contact rough, against sliding and rotation, and confinement-dependent. The combination of these mechanisms is the reason for the steeper failure envelope and the transition from cohesion-dominated to friction-controlled behavior at high pumice contents.

3.3.2 Group 2 (Reinforced Samples)

The Mohr-Coulomb failure envelopes shown in Fig. 8 and the parameters in Table 4 have a strong linear relationship with the normal stress to the peak shear stress, R^2 from 0.9567 to 0.9999. Increasing the pumice content from 0% to 50% allowed us to increase the friction angle from 16.3° to 31.0° , by a factor of 90.2%, while the cohesion fell from 19.69 kPa to 10.50 kPa, by a factor of 46.7%.

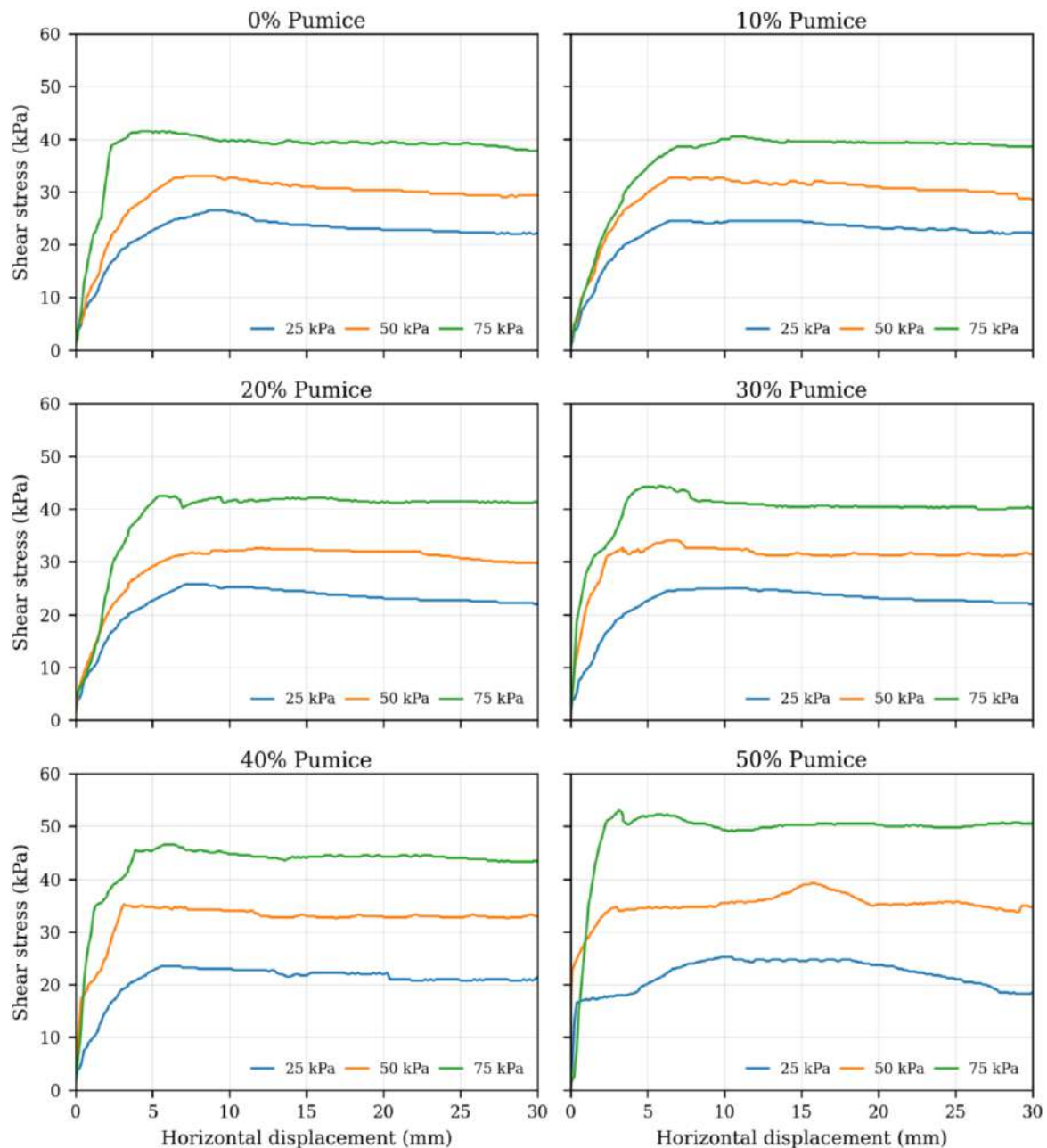


Fig. 9. Shear stress–horizontal displacement curves for reinforced clay–pumice mixtures

The reduction in apparent cohesion is because the continuous clay-clay bonding network was disrupted by the non-cohesive pumice particles as the fine-grained matrix became more and more

pumice. On the other hand, the angularity of the surface asperities and porous nature of pumice caused the contact of particles to be rough and the friction to get shifted and rotated towards each other to be less and less, and therefore the interlocking between particles to be less and less. These are the mechanisms that lead to the steeper failure envelope and the transition from cohesion-dominated to friction-controlled behavior at high pumice contents. The failure-envelope regressions in Fig. 10 and the parameters in Table 5 show coefficients of determination ranging from $R^2 = 0.9956$ to 0.9998 , indicating that normal stress is linear in the magnitude of interface shear. In Tables 4 and 5, the same stress levels, column sequence, units, and numerical precision are used to compare unreinforced and reinforced responses. The slope of the regression increased from 0.296 for natural clay to 0.555 for the 50% pumice mixture, corresponding to an interface friction angle that increased from 16.5° to 29.03° . This indicates that the thicker pumice content helped in the conversion of normal confinement to interface shear resistance through the roughness of the surface and particle interlocking. Reinforcement did not yield a uniform change in the fitted intercept. The apparent adhesion decreased from 19.69 to 19.38 kPa for the natural clay mixture and from 17.17 to 16.56 kPa for the 10% mixture, while small increases compared to the unreinforced values were found in 20-50% pumice. Their small increases are interpreted as localized locking of rough pumice particles against the polyethylene sheath, redistribution of clay near the strip, and regression variability corresponding to only three normal stress levels and not a systematic increase in cohesive bonding.

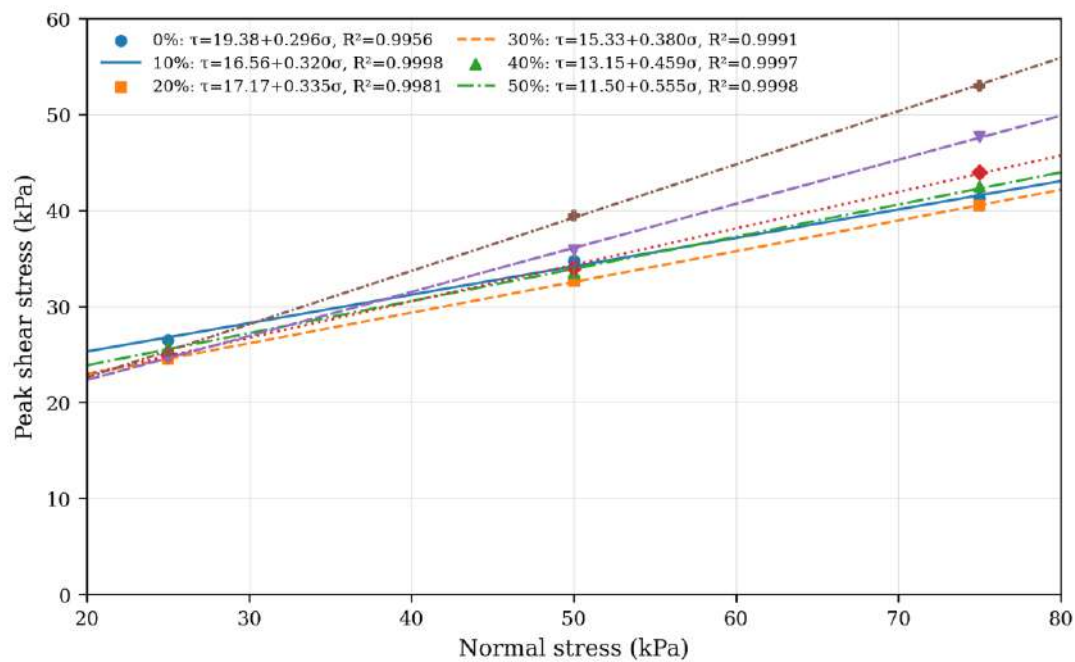


Fig. 10. Mohr-Coulomb failure envelopes for reinforced clay-pumice mixtures

Table 5. Peak interface shear-strength parameters of reinforced clay-pumice mixtures.

Pumice content (%)	Peak shear strength, τ (kPa)			Cohesion, c (kPa)	Friction angle, φ ($^\circ$)	R^2
	$\sigma = 25$ kPa	$\sigma = 50$ kPa	$\sigma = 75$ kPa			
0%	26.50	34.75	41.30	19.38	16.50	0.9956
10%	24.50	32.67	40.50	16.56	17.75	0.9998
20%	25.75	33.50	42.50	17.17	18.52	0.9981
30%	25.00	34.00	44.00	15.33	20.80	0.9991
40%	24.75	35.86	47.70	13.15	24.66	0.9997
50%	25.25	39.50	53.00	11.50	29.03	0.9998

3.3.3 Interface Shear Strength Coefficient

The interface interaction coefficient, η , was used to compare the peak interface shear strength of the reinforced specimen with the peak shear strength of the corresponding unreinforced mixture at the same normal stress [16]. It was calculated using Eq. 4, where τ reinforced is the peak shear stress measured along the soil–geostrip interface and τ unreinforced is the peak shear stress of the unreinforced mixture:

$$\eta = \frac{\tau_{\text{reinforced}}}{\tau_{\text{unreinforced}}} \quad (4)$$

where η is the interaction coefficient at a specified normal stress, τ reinforced is the peak interface shear strength of the reinforced specimen, and τ unreinforced is the peak shear strength of the corresponding unreinforced specimen.

Fig. 11 and Table 6 show that the average η values ranged from 0.94 to 0.99, indicating that the soil–Geostrip interface generally mobilized a shear resistance close to the internal strength of the unreinforced mixtures. Similar interface responses have been reported to depend on confinement level and contact conditions for soil–geosynthetic direct shear testing [16]. Moreover, the very stable mean coefficients at 30–50% pumice suggest that the increase in granular content did not lead to a systematic deterioration of stress transfer. However, experimental uncertainty arises from specimen preparation, local density variation, pumice distribution, reinforcement alignment, and peak stress identification, for example. Therefore, we consider the 0.87 value as a single figure that requires replicate testing and not the material trend.

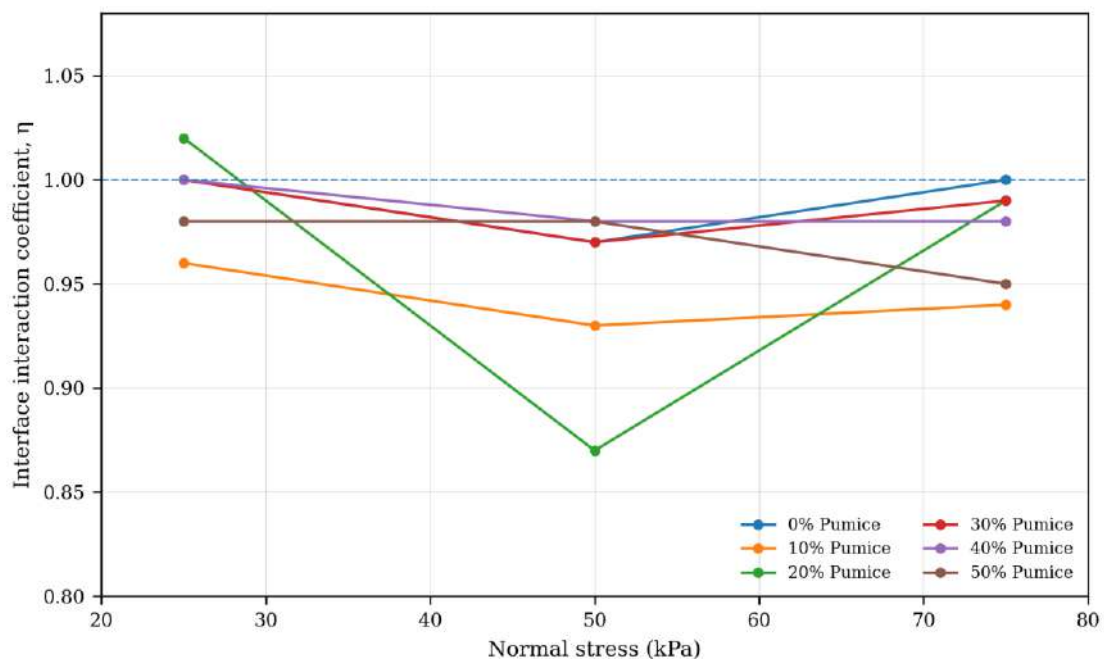


Fig. 11. Interface interaction coefficient versus normal stress for reinforced clay–pumice mixtures

The interface coefficient η reached 1.02 for the 20% pumice mixture under the lowest normal stress (25 kPa). Although interface coefficients are expected to be equal to or slightly less than unity, this increase may be due to localized mechanical interlocking between the pumice particles and the rough Geostrip surface under low confinement that can slightly increase the mobilized interface shear resistance. Since the increase above unity is only approximately 2%, it is also within the range of normal experimental variability. A localized decrease in the interface coefficient η was observed for the 20% pumice mixture at 50 kPa. It seems to be a case of transition of the mixture (the cohesive clay matrix but not the frictional Ponza skeleton is fully dominant). In intermediate confinement, the initial contact between the particles at low stress may be partially broken before

enough normal stress is available to fully mobilize interface friction, leading to the interface efficiency being temporarily lower. As the normal stress is increased to 75 kPa, frictional contact increases and interparticle interlocking is improved, and the interface coefficient is recovered. Since this behavior was only observed for the 20% Ponza mixture, it should be understood as a local response of the transitional soil fabric rather than a general trend.

Table 6. Peak interface interaction coefficients at different normal stresses.

Pumice content (%)	η at 25 kPa	η at 50 kPa	η at 75 kPa	Average η
0%	1.00	0.97	1.00	0.99
10%	0.96	0.93	0.94	0.94
20%	1.02 ^a	0.87 ^b	0.99	0.96
30%	1.00	0.97	0.99	0.98
40%	1.00	0.98	0.98	0.98
50%	0.98	0.98	0.95	0.97

Notes: ^a Slightly above unity; possible local locking or experimental scatter. ^b Isolated low value; replicate testing is required to assess specimen variability.

4. Conclusions

In this study, we have studied the combined effects of pumice replacement and ParaWeb 2D reinforcement on compaction characteristics, soaked CBR, shear strength and soil-Geostrip interface behavior of Al-Amiriya clay through modified Proctor compaction and large-scale direct shear testing. Based on these experimental findings, we can draw the following conclusions:

- The addition of pumice gradually reduced the maximum dry unit weight but raised the internal friction angle. The maximum dry density decreased by about 10.9% when a pumice replacement is 50% and the friction angle increased from 16.3° to 31.0°. The combined effect decreased the active earth pressure, which indicates that pumice-modified clay has great potential as a lightweight reinforced backfill for MSE walls.
- The large-scale direct shear testing demonstrated a gradual transition from cohesion-controlled behavior to friction-controlled behavior as the pumice content increased. This transition reduced post-peak softening and produced a more stable shear response under higher normal stresses.
- The results of the experiments suggest that the most appropriate pumice content is based on the performance-based value rather than on the optimal replacement ratio. Although the 50% pumice mixture retained the highest friction angle and soaked CBR values, the 30% mixture retained the natural-soil cohesion of 76.3% and increased the friction angle by about 31.7% and the soaked CBR by about 138.4%. Therefore, it is good if the 30% mixture is the right balance of cohesion retention and frictional resistance, but the higher the pumice content, the more appropriate it is for a design in which maximum bearing capacity and weight reduction are the focus.
- The reinforced specimens had interface interaction coefficients of 0.94-0.99, indicating excellent compatibility with ParaWeb 2D and clay modified by pumice. The interface transferred shear stresses effectively with no significant loss of resistance, which indicates the suitability of the proposed system for reinforced soil applications.
- The proposed pumice-Geostrip system is a technically promising solution for lightweight embankments and MSE wall backfills constructed on Al-Amiriya clay or similar soils by reducing self-weight, enhancing soaked bearing capacity and efficient soil-reinforcement interaction. However, the economic feasibility should be examined according to the local material availability, transportation distance and project-specific costs.

The conclusions are limited to the tested soil type, pumice gradation, replacement ratios, normal-stress range, specimen preparation method, and loading conditions adopted in this study. Because pore-water pressures were not monitored and the selected shear rate was not established from specimen-specific consolidation characteristics, the measured shear parameters should be regarded as rate-dependent large-scale shear parameters rather than fully drained or undrained

properties. Future research should investigate particle crushing, pullout resistance, long-term settlement, reinforcement creep, cyclic traffic and seismic loading, wetting–drying durability, freeze–thaw resistance where applicable, pore-water-pressure response, repeatability and uncertainty, and full-scale field performance.

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References

- [1] Shimobe S, Spagnoli G. Some generic trends on the basic engineering properties of fine-grained soils. *Environmental Earth Sciences*. 2019;78:281. <https://doi.org/10.1007/s12665-019-8266-4>
- [2] Zolfaghari Z, Mosaddeghi MR, Ayoubi S, Kelishadi H. Soil Atterberg limits and consistency indices as influenced by land use and slope position in western Iran. *Journal of Mountain Science*. 2015;12(6):1471-1483. <https://doi.org/10.1007/s11629-014-3339-z>
- [3] Önalp A, Bol E, Özocak A, Sert S, Ural N, Arel E. Influence of index properties on the cyclic failure of fine-grained soils. *Engineering Geology*. 2023;317:107056. <https://doi.org/10.1016/j.enggeo.2023.107056>
- [4] Lambe TW. The structure of compacted clays. *Journal of the Soil Mechanics and Foundations Division*. 1958;84(2):1654-1-1654-34. <https://doi.org/10.1061/JSFEAQ.0000114>
- [5] Sherwood PT. *Soil Stabilization with Cement and Lime: State-of-the-Art Review*. Crowthorne: Transport Research Laboratory; 1993.
- [6] Al-Waily MJ. *Principles of Foundation Engineering*. 4th ed. 2024. (In Arabic).
- [7] Choo H, Hwang J, Choi Y, Lee C, Lee W. Geotechnical characteristics of volcanic beach sands with varying iron contents. *Marine Georesources & Geotechnology*. 2016;34(6):571-580. <https://doi.org/10.1080/1064119X.2015.1043476>
- [8] Sayyah MA, Abrishami S, Dastpak P, Dias D. Behavior of volcanic ash-soil mixtures under one-dimensional compression testing. *Scientific Reports*. 2022;12:14524. <https://doi.org/10.1038/s41598-022-18767-8>
- [9] Beyranvand A, Ghazavi M, Lajevardi SH, Mirhosseini SM. Laboratory large-scale pullout tests for evaluation of waste plastic bottles as transverse members on geogrid reinforcement. *International Journal of Geosynthetics and Ground Engineering*. 2022;8:43. <https://doi.org/10.1007/s40891-022-00380-4>
- [10] Zhang WH, Wang BT, Zhang FH, Li SD. Test study on interaction characteristics between two-way geogrids and clay. *Rock and Soil Mechanics*. 2007;28(5):1031-1034.
- [11] Coduto DP, Yeung MCR, Kitch WA. *Geotechnical Engineering: Principles and Practices*. Upper Saddle River: Prentice Hall; 1999.
- [12] Budhu M. *Soil Mechanics and Foundations*. 3rd ed. Hoboken: John Wiley & Sons; 2011.
- [13] Das BM. *Advanced Soil Mechanics*. 4th ed. Boca Raton: CRC Press; 2014.
- [14] Das BM, Sobhan K. *Principles of Geotechnical Engineering*. 8th SI ed. Boston: Cengage Learning; 2014.
- [15] Shukla SK, Yin JH. *Fundamentals of Geosynthetic Engineering*. London: Taylor & Francis; 2006.
- [16] Liu CN, Zornberg JG, Chen TC, Ho YH, Lin BH. Behavior of geogrid-sand interface in direct shear mode. *Journal of Geotechnical and Geoenvironmental Engineering*. 2009;135(12):1863-1871. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0000150](https://doi.org/10.1061/(ASCE)GT.1943-5606.0000150)
- [17] Koerner RM. *Designing with Geosynthetics*. 6th ed. Bloomington: Xlibris; 2012.
- [18] Palmeira EM, Milligan GWE. Large-scale direct shear tests on reinforced soil. *Soils and Foundations*. 1989;29(1):18-30. <https://doi.org/10.3208/sandf1972.29.18>
- [19] Zekkos D, Athanasopoulos GA, Bray JD, Grizi A, Theodoratos A. Large-scale direct shear testing of municipal solid waste. *Waste Management*. 2010;30(8-9):1544-1555. <https://doi.org/10.1016/j.wasman.2010.01.024>
- [20] Choudhary AK, Krishna AM. Experimental investigation of interface behaviour of different granular soils and geosynthetics. *International Journal of Geosynthetics and Ground Engineering*. 2016;2:4. <https://doi.org/10.1007/s40891-016-0044-8>
- [21] Banyhussan QS, Mosa MM, Hussein AN, Sigar EJ. Evaluating the shear strength of subbase-subgrade interface using a large-scale direct shear test. *International Journal of Innovation in Engineering*. 2023;3(1):35-47. <https://doi.org/10.59615/ijie.3.1.35>

- [22] Sreelekshmypillai G, Vinod P. Prediction of CBR value of fine-grained soils at any rational compactive effort. *International Journal of Geotechnical Engineering*. 2019;13(6):560-565. <https://doi.org/10.1080/19386362.2017.1374495>
- [23] Nagaraj HB, Suresh MR. Influence of clay mineralogy on the relationship of CBR of fine-grained soils with their index and engineering properties. *Transportation Geotechnics*. 2018;15:29-38. <https://doi.org/10.1016/j.trgeo.2018.02.004>
- [24] Yin K, Fauchille AL, Di Filippo E, Kotronis P, Sciarra G. A review of sand-clay mixture and soil-structure interface direct shear tests. *Geotechnics*. 2021;1(2):260-306. <https://doi.org/10.3390/geotechnics1020014>
- [25] Siahkouhi M, Pletzer C, Marcher T, Schneider-Muntau B. Investigation of shear-strength parameters of soil and soft-rock materials in the low-stress range. *Geosciences*. 2021;11(4):169.
- [26] Aljboori W, Al-Soud M, Ali AM. Geotechnical properties of soil-lightweight aggregate mixtures. *Engineering, Technology & Applied Science Research*. 2025;15(1):20401-20408. <https://doi.org/10.48084/etasr.9419>
- [27] Maccaferri. ParaWeb 2 Technical Data Sheet: Polymeric Geostrips for Reinforced Soil Applications. Revision 08; 2015. Available at: https://gpil.co.nz/wp-content/uploads/2022/03/TDS-PARAWEB-2_rev08-20151016.pdf.
- [28] Deiminiat A, Li L, Zeng F. Experimental study on the minimum required specimen-width-to-maximum-particle-size ratio in direct shear tests. *CivilEng*. 2022;3(1):66-84. <https://doi.org/10.3390/civileng3010005>
- [29] Saltan M, Kavlak Y, Ertem FS. Utilization of pumice waste for clayey subgrade of pavements. *Journal of Materials in Civil Engineering*. 2011;23(12):1616-1623. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000336](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000336)
- [30] Nasiri H, Khayat N, Nazarpour A. Assessing one-year effects of 10% pumice powder on strength and microstructure of contaminated clay (case study: Ahvaz Project, Iran). *Case Studies in Construction Materials*. 2024;21:e03892. <https://doi.org/10.1016/j.cscm.2024.e03892>
- [31] Chai J-C, Saito A. Interface shear strengths between geosynthetics and clayey soils. *International Journal of Geosynthetics and Ground Engineering*. 2016;2:19. <https://doi.org/10.1007/s40891-016-0060-8>
- [32] Muluti SS, Kalumba D, Sobhee-Beetul L, Chebet F. Shear strength of single- and multi-layer soil-geosynthetic and geosynthetic-geosynthetic interfaces using large direct shear testing. *International Journal of Geosynthetics and Ground Engineering*. 2023;9:33. <https://doi.org/10.1007/s40891-023-00450-1>

Comparative laboratory study on the stabilization of clayey sand under strip foundations via micro cement and fly ash injection

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Article Info	Abstract
Article History: Received 21 May 2026 Accepted 27 July 2026 Keywords: Ground improvement; Clayey sand (SC); Micro cement injection; Fly ash injection; Injection pressure; Bearing capacity; Cost-effectiveness; Sustainability	This study compares micro cement and fly ash injection for improving clayey sand soil (SC) beneath a model strip foundation. A total of 54 laboratory model tests were conducted using injection pressures of 100, 200, and 300 kPa and curing periods of 7, 14, and 28 days. Performance was evaluated from load-settlement response, model test capacity, settlement at a common comparison stress of 100 kPa, unconfined compressive strength, grout consumption, cost, and a screening environmental inventory. The untreated soil had a model test capacity of 185 kPa. After 28 days, micro cement increased this capacity by 162% to 219% at pressures of 200 to 300 kPa, whereas fly ash produced gains of 146% to 189%. At the same pressures and curing age, settlement was reduced by 65% to 72% with micro cement and by 56% to 63% with fly ash. At 200 kPa and 28 days, fly ash achieved approximately 94% of the micro cement model test capacity and 90% of its unconfined compressive strength. The 28-day material-cost index was 3.41 times more favorable for fly ash (\$0.044 versus \$0.150 per kPa of capacity gain), while the screening inventory indicated an approximately 77% lower binder-related carbon burden. The highest measured responses occurred at 300 kPa; however, grout spread reached the chamber boundaries, so these values may include confinement effects. For the tested soil and model geometry, 200 kPa provided the most defensible balance among performance, grout consumption, cost, and experimental reliability. The results are laboratory-specific and require field trial sections and verification testing before design application.

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1. Introduction

Settlement of shallow foundations on soft soils can be excessive and lead to bearing capacity and serviceability problems. Thus, ground improvement is generally the most cost-effective solution when the cost of foundation replacement or deep foundations would be excessive or the required depth is considerable [1,3]. Stabilization injection is desirable in that it allows treatment to be applied under new or existing foundations with minimal excavation. The technique injects a pumpable binder into the pore space in soil, filling voids, bonding soil particles, and modifying stiffness and strength. With strip foundations, continuity of treatment and the dimensions (width and depth) of the strengthened body of ground are particularly important, because imperfect treatment may result in unacceptable differential settlement [4-8]. Micro cement is a super-fine cementitious paste, generally composed of particles at the size of micrometers, not millimeters. Due to the small size of its particles and its high specific surface, under certain conditions micro cement particles can enhance penetrability and activity, providing it does not adversely affect water-to-cement ratio, suspension stability, filtration, and injection pressure [8-14]. Fly ash is a by-product of powdered coal combustion in thermal power stations and consists of pozzolanic materials which can react with water and a calcium-bearing activator to create cementing

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materials. Its use can lessen the need for virgin cement and the disposal of waste, but Class F fly ash usually gains strength more slowly and needs an activator and adequate curing [15-19]. Micro cement and fly ash have each been used in the literature for ground improvement, but direct comparisons on the same soil under the same injection, curing, and loading conditions are rare. There is also relatively little published work providing guidance with respect to the influence of injection pressure on the coupled competing effects of grout spread, bearing response, settlement, cost and boundary disturbance in clayey sand under strip foundations.

This study fills that gap with a controlled laboratory comparison of micro cement and activated fly ash injection for a single clayey sand and model foundation system. Specifically, the aim is to measure the pressure- and curing-dependent variations of the load-settlement curves and unconfined compressive strength, to contrast the grout usage and the price of materials, and to construct a first-approximation decision-framework on binder selection. As the analysis employs a finite laboratory chamber, the interpretation makes a clear distinction between measured model response and design capacity in the field and discusses confinement and scale effects.

1.1. Soil Stabilization Fundamentals

Soil stabilization is to enhance the resistance to bearing and stiffness, decrease the compressibility, and control the deformation. The behavior of injected soils is influenced by their gradation, fines content, plasticity, permeability, initial density, and stress state. When the fines content is greater than 12%, the plasticity of the fines determines the Unified Soil Classification System (USCS) classification and also has an effect on the grout filtration and accessibility of pores. Here the soil is a sand-dominated coarse fraction with non-plastic or low-plasticity fine fraction. So it is classified as SC soil, not as SM. The main injection problems for these soils are confined pore throats, filtration of suspending particles, preferential flow, and possible hydraulic fracturing or heave at excessive pressure. Therefore, the enhancement must be measured in terms of the mechanical behavior and the treated zone size and homogeneity.

1.2. Cement-Based Stabilization

Cement grouting densifies soil, where hydration products, mainly calcium-silicate-hydrate phases, coat particles and fill up pores. Ultra-fine micro cement allows the application of suspension grouting to soils with smaller pore sizes than those which can be treated with ordinary Portland cement. The behavior of micro cement-treated soil is controlled by cement fineness, w/c ratio, injection pressure and rate, soil permeability, filtration, and curing condition. Lower water-to-cement ratios tend to enhance strength, but decrease penetrability, higher water-to-cement ratio suspension can travel farther, but may bleed or produce weaker cemented zones [8,10,12,13]. Injection pressure should be sufficient to overcome hydraulic resistance but without promoting unwanted fissuring, soil heaving, or boundary-driven confinement. Effective control is also a matter of continuously mixing and watching for plugging, agglomeration of particles, filtration, and premature gelation. Such thinking encourages the consideration of pressure as one of a set of variables, together with grout volume and the observed treated geometry rather than simply viewing higher pressure as inherently better for soil conditions.

1.3. Fly Ash Stabilization

Class F fly ash has reactive silica and alumina, and a very low amount of self-cementing potential. The rationale is that pozzolanic reactions, when a calcium-bearing activator is also present, gradually produce cementitious compounds that strengthen and stiffen the soil over time. The stronger time dependence of fly ash treatment can be attributed to the slower rate of its reaction compared to cement hydration [15,17,19]. In many of the published reports on stabilization of soil with fly ash, fly ash is expressed as a percentage of the dry soil mass in the combined sample. That dosage cannot be converted directly for injection, as performance is dictated by concentration, water-to-solids ratio, viscosity, stability, and pumpability of the suspension. Therefore, this work investigates an injectable fly ash suspension activated with a minor Portland-cement portion rather than decoding mixing percentages as injection dosages. In terms of sustainability, the use of fly ash enables the substitution of a significant amount of virgin cementitious binder and provides a means to valorize an industrial by-product. However, environmental advantages depend on

transportation distance, processing, allocation of upstream impacts, activator content, and the level of performance achieved in service. Correspondingly, this work reports a transparent screen inventory in lieu of a full life-cycle assessment [20].

1.4. Alternative Injectable Stabilization Materials

Other satisfactory mineral and cementitious powders, such as silica fume and multicomponent ultra-fine binders, have been studied to vary stability of grouts, pore filling, strength and durability. For an injection use, a different material is meaningful only if the particle size distribution, bleeding, setting behavior, and filtration enable a controlled delivery within the soil [21,22]. The review does not include comparison with dissimilar concrete, asphalt, fiber-reinforced structural members, or soil-mixing techniques. This more focused topic ensures a close literature-experimental study linkage.

1.5. Foundation Improvement and Model-Scale Considerations

Soil injection under a strip footing may enhance its bearing capacity by forming a stiffer and stronger composite soil layer that helps to spread the contact stress over a wider area and reduces the settlement. Injectable techniques may be used with minimal impact to the surface as opposed to excavation-based techniques; however, the resulting geometry is influenced by soil heterogeneity, injection spacing, pressure, and stop criteria [4,6,19]. While laboratory apparatus allows closely controlled comparisons, they can lead to overestimates of response when the treated zone or failure mechanism co-acts with rigid boundaries. Hence, model dimensions should be presented as ratios to the foundation width and findings obtained at a wall should be regarded as boundary limited rather than as unconstrained expansion of grout or bearing capacity at the field scale. The rates of hydration and pozzolanic reaction are affected by curing temperature and moisture supply. Comparative tests should be carried out under the same curing conditions, while the field application should consider ground water, seasonal temperature, drainage, and construction sequence. These concerns are baked in to the results and the limitations of the current study.

2. Materials and Methods

2.1 Soil Characterization

In all, 54 specimens were tested, 18 untreated reference specimens and 36 treated specimens for the material-injection pressure-curing period combination in duplicate tests. The soil was collected from Baghdad, Iraq. Preliminary tests were particle-size distribution from sieving and hydrometer (ASTM D6913/D6913M, ASTM D7928), Atterberg limits (ASTM D4318), water content (ASTM D2216), specific gravity (ASTM D854), compaction by Standard Proctor (ASTM D698), direct shear (ASTM D3080/D3080M), and California bearing ratio (ASTM D1883). The classification is based on ASTM D2487. The gradation fraction was 12% gravel, 45% sand, 28% silt, and 15% clay. Since 57% of the soil is coarse grained and sand is the predominant coarse fraction, the primary symbol is S. The fines content is more than 12%; with LL = 32% and PI = 14%, the measured PI lies above the A-line with a value of 8.76, means that the fines are clayey and the soil is classified as a clayey sand (SC). The other index, compaction, shear, CBR, and chemical attributes are not reiterated in the text but are presented in Table 1.

2.2 Stabilization Materials

The micro cement was a ready-to-use ultra-fine cement grout with $d_{95} < 15 \mu\text{m}$, a specific gravity of 3.05, and a Blaine fineness of $8500 \text{ cm}^2/\text{g}$. Micro cement suspensions were passed through a screen at w/c ratios of 0.6, 0.8, and 1.0; 0.8 was selected for the major pressure and curing evaluation because it represented a best compromise between pumpability and strength. The bulk physical and oxide composition are listed in Table 1. The fly ash was obtained from a coal-fired power plant and satisfied the chemical criteria for Class F fly ash in ASTM C618. The reported chemical compositions of both binders were determined by X-ray fluorescence (XRF) analysis. For injection, fly ash was activated with ordinary Portland cement equal to 3% of the fly-ash mass. Suspensions were screened at water-to-solids ratios of 0.8, 1.0, and 1.2; a ratio of 1.0 was used in

the main comparison following preliminary injectability trials. The physical and chemical properties of the soil and stabilization materials are summarized in Table 1.

Table 1. Physical and chemical characteristics of the soil and stabilization materials

Property	Soil	Micro cement	Fly Ash
USCS classification (ASTM D2487)	SC	-	-
Gravel fraction (%)	12	-	-
Sand fraction (%)	45	-	-
Silt fraction (%)	28	-	-
Clay fraction (%)	15	-	-
Specific gravity (ASTM D854)	2.67	3.05	2.15
Natural water content (%)	8.5	-	-
Liquid limit (%)	32	-	-
Plastic limit (%)	18	-	-
Plasticity index (%)	14	-	-
Optimum moisture content (%)	12.5	-	-
Maximum dry unit weight (kN/m ³)	18.4	-	-
Effective cohesion (kPa)	15	-	-
Effective friction angle (°)	28	-	-
Unsoaked CBR (%)	8.5	-	-
Soaked CBR (%)	5.2	-	-
SiO ₂ (%)	-	21	52
Al ₂ O ₃ (%)	-	5	28
Fe ₂ O ₃ (%)	-	3	8
CaO (%)	-	64	3
Blaine fineness (cm ² /g)	-	8500	3200
pH	7.8	12.5	9.2
Sulfate content (%)	0.08	-	-
Organic content (%)	<1	-	-

2.3 Experimental Setup

A physical laboratory model was developed to simulate injection-driven improvement beneath a strip foundation. The steel test chamber measured 600 mm × 300 mm × 400 mm internally, and transparent acrylic side panels were used to observe the flow of grout.

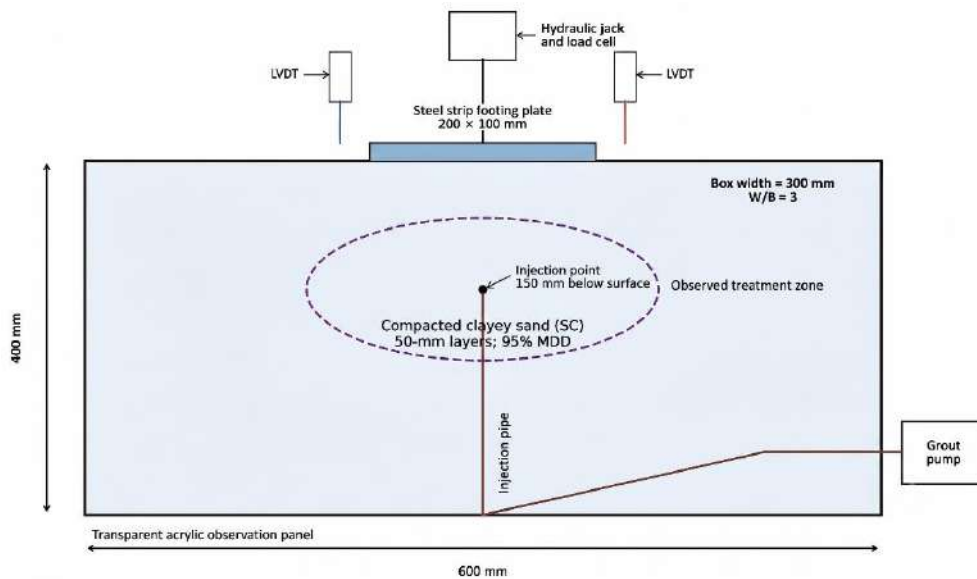


Fig. 1. Schematic of the laboratory injection and plate-load test setup (dimensions in millimetres)

Compaction was achieved by placing the soil in 50-mm thick layers and compacting each layer to 95% of maximum dry unit weight at the optimum moisture content. The model strip footing was a rigid steel plate with a length of 200 mm and a width of 100 mm and the chamber-width-to-footing-width ratio was set at $W/B = 3$. A single injection point was located 150 mm under the surface of the soil below the plate centerline. Grout was pumped under pressure through a pressure-regulated pump with depth measurement and a calibrated flow meter. The plate was subjected to a vertical load by the loading frame and LVDTs at the center and edges of the plate were used to measure settlement. The device, as well as the main dimensions, is schematically illustrated in Fig. 1.

2.4 Injection Procedure

Micro cement and fly ash slurries were injected at the controlled pressures of 100, 200 and 300 kPa. The basal comparison was with 0.8 water-to-cement ratio for the micro cement and 1.0 water-to-solids ratio for the activated fly ash. Pressure was raised stepwise to the target value, maintained and the injected volume, time, surface response and observed grout spread recorded. Injection was stopped when the target volume was injected, or a sudden sustained pressure increase was observed, or when substantial surface heaves or leak of grout from the surface was visually appreciated. Before injection, a 10-kPa seating stress was applied to establish the initial plate position. After injection, specimens were cured for 7, 14, or 28 days under nominal laboratory conditions of $23 \pm 2^\circ\text{C}$. The surface was covered to minimize moisture loss and all specimens were handled using the same curing procedure. The reported grout volumes are actual accepted volumes and therefore vary with material and pressure.

2.5 Loading and Strength Tests

After curing, vertical contact stress was applied in 10-kPa increments. Each increment was maintained for at least 15 min and until the settlement rate had substantially stabilized before the next increment. Loading continued until pronounced nonlinear deformation was observed or the planned terminal settlement was reached. Load and settlement were recorded continuously to construct the model load-settlement curves. Because the plotted curves do not support a consistent 10%-of-footing-width failure criterion, the reported capacity is defined here as the maximum applied contact stress reached in the model test at the adopted termination condition. It is therefore termed “model test capacity” and must not be interpreted as an allowable field bearing capacity. Unconfined compressive strength (UCS) was measured on samples recovered from the central and peripheral portions of the treated zones after plate loading; the sampling location and extraction method, and the potential disturbance caused by prior loading, are considered when interpreting the UCS results.

2.6 Analysis Methods

The results were evaluated using descriptive comparisons. Model capacity improvement was calculated as $100(q_t - q_u)/q_u$, where q_t and q_u are the treated and untreated model test capacities. Settlement reduction at 100 kPa was calculated as $100(s_u - s_t)/s_u$, where s_t and s_u are the treated and untreated model test settlement. Cost effectiveness was expressed as material cost divided by the increase in model test capacity. Duplicate measurements and reported coefficients of variation were used to describe repeatability; no inferential ANOVA claim is made because the available replicate structure is insufficient for a robust multifactor significance test. The charts are empirical summaries of the tested conditions and are not predictive design equations. Figure 2 compares the 28-day load-settlement response of untreated soil with micro cement- and fly-ash-treated soil at injection pressures of 100, 200, and 300 kPa. The curves show progressively lower settlement and higher terminal model stress with increasing injection pressure, while the 300-kPa results require caution because the treatment zones approached the chamber boundaries.

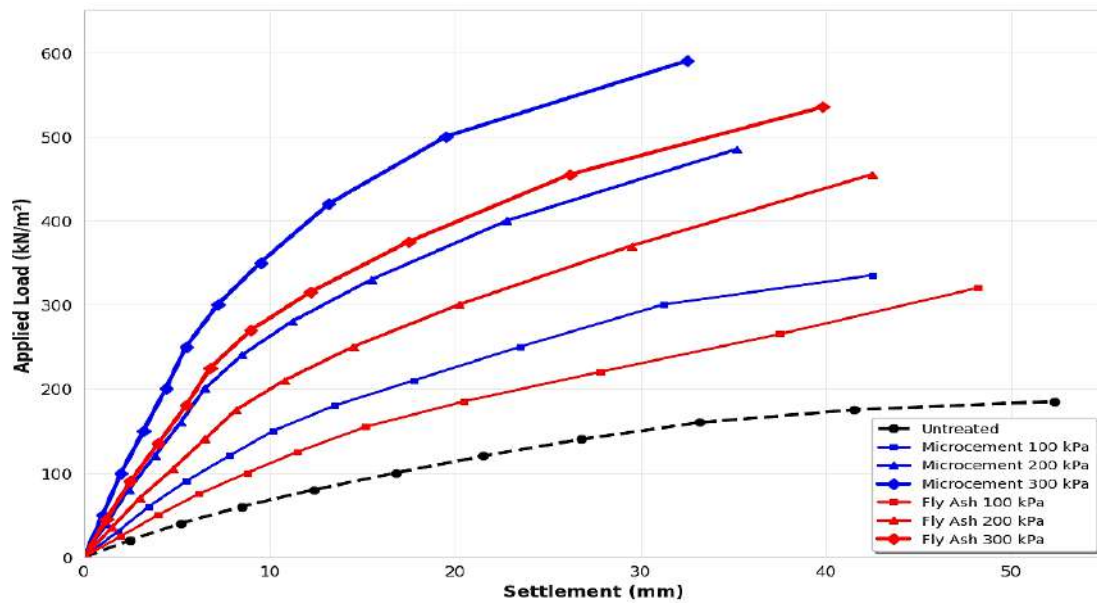


Fig. 2. Load-settlement response of untreated and treated clayey sand after 28 days of curing

3. Results and Discussion

3.1 Injection Characteristics

Micro cement and fly ash demonstrated different injection performances. Micro cement achieved the termination state at 18.5 to 24.5 min at pressures of 100 to 300 kPa, while fly ash took 32.0 to 38.0 min for the two said pressures. Through the acrylic windows visual observations showed a relatively sharper and more regular micro cement front, with the fly ash suspension yielding a more irregular interface and larger local accumulation. These results seem also in line with the finer micro cement particles and the stronger filterability of the coarser fly ash suspension, albeit microstructural imaging was not undertaken. Injection pressure had the effect of increasing both the volume of the accepted grout and the apparent size of the treatment zone (Table 2).

Table 2. Injection parameters and observed treatment-zone dimensions

Material	Injection pressure (kPa)	Injection time (min)	Accepted grout volume (L)	Horizontal extent (mm)	Vertical extent (mm)
Micro cement	100	18.5	5.5	180	120
	200	22.0	10.0	280	190
	300	24.5	14.5	≥300*	250
Fly ash	100	32.0	6.5	150	100
	200	36.5	11.5	240	160
	300	38.0	16.5	≥300*	210

* The treatment front reached the chamber wall; the unconstrained horizontal extent could not be measured.

At the levels of 100 and 200 kPa the measured horizontal extents had not yet exceeded the 300-mm chamber width. At 300 kPa the nominal horizontal distances from visual tracing are 350 mm for micro cement and 310 mm for fly ash, which is larger than the available chamber width. Hence, these values are reported as boundary-censored (≥300 mm), indicating interaction with the side walls rather than unconstrained radial spreading. Additionally, significant surface heave and preferential pathways for the highest pressure suggests that 300 kPa was close to the upper limit of the model. The volume of accepted grout ranged between 5.5 and 14.5 L for the micro cement and between 6.5 and 16.5 L for the fly ash. The chamber volume is about 72 L, so the maximum fly ash injection is about 23% of the chamber volume. This large fraction of replacement and the wall interaction at 300 kPa, are significant scale limitations. Fly ash needed a greater volume and a longer injection time at all pressures due to its lower penetrability and even higher void filling request.

3.2 Model Test Capacity

The terminal model test strength of the clayey sand without treatment was 185 kPa. At 28 days, micro cement treatment resulted in capacities of 335, 485, and 590 kPa for injection pressures of 100, 200, and 300 kPa, respectively. These represent increases of 81%, 162%, and 219% over the baseline untreated. The result demonstrates a strong pressure-dependent improvement and is very encouraging when considering this model, but the 590-kPa should be viewed as a maximum laboratory response and not an unconstrained field capacity since the grout front reached the chamber boundary. Resistance at the end of treatment by fly ash stabilized soil increased gradually, however; it got closer to the micro cement response with curing. At 28 days, the capacities were 320, 455, and 535 kPa at 100, 200, and 300 kPa, respectively, which represent increases of 73%, 146% and 189%. At 200 kPa the strength of the fly ash was nearly 94% that of the micro cement. The change from 100 to 200 kPa was more effective than from 200 to 300 kPa for both materials. This latter increase required much more grout and was accompanied by wall effect, preferential flow, and soil heave. Therefore, the maximum observed at 300 kPa, and 200 kPa is more justifiable compromise-performance efficiency. The model capacity enhancement factor at 7, 14, and 28 d is summarized by Fig. 3. Micro cement exhibited the best early-age response, followed by the fly ash curves that increased more pronouncedly with curing age. The figure should be read as a contrast among the tested model scenarios, not as a regression equation for field design.

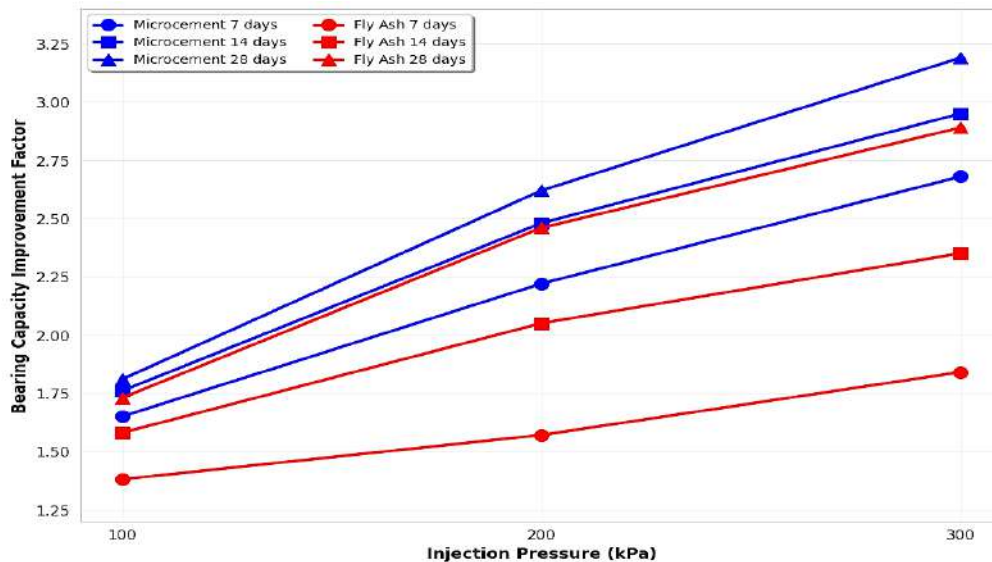


Fig. 3. Model capacity improvement factor versus injection pressure at different curing periods

3.3 Settlement Reduction

Settlement was compared at the same applied stress of 100 kPa to enable the evaluation of all specimens at the same load level. This is a test stress for laboratory comparison only and is not recommended as allowable design pressure. The untreated consolidation was 18.5 mm. At an injection pressure of 200 kPa and a curing time of 28 days, micro cement reduced the consolidation to 6.5 mm (65%), while fly ash reduced it to 8.2 mm (56%). At the injection pressure of 100 kPa, the 28-day consolidation reductions were 42% and 35% for micro cement and fly ash, respectively. When the injection pressure was raised to 300 kPa, the reductions were 72% and 63%, respectively. Yet the further improvement was relatively small compared to the extra grout consumption under conditions that the treatment front reached the chamber walls. For this reason, the 300-kPa result can be considered as an upper model response, but not as the automatically recommended field pressure. Curing had a larger effect on settlement for fly ash. At 200 kPa, the reduction obtained with micro cement was 58% (7 days) and 65% (28 days), while the rates for fly ash rose from 38% to 56% during the same interval. The distinction favors micro cement when

rapid loading is needed and fly ash when longer curing is permissible. The full settlement results are tabulated in Table 3.

Table 3. Settlement at a common comparison stress of 100 kPa

Material	Injection pressure (kPa)	7-day settlement (mm)	14-day settlement (mm)	28-day settlement (mm)	28-day reduction (%)
Untreated reference	N/A	18.5	18.5	18.5	N/A
Micro cement	100	12.0	11.2	10.8	42
	200	7.8	6.9	6.5	65
	300	6.2	5.5	5.2	72
Fly ash	100	13.5	12.5	12.0	35
	200	11.5	9.2	8.2	56
	300	9.8	7.5	6.8	63

3.4 Strength Development

UCS testing offered a straightforward measure of cementation in the treated zones. The reference UCS for untreated soil was 145 kPa. For the samples retrieved from the 200-kPa micro cement treatment, the average strength values were 680, 1150, and 1420 kPa at 7, 14, and 28 days. The rapid early gain is consistent with cement hydration. Since the UCS specimens were taken after plate loading, the values may be sampling and prior-loading disturbed and should be considered post-test indices, not undisturbed in situ strengths. The fly ash equivalent strengths were 385, 825 and 1285 kPa at 7, 14, and 28 days. The lower 7 days result and higher gain from 7 to 28 days agrees with delayed pozzolanic reaction. At 28 days, fly ash achieved about 90% of the micro cement UCS, which confirms that the difference in performance reduces when sufficient curing is supplied [15,16,19]. The central samples were said to be 15% to 25% stronger than the peripheral ones. Coefficients of variation (reported) were 12% to 18% for micro cement and 18% to 28% for fly ash, suggesting higher spatial variability in the fly ash treatment. These results justify the conservative field acceptance criteria and multiple verification locations. No scanning electron microscopy or full post-test core-photograph suite exist, so any statements concerning bonding mechanisms and homogeneity in are derived from the measured macroscopic response and visual chamber observations. Figure 4 shows the UCS development at 200 kPa. Micro cement dominates at early age, whereas the fly ash curve rises more steeply between 7 and 28 days and approaches the micro cement strength at the final curing age.

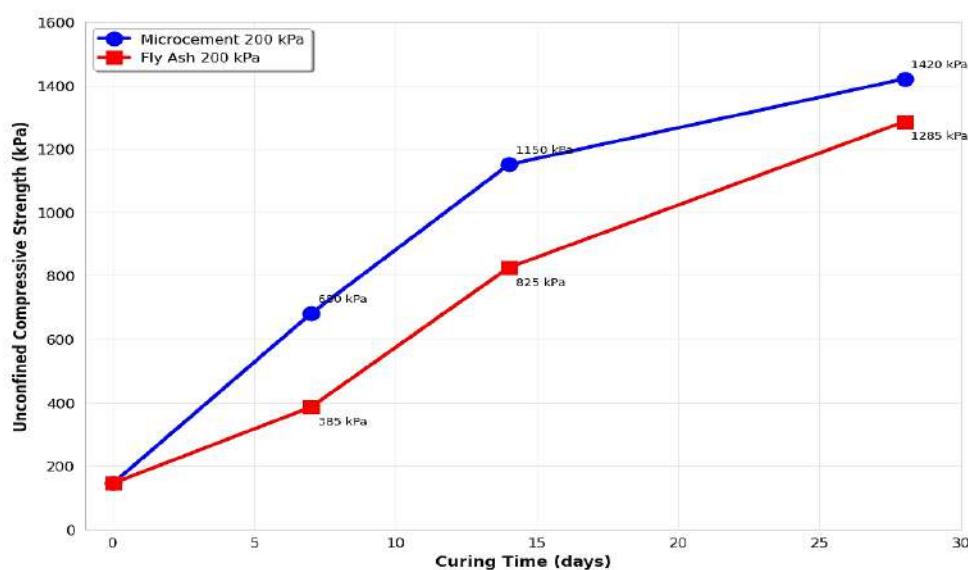


Fig. 4. Unconfined compressive strength development at an injection pressure of 200 kPa

3.5 Cost-Effectiveness Analysis

Economic analysis indicates that the selection of stabilization materials should not rely solely on technical performance parameters but must also integrate material and operational costs. The material-only economic comparison was performed for the 200-kPa condition using contemporaneous laboratory estimates for procurement in Baghdad. The estimated treatment material cost was \$45 for 10.0 L of micro cement grout and \$12 for 11.5 L of activated fly ash grout. Because absolute prices vary by location, supplier, currency, transport, and date, Table 4 presents normalized ratios, with fly ash assigned a material-cost index of 1.00. The two methods used broadly similar pumping equipment, although micro cement required tighter control of mixing, suspension stability, and potential early gelation. Fly ash required longer injection time and a slightly larger volume. Labor, mobilization, quality-control, and schedule costs were not included; thus, the comparison is a material-cost screening rather than a complete project estimate. At 28 days, material cost per unit increase in model test capacity was \$0.150/kPa for micro cement and \$0.044/kPa for fly ash. On this basis, fly ash was 3.41 times more cost effective for the 28-day response. At 7 days, the difference was smaller because micro cement developed strength more rapidly. Selection should therefore consider the value of schedule acceleration in addition to binder cost.

Table 4. Material-cost screening for the 200-kPa injection condition

Parameter	Micro cement	Fly ash	Ratio (MC/FA)
Material cost (\$/treatment)	45	12	3.75
Normalized material-cost index (FA = 1.00)	3.75	1.00	3.75
Accepted grout volume (L)	10.0	11.5	0.87
7-day model test capacity (kPa)	410	290	1.41
28-day model test capacity (kPa)	485	455	1.07
Cost per kPa capacity gain, 7 days (\$/kPa)	0.200	0.114	1.75
Cost per kPa capacity gain, 28 days (\$/kPa)	0.150	0.044	3.41
28-day settlement reduction (%)	65	56	1.16

Notes: Costs represent laboratory-scale material estimates at the time of testing and exclude labor, equipment, transport, overhead, and quality-control costs. The normalized cost index is included to improve transferability across markets.

3.6 Sustainability Screening

A transparent binder-inventory calculation was used instead of a qualitative “overall impact score.” For 10 L of micro cement grout at $w/c = 0.8$ and a micro cement specific gravity of 3.05, the estimated dry micro cement mass is approximately 8.9 kg. Using a representative screening factor of 0.85 kg CO₂e/kg gives about 7.5 kg CO₂e per treatment. For 11.5 L of fly ash grout at a water-to-solids ratio of 1.0, the estimated solids are approximately 7.6 kg fly ash and 0.24 kg Portland-cement activator. Using screening factors of 0.20 kg CO₂e/kg for processed fly ash and 0.85 kg CO₂e/kg for cement gives approximately 1.7 kg CO₂e, or about 77% less than the micro cement treatment. The same analysis shows about 8.9 kg to 9.4 kg of virgin cementitious binder is needed for micro cement as compared to about 0.24 kg of virgin cement activator for the fly ash mixture, and about 7.6 kg of fly ash is recovered. The results thus indicate a potential lower binder and waste-based benefit for fly ash given the specified assumptions.

This indeed is a screening comparison, not a full life-cycle assessment. It does not consider transport, plant energy, allocation of upstream burdens of fly ash, equipment, field losses, duration of construction, and effects of service life. Sulfate, acid, wet-dry and other aggressive exposures testing was not conducted and therefore no claim of superior durability is made. A project life-cycle assessment and durability program are needed before the screening result can be applied to procurement or design decisions. Figure 5 presents the underlying inventory quantities rather than a composite 0-to-100 score. The separation of carbon burden, virgin binder consumption, and waste utilization makes the assumptions and trade-offs directly auditable.

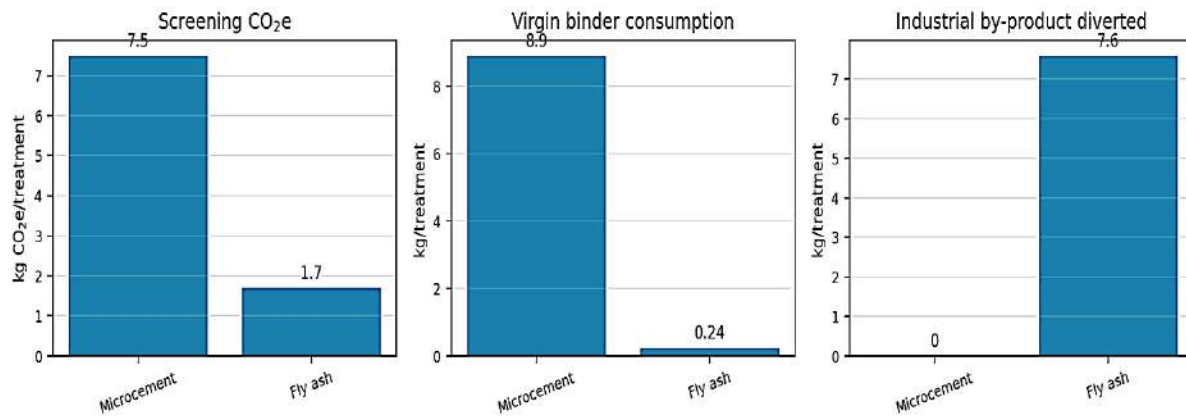
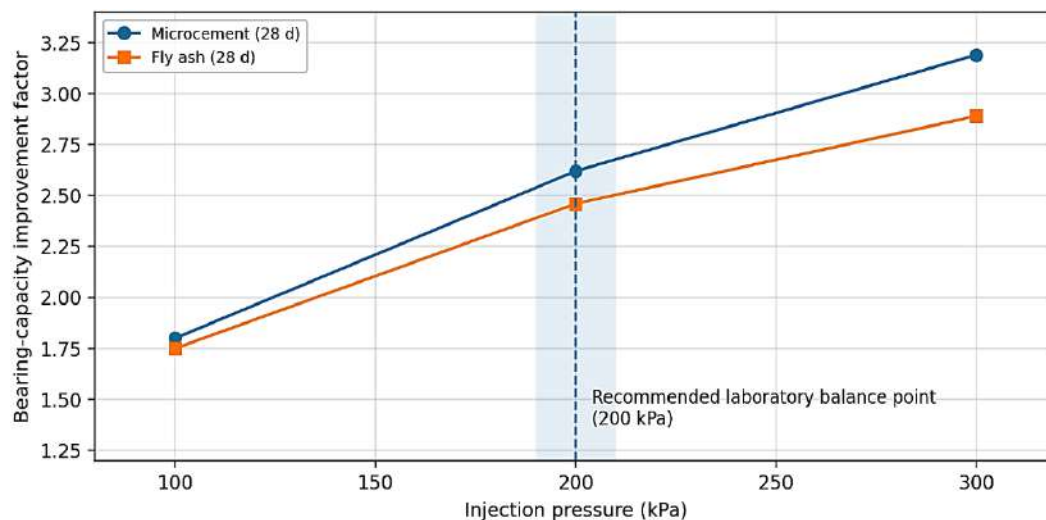


Fig. 5. Screening environmental inventory for the 200-kPa, 28-day laboratory treatments. Values depend on the stated material quantities and emission-factor assumptions

3.7 Practical Implementation Considerations

Natural soils are more variable than soil prepared in the laboratory and can include layers, fractures, water, utilities, and non-uniform stress fields. A field program, therefore, has to start from site investigation and a trial section for gathering information on grout ability, pressure limits, spacing, target volume, heave response, and the volume injected vs. the verified improvement [4,15,23]. Quality control should be a combination of direct and indirect testing. Direct methods are recovered cores or treated-soil samples, and laboratory strength tests. Indirect procedures are such as pressure-flow-volume records, measurement of surface heave, penetration tests, if applicable geophysical measurements and plate-load tests after treatment. The laboratory results showed some central-to-peripheral variability, especially for fly ash; it should be accepted on a number of locations. Micro cement is preferred in applications requiring early loading, better grout ability or more uniform treatment. Activated fly ash is preferred for at least 28 days of curing, and low material cost and embodied-carbon reduction are the benefits. In both cases, choose maximum pressure not based only on laboratory values, but determine an in-field limit to prevent hydraulic fracture, un-controlled flow, or damaging heave from developing. Figure 6 is the preliminary decision flow chart derived from the observed 28-day model capacities, grout consumption and cost screening. It is not a regression model. The 200-kPa point is recommended as the optimum value for the tested clayey sand, as it achieved significant improvement, and was the least influence by wall effect when compared to the 300-kPa condition. The chart needs to be recalibrated for other soils, foundation sizes and field conditions.



LABORATORY-SPECIFIC: not valid for other soil types or field design without trial sections.

Fig. 6. Preliminary laboratory decision chart for the tested clayey sand. The chart is not a field design equation

3.8 Limitations and Future Research

The main restriction is the finite model geometry. The width of the chamber was only three footing widths ($W/B = 3$), and the 300 kPa grout fronts hit the lateral boundaries. The nominal horizontal spreads at that pressure are therefore censored, and the associated capacity and settlement enhancements could have been further amplified by lateral confinement. The maximum volume injected also accounted for about 23% of the chamber volume, which is significantly larger than the local treatment ratio commonly employed in field applications. Only one clayey sand, one chamber density, one injection depth and curing ages up to 28 days were investigated. The findings cannot be directly extended to clean sand, high plastic clay, collapsible soil, stratified ground or water bearing ground. Longer curing ages and durability testing under wet-dry cycling, sulfate and acid attack and under hydraulic gradients are especially necessary, in particular for activated fly ash.

The double test structure precludes robust inferential statistics and prevents meaningful multifactor ANOVA without the entire replicate-level data set. If possible, show error bars and test for the significance of raw readings in the original. Specimens of UCS were retrieved post plate loading; thus, the sampling disturbance and the pre-shear history might affect the strength presented of the sample. Dynamic and cyclic loading, repeated foundations and full-scale trial section developments were beyond the scope of this study. Figure 7 summarizes exploratory mix-ratio tests. For micro cement, lower w/c increased strength and decreased penetration, whereas for fly ash, higher water-to-solids ratio resulted in higher penetration and lower strength. These tests are indicative of the strength-grout ability trade-off and the above hold for the chosen main-test ratios of 0.8 and 1.0, respectively. Future work needs to explicitly measure viscosity, bleeding, setting time, and filtration, and employ these results to establish a soil-specific grout ability criterion.

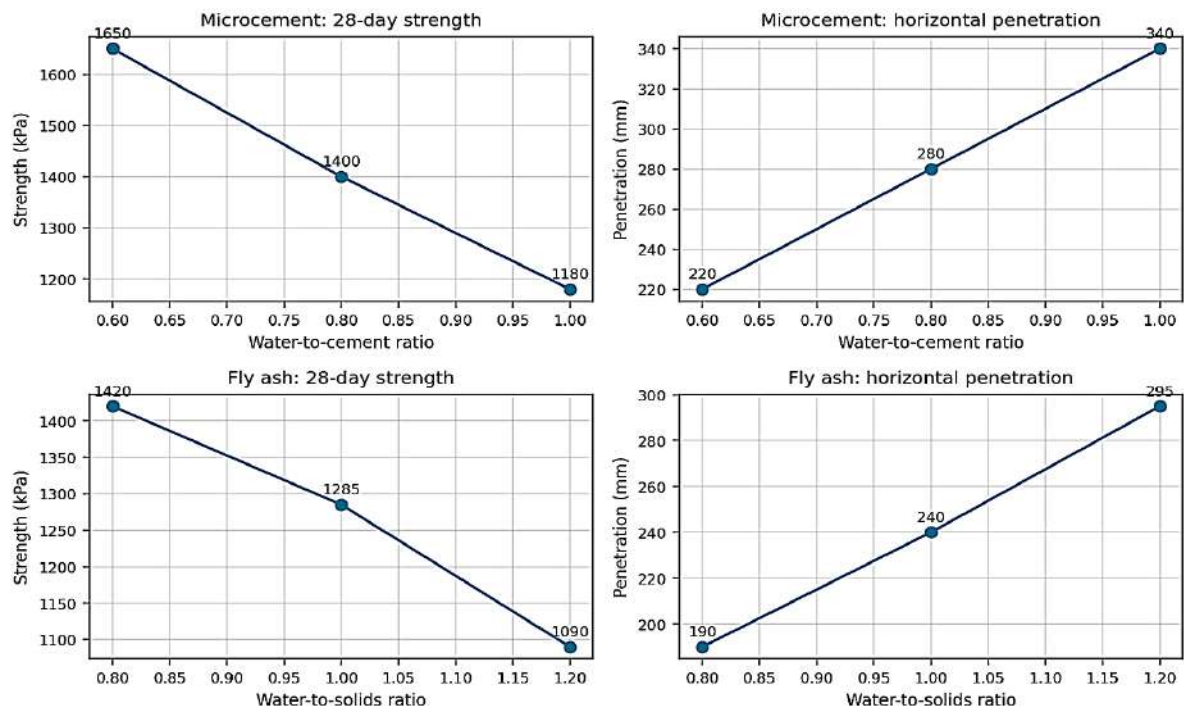


Fig. 7. Exploratory effect of suspension ratio on 28-day strength and horizontal penetration. Each panel uses a single response axis to avoid dual-axis ambiguity

This study is constrained by ($W/B = 3$), 300 kPa boundary-censored spread, one soil type, short-term curing, duplicated tests, UCS sampling after loading, and lack of durability and dynamic loading. Advancing the research requires larger chambers or centrifuge/full-scale models preserving entire replicate data for statistical analysis, communication of grain size distribution parameters, as well as microstructural and core evidence, and evaluation of long-term curing and excessive exposure. Within these constraints the results indicate that pressure optimization and

curing time are at least as important as binder type and that activated fly ash can be competitive with micro cement performance whilst having a significantly superior 28-day cost and screening-carbon material profile. Table 5 presents decision-support summary for preliminary material selection

Table 5. Decision-support summary for preliminary material selection

Project priority	Preferred option	Reason and required control
Early loading or accelerated construction	Micro cement	Faster hydration and greater early-age strength; verify grout ability and control gelation.
Restricted pore access or treatment uniformity	Micro cement	Finer particles and more regular laboratory spread; confirm by a field trial and multiple verification points.
Lower binder cost and carbon with ≥ 28 days curing	Activated fly ash	Near-micro cement 28-day performance in this study; verify activator, supply consistency, and curing.
Uncertain or heterogeneous field conditions	Trial both or use staged selection	Establish pressure, grout take, heave, spread, and acceptance criteria before production treatment.

4. Conclusions

This paper is part of a series of investigations on micro cement and activated fly ash grouting in which injection into clayey sand through a strip footing is considered under strip-footing pressure application and curing conditions. Both binders showed an improved load-settlement response measured with respect to the untreated reference as drawn in the following points:

- The unmodified model achieved a terminal capacity of 185 kPa. In 28 days, micro cement yielded 335, 485 and 590 kPa for injection pressures of 100, 200 and 300 kPa that represented increments of 81%, 162% and 219%. Fly ash achieved 320, 455, and 535 kPa, representing increments of 73%, 146%, and 189%. These are model test capacities, as defined by the adopted loading termination criterion, and should not be interpreted as allowable bearing soil pressures in the field. Micro cement developed strength at a faster rate leading to shorter injection times and more regular, predictable fronts of treatment. This is advantageous for projects with early loading or greater penetrability in tight pores space.
- Fly ash-based products were more dependent on curing, however the difference between outputs decreased significantly over time. At 200 kPa and 28 days, fly ash was about 94% of micro cement model capacity and 90% of its UCS. This indicates that activated fly ash, given sufficient curing time, could be a technically competitive long-term option for when the construction schedule can be extended long enough to allow for adequate curing.
- The settlement at a common comparison stress of 100 kPa was reduced from 18.5 mm in untreated soil to 8.2 mm with fly ash and 6.5 mm with micro cement at 200 kPa after 28 days. The reductions were 65% and 56%, respectively. Although maximum capacity and settlement reduction were also measured at 300 kPa, the grout fronts reached the borders of the chamber and the injected volumes became large relative to the chamber volume. Thus, the 300-kPa results might incorporate lateral-confinement effects and should be considered as maximum laboratory responses rather than proof that the highest tested pressure is universally best.
- Under the test model with the given parameters, 200 kPa was the most reasonable compromise in terms of mechanical enhancement, grout consumption, cost and experimental reliability. For the minimum tested quality, cement cost screening at this pressure gave the values of 0.150\$/kPa of capacity gain for micro cement and 0.044/kPa for fly ash at 28 days, which indicates that using fly ash was 3.41 times more profitable on this metric.
- The binder-resource screening, meanwhile, also predicted roughly 7.5 kg CO₂e for micro cement and 1.7 kg CO₂e for fly ash, a 77% reduction under the given assumptions. These

figures do not account for labor, equipment, transportation, field losses and service-life effects and thus should help, not hinder, a project-specific life-cycle cost and assessment.

The results are consistent with an opportunistic selection model. Micro cement is the choice when early strength development, high pumpability, or finer consistencies are the main needs. Activated fly ash is the choice if more than 28 days of curing are available and considerations of the lowest binder cost, the least virgin-cement demand, and the use of an industrial by-product are paramount. Neither should be based on laboratory pressure values alone. Test sections in the field must determine the pressure limitations, grout take, treatment spacing, heave tolerance, and acceptance criteria for the encountered soil.

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References

- [1] Al-Kalili A, Ali AS, Al-Taie AJ. A review on expansive soils stabilized with different pozzolanic materials. *Journal of Engineering*. 2022;28(1):1-8. <https://doi.org/10.31026/j.eng.2022.01.01>
- [2] Karkush M, Salman A D, Abbood A N, Emad R. Evaluating the role of nano-clay and nano-metakaolin in mitigating the collapsibility of gypseous soils. *Res. Eng. Struct. Mater.*, 2026; 12(2): 1139-1151. <http://dx.doi.org/10.17515/resm2026-1351ic1120rs>
- [3] Mohsen AH, Albusoda BS. The collapsible soil, types, mechanism, and identification: a review study. *Journal of Engineering*. 2022;28(5):41-60. <https://doi.org/10.31026/j.eng.2022.05.04>
- [4] Al-Khadaar RM, Ahmed MD. Review of jet grouting practice around the world. *Journal of Engineering*. 2023;29(7):48-70. <https://doi.org/10.31026/j.eng.2023.07.04>
- [5] Mohammed MS, Ahmed MD. Performance assessment of pile models chemically grouted by low-pressure injection laboratory device for improving loose sand. *Journal of Engineering*. 2023 Dec 2;29(12):123-38. <https://doi.org/10.31026/j.eng.2023.12.08>
- [6] Hussein SH, Ahmed MD. Stiffness characteristics of pile models for cement-improved sandy soil by a low-pressure injection laboratory setup. *Journal of Engineering*. 2023;29(3). <https://doi.org/10.31026/j.eng.2023.03.11>
- [7] Ali HA, Yousuf YM. Improvement of shear strength of sandy soil by cement grout with fly ash. *Journal of Engineering*. 2016;22(12):16-34. <https://doi.org/10.31026/j.eng.2016.12.02>
- [8] Sabermahani M, Elahi H R, Bilal A M, Khodaverdian A, Karkush M. Stabilization of excavation using the diaphragm wall method in Karbala. *Res. Eng. Struct. Mater.*, 2026; 12(2): 1153-1164. <http://dx.doi.org/10.17515/resm2026-1352ic1120rs>
- [9] Axelsson M, Gustafson G, Fransson Å. Stop mechanism for cementitious grouts at different water-to-cement ratios. *Tunnelling and Underground Space Technology*. 2009;24(4):390-397. <https://doi.org/10.1016/j.tust.2008.11.001>.
- [10] Mozumder RA, Laskar AI, Hussain M. Penetrability prediction of microfine cement grout in granular soil using artificial-intelligence techniques. *Tunnelling and Underground Space Technology*. 2018;72:131-144. <https://doi.org/10.1016/j.tust.2017.11.023>.
- [11] Mollamahmutoglu M, Avci E. Strength and permeability of boric-acid-tempered ultrafine-cement-grouted sand. *Construction and Building Materials*. 2021;284:122812. <https://doi.org/10.1016/j.conbuildmat.2021.122812>.
- [12] da Rocha Gomes S, Ferrara L, Sánchez L, Moreno MS. A comprehensive review of cementitious grouts: composition, properties, and applications. *Construction and Building Materials*. 2023;375:130991. <https://doi.org/10.1016/j.conbuildmat.2023.130991>.
- [13] Bantralexis KE, Markou IN, Zografos GI. Use of sand pore-size distribution to predict cement-suspension groutability. *Developments in the Built Environment*. 2023;14:100138. <https://doi.org/10.1016/j.dibe.2023.100138>.
- [14] Dano C, Hicher PY, Tailliez S. Engineering properties of grouted sands. *Journal of Geotechnical and Geoenvironmental Engineering*. 2004;130(3):328-338. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:3\(328\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:3(328)).
- [15] Karim HH, Samueel ZW, Jassem AH. Influence of fly ash addition on behavior of soft clayey soil. *Engineering and Technology Journal*. 2020;38(5):698-706. <https://doi.org/10.30684/etj.v38i5A.426>
- [16] Amran M, Fediuk R, Murali G, Avudaiappan S, Ozbakkaloglu T, Vatin N, Karelina M, Klyuev S, Gholampour A. Fly ash-based eco-efficient concretes: A comprehensive review of the short-term properties. *Materials*. 2021 Jul 30;14(15):4264. <https://doi.org/10.3390/ma14154264>

- [17] Khare A, Gupta SK, Sah S, Kumar M, Toppo A, Jain A, Jaiswal SK. Implication of Fly Ash in Stabilizing Expansive Soil. *Intersect: The Stanford Journal of Science, Technology, and Society*. 2023 Apr 26;16(2).
- [18] Sengul T, Akray N, Vitosoglu Y. Investigating the effects of stabilization carried out using fly ash and polypropylene fiber on the properties of highway clay soils. *Construction and Building Materials*. 2023 Oct 12;400:132590. <https://doi.org/10.1016/j.conbuildmat.2023.132590>
- [19] Pekrioglu Balkis A. Properties and performance of a high-volume fly ash grout. *Marine Georesources & Geotechnology*. 2020;38(1):73-82. <https://doi.org/10.1080/1064119X.2018.1552999>.
- [20] Shi X, Zhang C, Liang Y, Luo J, Wang X, Feng Y, Li Y, Wang Q, Abomohra AE. Life cycle assessment and impact correlation analysis of fly ash geopolymer concrete. *Materials*. 2021 Dec 1;14(23):7375.. <https://doi.org/10.3390/ma14237375>.
- [21] Al-Gharbawi AS, Najemalden AM, Fattah MY. Expansive soil stabilization with lime, cement, and silica fume. *Applied Sciences*. 2023;13(1):436. <https://doi.org/10.3390/app13010436>
- [22] Almurshedi AD, Karkush M. Experimental and numerical modeling of load settlement behavior of gypseous soils improved by MICP. In *Smart Geotechnics for Smart Societies 2023* Aug 4 (pp. 583-589). CRC Press. <https://doi.org/10.1201/9781003299127>
- [23] Bilal AM, Jebur MM, Zhussupbekov A, Karkush M. Evaluation of in situ loading tests conducted on micropiles used to enhance the bearing capacity of Karbala sandy soil. *Soil Mechanics and Foundation Engineering*. 2025 Mar;62(1):65-72. <https://doi.org/10.1007/s11204-025-10028-w>

Using a non-homogeneous Markov Model to predict the load-deflection curve of concrete-encased steel I-beams

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Abstract

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A non-homogeneous Markov chain framework predicts the load-deflection curve in relation to different loads and deflections for the beams being tested. The load-deflection response was modeled using six discrete load levels and seven deflection states, where the deflection at midspan (discrete-state) can occur in a defined range or "delta c" (minimum to maximum). The transition matrices are derived from characteristics of the specimen beams (using ACI and AISI) and from the experimental measurement of the epochal test results of load and deflections, along with the associated time sequences through the "Markov" model. Incremental deflection was modeled using a normal distribution as a simplifying probabilistic assumption to estimate transition probabilities between discrete deflection states. Furthermore, the distribution has been evaluated by analyzing the mean-square-error (MSE). Two specimen beams were included in the evaluation (C1 and C2), and the performance was assessed using root mean square error (RMSE), normalized root mean square error (NRMSE), mean absolute percentage error (MAPE), accuracy acceptance (AA), coefficient of determination (R^2), and Pearson correlation coefficient (Pearson R). Beam C1 RMSE was 3.03 mm, AA = 97.62%, and $R^2 = 0.9920$. Beam C2 RMSE was 6.64 mm, AA = 93.03%, and $R^2 = 0.9783$. Within the proposed Markov framework, failure probability is defined as the probability that the beam reaches the terminal deflection state associated with a predefined limit state.

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1. Introduction

Composite steel-concrete structural systems exhibit significant structural advantages when the steel section is encased in or connected to concrete, allowing both materials to act together under applied loading. The structural performance of composite steel and concrete structural systems can also be considered. Composite members include steel 'I' sections, as well as other structural members, encased and/or connected to concrete and allowing for the materials to work together and respond to the applied load jointly. The combination of steel and concrete results in a high-performance structural system, leveraging the strengths of both materials. Steel is primarily known for its advantageous tensile strength and, more importantly, ductility. In contrast, concrete is mainly known for its compressive resistance and stiffness [1-3]. When properly bonded, steel and concrete working together to form an effective composite structural system, which will be discussed shortly. The outcome is a more, ductile, energy-efficient and effective structure. In structural engineering practice, the metals and the cementitious family, steel and concrete, respectively, the load-deflection behavior of steel-concrete composite sections is significantly influenced by the shear connection and the degree of composite interaction between the steel section and the surrounding concrete [4].

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Understanding serviceability and failure mechanisms in composite beams necessitates a well-characterized load-deflection curve. It has been shown that the composite I-beam stiffness and deflection characteristics are sensitive to longitudinal reinforcement, degree of composite action, and concrete confinement by several researchers [5-7]. Traditional methods for estimating deflection, such as differential equations of elastic behavior and finite element analysis, are useful for deterministic response prediction. The majority of traditional methods generate a single response curve using fixed properties of the material, fixed boundary condition, and fixed load assumption. It may be possible to define the relationship between the increasing load on the structure and the transition between ranges of deflection; however, these methods do not quantify the uncertainty associated with cracks, loss of stiffness and the variability between specimens. Therefore a Markov chain is established as the model to represent the load-deflection relationship as a series of probabilistic transitions through discrete states of deflection [8].

The Markov chain model has recently been used to evaluate parameters such as the life expectancy of a building, how far a crack will spread, and its reliability. The Markov chain model belongs to a larger group of models called stochastic models, which use probability and statistical methods to predict future behaviors. In the case of the Markov chain model, the future behavior is predicted by looking at previous behaviors or changes among different states of a system. These different states must be defined and fixed to different parts of the system. Once these rules are set they will not change from there on. The Markov chain model evaluates the changes of a system's state in a given time and records the result [9,10]. The Markov model has been used extensively in modeling the detailed actual load-deflection behavior of composite I-beams consisting of steel and concrete. Most of the work until now concentrated either on deterministic models or probabilistic deterioration under long-term environmental exposure [11,12]. The discrete state transitions to capture the progressive deflection behavior of steel-concrete composite beams under monotonic or incremental loading have not been sufficiently addressed [13].

An approach for estimating the monotonic load-deflection behaviour of encased composite beams has been developed as a Markov chain-based stochastic model. The primary basis for this approach is to model structural responses as a sequence of discrete deflected states (i.e., deflection state transitions associated with given incremental load steps) in which the current state of the structure and the magnitude of the applied incremental loading control the probability of transitioning from the currently observed state to another state in the subsequent incremental loading. While probabilistic structural models have been successfully employed for assessing both time-dependent degradation of structures and time-dependent reliability of structural components, their application to predicting response curves (i.e., developing a continuous load-deflection response over the entire range of increasing loads on a monotonically loaded structure) is limited, particularly with respect to encased composite structures. This limitation is significant because the traditional code-based methods for determining monotonic load-deflection responses (e.g., ACI and AISC sectional equations) are based on particular assumptions regarding the behavior of the structure during load application and only provide point estimate response curves; therefore, they do not provide any indication of the uncertainty or variability in the evolving structure due to stiffness degradation throughout the allocated timeframe of a structural component. A Markov-based model has been developed for predicting the load-deflection curve from the transition structure learned from the averages of the responses generated by the ACI and AISC equations, providing an additional "simplified" probabilistic approximation to non-linear stiffness evolution using a discrete data set that may still be used in conjunction with existing code based mechanics.

Most previous studies on the behavior of steel-concrete composite beams mainly relied on either deterministic analytical models or the more detailed representation provided by finite element (FE) analyses. While unimodal analytical models provide detailed local responses, they are computationally expensive when performing the progressive transition between deflection states, FE analysis does not capture the probability associated with transitioning from one deflection state to another. Thus, this study fills a gap in the literature whereby no simple, probabilistic framework has been developed to track the progressive load-deflection response of concrete encased steel I-beams under monotonic loading and express uncertainty regarding deflection state transitions. A Markov-chain model has been proposed to provide a solution to this gap by transforming the load-

deflection response into a series of discrete probabilistic states such that the model may predict the probability of transitioning from one deflection state to another, as load is increased.

2. Materials and Sample Preparation

An attempt was made to assess the accuracy and predictability of the proposed model against two other existing models that have been subjected to experimentation. Previous experimental evaluations conducted by Xian et al. [14] and Mahmood [15] were performed using two individual concrete-encased composite beams (i.e., Case C1 and Case C2) in a flexural test when a single point load was applied at mid-span along a simply supported beam placed in a monotonic, three-point bending configuration. Clear spans were equal to 2,000 mm for Beam C1 and 2,750 mm for Beam C2, which is the reason for differentiating between how these two experiments were referenced in this study. For beam C1, the spans of the supports were 2000 mm, while those for beam C2 were 2750 mm. The works reported by these authors will be referred to as Case C1 (for beam C1) and Case C2 (for beam C2).

The C1 beam tested by Xian et al. has a concrete cross-section of 160 mm × 250 mm. The C1 beam includes a steel profile with a height of 140mm; it has a web with a thickness of 5.5mm and flanges 80 mm wide x 9.1 mm thick. The concrete was reinforced with a 40 mm overlay of concrete rebar over the top and bottom of the beam, the top rebar having an area of 85 mm² and a bottom rebar area of 57mm². The C1 beam had a total span of 2000 mm with a shear span of 120mm (0.12m). The concrete had a compressive strength of 48.4MPa, tensile strength of 4.8 MPa, and elastic modulus of 33.23GPa. The assumed ultimate strain at failure is 0.002*, and the cracking strain is 0.0004*. The yield strength of the reinforcing steel is 276 MPa, the elastic modulus is 200 GPa; whereas the yield strength of the steel profile is 315 MPa and the ultimate strength is 410 MPa. Parameters identified with an asterisk (*) were not reported directly from the original source required for analytical purposes; however, the following values referenced below are from related design codes or literature in support of other similar steel/concrete composite elements. No unmarked estimated values were included in the analysis.

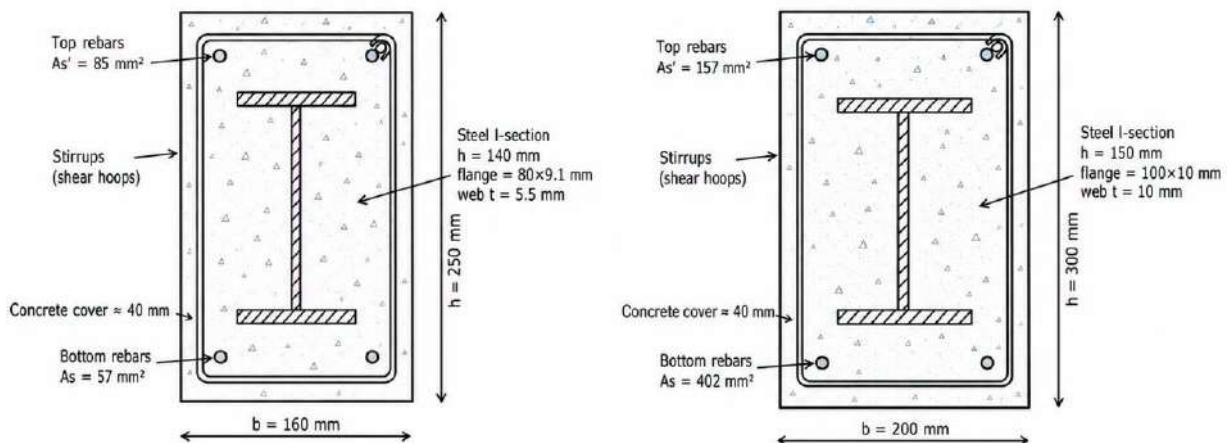


Fig. 1. Cross-section details and three-point bending configuration of Beams C1 and C2

By comparison, C2 from Mahmood et al. section with dimensions of 200 mm × 300 mm and a steel section 150 mm high, 10 mm thick web and 100 mm broad flanges with 10 mm thick flanges, reinforced with 40 mm of concrete and a much larger area for reinforcement than for its counterpart. In addition to the larger area (157 mm² and 402 mm² for tension and compression, respectively), the span length is 2750 mm, and the shear span is 150 mm. The compressive strength of concrete was lower (25.75 MPa), and the tensile strength (2.78 MPa) and elasticity (23.98 GPa) of concrete were lower than in Beam C1. The strain at cracking was significantly greater (0.004) than in C1. Beam C2 has a yield strength of 520.7 MPa of reinforcing steel (in the original experimental source of 600 MPa ultimate strength) as being unique for C2. The constituent materials for C2 are not characterized as similar to those in C1 (the yield and ultimate strength for C1 are still 315 and 410, respectively).

To clarify, Beam C1 was simply Supported three-point bending conditions with a concentrated load applied at midspan and a clear support span of 2000 mm. The same was true for Beam C2 but the point load was at the midpoint and had 2750mm between the supports. The attached schematic includes the cross-section and loading configuration for both Beam C1 and C2 together with details of concrete dimensions, concrete covering, embedded steel I-section, stirrups, and reinforcement position. The parameters denoted by an asterisk (*) were either unreported in the original experimental source or were simply extrapolated from design code recommendations or literature on comparable-sized steel-concrete composite structural members. Only those parameter values that were required for analytical purposes were utilized; no estimates (unmarked parameters) were included in the analytical calculation.

3. Markov Chain Modeling and Code Integration (ACI & AISC)

When reinforced concrete beams designed to be nominally identical are subjected to monotonically increasing loads, their load–deflection responses exhibit noticeable variability between specimens. This variation, illustrated in Figure 2, reflects the stochastic nature of the deflection behavior, which is referred to as the “deflection process” in this study. Incremental deflections between subsequent load levels are controlled by the normal distribution assumption. The normal distribution assumption provides a reasonable probabilistic representation of incremental deflections, where transition probabilities for each load step are estimated using the deflection mean and standard deviation. While the normal distribution assumption simplifies the evaluation, it does not accurately represent highly nonlinear response phases where deflections might be skewed. Therefore, any interpretation of the results will be dependent upon the calibrated limits of each tested beam. The deflection process is represented as $(y_{\max L}, L \text{ belongs to } Z)$. For each load L belongs to Z , the maximum deflection, $y_{\max L}$. At each load level L , the corresponding maximum mid-span deflection $y_{\max L}$ is treated as a random variable. The index set Z defines the sequence of load applications, while the deflection state space consists of all possible values that $y_{\max L}$ may assume [9,11]. The state space for modeling can be divided by defining a finite range for each resulting deflection state, since the ability of the states to be defined on a continuum or on a discrete set of values is a consequence of the process index. The deflection is therefore a discrete stochastic process and the beam itself is the system, where, at any load level, the state of the system is defined by the maximum deflection [10].

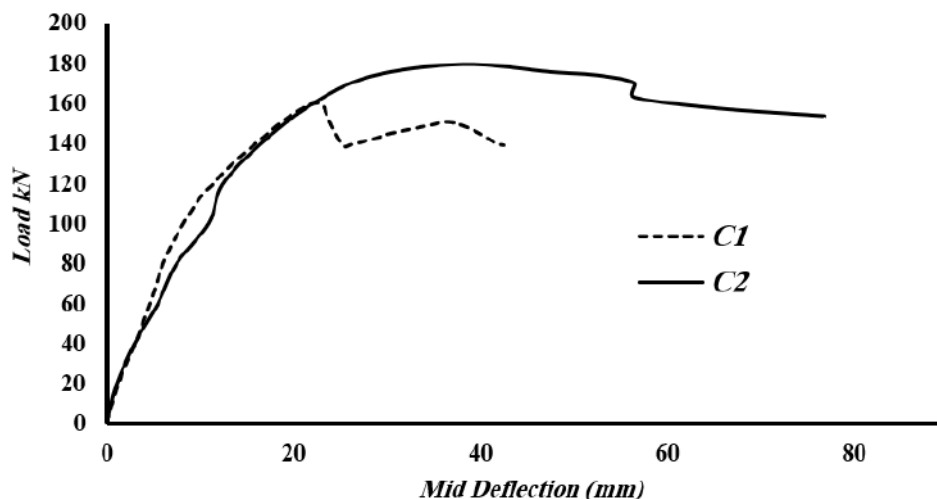


Fig. 2. Load-deflection response of Beams C1 and C2 used to illustrate variability in the composite beam response

Under the current level of applied load, when $y_{\max L} = i$, the system is in state i . In the presence of increasing load, the system switches between discrete states, and the likelihood of these switches is determined by the transition probabilities, giving rise to probabilistic modeling frameworks [16]. In general, in a stochastic process, the probability of the system visiting a particular future state may not only depend on the current state but also on the whole sequence of earlier states (full

history), and this is represented by the conditional probability of future states given the whole load-deflection history of the system

$$P[y_{max}(L_a) = j: y_{max}(L_{a-1}) = i, \dots, y_{max}(L_o) = 1] \quad (1)$$

Table 2. System state classification

State of the system	Implication	Mathematical representation
1	The maximum deflection is between y_1 and y_2	$y_1 < y_{max} \leq y_2$
2	The maximum deflection is between y_2 and y_3	$y_2 < y_{max} \leq y_3$
i	The maximum deflection is between y_i and y_{i+1}	$y_i < y_{max} \leq y_{i+1}$
n	The maximum deflection is between y_i and y_{i+1}	$y_n < y_{max}$

Let $Z (L^0, L^1, \dots, L_a^{-1}, L_a, \dots, L_m)$ represent the index set of the applied loads, and $(1, 2, \dots, i, j, \dots, n)$ represent the deflection statespace, which is the possible values of $[y_{max}(L), L \in Z]$ for each load L_a . The process of deflection goes through the intermediate state rules, which constitute the core modelling assumptions. The deflection process is modeled as a non-homogeneous Markov chain because the future value of the deflection only depends on the present value of the deflection. The non-homogeneity arises because the deflection transitions are associated with the different levels of load so that the discrete values index the process of the deflection $(L^1, L^0, \dots, L_N)$ of the levels of load, and a finite number of deflection states is assumed. The deflection is said to have N distinct values if it is categorized into N states, as shown in Table 2. Hence, the conditional transition probability of the deflection process is formulated appropriately as shown in equation (1).

$$PP_{ij}(L_{a-1}, L_a) = [y_{max}(L_a) = j: y_{max}[(L)_{a-1}] = i]; L_{a-1} < L_a \quad (2)$$

The probability of the system transitioning from state i to state j under the load increment under the load of $L_a - 1$ to L_a . Let $P_{ij}(L_{a-1}, L_a)$ denote this transition probability. This probability captures the system's response as the applied load increases. The corresponding transition probability matrix, denoted by $[II(L_{a-1}, L_a)]$, defines the complete set of transition probabilities among n discrete states during the load increment or decrement from L_{a-1} to L_a . As the load progresses, the loads are monotonically increasing. Thus, no transitions may occur between any of the lower deflection states. Consequently, the matrix will not include any entries below the diagonal ($p_{ij} = 0$) for rows above their column (i.e., for all $j < i$).

$$[II(L_{a-1}, L_a)] = \begin{matrix} & \begin{matrix} 0 & 1 & 2 & 3 & 4 & 5 & . & n \end{matrix} \\ \begin{matrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ . \\ n \end{matrix} & \begin{bmatrix} p_{00} & p_{01} & p_{02} & p_{03} & p_{04} & p_{05} & . & . \\ p_{10} & p_{11} & p_{12} & p_{13} & p_{14} & p_{15} & . & . \\ p_{20} & p_{21} & p_{22} & p_{23} & p_{24} & p_{25} & . & . \\ p_{30} & p_{31} & p_{32} & p_{33} & p_{34} & p_{35} & . & . \\ p_{40} & p_{41} & p_{42} & p_{43} & p_{44} & p_{45} & . & . \\ p_{50} & p_{51} & p_{52} & p_{53} & p_{54} & p_{55} & . & . \\ . & . & . & . & . & . & . & . \\ n & . & . & . & . & . & . & . \end{bmatrix} \end{matrix} \quad (3)$$

To make it easier to visually understand the suggested non-homogeneous Markov model, a state-transition diagram has been created (i.e., Figure 3). The seven deflection states are the nodes, while the arrows represent the feasible transition probabilities (p_{ij}) between the states. Since the primary loads applied to the beams are monotonically increasing, the transition from one downward state to another is irretrievable in the downward direction; thus, the only possible way for the system to remain in the same state or to go upward in deflection (but not downward). The final state is defined as the terminal high-deflection state.

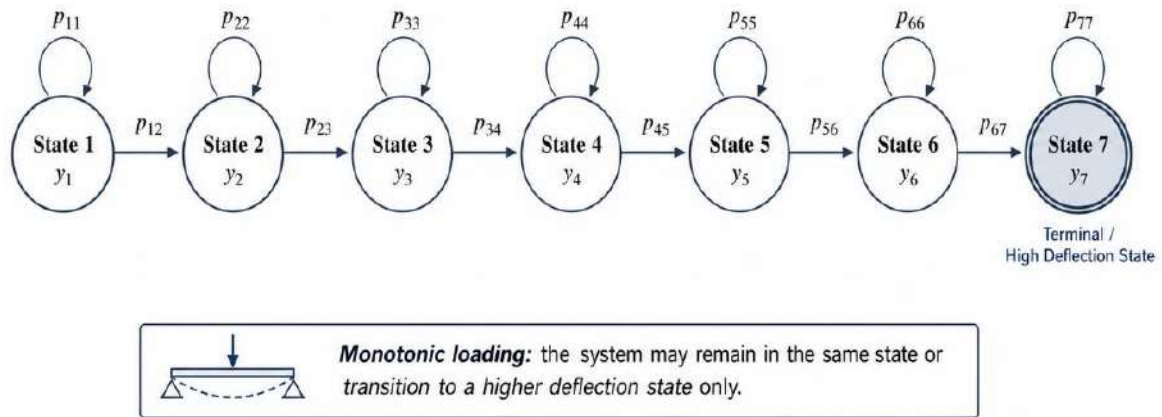


Fig. 3. Markov state-transition diagram for the seven deflection states under monotonic loading

The diagram includes State 1 through State 7, self-transition arrows p_{ii} , and forward transition arrows p_{ij} for $j \geq i$. No backward arrows are shown because $p_{ij} = 0$ when $j < i$. Given that the deflection states defined in Table 2 are mutually exclusive and collectively exhaustive, the sum of each row in the transition probability matrix must equal one. As the load increases from (p_{00}) , from L_{a-1} to L_a , the system's state, defined by the maximum deflection, can either remain unchanged or progress to a higher deflection level. A reversal to a lower deflection state is not possible. Consequently, the transition matrix reflects this non-decreasing behavior. To compute the transition probability $P_{ij}(L_{a-1}, L_a)$, it is necessary to consider both the marginal probability distribution of the maximum deflection at L_{a-1} and L_a and the joint probability distribution of deflections at L_{a-1} and L_a . Let M_{a-1} and M_a represent the maximum deflection at the respective load levels. Given the joint and individual probability density functions of these variables, the transition probability can be derived accordingly.

$$p_{12}(L_{a-1}, L_a) = \frac{\int_{y_1}^{y_2} \int_{y_2}^{y_3} f_{M_{a-1}, M_a}(M_{a-1}, M_a) \cdot dy_{a-1} \cdot dy_a}{\int_{y_1}^{y_2} f_{M_{a-1}}(M_{a-1}) \cdot dy_{a-1}} \quad (4)$$

Let $f_{M_{a-1}, M_a}(M_{a-1}, M_a)$ denote the joint probability density function of the random variables M_{a-1} and M_a , representing the maximum deflection at two successive load levels. Additionally, the marginal density function of M_{a-1} is denoted as $f_{M_{a-1}}(M_{a-1})$. It is unrealistic to represent the behaviour of maximum deflection using an extreme-value distribution for M_{a-1} and M_a , as previously described. It also will cause many difficult calculations. Calculating joint distributions, estimating parameters can be computationally challenging, especially when large experimental datasets are unavailable. For simplicity and computational feasibility, the individual and joint deflection variables were represented by normal and bivariate normal distributions, respectively, so that transition probabilities between consecutive deflection states could be determined from the mean deflection, standard deviation, and correlation coefficient as the loads were applied. While the present assumption is appropriate given the limitations of the current model as well as the smaller data set used to calibrate it, it does still constitute a form of modeling simplification. Since all deflection values are positive and may exhibit right-tailed skewness during advanced stiffness degradation, lognormal or other right-skewed distributions should be considered for future studies of future modeling studies with larger data sets. Thus the results from this model only relate to those tested beams which have been calibrated:

$$f_{M_{a-1}}(M_{a-1}) = \frac{1}{\sqrt{2\pi} \cdot \sigma_{a-1}} \cdot e^{-\left(\frac{1}{2}\right) \cdot \left[\frac{M_{a-1} - G_{a-1}}{\sigma_{a-1}}\right]^2} \quad (5)$$

$$f_{M_a}(M_a) = \frac{1}{\sqrt{2\pi} \cdot \sigma_a} \cdot e^{-\left(\frac{1}{2}\right) \cdot \left[\frac{M_a - G_a}{\sigma_a}\right]^2} \quad (6)$$

$$f_{M_{a-1}, M_a}(M_{a-1}, M_a) = \frac{1}{2\pi \cdot \sigma_{a-1} \cdot \sigma_a \cdot \sqrt{1 - \rho^2}} \cdot e^{-\left(\frac{1}{2(1-\rho^2)}\right) \left\{ \left[\frac{M_{a-1} - G_{a-1}}{\sigma_{a-1}} \right]^2 - 2\rho \left[\frac{M_{a-1} - G_{a-1}}{\sigma_{a-1}} \right] \left[\frac{M_a - G_a}{\sigma_a} \right] + \left[\frac{M_a - G_a}{\sigma_a} \right]^2 \right\}} \quad (7)$$

It can be assumed that the average max displacement measured at two sequential load levels, La-1 and La, denotes the respective mean values Ga and Ga-1. The measurements σ_{a-1} and σ_a will represent the respective standard deviations of the max displacement measured at La-1 and La. The correlation coefficient, ρ , denotes a statistical relationship established between the two maximum displacements measured at load levels La-1 and La.

In order to estimate the average deflection at a given loading level, the methodology combines the provisions of ACI 318 [17] and AISC 2016 [18]. Specifically, elastic analysis is applied to both uncracked and cracked transformed sections, as outlined in ACI 318-17. In contrast, the internal moment capacity of the composite section is determined using the plastic stress distribution concept described in AISC 2017. This combined approach enables accurately calculating mid-span deflection by accounting for both elastic and plastic behavior in steel-concrete composite sections.

$$y_{max} = \frac{p \cdot (\text{span})^3}{48 \cdot E_c \cdot I_g} \quad \text{for the uncracked section} \quad (8)$$

$$y_{max} = \frac{p \cdot (\text{span})^3}{48 \cdot E_c \cdot \left\{ \left[\left(\frac{M_{cr}}{M_a} \right)^3 \cdot I_g \right] + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] \cdot I_{cr} \right\}} \quad \text{for the cracked section} \quad (9)$$

Where M_a , the moment at any applied load La, and M_{cr} is the moment at cracked load, I_g is the moments of inertia of the gross (Ig) and cracked (Icr) sections, respectively. The variable in the equation is the moment value at each point, and it will be in two forms, which are (M_{a-1}, M_a) and these equations yield two deflection values, which are $(y_{\max(a-1)}, y_{\max(a)})$ used for training in addition to the values resulting from practical experiments.

The validation of the proposed Markov Framework is based on the defined training domain boundaries and state discretization employed. The Transition Probabilities considered were established from both the (i) Experimental Load-Deflection points obtained from testing, and (ii) Theoretical Response Points from the ACI318 and AISC provisions previously described in Section 3. Therefore, the application of the Markov Model to beams requires an alignment with the calibrated parameters of the training domain with regard to the section configuration (encased composite beams with embedded Steel I-sections), material classifications, reinforcement layouts, span-to-depth ratio, and Monotonic Load Protocol. Generalizing the model to new datasets outside of these parameters requires recalibration of the model. The discretisation of the deflections into a defined number of states will directly affect the smoothness and uncertainty of predictions; for example, if the state grid is coarse, the resolution for detecting changes in stiffness will be lower than that of a refined state grid. Conversely, if the state grid is refined, then the number of states will be high, resulting in sparse transition counts or unstable estimates. Therefore, state definitions were determined to provide an adequate number of samples per state to detect the primary curvature changes from the measured response. A sensitivity assessment of state size (number of deflection bins) is recommended when applying the model to new datasets, and the discretisation should be considered a fixed modelling parameter for each calibration set to which it is applied and should be reported as such.

4. Beam C1

To demonstrate the application of the proposed model, the analysis steps for Beam C1 are presented in detail, while Beam C2 is similarly evaluated, with only the final results reported. The specifications of Beam C1 have been previously outlined, and its corresponding load-deflection response is illustrated in Figure 4.

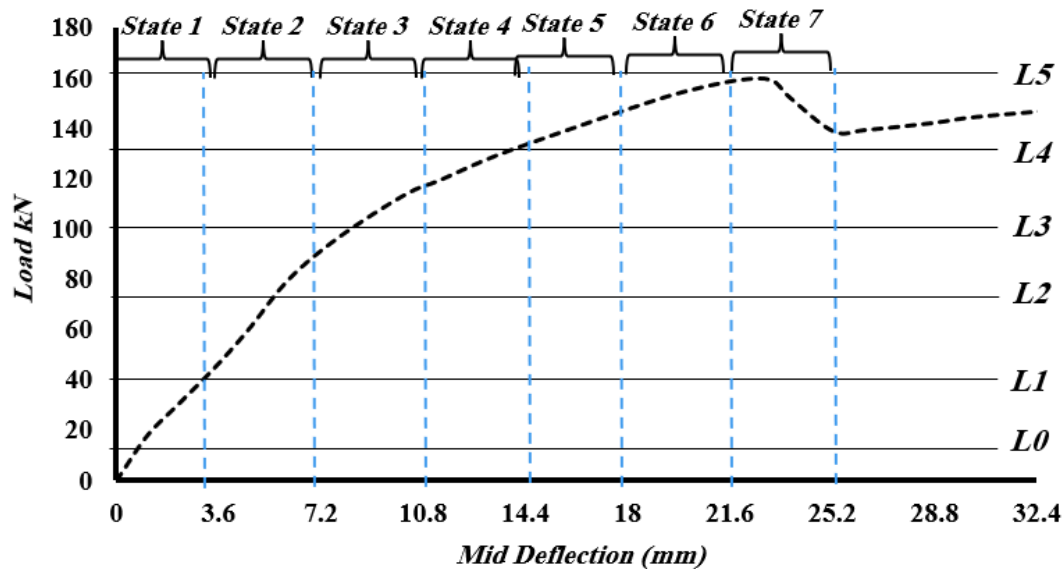


Fig. 4. Load deflection plot for beam C1

4.1 Step 1

In order to develop the transition probability matrix for the load–deflection behavior, it is necessary first to establish the index set, which represents discrete load levels, and the state space, which defines the corresponding deflection intervals. The index set captures the applied loads being stepped through, and the state space discretises the structural response into distinct deflection ranges [19]. These two combined form the basis for modeling a beam's behavior as a stochastic process with the Markov chain. For this application, the index set is broken up into discrete load levels 10, 40, 70, 100, 130, and 160 kN. The load points can be arbitrary as long as they fall within the range of interest and are suitable to capture the deflection progression during a load increase.

For the presented example, there are no specific rules guiding the choice of load points, and they do not have to meet any specific restrictions beyond spanning the range of interest, and should each correspond to a different measurable deflection state, except when multiple load levels induce the same deflection state.

- State 1: $0 < y_{\max} < 3.6$ mm
- State 2: $3.6 < y_{\max} < 7.2$ mm
- State 3: $7.2 < y_{\max} \leq 10.8$ mm
- State 4: $10.8 < y_{\max} \leq 14.4$ mm
- State 5: $14.4 < y_{\max} \leq 18.0$ mm
- State 6: $18.0 < y_{\max} \leq 21.6$ mm
- State 7: $y_{\max} > 21.6$ mm

Table 3. The means and standard deviations of deflection at different loads of beam C1 (experimental and analysis data), assuming $\Omega = 0.25$

Load (kN)	Mean (mm)	Standard deviation (mm)
10	7.090	1.770
40	9.785	2.445
70	12.475	3.120
100	15.170	3.795
130	17.865	4.465
160	20.555	5.140

A framework that is achieved through the use of seven designated deflection states, which are mutually exclusive. In addition, the seven deflection states are constructed to be collectively exhaustive so that every possible structural response can be captured in one state, and so that each

deflection value uniquely relates to one, and only one, possible outcome. Consequently, the load-deflection relationship can be effectively represented as a nonhomogeneous Markov chain, consisting of seven distinct deflection states and six discrete load levels: 10, 40, 70, 100, 130, and 160 kN. These six load levels produce five successive load intervals used to construct the transition probability matrices [20].

4.2 Step 2

To compute the transition probability matrices across successive load intervals, it is assumed that the deflection at any given load level L_k is a normally distributed random variable. Furthermore, the joint probability density of deflections at two consecutive load levels, L_{k-1} and L_k , is modeled as a bivariate Gaussian distribution with an assumed correlation coefficient of 0.95 [11].

Equations 5–7 can be utilized to calculate the transition probabilities defined in Equation 3, provided that the relevant distribution parameters are known. While the mean deflections at load levels L_{k-1} and L_k are computed using Equations 8 and 9, the corresponding standard deviations were calculated using the adopted coefficient of variation (CV), as follows:

$$\sigma_{k-1} = CV \times G_{k-1} \quad (10)$$

$$\sigma_k = CV \times G_k \quad (11)$$

The values of G_{k-1} and G_k are the mean of the deflections representing two load levels, while σ_{k-1} and σ_k represent the standard deviation of the respective mean deflection for the same two load levels. For this study, a coefficient of variation of (CV) of 0.25 was adopted based on the statistical analysis of available deflection data associated with Beams C1 and C2.

$$[II(L_0, L_1)] = \begin{bmatrix} 0.325 & 0.657 & 0.018 & 0 & 0 & 0 & 0 \\ 0 & 0.19 & 0.739 & 0.071 & 0 & 0 & 0 \\ 0 & 0 & 0.147 & 0.6876 & 0.165 & 0.0004 & 0 \\ 0 & 0 & 0 & 0.085 & 0.6799 & 0.234 & 0.0011 \\ 0 & 0 & 0 & 0 & 0.0199 & 0.6891 & 0.291 \\ 0 & 0 & 0 & 0 & 0 & 0.0013 & 0.998 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (12)$$

$$II(L_1, L_2) = \begin{bmatrix} 0.343 & 0.617 & 0.029 & 0.0111 & 0 & 0 & 0 \\ 0 & 0.256 & 0.673 & 0.066 & 0.005 & 0 & 0 \\ 0 & 0 & 0.221 & 0.647 & 0.131 & 0.0011 & 0 \\ 0 & 0 & 0 & 0.159 & 0.652 & 0.188 & 0.001 \\ 0 & 0 & 0 & 0 & 0.108 & 0.642 & 0.249 \\ 0 & 0 & 0 & 0 & 0 & 0.071 & 0.929 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1.00 \end{bmatrix} \quad (13)$$

$$II(L_2, L_3) = \begin{bmatrix} 0.379 & 0.548 & 0.049 & 0.024 & 0 & 0 & 0 \\ 0 & 0.318 & 0.586 & 0.079 & 0.017 & 0 & 0 \\ 0 & 0 & 0.277 & 0.61 & 0.102 & 0.011 & 0 \\ 0 & 0 & 0 & 0.2197 & 0.6097 & 0.1622 & 0.0084 \\ 0 & 0 & 0 & 0 & 0.171 & 0.612 & 0.217 \\ 0 & 0 & 0 & 0 & 0 & 0.141 & 0.859 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (14)$$

Simulations, which were based on the coefficient of variation, were used in lieu of a limited number of the available experimental deflection points. Statistical analysis performed on the available deflection data for Beams C1 and C2 produced coefficients of variation that ranged from approximately 0.16 to 0.25. The conservative coefficient of variation selected to estimate the maximum allowable is proposed to be used as a representative sample value (the upper limit). This selection was made due to the relatively high level of uncertainty that was associated with the limited amount of experimental data available, the presence of nonlinear reductions in stiffness, the continuing presence of cracks, and the variability of response from one specimen to another. Therefore, the $\Omega = 0.25$ coefficient of variation provides a conservative distribution for determining the transition probability matrices. The mean value as well as the standard deviation of the parameters for various load stages are exhibited in Table 3. Based on these values, transition probability matrices were created with regard to Equation 3 as well as Equations 5 to 7. The required integrals for Equation 3 were calculated using Gaussian quadrature, implemented in standard Python code from the NAG numerical library.

4.3 Step 3

By having the transition probability matrices for each load, we are now able to calculate multi-step transition probabilities. For example, the two-step transition matrix from L_0 to L_2 is obtained by multiplying the two consecutive one-step transition matrices:

$$[II(L_0, L_2)] = [II(L_0, L_1)] \times [II(L_1, L_2)] \quad (15)$$

This notation indicates the probability of moving between deflection states from load level L_0 to load level L_2 through the intermediate load level L_1 . These calculations involve matrix multiplication of the transition probability matrices. The required transition probabilities were obtained using Gaussian quadrature, and the simulation program was implemented in standard Python, and the resulting matrix based on Eqs. 12 and 13 is:

$$[II(L_0, L_2)] = \begin{bmatrix} 0.111 & 0.369 & 0.456 & 0.059 & 0.006 & 0 & 0 \\ 0 & 0.049 & 0.291 & 0.502 & 0.144 & 0.014 & 0 \\ 0 & 0 & 0.032 & 0.204 & 0.486 & 0.235 & 0.042 \\ 0 & 0 & 0 & 0.014 & 0.129 & 0.469 & 0.388 \\ 0 & 0 & 0 & 0 & 0.002 & 0.062 & 0.936 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (16)$$

Similarly, three-step, four-step, and five-step transition probability matrices are obtained.

4.4 Step 4

Since the calculated probabilities are conditional, determining the unconditional probabilities of the beam transitioning from the load level L_0 to L_2 requires first establishing the initial state distribution. In the initial state vector, the probabilities of the seven possible deflection states must sum to 1, not the probabilities of the load levels [21]. These initial probabilities act as the starting position for propagating probabilities across multiple load steps using the transition matrices. Figure 2 plots the experimental deflection data at the various loads from L_0 to L_5 ; deflections are clearly seen at each load. Deflection at intermediate loads is estimated through linear interpolation; it is now possible to observe the system at each load on the graph. The observed data show that the beam is in state 1 at L_0 . The initial state probability vector is then:

$$[II(0)] = (1, 0, 0, 0, 0, 0, 0) \quad (17)$$

Table 4 demonstrates the unconditional probabilities based on the input load levels and was computed with equations (17) and the initial system probability vector, which was given in Equation $[II(0)]$, shown in the last section. Additionally, to provide for a comparison, Table 4 also

contains the corresponding deflection values from Figure 2. This analysis provides a probabilistic framework for evaluating of the structural response of Beam C1 as the load level is increased, based upon a correlation coefficient of $p = 0.95$.

Table 4. Unconditional probabilities of different states of beam C1 for $p = 0.95$

Increase in Load		Load step (m)	Vector	Unconditional Probabilities of Different States							Range of deflection having most significant probability of occurrence (mm)	Experimental deflection at load Lk (mm)
From Lk-1 (kN)	To Lk (kN)			1	2	3	4	5	6	7		
10	40	1	(IIU [L0,L1])	0.325	0.657	0.018	0	0	0	0	3.6-7.2	3.05
10	70	2	(IIU [L0,L2])	0.111	0.369	0.456	0.059	0.006	0	0	7.2-10.8	5.05
10	100	3	(IIU [L0,L3])	0.013	0.065	0.189	0.291	0.273	0.128	0.041	10.8-14.4	8.02
10	130	4	(IIU [L0,L4])	0.001	0.003	0.014	0.037	0.076	0.127	0.741	14.4-18	14.6
10	160	5	(IIU [L0,L5])	0	0	0	0.001	0.002	0.005	0.992	18-21.6	22.6

- The sub-entry $P_{11(L_0, L_1)}$ represents, and it means the possibility that the system will stay at state 1 when the load changes from 10kN to 40kN. This value, 0.325, indicates a 32.5% likelihood that deflection remains within the 0–3.6 mm range at 40 kN, given it was within the same range at 10 kN. Other elements in the transition matrices $[II(L_0, L_1)]$, through $[II(L_1, L_2)]$, ... , $[II(L_4, L_5)]$ follow similar interpretations. Notably, state 7 serves as the absorbing state in all matrices. An examination of Equations 10–12 reveals a consistent decline in the diagonal and first super-diagonal probabilities as the load increases from (L_0, L_1) to (L_1, L_2) and (L_2, L_3) . This is the trend in deflection magnitude in the deflection between any subsequent load steps not linked with a change in direction of applied load.
- The first element of the two-step transition probability matrix $II(L_0, L_2)$, specifically $p_{11(L_0, L_2)}$, is calculated to be 0.111. This indicates that, starting in state 1 at a load of 10 kN, there is an 11.1% probability that the system remains in state 1 when the load reaches 70 kN. In other words, the likelihood of deflection staying within the 0–3.6 mm range under this load increase is relatively low. Multi-step transition probability matrices such as this are effective methods for estimating deflection at elevated load levels based on deflection observations at earlier stages.
- Assuming the system initially occupies state 1, the unconditional probabilities corresponding to each state at successive load levels are computed as part of Step 4. The resulting probability vector $[II(L_0, L_2)]$ reflects the likelihood of the system being in each possible state at load L_2 , given it began in state 1 at L_0 , as described by Equation 15.
- As shown in Table 4, the model estimates that at 40 kN, the beam will be in state 1 with 32.5% probability, state 2 with 65.7%, and state 3 with 1.8%, based on the initial state vector from Equation 15. The table also highlights, for each load increment, the most probable state. At 40 kN (after the first load step), the most likely deflection range is 0–3.6 mm, which aligns well with the observed deflection of 3.05 mm from Figure 2. This indicates good model accuracy at early stages. Similarly, the model's prediction remains consistent at 70 kN (second load step), though it forecasts a higher likelihood (29.1%) for state 3 (10.8–14.4 mm), while the actual deflection was only 3.02 mm. The states predicted at the higher levels of loading (130kN and 160kN) were slightly less than what was experimentally measured for the same load levels. Specifically, for 130kN, the 14.4-18.0mm range was predicted, while the measured deflection was measured to be 14.6mm which was at the lower bound of the same non-linear transition range. Then, at 160kN, the measured deflection was 22.6mm and well above its predicted value of 18.0-21.6mm. The difference indicates that at this load level the beam had begun to respond much more strongly in a non-linear way; i.e. cracking formation, stiffness degradation and re-distribution of internal forces were increasingly pronounced. The differences are also indicative of the fact that, using a Markov-chain methodology, there

is a non-homogeneous nature in the transitions as a result of large load levels, and therefore the probability of moving from one deflection state to another changes as a result of the load level – therefore the difference at the higher load levels represents an inadequacy of using fixed intervals between deflection states and/or assuming a normal transition during advanced non-linear behaviour.

5. Beam C2

In addition to Beam C1, Beam C2 was also analysed following the methodology established for Beam C1, and only the final results for Beam C2 are presented herein. Beam C2 specifications are, as already defined, and the load-deflection response for Beam C2 is illustrated in Figure 5.

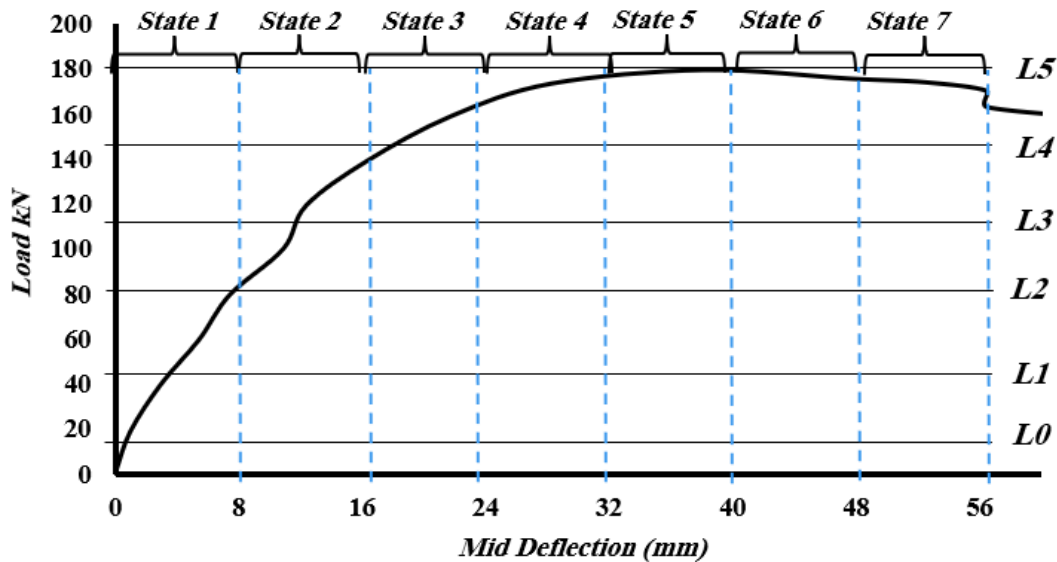


Fig. 5. Load- deflection plot for beam C2

Table 5. Unconditional Probabilities of Different States of Beam C2 for $\rho = 0.95$.

Increase in Load		Load step (m)	Vector	Unconditional Probabilities of Different States							Range of deflection having the most significant probability of occurrence (mm)	Experimental deflection at load Lk (mm)
From Lk-1 (kN)	To Lk (kN)											
				1	2	3	4	5	6	7		
13	44	1	(IIU (L0,L1])	0.098	0.368	0.513	0.018	0.002	0.000	0.000	0-8	4.53
13	81	2	(IIU (L0,L2])	0.031	0.152	0.389	0.345	0.078	0.005	0.000	16-24	8.03
13	110	3	(IIU (L0,L3])	0.010	0.056	0.199	0.302	0.305	0.108	0.021	24-32	12.4
13	145	4	(IIU (L0,L4])	0.003	0.019	0.085	0.170	0.277	0.255	0.191	32-40	18.3
13	180	5	(IIU (L0,L5])	0.098	0.003	0.011	0.018	0.368	0.513	0.000	40-48	38.6

Table 5 lists the results of the revised C2 deflection-state intervals for compatibility with the current loading progressions, and will display continuous state definitions starting from the first state defined as 0-8 mm of deflection. The results display good correspondence between the overall trend of deflection (increase) as a load increases, but do not correspond consistently with the amount of deflection shown at each load. At 44 kN, the measured deflection (4.53 mm) falls into state 1; consequently, the higher predicted probability of being in the upper state compared to the coefficient of variation used in the calculation, indicates that the increased variance of the probability distribution associated with the normal transition rule would yield a higher state assignment when applied to the first loading period (44 kN). At 180 kN, the measured deflection (38.6 mm) lies in the 32-40 mm interval and in contrast, the most probable interval given the

Markov process would provide a predicted maximum deflection at 40-48 mm. Additionally, using only a fixed number of deflections for determining states results in an overestimator for the predicted maximum deflection and emphasizes the influence of the non-linear change in stiffness and lack of resolution associated with each state suggesting that the C2 results primarily follow the general trend rather than providing exact state-by-state predictions rather than specifying an exact state-by-state prediction[21]. This can help in predicting the behavior of reinforced concrete beams under monotonic increasing loads through a non-homogeneous Markov model, where the consecutive deflection variables $Ma-1$ and Ma are assumed to be bivariate normally distributed with a correlation coefficient $\rho = 0.95$.

Many current codes provide recommendations on detailing practices to limit deflection, and some also provide procedures to estimate deflection under service loads. The probabilistic framework that is presented herein is an alternative to these codes. It is a statistically based methodology that allows the determination of deflection, taking into account a validated elastic deflection calculation method (with some accuracy). This model allows the user to estimate the deflection likely to be reached at higher load levels, given the response at a lower load stage. This is particularly useful when trying to assess beam behavior under potential overloading scenarios, especially when deflections approach or exceed serviceability limit states.

6. Predictive Performance

In order to objectively assess the predictive performance of the proposed non-linear Markov model (NLMM), load-deflection predictions from the model were compared with experimental responses from Beams C1 and C2, which were used as the benchmark datasets. Model accuracy was quantified at matched load levels using six commonly accepted statistical indicators: root mean square error (RMSE), normalized root mean square error (NRMSE), mean absolute percentage error (MAPE), accuracy acceptance (AA), coefficient of determination (R^2), and Pearson's correlation coefficient (R). The purpose of these indicators is to provide complementary information related to the model's predictive accuracy: RMSE emphasizes large deviations, NRMSE allows for a scale-independent comparison of the two beams, MAPE and AA provide relative error reported as a percentage, and R^2 and R describe the strength of linear correlation between the predicted and measured responses. Together, these indicators provide an objective and reproducible quantitative assessment of the predictive performance of the proposed Markov model. Equations (18)-(22) were used to compute the values of the six statistical indicators evaluated in this study. This study used the following definitions: N =number of pairs of Measured-Predicted deflections, A_n =measured deflection at Load Level n , P_n = predicted value as obtained from the Newly Proposed Markov Model at Load Level n , and $\bar{A}$ =Average of the Measured Deflections. Thus, all Statistical Indicators utilized paired Measured and Predicted Deflections on Beams C1 & C2. The statistical indicators, including RMSE, NRMSE, R^2 , MAPE, and AA, were calculated as follows:

$$RMSE = \sqrt{\frac{1}{N} \sum_{n=1}^N (A_n - P_n)^2} \quad (18)$$

$$NRMSE = \frac{RMSE}{S} \quad (19)$$

$$R^2 = 1 - \frac{\sum (A_n - P_n)^2}{\sum (A_n - \bar{A})^2} \quad (20)$$

$$MAPE = \frac{\left(\sum \frac{|A_n - P_n|}{A_n} \right) * 100}{N} \quad (21)$$

$$AA \% = 100 \% - MAPE \quad (22)$$

Where S is the range of the measured deflection values and is calculated as $S = A_{max} - A_{min}$. Here, A_{max} and A_{min} are the maximum and minimum measured deflections, respectively, $\bar{A}$ is the mean of the measured deflection values. Pearson's R was computed using the standard correlation formulation between A_n and P_n . Notation was kept consistent across beams, and all metrics were computed from the same paired datasets to ensure comparability.

Table 6. Statistical performance evaluation of the experimental results and the Markov model

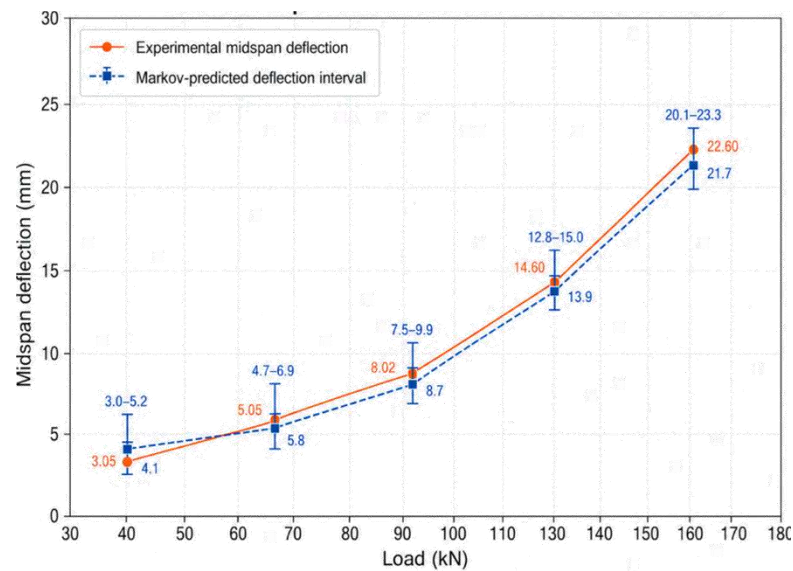
Beam	RMSE (mm)	NRMSE	MAPE (%)	AA% (%)	R ²	Pearson R
Beam C1	3.0252	0.0236	2.36	97.62	0.9920	0.9931
Beam C2	6.6379	0.0483	6.91	93.03	0.9783	0.9906

Table 6 lists the values of the indicators for both beams. The absolute error for Beam C1 produced by the Markov model was low (RMSE = 3.03 mm; NRMSE = 0.0236), producing a very low MAPE of 2.36% (AA = 97.62%). The strong correlation and goodness-of-fit ($R^2=0.9920$, $R=0.9931$) between the two data sets indicate that the model replicated the entire non-linear evolution of the experimental response at a high degree of accuracy. In contrast, the absolute and relative errors for Beam C2 were higher than those produced by Beam C1 (RMSE = 6.64 mm; NRMSE = 0.0483; MAPE = 6.91%; AA = 93.03%); however, the correlation metrics remained at a relatively high level ($R^2=0.9783$, $R=0.9906$). This suggests that the model was still able to track the trend and monotonic progression of the response; however, the discrepancies in pointwise comparison of the two beams had become larger. The larger pointwise discrepancies for Beam C2 correspond to the increased degree of complexity in Beam C2's stiffness degradation and transitional behavior, where local cracking progression and interactions between materials may exist such that they produce greater curvature changes that may be difficult to capture accurately using a finite-state structure.

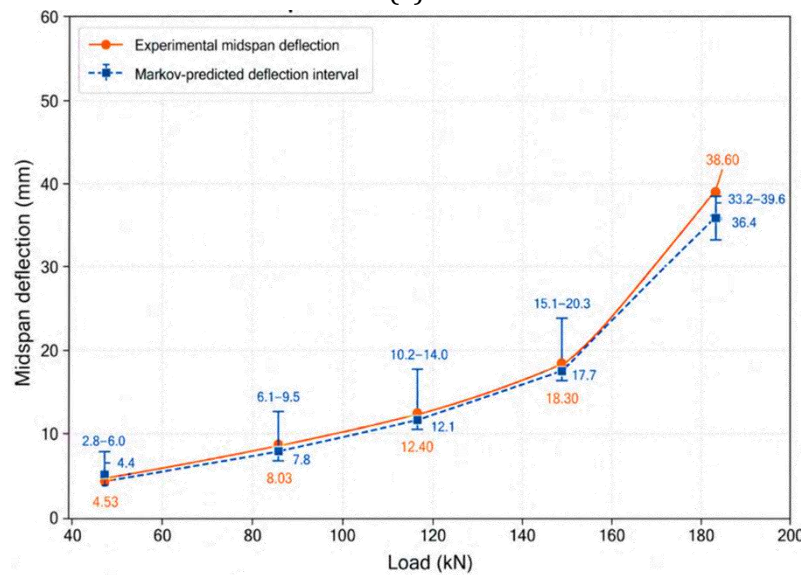
The quantitative interpretation of the predictions for both configurations also allows for additional understanding via a comparative analysis between the predicted results of C1 (3.03 mm RMSE) and C2 (6.64 mm RMSE), in addition to the qualitative (overlay of curves). The increase in the RMSE value from C1 to C2 (e.g., 3.03 kN to 6.64 kN) indicates that in C2, the larger predictions tended to occur more frequently and/or at a greater magnitude than in C1. The greater NRMSE and MAPE values in C2 also indicate that the lack of accuracy from C1 to C2 is not solely due to scaling but reflects an increase in the relative amount of prediction errors. The consistently high Pearson R values (> 0.99) between both tests show that the Markov predictive model preserved the appropriate monotonic ordering and the overall response shape; as such, the errors were primarily due to localized increases in the prediction error. Ultimately, the decrease of the R^2 value (0.9920 for C1 to 0.9783 for C2) suggests that although the measurement data still contained large localized amounts of nonlinearity, the proportion of variance accounted for still experienced a slight reduction. Collectively, these quantitative results indicate that the proposed predictive model provides an accurate and statistically demonstrable way to predict the monotonic load-deflection response and provides an objective basis for a performance comparison of the alternate beam systems.

To facilitate a better visual interpretation of the numerical outcomes listed in Tables 4 and 5, Figures were added to compare the Beams C1 and C2 experimentally with their respective Markov predictions. In Figure (6); the experimental deflection data points of Beam C1 are plotted against the Markov prediction of their most probable deflection interval for each of the specified loading levels (or steps). Figure (6) displays a similar comparison for Beam C2. For both beams, the experimental deflection data points are displayed alongside the probability band/error bar of the most likely State of Deflection according to Markov; therefore visually showing the deviations from the Markov prediction associated with each level of Loading deformation applied.

Overall, the graphical comparison for Beam C1 shows that experimental deflection tends to match the anticipated state transition with increasing deviation towards the final loading stage; however, for Beam C2 there is an obvious and significant difference between measured and predicted deflection states at 180kN, with measured deflection being measured as 38.6mm compared to predicted range of 40-48mm. The model indicates a slight overestimation of the final deflection state of Beam C2 based on fixed increment discrete state representation and stronger nonlinear stiffness reduction at higher levels of load.



(a)



(b)

Fig. 6. Graphical comparison between experimental deflection and Markov-predicted deflection interval for: (a)- Beam C1, (b)- Beam C2

For the quantitative evaluation of each Markov model's ability to predict deflection, multiple metrics were used: RMSE, NRMSE, MAPE, AA, R^2 and Pearson R. Each metric provides a basis for the predicted deflection states can be compared to the experimental deflections for Beams C1 and C2. As previously reported in Tables 4 and 5, predicted deflection intervals that corresponded to each experimental deflection value were identified, while Table 6 presented all statistical evaluations. Therefore. The parameters specified here are to be understood by reference to the assumptions used for this model: first, transition probabilities were obtained using a normal distribution assumption; second, fixed deflection-state intervals were defined; third, each of the experimental load and deflection measurements was taken at only a limited number of load-deflection points; and lastly, certain loads required interpolation for estimation. Therefore, the performance metrics obtained and communicated in this report reflect the ability to follow trends rather than providing absolute predictive accuracy over the entire range of loading conditions.

7. Conclusions

This research establishes and assesses a non-homogenous Markov chain method for predicting the load-deflection response of concrete-encased steel I-beams under monotonic three-point bending.

Structural response is defined as a series of discrete deflection states corresponding to increasing loads, rather than a single deterministic load-deflection curve. Through combining transition probabilities and response estimates based on the American Concrete Institute (ACI) and the American Institute of Steel Construction (AISC) calculations, a simplified probabilistic model can be used to track the degradation of stiffness and deflection of composite steel-concrete beam systems over time.

The study concluded that the Markov approach can follow approximate experimental values for the load-deflection performance of Beam C1 and Beam C2. Specifically, the RMSE of 3.03 mm, AA of 97.62%, and R^2 of 0.9920 indicated a high correlation with the experimental results for beam C1, while the RMSE of 6.64 mm, AA of 93.03%, and R^2 of 0.9783 showed some correlation with the experimental values for Beam C2. There was a significant overestimation of the deflection response of Beam C2 at the last level of applied load, which suggests that the accuracy of the model is dependent upon the selected deflection-state intervals and the amount of nonlinear degradation of stiffness due to high levels of applied load.

This research provides a new approach to modelling the load-deflection behaviour of concrete-encased steel I-beams by using a Markov chain based probabilistic model. Traditional deterministic models provide a single predicted curve, while the proposed Markov chain model estimates the probability of transition between deflection states as load increases. This provides more information about the uncertainty in a structure's behaviour and can aid in determining serviceability of composite beams. However, this study's model should only be applied within the calibrated range of specimens studied, and further validation should be performed prior to application to different geometry beams, loading protocols, or composite sections.

Future research is needed to confirm the proposed Markov model by utilising a larger sample of concrete-encased steel beam specimens of differing spans/reinforcements/materials/reinforcement spacing and concrete strengths. Other research efforts should explore how varying the size of the intervals/states, load/steps and correlation coefficient (ρ) will affect the stability of the transition probabilities. Additionally, expanding this model to other load protocols (e.g., repeated, cyclic and long-term) will provide further insight into the general utility of serviceability prediction analysis for composite beams.

References

- [1] Bai H, Xu J, Demartino C, Zhao X. Probabilistic code-based shear capacity model for I-shaped steel reinforced concrete (SRC) beams. *Engineering Structures*. 2024;301:117180. <https://doi.org/10.1016/j.engstruct.2023.117180>
- [2] Abdullatef A D, Ali S I, Aldarraji S Y H, Ali M I. Evaluating the Ramberg–Osgood Model for nonlinear moment–curvature analysis of reinforced concrete beams. *Res. Eng. Struct. Mater.*, 2026; 12(2): 1165-1177. <http://dx.doi.org/10.17515/resm2026-1518st0217rs>
- [3] Yu Y, Xie T, Xu J, Lai Z, Elchalakani M. Probabilistic models for characteristic bond stresses of steel-concrete in steel reinforced concrete structures. *Engineering Structures*. 2024;300:117167. <https://doi.org/10.1016/j.engstruct.2023.117167>
- [4] Ahmed AM, Ali SI, Ali MI, Jamel AAJ. Analyzing Self-Compacted Mortar Improved by Carbon Fiber Using Artificial Neural Networks. *Matrix*. 2023;9:11. <https://doi.org/10.18280/acsm.470602>
- [5] Alkayem NF, Shen L, Mayya A, Asteris PG, Fu R, Di Luzio G, et al. Prediction of concrete and FRC properties at high temperature using machine and deep learning: a review of recent advances and future perspectives. *Journal of Building Engineering*. 2024;83:108369. <https://doi.org/10.1016/j.jobbe.2023.108369>
- [6] Jabbar AA, Irzooki RH, Al-Obaidi AA. Improving resistance of gravity dam to seismic wave by using stepped base. In: *E3S Web of Conferences*. EDP Sciences; 2021. p. 01007. <https://doi.org/10.1051/e3sconf/202131801007>
- [7] Richard D, Hong T, Hastak M, Mirmiran A, Salem O. Life-cycle performance model for composites in construction. *Composites Part B: Engineering*. 2007;38(2):236-246. <https://doi.org/10.1016/j.compositesb.2006.04.001>
- [8] Ali MI, Allawi AA. An Artificial Neural Network Prediction Model of GFRP Residual Tensile Strength. *Engineering, Technology & Applied Science Research*. 2024;14(6):18277-18282. <https://doi.org/10.48084/etasr.9107>

- [9] Loebens N, de Andrade Porto JV, de Andrade Porto KR, Souza JS, Pistori H. Types of Markov random fields in deep learning: A review. *Neurocomputing*. 2025:132353. <https://doi.org/10.1016/j.neucom.2025.132353>
- [10] Ortner R. An identity for the bias in Markov reward processes with applications to Markov chain perturbation and Kemeny's constant. *Statistics & Probability Letters*. 2025:110592. <https://doi.org/10.1016/j.spl.2025.110592>
- [11] Feng M, Pi B, Deng LJ, Kurths J. An evolutionary game with the game transitions based on the Markov process. *IEEE Transactions on Systems, Man, and Cybernetics: Systems*. 2023;54(1):609-621. <https://doi.org/10.1109/TSMC.2023.3315963>
- [12] Mousavirad SJ, Ebrahimpour-Komleh H, Schaefer G. Convergence Analysis of the Human Mental Search Algorithm by a Markov Model. *Knowledge-Based Systems*. 2025:115073. <https://doi.org/10.1016/j.knosys.2025.115073>
- [13] Geiger BC. Information-theoretic reduction of Markov chains. *arXiv preprint arXiv:2204.13896*. 2022.
- [14] Xian G, Zhang Y, Wang Y. Experimental study on the flexural behavior of steel-concrete composite beams. *Journal of Constructional Steel Research*. 2012;77:59-67.
- [15] Mahmood EM, Ibrahim TH, Allawi AA, El-Zohairy A. Experimental and numerical behavior of encased pultruded GFRP beams under elevated and ambient temperatures. *Fire*. 2023;6(5):212. <https://doi.org/10.3390/fire6050212>
- [16] Watson SR. *Probability Concepts in Engineering Planning and Design Vol 1-Basic Principles*. AH-S. Ang and WH Tang. Wiley, New York. 1975;79(779):518-518. <https://doi.org/10.1017/S0001924000036162>
- [17] American Concrete Institute (ACI). *Building Code Requirements for Structural Concrete (ACI 318-17) and Commentary*. Farmington Hills, MI: ACI; 2017.
- [18] American Institute of Steel Construction (AISC). *Steel Construction Manual*. 15th ed. Chicago, IL: AISC; 2017.
- [19] Zhao P, Kang Y, Niu B, Song X. Stochastic stability of positive Markov jump sub-homogeneous and homogeneous non-linear systems. *Automatica*. 2026;184:112730. <https://doi.org/10.1016/j.automatica.2025.112730>
- [20] Basnarkov L. Semi-Markov random walk on complex networks. *Chaos, Solitons & Fractals*. 2026;202:117578. <https://doi.org/10.1016/j.chaos.2025.117578>
- [21] İzgi B, Özkaya M, Üre NK, Perc M. Machine learning tree trimming for faster Markov reward game solutions. *Journal of Computational Science*. 2025:102726. <https://doi.org/10.1016/j.jocs.2025.102726>

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Comparative investigation of dynamic resilience and vibration characteristics of one-way concrete slabs reinforced with steel bars, GFRP bars and steel wire ropes

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Abstract

The effect of reinforcement material type and bar diameter on the dynamic response of one-way slab panels is studied in this research work. The experimental program included testing twelve one-way, simply supported slab panels under a low-velocity impact through a 10 kg mass freely falling from a height of 1 m. Three systems of reinforcement were considered: deformed steel bars, GFRP bars, and steel wire ropes. Four specimens were cast for each type of reinforcement, each specimen was reinforced with one of four diameters (6, 8, 12, and 16 mm). Under dynamic excitation, the three-reinforcement series exhibited distinctly different vibration characteristics, with damping ratios, natural frequencies, and peak amplitudes varying in a manner directly related to the stiffness of each reinforcement type. The lowest midspan amplitude values and the highest natural frequency (18.5–16.67 Hz) were achieved by steel-reinforced panels; therefore, it is recommended to use deformed steel bars in reinforcing members when lower vibration amplitude is required. Panels reinforced by 12 mm diameter GFRP bars (G-12) recorded the highest damping ratio (12.5%), meaning that the vibration diminished faster than any other among the twelve specimens. This behavior affirms that reinforcement selection governs not only static load-carrying behavior but also the dynamic resilience of the structural element.

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1. Introduction

In some specific conditions, traditional steel bars could be replaced by fiber-reinforced polymer bars [1]. In concrete members exposed to high levels of chloride [2] or bridge slabs where the use of deicing salts promotes the rapid corrosion of steel reinforcement [3,4]. It is recommended to use composite reinforcing bars due to their resistance to aggressive environments. Extensive research has been conducted in the field of reinforcement alternatives and their use in concrete elements. Below are some of the research efforts focused on the behavior of reinforced concrete elements reinforced by steel and steel alternatives under dynamic loading conditions.

According to Liu and Zou, dynamic energy dissipation can be significantly enhanced without compromising stiffness when reinforcement and transverse detailing are properly chosen. For members, the damping ratio increases with GFRP ratio up to an optimum of about 0.9 %, then decreases as section rigidity dominates. Closely spaced stirrups further increase damping. Pull-out tests indicate that average bond stress improved with smaller bar diameter, shorter embedment, thicker concrete cover and ribbed rather than smooth surface [5]. Concrete slabs reinforced with steel and GFRP bars were tested using a 105 kg conical impactor dropped from 2.5 m and responses were recorded via LVDTs, strain gauges and accelerometers. GFRP reinforced slabs generally

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showed larger displacements and more cracking but could match or outperform steel reinforced slabs when a higher GFRP ratio was provided [6].

FRP primarily limits crack width and concrete damage by remaining elastic, whereas steel governs energy dissipation; consequently, the FRP-steel volume ratio controls both residual deformation and absorbed impact energy, with an optimal balanced configuration at approximately 50% FRP. Hybrid slabs exhibited more distributed, partially recoverable cracking than regular slabs. Although over-reinforcement is effective in limiting deflection, it is discouraged as it diminishes energy absorption and may increase damage demand on concrete [7]. In full-scale one-way concrete slabs reinforced either with GFRP bars or conventional steel, strains are highest along the short span and near the impact point, confirming that flexural response is governed by the primary spanning direction under local impact. GFRP-reinforced slabs exhibit lower maximum strains and significantly shorter strain-decay time with pulse suppression time lower than steel-reinforced slabs at the same impact height, indicating improved impact resistance and faster dynamic recovery despite similar concrete mixes [8].

UHPC/SFR-UHPC slabs with 8 mm CFRP tension bars achieved similar or slightly lower peak displacements than their steel reinforced counterparts but exhibit notably smaller crack widths, limited scabbing, and superior impact damage resistance since neither CFRP nor steel yields in high strength concretes. Normal-strength concrete control slab fails through severe localized punching-shear, diagonal cracking, bond deterioration, bar yielding and concrete perforation beneath the impactor. Using equivalent area of CFRP bars of smaller diameter 4.66 mm in normal strength concrete increases the slab maximum displacement with widened diagonal cracks on the tension zone. Slab impact resistance is governed primarily by concrete compressive and shear strength rather than reinforcement strength [9]. Stainless steel wire reinforcement markedly enhanced impact toughness and suppressed crack propagation through the formation of inter-anchored among bundled wires. Notably, however, the addition of stainless-steel wire attenuated the inherent strain-rate strengthening effect of a plain reactive powder concrete [10].

The three-dimensional micron-diameter wire network effectively suppressed crack initiation and propagation, with dynamic impact toughness and energy dissipation increased by 43.5% and 58.2%, respectively, relative to ordinary concrete at 1.5% volume wire content. Slab thickness is the most influential parameter governing maximum deflection, followed by scaled distance, reinforcement ratio and slab length. Increasing thickness from 40 to 90mm reduced deflection by 82% [11]. Stainless steel wire-reinforced UHPC door panels substantially outperformed normal concrete, HPC and SFR-UHPC, reducing spalling damage and fragment ejection under close-range explosions. At merely 62.5% of conventional panel thickness, stainless steel wire-reinforced UHPC offered 2.9 times greater projectile impact resistance at 600 m/s [12].

According to ACI code, detailing of steel reinforcement is important to control cracking. Many fine cracks are preferable to a few wide cracks for better durability and appearance. Spacing of reinforcement and concrete cover were found to be significant variables reflecting detailing of the reinforcement. Well-distribution of reinforcement in the maximum tension zone helps in controlling cracking. For that, several bars at moderate spacing are much more effective in controlling cracking than one or two large bars of equivalent area [13]. Theoretically, changing reinforcement bar diameter affects the cumulative bond parameter per unit length, specific shear bond stress per unit contact area and distribution of reinforcement. Specimens reinforced with smaller-diameter bars possess substantially larger cumulative bond parameter per unit length, generating more extensive micro-frictional interaction at the steel-concrete interface. Assessing how the reinforcement configuration affects the behavior of concrete structures is important in making the right decision in the design of concrete members. This project aims to understand the impact of changing reinforcement bar diameters on the behavior of one-way slab panels reinforced by steel alternatives and compared with panels reinforced by traditional steel. An experimental program is applied casting several slab panels, changing reinforcement type and bar diameter and assessing their behavior under dynamic loading conditions.

2. Experimental Program

The experimental program included casting 12 reinforced concrete slab panels with dimensions. 4000×1000×180 mm. The panels were tested under dynamic impulse and measuring the dynamic response due to low velocity dropped object [9, 14, 15]. Slab panels were reinforced by three different types of reinforcement individually and every panel was reinforced by different reinforcement diameter with the same reinforcement ratio for the specimens of every series as shown in the table below.

Table 1. Specimen reinforcement details

symbol	No of bars	Reinforcement area	description
S-6	20 Φ 6	565.48	Slab panel reinforced by 6 mm diameter steel bars
S-8	12 Φ 8	603.19	Slab panel reinforced by 8 mm diameter steel bars
S-12	5 Φ 12	565.48	Slab panel reinforced by 12 mm diameter steel bars
S-16	3 Φ 16	603.19	Slab panel reinforced by 16 mm diameter steel bars
G-6	20 Φ 6	565.48	Slab panel reinforced by 6 mm diameter GFRP bars
G-8	12 Φ 8	603.19	Slab panel reinforced by 8 mm diameter GFRP bars
G-12	5 Φ 12	565.48	Slab panel reinforced by 12 mm diameter GFRP bars
G-16	3 Φ 16	603.19	Slab panel reinforced by 16 mm diameter GFRP bars
W-6	20 Φ 6	372.655	Slab panel reinforced by 6 mm diameter steel wire rope
W-8	12 Φ 8	397.45	Slab panel reinforced by 8 mm diameter steel wire rope
W-12	5 Φ 12	373.25	Slab panel reinforced by 12 mm diameter steel wire rope
W-16	3 Φ 16	397.45	Slab panel reinforced by 16 mm diameter steel wire rope

Note: The gross cross-sectional area of wire rope reinforcement is lower than steel and GFRP counterparts due to the 65% fill factor of wire ropes.

Note: Only a single specimen for each configuration was cast and tested, it is recommended to test multiple specimens to improve statistical reliability in future studies.

2.1. Material

The same ready-mixed concrete was used in casting all slabs in this research at the same time and under the same conditions. To achieve this a ready mixed concrete truck with capacity of ten cubic meters was delivered to the worksite to cast the twelve slabs together at one time. Concrete targeted compressive strength was 25.2 MPa. Three concrete cylindrical samples were taken from the truck to test the compressive strength according to [16].

Table 2. Concrete mix mechanical properties

Type	Compressive strength (MPa)	Tensile strength (MPa)	Modulus of rupture (MPa)
C 25	25.2	2.83	2.61

The control specimens were reinforced by traditional deformed steel bars matching ASTM A615 [17]. Four slab panels were reinforced by deformed steel bars with different bar diameters for each panel maintaining the same reinforcement ratio (the number of bars will be different). Four specimens were reinforced by ribbed GFRP bars using four different bar diameters. Every single slab was reinforced by different bar diameters maintaining the same area of reinforcement. Table 3 presents the mechanical characteristics of GFRP bas provided by the manufacturer who tested specimens according to ISO10406 (FRP reinforcement of concrete. Test methods) [17]. Steel wire ropes with four wire diameters were used in four slab specimens. Every slab was reinforced with one different diameter provided that the steel area is fixed (the number of wires will be different). The type used in this study is the (7x19) IWRC. Table 4 presents the mechanical characteristics of deformed steel bars and steel wire ropes tested in Tikrit University mechanical laboratories according to ASTM A615 and ASTM A931 respectively. [18], [19].

Table 3. GFRP bars mechanical properties

Reinforcement	Ultimate Tensile Strength (MPa)	Modulus of Elasticity (MPa)
G-6	1300	51125
G-8	1840	72521
G-12	1331	51197
G-16	1204	52725

Table 4. Steel bars and wire ropes mechanical properties

Reinforcement	Yield Strength (MPa)	Ultimate strength (MPa)	Elongation %	Modulus of Elasticity (MPa)
S-6	407	491	4.8	185000
S-8	422	552	6.5	215350
S-12	430	608	14.2	193650
S-16	626	740	15.4	207150
W-6	350.62	561.1	3.71	102000
W-8	351.05	437.36	5.7	105500
W-12	361.2	535.7	6.13	100180
W-16	345.24	350	6.22	108250

2.2. Stages of Work

2.2.1 Preparation of The Specimens

The panels were cast on two parallel supporting walls built for this purpose with 3600 mm clear distance between them. Specimens were cast using the same concrete mixture with 25.2 MPa compressive strength.



Fig. 1. Supporting walls



Fig. 2. Panels formwork

2.2.2 Dynamic Test

The dynamic test involved evaluating the dynamic response of the slabs through a 10 kg metal mass (ball) freely dropped - from a height of 1 m to the midspan of the panel and recording the resulting

response. The mass dropped without any guideway to eliminate any effect of friction losses. After the drop, the vibration amplitude and acceleration at midspan were measured as shown in figure 3. The accelerometer sensor used was (GY-61 ADXL335 3-axis) and the LVDT used was (SOPG20). The dynamic LVDT end was firmly connected to the bottom of the panel to ensure compatible vibration characteristics as shown in figure 4. The amount of the falling mass (10 kg) and the height (1 m) is chosen to ensure no cracks were propagated in the panel due to the impact. However; this goal was verified through an Ultrasonic pulse velocity test on the bottom face of the panel, which showed no significant variation in pulse velocity was recorded before and after the impact was applied, which confirmed that no cracks were propagated due to the impact.

Table 5. LVDT sensor specification

Reinforcement	Specification
Supply voltage	9-12VDC
Operating current	$\leq 12\text{mA}$
Measuring range	50mm, 100mm
Output signal	0-5v
Linearity	0.25%, 0.5%
Stok length	50mm, 100mm
Resolution	$\leq 0.1\mu\text{m}$, 16bit

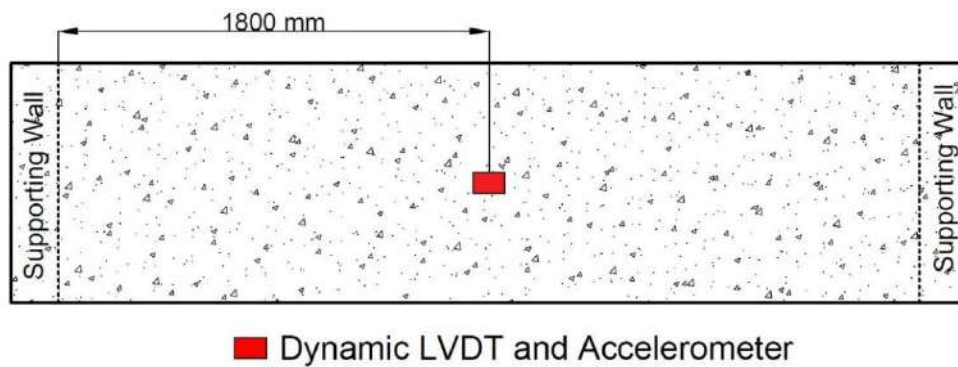


Fig. 3. Position of LVDT



Fig. 4. Dynamic LVDT and accelerometer

3. Results and Discussion

The dynamic response of reinforced concrete slab panels under transient impact excitation is governed by the complex interaction of the panels' flexural stiffness, mass, internal damping mechanism and the spatial distribution of the reinforcement. In the present experimental program, twelve one-way slab panels were subjected to the free fall of 10 kg mass from height of 1m to the midspan of the panel. Midspan dynamic displacements and midspan accelerations were measured for each panel. Downward deflections are defined as negative in accordance with the physical direction of loading. The results are presented and discussed individually and comparatively in the sections that follow.

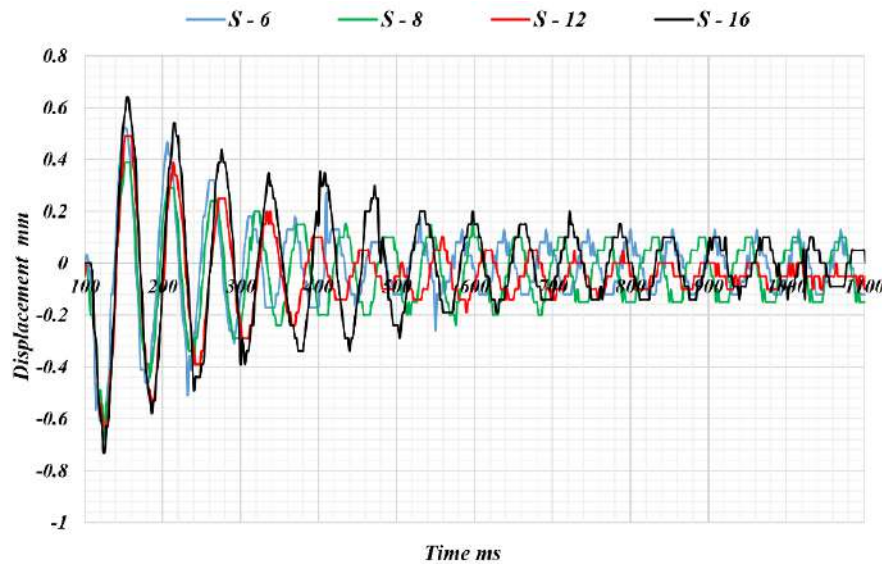


Fig. 5. Midspan displacement for deformed steel specimens. (Upward Disp. +)

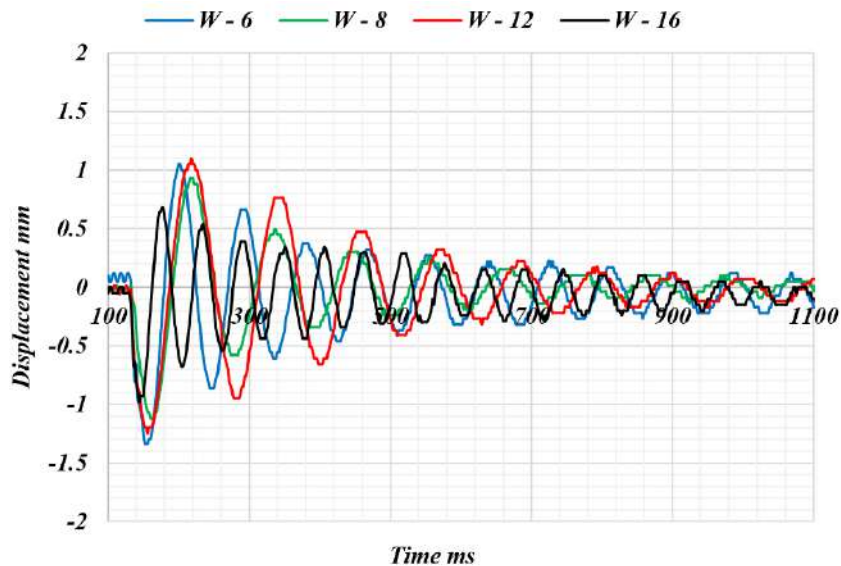


Fig. 6. Midspan displacement for wire rope reinforced panels. (Upward Disp. +)

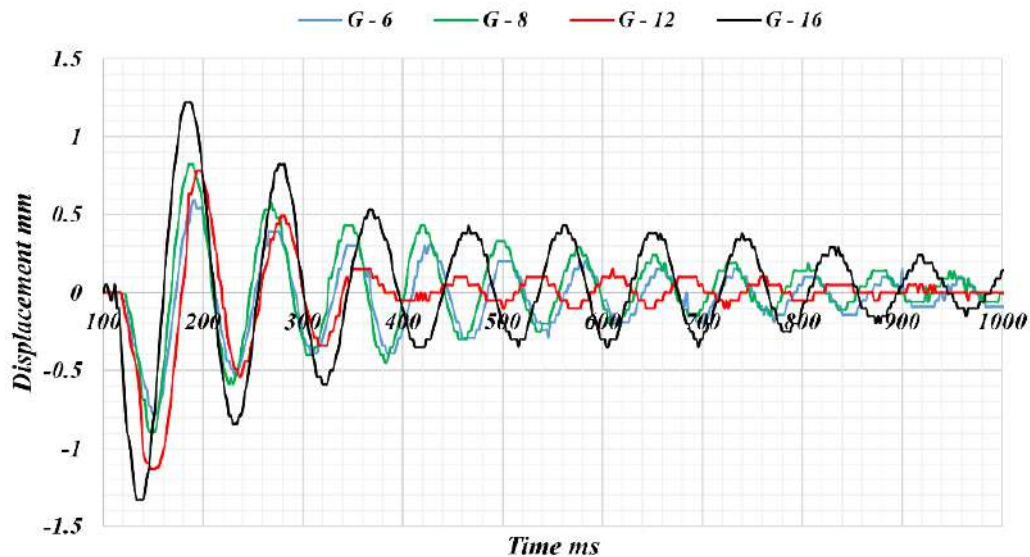


Fig. 7. Midspan displacement for GFRP reinforced panels. (Upward Disp. +)

3.1 Natural Frequency

Since the impact-introduced displacements were of the order of fractions of millimeters - well below any cracking threshold - the section responded in its uncracked elastic state throughout the free-vibration phase, and the governing stiffness is therefore the gross uncracked section stiffness rather than any post-crack effective stiffness. Under these conditions, the natural frequency is primarily controlled by the section geometry, concrete elastic modulus and total mass with reinforcement contributing through its modular ratio effect on the gross transform section stiffness.

Traditional steel reinforced panels exhibited the highest natural frequency values attributed to the high modulus of elasticity of steel reinforcement and the efficient bond between steel bars and the surrounding concrete. Generally, natural frequency decreased with increasing bar diameter due to the increase in the total steel-concrete interfacial contact area resulting to better distribution of bond stresses on reinforcement concrete interfaces.

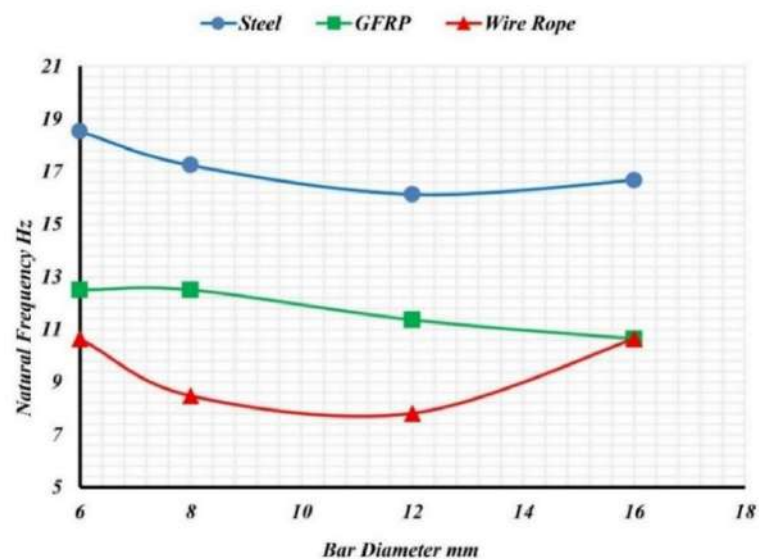


Fig. 8. The effect of bar diameter on natural frequency

In the wire rope reinforced series, the initial decreasing trend from W-6 to W-12 is attributable to the progressive reduction in the number of wire ropes as diameter increases. W-6 contains the greatest number of rope elements, providing more distributed composite interaction with the

surrounding concrete and thus a higher cumulative interfacial stiffness contribution. The notably higher frequency of W-16, which departs from the otherwise decreasing trend, warrants careful consideration. A single wire rope of 16mm diameter possesses substantially greater individual flexural rigidity than the smaller diameter ropes used in the other specimens.

The observed trend of decreasing natural frequency with increasing bar diameter in GFRP-reinforced series is physically meaningful and warrants careful interpretation. Since all four panels share identical geometry, concrete grade and reinforcement ratio, the variation in frequency cannot be attributed to differences in mass or gross cross-sectional stiffness. Rather, it reflects the influence of the effective flexural stiffness contributed by the GFRP enforcement itself. Specimens G-6 and G-8 exhibited higher frequencies whereas G-12 and G-16 yielded reduced frequencies. The greater number of bars in G-6 and G-8 distributes the reinforcement more uniformly across the panel width, potentially improving composite action and effective moment of inertia of the transformed section, further supporting the observed frequency values.

A higher natural frequency is preferable as it means that the section is stiffer and this reduces the likelihood of resonance with common dynamic excitation sources (machinery, traffic, people and impact). Slab natural frequency should be well above the dominant excitation frequency to maintain a large frequency separation margin.

3.2 Damping Ratio

The effective viscous damping ratio estimated by the logarithmic decrement method [20] from successive free-vibration peaks ranged from 5.39% for S-6 to 3.04% for S-16, revealing a consistent and monotonically decreasing trend as increasing bar diameter. The inverse relationship between bar diameter and damping ratio in steel reinforced specimens can be attributed to the total steel-concrete interfacial contact area. Specimens reinforced with smaller-diameter bars, possess substantially larger cumulative bond parameter per unit length, generating more extensive micro-frictional interaction at the steel-concrete interface.

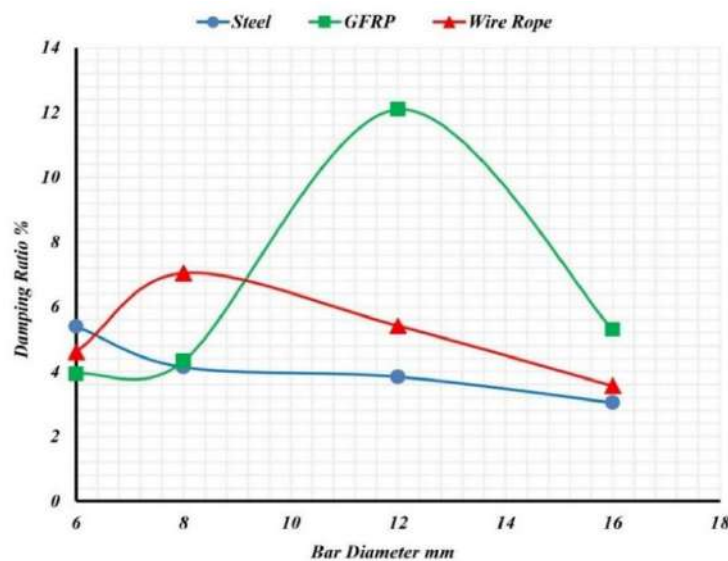


Fig. 9. The effect of bar diameter on damping ratio.

The non-monotonic trend observed across the four specimens reinforced by steel wire ropes is particularly noteworthy. Given that the reinforcement ratio is held constant and the cross-sections remain uncracked, this pattern is unlikely to be governed by crack-induced energy dissipation, which is the dominant mechanism in conventional reinforced concrete. Rather, it may be attributed to the mechanical interaction between the wire rope and the surrounding concrete matrix, whereby the multi-wire helical geometry of the rope introduces frictional inter-wire slip at the strand level. The lowest damping ratio is recorded for W-16 ($\zeta = 3.56\%$). The three 16 mm wire ropes provide a limited total steel-concrete interface parameter. Consequently, the panel oscillates

with low damping and sustains amplitude - a response pattern consistent with the behavior of near elastic, lightly damped structural members.

The notably elevated damping ratio of specimen G-12 is a physically significant observation that warrants careful consideration. Given that the enforcement ratio is held constant across all the specimens and the sections remain uncracked, this anomaly cannot be attributed to crack-induced energy dissipation. Rather, it's more appropriately interpreted in terms of the relatively lower elastic modulus of the 12 mm GFRP bars combined with the reduced bar count, which together yield a more flexible reinforcement arrangement.

Panels reinforced by large-diameter GFRP bars consistently exhibited higher damping ratio than their steel bar and wire rope counterparts, meaning that the use of coarse configuration of GFRP reinforcement is preferable in cases where high damping is required. Despite the high cost of GFRP reinforcement, it's beneficial to be used in structural elements exposed to dynamic vibrations and severe corrosive environment like bridge decks and girders.

A high damping ratio means that the slab dissipates dynamic energy more rapidly, causing vibrations to decay faster after excitation ceases. Higher damping reduces dynamic amplification and limits peak displacement, and stress under dynamic loading. Additionally, higher damping improves serviceability and fatigue performance due to the reduction in sustained dynamic stresses.

3.3 Dynamic Amplitude

The larger amplitude of S-16 compared to other steel reinforced specimens is attributable to the concentration of bending stiffness into only three bars, which simultaneously reduces the total steel concrete interface parameter, allowing a greater fraction of the impacted kinetic energy to be converted into elastic strain energy and peak displacement. The relatively modest amplitude of S-12, despite its intermediate diameter, reflects the beneficial interaction between its five-bar configuration and the concentrated bond stress intensity per bar. The narrowness of the amplitude range across the series (0.62-0.75 mm) compared to GFRP series (0.78-1.33 mm), reflects the uniformly high elastic moduli of the steel bars relative to the more variable and generally lower GFRP moduli.

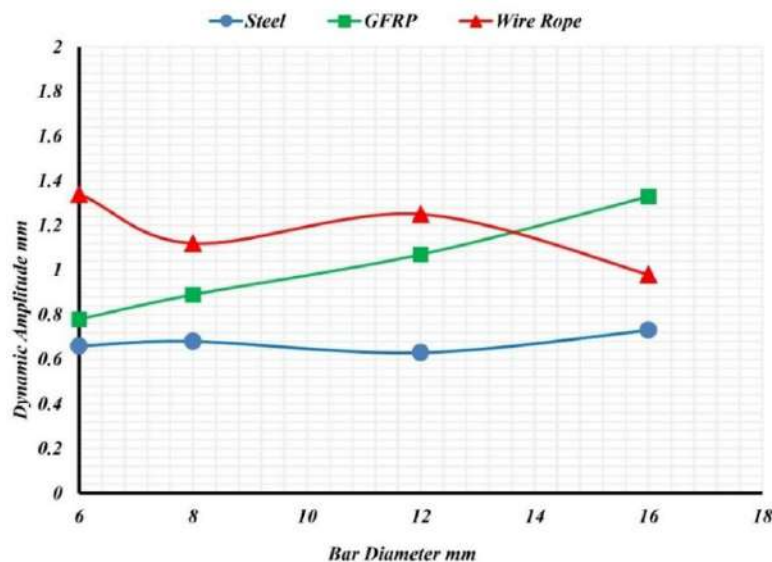


Fig. 10. The effect of bar diameter on dynamic amplitude

Except for the slight reversal between W-8 and W-12, the general trend is one of decreasing peak displacement with increasing wire rope diameter, which is physically consistent with the expected dynamic stiffness behavior of uncracked composite sections. The number of wire ropes decreases substantially from W-6 to W-16, progressively reducing the total steel-concrete interfacial contact area and, consequently, the distributed composite action across the slab width. Specimen W-6

benefits from the most extensive interfacial engagement which enhances the effective flexural rigidity of the section and thereby limits the peak displacement response to the dynamic impulse. The slight departure from the monotonic trend at W-12, which registers a higher peak displacement than W-8, may be attributed to the competing influence of the individual rope flexural rigidity.

G-16 records the largest initial dynamic amplitude in the series, while G-6 records the smallest. This monotonic increase with bar diameter follows the same physical mechanism as observed in the steel series: as bar diameter increases, the total GFRP concrete interface parameter decreases progressively. The reduction in total interface parameter allows a greater fraction of the imparted kinetic energy to be converted into elastic strain energy and manifest as peak midspan displacement. The lower dynamic amplitude is preferred as it refers to the smaller displacements and stresses under dynamic loading protecting structural integrity and occupant comfort. Excessive dynamic amplitude causes human discomfort, equipment malfunction and potential fatigue cracking in the slab.

Table 6. Summary of test results

Reinforcement	Damping ratio %	Natural frequency Hz	Dynamic amplitude mm
S-6	5.39	18.52	0.66
S-8	4.14	17.24	0.68
S-12	3.84	16.13	0.63
S-16	3.039	16.67	0.73
W-6	4.611	10.64	1.34
W-8	7.04	8.47	1.12
W-12	5.414	7.8	1.25
W-16	3.56	10.64	0.98
G-6	3.934	12.5	0.78
G-8	4.323	12.5	0.89
G-12	12.1	11.36	1.07
G-16	5.3	10.64	1.33

3.4 Midspan Acceleration

Upon impact, all steel reinforced panels displayed an initial sharp acceleration peak, followed by a free-vibration decay phase whose rate varied considerably among the tested configurations. S-6 recorded the highest initial positive acceleration peak with relatively rapid attenuation reflecting a higher damping. This behavior suggests that the more distributed reinforcement arrangement associated with smaller diameter bars promotes greater energy dissipation other reinforcement concrete interface, resulting in a higher effective damping ratio. Midspan acceleration data in Figure 12 indicate that, within the deformed steel bar series, the viscous damping ratio decreases as bar diameter increases. This trend is physically consistent with the reduction in the number of reinforced reinforcing bars and the corresponding decrease in total reinforcement-concrete interfacial area as bar diameter decreases, which in turn limits the frictional and bond-related energy dissipation mechanisms that governs the damping response under low energy impact loading.

The non-monotonic relationship between bar diameter and damping ratio observed in the GFRP series - where G-12 outperforms both smaller and larger diameter configurations - suggests that the energy dissipation mechanism in GFRP reinforced panels is governed by a complex interaction of factors rather than bar diameter alone. At intermediate diameters such as 12 mm, the adhesion characteristics between the GFRP bar and the concrete matrix and the local confinement effect of the surrounding concrete produce a more favorable condition for interfacial energy dissipation. In contrast, the finer reinforcement distribution of G-6 and G-8, while providing greater contact area, appears to generate a stiffer post impact response, whereas the larger G-16 bars tend to concentrate damage without a proportional increase in distributed energy dissipation.

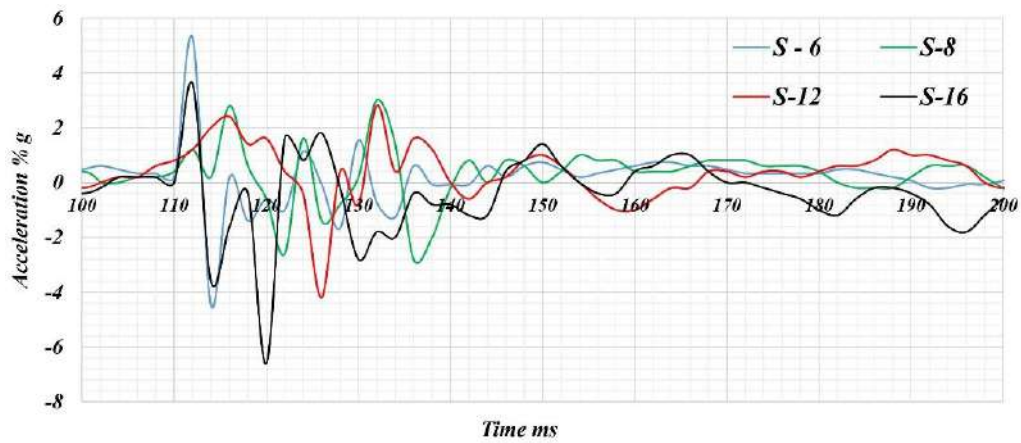


Fig. 11. Midspan acceleration for deformed steel specimens

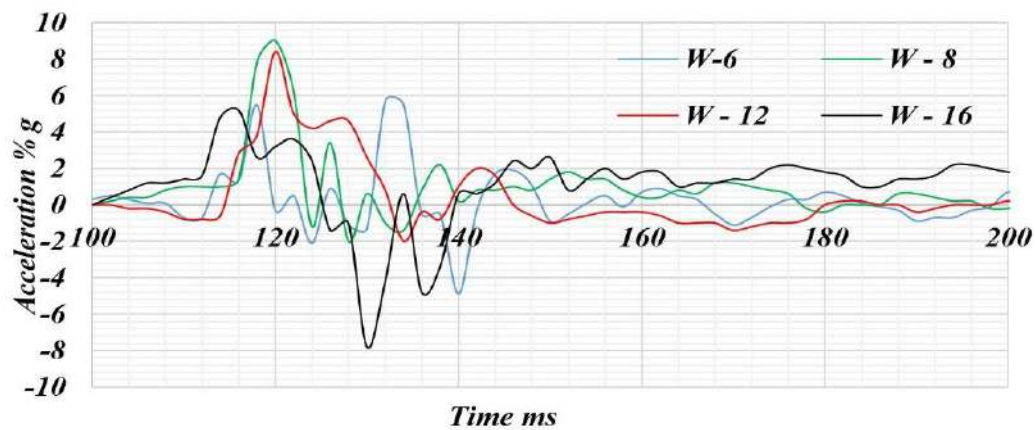


Fig. 12. Midspan acceleration for wire rope reinforced panels.

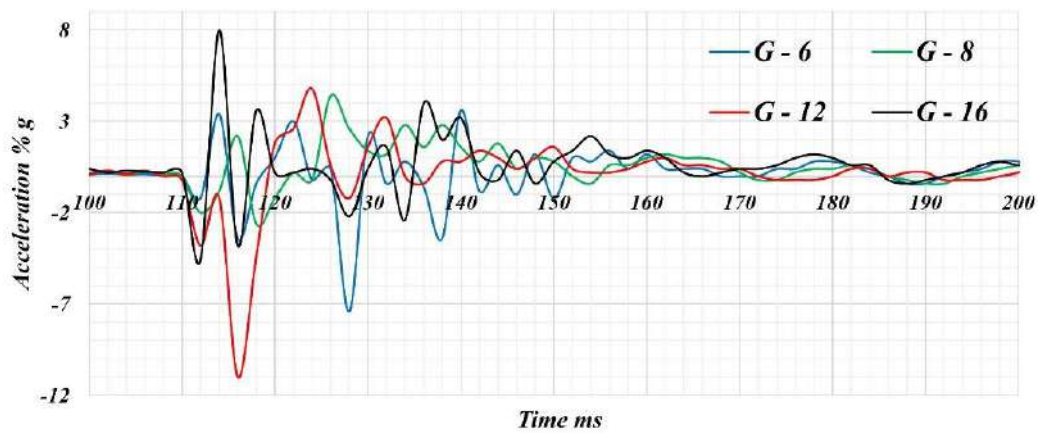


Fig. 13. Midspan acceleration for GFRP reinforced panels.

The observation that W-8 outperforms W-6 in terms of energy dissipation, despite having larger bar diameter, indicates that the relationship between wire rope diameter and damping capacity is not strictly monotonic within this series. This behavior may be partly attributed to the diameter-dependent variation in internal wire-to-wire friction within the rope strands.

Overall, the midspan acceleration data showed the same behavior of the three series in terms of damping ratio calculated from the midspan displacement graphs. In deformed steel reinforced panels, the damping ratio decreases when increasing bar diameter. In wire rope reinforced specimens, G-8 exhibited the faster acceleration diminishing, and in GFRP reinforced panels, G-12 showed relatively rapid acceleration wave excitation.

3.5 Feasibility and Applicability Analysis

The practical applicability of the three reinforcement types investigated in this study varies considerably in terms of cost, workability, anchoring, bond characteristics and durability. Deformed steel bars remained the most established and cost-effective reinforcement option, benefiting from standardized production, widely available skilled labor, and well-developed design codes. However, their susceptibility to corrosion in aggressive environments - particularly in chloride-rich or carbonated concrete - represents a long-term durability concern that may lead to significant maintenance and repair costs over the service life of the structure.

GFRP bars offer a compelling alternative in corrosive environments due to their inherent resistance to electrochemical corrosion, significantly reducing long-term maintenance costs. Their high tensile strength to weight ratio also facilitates handling and placement. However, the initial material cost of GFRP bars is considerably higher than that of conventional steel. Additionally, GFRP bars exhibit linear elastic behavior with no yielding plateau. Cutting and bending GFRP bars onsite also require specialized tools, adding to labor complexity.

Steel wire rope presents a fundamentally different set of practical challenges. While it offers flexibility and high tensile capacity, its use as conquering reinforcement introduces several significant workability concerns. The absence of standardized anchoring system for wire rope embedded in concrete creates difficulties in ensuring adequate force transfer at the end of structural elements, potentially leading to premature anchorage failure. The helical geometry of wire rope results in a lower effective modulus of elasticity compared to solid steel bars and its smooth surface finishes produces inferior bond characteristics with the surrounding concrete matrix. Furthermore, wire rope is susceptible to internal corrosion between individual wires, which is difficult to detect and may progress unnoticed within the concrete section, posing a long-term durability risk comparable to or exceeding that of conventional steel in aggressive environments. The higher labor costs associated with cutting, positioning and securing wire rope within formwork, combined with the lack of established design provisions, further limits its immediate applicability as a primary reinforcement system.

4. Conclusions

Depending on the results of the tests performed on the twelve specimens included in the experimental program, it is important to summarize the main conclusions as follows:

- Natural frequency followed the ranking deformed steel > GFRP > wire rope. The highest natural frequency was recorded by S-6 and most of the natural frequency values decreased when increasing reinforcement diameter.
- Specimens reinforced with the large-diameter bars produced sharp, high amplitude impact spikes and pronounced rebound pulses. In contrast, reducing reinforcement bar diameter results in lower peak amplitudes but more sustained pulse durations.
- In deformed steel reinforced panels, the viscous damping ratio decreased when increasing reinforcement diameter, demonstrating that panels with finer reinforcement distributions achieve significantly higher damping. While in steel wire rope reinforced panels the damping ratio was higher in specimens reinforced by large bar diameter.
- In GFRP series, however, the damping ratio did not follow a consistent linear trend with bar diameter. G-12 exhibited the highest damping ratio, exceeding both the G-6 and G-16 specimens. This anomalous behavior is attributed to the interplay of several physical mechanisms specific to GFRP bars. At intermediate diameters, greater frictional energy dissipation is generated; the adhesion characteristics between the GFRP bar surface and the surrounding concrete matrix are diameter-dependent, with the G-12 bar producing an optimal bond condition.
- Distributed fine-bar configurations should be preferred over concentrated large-bar configurations, even when both provide equivalent static strength as finer reinforcement distributions provide substantially greater total interface parameter at an equivalent reinforcement area, thereby producing higher system-level damping.

- GFRP reinforced panels consistently exhibited higher damping ratio than their steel and wire-reinforced panels in the coarse reinforcement configuration (12 and 16 mm reinforcement bars). Specimen G-12 achieved the most rapid extinction outperforming both the stiffer 16 mm configuration and the more flexible 6 mm configuration.
- Natural frequency of wire rope-reinforced panels is the lowest in the three series. However; the effect of bar diameter on natural frequency is almost identical between steel and wire- reinforced specimens.
- Midspan acceleration showed agreement with displacement readings in terms of vibration damping in all the three series.

It is recommended to extend the study in the future by using more reinforcement systems (i.g. hybrid reinforcement system) and different dynamic impacts (i.g. high velocity impact). The effect of changing concrete mixture also could be studied in future. Additionally, it is recommended to apply theoretical analysis of the dynamic response if slab panels are reinforced by steel and alternatives.

References

- [1] Firmo JP, Fibre-Reinforced Polymer Reinforcement Towards More Durable and Sustainable Concrete Structures. In: Janberg N, editor. Sustainability in Structural Concrete Design. Zurich: International Association for Bridge and Structural Engineering; 2024. p. 320-360. <https://doi.org/10.2749/sed020.ch14>
- [2] Ruiz Emparanza A, Durability assessment of GFRP rebars in marine environments. Constr Build Mater. 2022;329:127028. <https://doi.org/10.1016/j.conbuildmat.2022.127028>
- [3] Ahmed E, Design of Bridge Deck Slabs Using Glass Fiber-Reinforced Polymer (GFRP) Bars of Different Grades. In: The 9th International Conference on Short and Medium Span Bridges. Calgary: SMSB; 2014. p. 1-10.
- [4] Carvelli V, Fatigue behaviour of concrete bridge deck slabs reinforced with GFRP bars. Compos Part B Eng. 2010;41:560-567. <https://doi.org/10.1016/j.compositesb.2010.06.006>
- [5] Liu T, Experimental Research on Damping and Bonding Properties of Concrete Reinforced with GFRP Bars. In: 12th Biennial International Conference on Engineering, Construction, and Operations in Challenging Environments. Honolulu: American Society of Civil Engineers; 2010. p. 2793-2805. [https://doi.org/10.1061/41096\(366\)264](https://doi.org/10.1061/41096(366)264)
- [6] Sadraie H, Dynamic performance of concrete slabs reinforced with steel and GFRP bars under impact loading. Eng Struct. 2019;62-81. <https://doi.org/10.1016/j.engstruct.2019.04.038>
- [7] Mousavi T, Impact response of hybrid FRP-steel reinforced concrete slabs. Structures. 2019;436-448. <https://doi.org/10.1016/j.istruc.2019.02.013>
- [8] Salih YA, Impact Resistance of GFRP Reinforced Concrete One-Way Slabs. Inf Sci Lett. 2022;1267-1275. <https://doi.org/10.18576/isl/110426>
- [9] Anas SM, Ultra-high-performance concrete and C-FRP tension Re-bars: A unique combination of materials for slabs subjected to low-velocity drop impact loading. Front Mater. 2022;1061297. <https://doi.org/10.3389/fmats.2022.1061297>
- [10] Dong S, Dynamic impact behaviors and constitutive model of super-fine stainless wire reinforced reactive powder concrete. Constr Build Mater. 2018;602-616. <https://doi.org/10.1016/j.conbuildmat.2018.07.027>
- [11] Wu Q, Blast-resistance characteristics and design of steel wire reinforced ultra-high performance concrete slabs. Int J Impact Eng. 2024;105059. <https://doi.org/10.1016/j.ijimpeng.2024.105059>
- [12] Wu Q, Blast and impact resistance of stainless steel wire-reinforced ultra-high performance concrete door panels: Numerical analyses and design insights. Thin-Walled Struct. 2025;113485. <https://doi.org/10.1016/j.tws.2025.113485>
- [13] American Concrete Institute. Building Code Requirement for Structural Concrete (ACI 318-19). Farmington Hills: American Concrete Institute; 2019.
- [14] Abdulsahib WS, Low Velocity Impact Response of Reinforced Concrete Flat Slabs. Open Civ Eng J. 2023;17:E230113. <https://doi.org/10.2174/18741495-v17-e230113-2022-75>
- [15] Anas SM, Influence of Supports on the Low-Velocity Impact Response of Square RC Slab of Standard Concrete and Ultra-High Performance Concrete: FEM-Based Computational Analysis. Buildings. 2023;13051220. <https://doi.org/10.3390/buildings13051220>
- [16] ASTM International. ASTM C39/C39M-21: Standard test method for compressive strength of cylindrical concrete specimens. West Conshohocken: ASTM International; 2021.

- [17] International Organization for Standardization. ISO 10406-1: Fibre-reinforced polymer (FRP) reinforcement of concrete - Test methods. Geneva: ISO; 2025.
- [18] ASTM International. ASTM A615/A615M-18: Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement. West Conshohocken: ASTM International; 2018.
- [19] ASTM International. ASTM A931/A931M-18: Standard test method for tension testing of wire rope and strand. West Conshohocken: ASTM International; 2018.
- [20] Chopra AK. Dynamics of Structures: Theory and Applications to Earthquake Engineering. Pearson Education; 2017.

Strength and deformation characteristics of stabilized fine-grained soil cured under overburden stress

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Abstract

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This study investigates the effects of overburden stress on the physical and mechanical properties of stabilized fine-grained soil. Unconfined compressive strength (UCS) tests were conducted to evaluate the influence of overburden stress on stabilized soil specimens cured for 7, 14, 28, 60, and 90 days. The soil was treated with various binder contents of 1%, 2%, 4%, and 7% by soil dry weight. Two sets of stabilized soil specimens were prepared: one cured under atmospheric pressure and the other cured under stress of 10.5 kPa. The results indicate that the specimen density increased with both binder content and curing stress. The applied stress had the effect of increasing the unconfined compressive strength and stiffness, while a reduction in failure strain was observed compared to stabilized soil specimens cured without stress. The failure behavior is more brittle in stabilized soil with 7% binder content compared to soils with lower binder contents and natural soil, which exhibit more ductile behavior.

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1. Introduction

Chemical soil stabilization involves adding different stabilizing agents to enhance the properties of weak soils, which exhibit low shear strength and stiffness. Generally, such soils are unsuitable for construction and are therefore excavated and replaced with good-quality geomaterials from quarries. Excavation and transportation generate large volumes of waste soil and contribute significantly to CO₂ emissions. In recent years, the lack of good-quality geomaterials and limited disposal options due to stricter environmental restrictions has made this approach increasingly impractical. Therefore, the use of stabilized soil in various construction projects has become widespread, as it improves soil performance while reducing environmental impact [1].

The use of hydraulic binders, such as cement, cement kiln dust, and their combinations, has been extensively studied for improving the physical and mechanical properties of fine-grained soils [2–11]. Moreover, the influence of other factors such as binder type and dosage, soil properties, mixing procedure, and curing time and conditions on the behavior of stabilized soils has been widely studied [12–17]. Previous studies have shown that improvements in cement stabilized soils are primarily attributed to the hydration and pozzolanic reactions of cement-based stabilizers, which produce primary and secondary cementitious materials. Hydration reactions start immediately after cement is mixed with moist soil, generating three types of primary cementitious products: calcium-silicate hydrate (CSH; C₂SH_x and C₃S₂H_x), calcium-aluminate hydrate (CAH; C₃AH_x and

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C_4AH_x), and calcium hydroxide ($Ca(OH)_2$). The symbols, C, S, A and H refer to calcium (CaO), silicate (SiO_2), aluminate (Al_2O_3) and water (H_2O) respectively. CSH and CAH gel bind soil particles together to form a strong, hard matrix over time, while $Ca(OH)_2$ deposited on the surface of soil particles increases the pH of the mixture. Under certain conditions, when the pH rises to around 12, calcium hydroxide can react with components of the soil, such as silica and alumina from clay minerals, through pozzolanic reactions, leading to the formation of additional of CSH and CAH gel over time (secondary cementitious materials).

To predict the improvement in the engineering properties of stabilized soil, a series of experimental tests are typically conducted in a laboratory under certain conditions, which in principle should simulate the in-situ conditions as much as possible. In a laboratory, the experimental testing program is carried out according to the standard protocols, starting from specimen preparation, curing conditions and period until testing of stabilized soil specimens [18]. Almost all standards recommend that the specimens of stabilized soils be cured under controlled conditions such as at temperatures around 20 °C, relative humidity exceeding 95%, and atmospheric pressure. However, in the field, stabilized soil layers are typically subjected to overburden stress immediately after treatment due to the placement of a cover layer 0.5–1.0 m thick. This loading contributes to densification and improved homogeneity of the treated soil by expelling some of the air entrapped during the mixing process. Moreover, the initial loading is considered a necessary step to create a leveled and more homogenized stabilized mass which can improve the physical and mechanical characteristics of stabilized soil [19].

Several researchers have reported differences between laboratory-cured specimens and field specimens taken from deeper depths for cemented paste backfill, cemented sand, and stabilized peat [20–22]. A few studies have pointed out that the mechanical behavior of stabilized soils cured under overburden stress can be altered due to the compression effect during the formation of cementitious products. The effect of the application of various vertical stresses (9 and 18 kPa) on top of stabilized peat specimens during curing was investigated by [19]. The UCS specimens were prepared by pouring the stabilized peats by hand inside the specimens. The results indicated that stabilized soil specimens cured under vertical stress showed a significant increase in strength compared to specimens cured without vertical stress. Moreover, the results show that the compression of stabilized peat during curing decreases with increasing binder content. Reference [23] pointed out that the pre-consolidation pressure of stabilized soils was highly affected by large cementation materials generated due to the use of higher binder content. Reference [24] investigated the effect of overburden stress (7.5 kPa) applied during curing on the strength behavior of silt soil treated with cement. Cement content ranged between 5%, 7% and 9%, and the specimens were cured for 7, 14 and 28 days. The UCS specimens were prepared by pouring the stabilized soil in five layers and tapping each layer with a rubber hammer to remove any air bubbles. The strength of stabilized soil increased due to the application of overburden stress.

The effect of applying 5 kPa overburden stress on stabilized silty clay soil treated with 8%, 10%, and 12% Portland pozzolanic cement has been investigated by [25]. The specimen was prepared and compacted at the optimum water content and cured for 56 days. The results indicated that the specimens cured under stress show an increase in soil strength compared to specimens cured without stress. A recent study investigated the effect of curing stress on low-plasticity silty soil treated with a mixture of 1% lime (90% quicklime (CaO)) and 5% hydraulic binder (35% Portland cement clinker and 65% blast furnace slag). The testing program was limited to only nine prepared specimens, which were compacted using an isotropic confining pressure of 500 kPa at optimum water content, and only one specimen was cured under 500 kPa curing stress for 70 days. The results showed that curing stress has an effect on further increasing the shear modulus of stabilized soil compared to specimens cured without stress [1]. As clarified by previous studies, higher binder content generates more cementitious materials compared to lower binder content, which subsequently results in greater improvement in soil strength and a lower compression effect under overburden stress during curing period. Despite these findings, most previous studies have not explicitly considered the combined influence of curing stress and standard compaction effort applied during specimen preparation. In addition, the majority of previous studies used higher binder contents, shorter curing periods, and did not compare the effect of overburden stress on the

natural soil. To address this limitation, laboratory tests were conducted to evaluate the effects of initial stress over time on the strength and stiffness of compacted natural and stabilized soil specimens.

This study aimed to investigate the effect of overburden stress on the strength and deformation characteristics of natural and stabilized soil specimens treated with various binder contents (from 1% to 7%) and compacted specimens using a Proctor hammer to achieve a compaction effort of 600 kJ/m³. The combined effects of binder amount, curing time and applied stress are evaluated using unconfined compressive strength tests.

2. Experimental Program

2.1. Materials

The natural soil used in this study was collected from a construction site in the southwestern part of Sweden. The standard identification laboratory tests were performed on the natural soil to determine its properties. The basic soil characteristics are illustrated in Table 1. The soil was classified as lean clay (CL) according to the Unified Classification System ASTM D 2487-11 [26].

The binder used in this study, named Multicem, was sourced from Cementa AB, Sweden. It is a mixture of Portland cement and cement kiln dust (CKD) in a 50:50 proportion at the plant site, where the CKD replaces limestone as a raw material during cement production. The chemical composition of the binder is presented in Table 2.

Table 1. The basic characteristics of studied soil

Description	Value
Specific Gravity	2.69
Liquid limit (%)	38
Plastic limit (%)	19
Plasticity index (%)	19
Sand content (%)	29
Silt content (%)	54
Clay content (%)	17
Optimum moisture content (%)	13
Maximum dry density (g/cm ³)	1.96
Organic content (Loss of ignition) (%)	4

Table 2. Chemical composition of the binder used in this study

Description	Value
Alkalis, Sodium oxide (Na ₂ O) (%)	3
Free lime, (CaO) (%)	14
Calcium oxide (CaO) total (%)	50
Magnesium oxide (MgO) (%)	2.1
Potassium oxide (K ₂ O) (%)	7.5
Silicon oxide (SiO ₂) (%)	19
Aluminum oxide (Al ₂ O ₃) (%)	4.5
Iron oxide (Fe ₂ O ₃) (%)	2.8
Sulfur trioxide (SO ₃) (%)	5
Loss of Ignition (%)	4.8

2.2. Preparation of Specimens

The unconfined compressive strength (UCS) specimens of the natural soil were prepared by homogenizing the natural soil using a laboratory mixer for 15 to 18 seconds [27]. After that, the homogenized soil was placed inside cylindrical plastic tubes in five compacted layers to achieve a height of 10 cm. The inner diameter of the tubes was 50 mm and the height was 170 mm. The standard compaction procedure was followed for preparing all specimens by compacting each

layer with 15 blows using a design hammer which provides energy per volume 600 kJ/m^3 as compaction effort. According to the curing conditions, the prepared specimens were divided into two sets: one cured under atmospheric pressure (without stress), and the other cured under a stress of 10.5 kPa. The applied curing stress of 10.5 kPa corresponds to an equivalent soil depth of about 0.6 m representing a low overburden stress near-surface soil.

For specimens cured under atmospheric pressure, the prepared specimens were sealed on both sides using covers to prevent water from entering. For the other specimens that were cured under overburden stress, the prepared specimens were subjected to loads on the top of each soil specimen. The load was applied using a filter plug as shown in Fig. (1). The filter plug was inserted from the top end of the PVC tube until it made contact with the upper surface of the specimen. The filter plug has a cylindrical shape with an outside diameter of 49 mm and contains a circular hole in the middle of both ends. The lower end (which makes contact with the upper surface of the specimen) contains a thick filter to allow water to drain through the hole during the compression period (application of overburden stress). The upper end is used to apply the vertical load. The main purpose of the filter plug was to transfer the applied stress to the soil specimens and allow drainage from the top surface during the curing period. Before inserting the plug and soil, the inner surfaces of the polyvinyl chloride (PVC) tubes used for specimen preparation were manufactured with a smooth finish layer to ensure very smooth surfaces and were also greased prior to testing to minimize side friction during the consolidation and extrusion of the soil specimens. The bottom end of the specimen was sealed with plastic covers and rubber lids. All prepared specimens were cured in glass containers partially filled with water to a certain level (cover half of the specimen height) and stored at a controlled room temperature of 20°C , as illustrated in Fig. 2. A load of 2 to 2.1 kg was placed on top of each filter plug for natural and stabilized soil specimens immediately after preparation. All prepared specimens were cured for 7, 14, 28, 60 and 90 days.

The stabilized soil specimens were prepared by mixing the homogenized soil with the binder for ten minutes using a laboratory mixer until a visually homogeneous consistency was achieved. The binder was added in various proportions of 1%, 2%, 4%, and 7% based on the dry weight of the soil. The mixtures were then placed by hand inside the plastic tubes. The preparation of UCS specimens of stabilized soil followed the same protocol and procedure described above for preparing and curing the UCS specimens of natural soil.

All prepared specimens (both cured under atmospheric and overburden pressures) were tested after 7, 14, 28, 60 and 90 days of curing. The filter plug was removed from the loaded specimens. The soil specimens were removed from the tubes and carefully trimmed to obtain parallel end surfaces. The specimen density was measured which can give an indication about the homogeneity of the specimens.

2.3. Testing of Specimens

Unconfined compression strength (UCS) tests were conducted according to the ASTM D2166/D2166M-16 standard [28] to evaluate the development of soil strength for various specimens. The UCS tests were performed on cylindrical specimens placed on the UCS apparatus without constraints with a constant height to diameter ratio of 2. Each specimen was axially loaded at a strain rate of 1 mm/minute until failure. During the test, the load and deformation of each specimen were continuously measured until failure occurred. The stress- strain curves were calculated by dividing the load by the specimen's cross-sectional area. The unconfined compressive strength was defined as the maximum peak stress causing failure and corresponding to the failure strain. The water content of the specimens was determined using the broken parts collected after the UCS test.

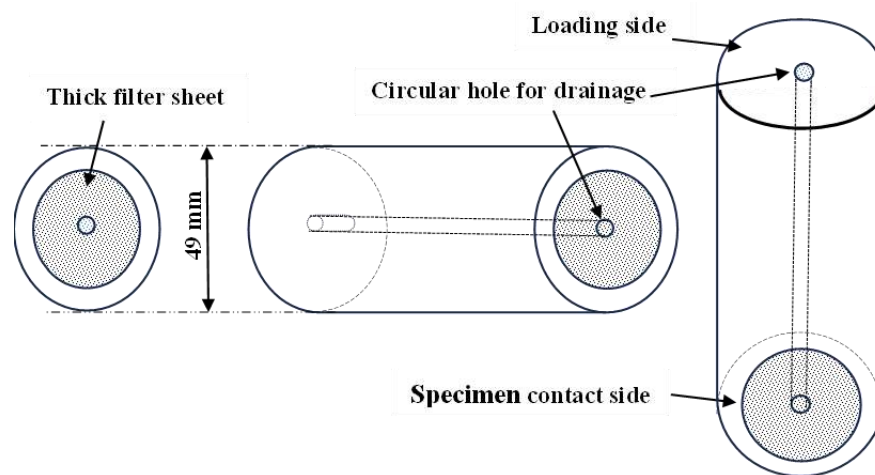
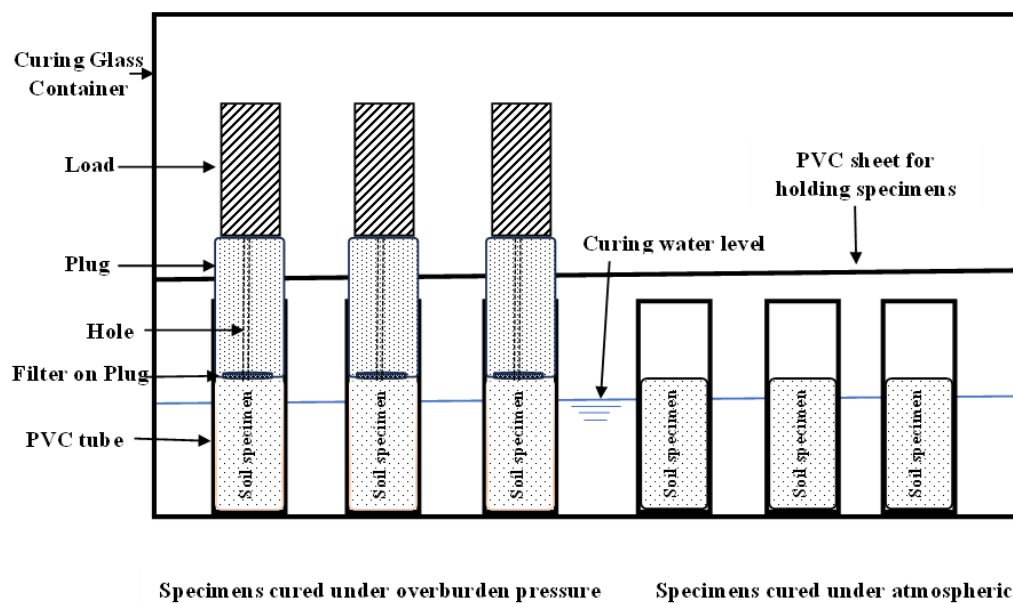


Fig. 1. Filter plug used in this study (schematic)



(a)



(b)

Fig. 2. Shown two diffrent curing conditions (a) Schematic sketch and (b) Two specimens sets cured under different conditions

3. Results and Discussion

3.1 Statistical Analysis

The unconfined compressive strength tests were conducted on specimens prepared in a similar manner and cured under either overburden or atmospheric pressure conditions. The specimen density after preparation was used to compare the two different curing conditions. A t-test was conducted to examine differences in specimen density for different binder contents. Table 3 presents the results of the t-tests, the ratio of standard deviation (SD) to the 95% confidence interval (CI), and the number of specimens (n) for all prepared specimens. The values of the t-test ranged between 0.029 and 0.444, and the p-values ranged between 0.661 and 0.977. The low values of the t-test, as well as all p-values being greater than 0.05 and close to one, indicate no significant differences in specimen density after preparation. The SD/CI ratios for both curing conditions also support this observation. For natural soil, the SD/CI values are highly consistent, whereas for stabilized soil, only small differences in SD/CI are observed. Generally, the similarity in SD/CI values indicates that specimen density after preparation is uniform under both curing conditions, and the process of specimen preparation was well controlled.

Table 3. Presents the results of the t-test analysis of specimen's density after preparation, binder content, and the values of SD/CI or all prepared specimens

Binder content %	Atmospheric Pressure Specimens			Overburden Pressure Specimens			T- test	
	n	Density (Mean Values)	Values of SD/CI	n	Density (Mean Values)	Values of SD/CI	T-value	P-value
0	11	1.94	1.48	13	1.96	1.49	0.406	0.688
1	10	1.98	0.96	10	1.99	1.19	0.359	0.725
2	10	2.01	1.04	10	2.02	1.21	0.268	0.792
4	12	2.02	1.57	11	2.02	1.48	0.444	0.661
7	12	2.02	1.53	10	2.02	1.41	0.029	0.977

Tables 4 and 5 present the ratio between the standard deviation (SD) and the 95% confidence interval (CI), and the number of specimens (n), for UCS, failure strain, density, and soil water content for two different curing conditions. The results shows that the majority of SD/CI values are around 0.1175 for both curing conditions. In contrast, slightly higher SD/CI values of around 0.4034 were noted in a few cases. These differences are infrequent and can be explained by minor experimental uncertainties; therefore, they do not significantly influence the overall trend. The consistency of the SD/CI values under the two curing conditions indicates that the experimental procedure was reliable, with only minimal variation in the measured results.

Table 4. presents the ratio between SD/CI and specimens' number for UCS, failure strain, density and soil water content for atmospheric pressure specimens

Binder content, %	Curing time, Days	n	Values of SD/CI					
			UCS	Strain	Density	Water Content		
						n	Before curing	After curing
0	7	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	14	3	0.4034	0.4034	0.4034	2	0.1175	0.1175
	28	3	0.1179	0.4034	0.1175	3	0.4034	0.4034
	60	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
	90	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
1	7	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	14	2	0.1175	0.1175	0.1178	2	0.1175	0.1175
	28	2	0.1175	0.1175	0.1174	1	-	-
	60	2	0.1175	0.1175	0.1179	0	-	-
	98	2	0.1175	0.1175	0.1174	1	-	-

Binder content, %	Curing time, Days	n	Values of SD/CI					
			UCS	Strain	Density	n	Water Content	
							Before curing	After curing
2	7	2	0.1175	0.1175	0.1178	2	0.1176	0.1175
	14	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	28	2	0.1175	0.1175	0.1176	1	-	-
	60	2	0.1175	0.1175	0.1172	2	0.1175	0.1175
	98	2	0.1175	0.1176	0.1175	2	0.1175	0.1175
4	7	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
	14	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	28	2	0.4034	0.1175	0.1175	4	0.6285	0.6286
	60	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
	90	2	0.1175	0.1175	0.1176	2	0.1175	0.1175
7	7	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	14	2	0.1175	0.1175	0.1175	1	-	-
	28	2	0.4034	0.1175	0.1170	5	0.8054	0.8054
	60	2	0.1175	0.1175	0.1175	2	0.1175	0.0831
	90	2	0.1175	0.1175	0.1176	2	0.1175	0.0831

Table 5. presents the values of SD/CI and specimens' number for UCS, failure strain, density and soil water content for overburden pressure specimens

Binder content, %	Curing time, Days	n	Values of SD/CI					
			UCS	Strain	Density	n	Water Content	
							Before curing	After curing
0	7	2	0.1175	0.1175	0.1175	3	0.4034	0.4034
	14	4	0.4034	0.4034	0.4029	4	0.4034	0.4034
	28	3	0.1175	0.4034	0.1175	3	0.4034	0.4034
	60	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
	90	2	0.1175	0.1175	0.1175	2	0.1174	0.1175
1	7	2	0.1175	0.1175	0.1174	2	0.1175	0.1175
	14	2	0.1175	0.1175	0.1178	2	0.1175	0.1174
	28	2	0.4034	0.1175	0.1177	2	0.1175	0.1175
	60	2	0.1175	0.1177	0.1172	0	-	-
	98	2	0.1175	0.1175	0.1178	2	0.1175	0.1175
2	7	2	0.1175	0.1175	0.1175	2	0.1174	0.1174
	14	2	0.1175	0.1175	0.1176	2	0.1175	0.1176
	28	2	0.1175	0.1175	0.1173	2	0.1176	0.1175
	60	2	0.1175	0.1175	0.1177	2	0.1175	0.1175
	98	2	0.1175	0.1175	0.1176	2	0.1175	0.1175
4	7	2	0.1175	0.1174	0.1173	2	0.1174	0.1175
	14	2	0.1175	0.1175	0.1175	0	-	-
	28	3	0.4034	0.1175	0.1171	5	0.8054	0.8056
	60	2	0.1175	0.1175	0.1179	2	0.1175	0.1175
	90	2	0.1175	0.1175	0.1243	2	0.1175	0.1175
7	7	2	0.1175	0.1175	0.1174	2	0.1177	0.1177
	14	2	0.1175	0.1175	0.1177	0	-	-
	28	3	0.1175	0.1175	0.1175	4	0.6286	0.6286
	60	2	0.1175	0.1175	0.1175	2	0.1175	0.1175
	90	2	0.1175	0.1175	0.1172	2	0.1175	0.1175

3.2 Specimens cured under atmospheric pressure

3.2.1 Effect of Binder Content and Curing Time on the UCS

Fig. 3 shows the effect of curing time on the unconfined compressive strength (UCS) of natural soil specimens cured under two different conditions. For specimens cured under atmospheric pressure, the natural soil exhibits low shear strength with an average value of 22 kPa.

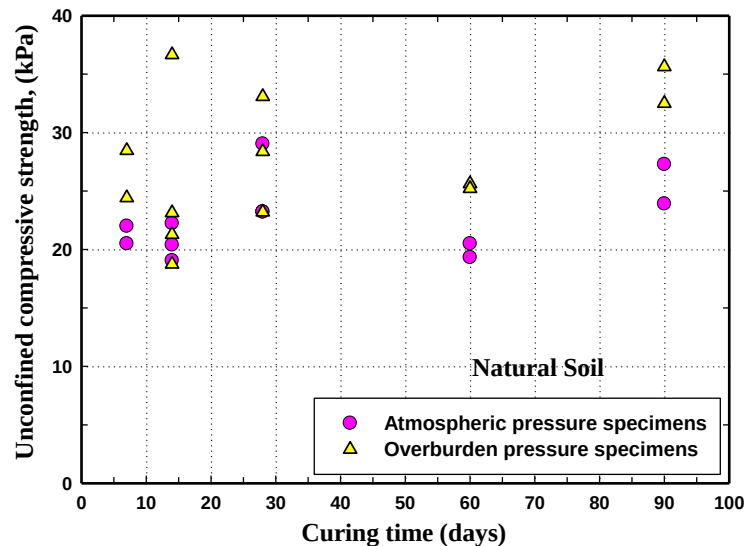


Fig. 3. Unconfined compressive strength of natural soil specimens for different curing times and conditions

Figs. 4 and 5 show the development of the unconfined compressive strength (UCS) over time for stabilized soil specimens treated with various binder contents. Table 6 summarizes the enhancement of soil strength for two curing conditions. Compared to natural soil, the increase in UCS of stabilized soil can be expressed as the UCS ratio, defined as the ratio of the UCS of stabilized soil to the average UCS of natural soil. For specimens cured under atmospheric pressure, the addition of 1% binder improved soil strength up to 5.6 times during the 28 days of curing, with only slight increases observed (6.2 times) at longer curing times of 60 and 98 days as illustrated in Fig. 4 and Table 6. Increasing the binder content to 2% further enhanced soil strength up to 8.5 times during the 28 days of curing, with a slight increase observed at longer curing times (9.5 times). Further increases in binder content to 4% and 7% (Fig. 5) resulted in higher soil strength due to the combined effects of increased binder content and curing time. For 4% binder content, UCS increased during the 28 days of curing up to 12.4 times, with a slight improvement to 15.5 times at 90 days of curing. A similar observation was found for 7% binder content, which showed rapid improvement in soil strength during 28 days (18.1 times) compared a slight increase up to 19.7 times at longer curing times.

In general, the addition of binder at different dosages improves the shear strength of the soil. The strength increases as both binder content and curing time increase. The dominant gain in soil strength was observed after 28 days of curing, compared to only slight improvement at longer curing times. This improvement was attributed to the formation of cementitious hydration products, primarily calcium silicate hydrate (CSH) and calcium aluminate silicate hydrate (CASH) gels, produced through hydration and pozzolanic reactions. A similar trend of increasing soil strength after treatment and over time is consistent with previous studies that used cement-based stabilizer to improve different soil types [7–15, 29–30, 48].

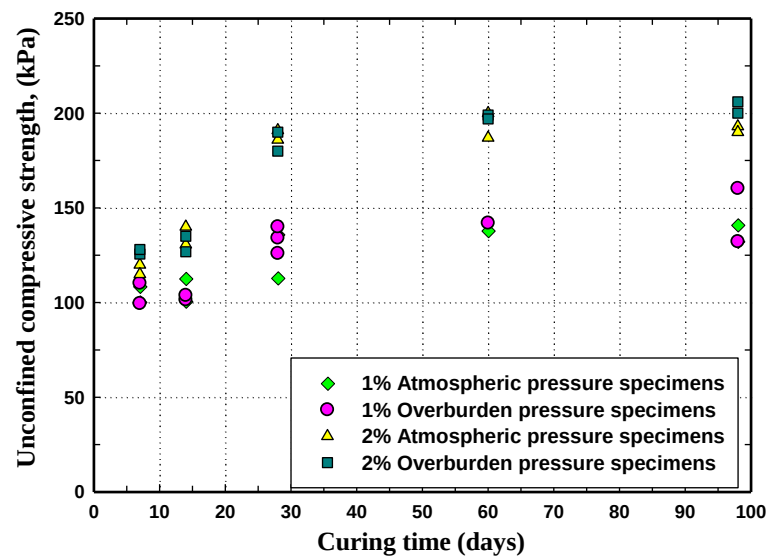


Fig. 4. Unconfined compressive strength of stabilized soil specimens treated with 1% and 2% binder content for different curing times and conditions.

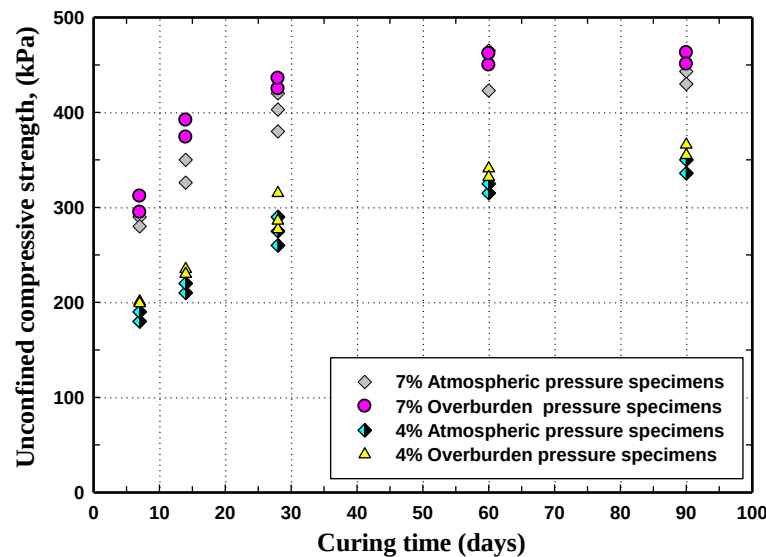


Fig. 5. Unconfined compressive strength of stabilized soil specimens treated with 4% and 7% binder content for different curing times and conditions

Table 6. Summarizes the increases in soil strength due to the addition of various binder content, curing time, and applied overburden pressure

Binder content %	Curing time, Days	Atmospheric Pressure Specimens		Overburden Pressure Specimens		Effect of Applying Overburden Stress on percentage increases in the UCS
		Unconfined compressive strength (UCS), KPa				
		Mean value	UCS ratio	Mean value	UCS ratio	
0	7	21.3	1	23.7	1.1	12
	14	20.6	1	25.0	1.2	21
	28	23.2	1	28.2	1.3	22
	60	20.5	1	25.4	1.2	28
	90	25.6	1	34.1	1.6	33
1	7	104.5	4.7	105.5	4.8	1
	14	106.7	4.8	108.7	4.9	4
	28	124.5	5.6	133.4	6.0	7
	60	136.5	6.2	145.3	6.6	6
	98	136.7	6.2	146.2	6.6	7

Binder content %	Curing time, Days	Atmospheric Pressure Specimens		Overburden Pressure Specimens		Effect of Applying Overburden Stress on percentage increases in the UCS
		Unconfined compressive strength (UCS), KPa				
		Mean value	UCS ratio	Mean value	UCS ratio	
2	7	117.5	5.3	122.0	5.5	4
	14	135.3	6.1	142.0	6.4	5
	28	188.5	8.5	196.0	8.9	4
	60	189.0	8.5	197.0	8.9	7
	98	190.0	8.5	199.0	9.0	5
4	7	185.0	8.5	196.0	8.9	6
	14	215.0	9.7	228.0	10.3	6
	28	275.0	12.4	292.7	13.2	6
	60	320.0	14.5	340.0	15.4	6
	90	343.0	15.5	366.0	16.6	7
7	7	285.0	12.9	303.5	13.7	6
	14	347.0	15.7	371.0	16.8	7
	28	401.0	18.1	430.5	19.5	7
	60	444.0	20.1	466.0	21.1	5
	90	436.5	19.7	457.0	20.7	5

3.2.2 Strain at Failure

The strain at failure, obtained from the stress–strain curves at peak UCS, is presented in Table 7. The failure strain ratio is defined as the ratio of stabilized soil failure strain to that of natural soil. For specimens cured under atmospheric pressure, the natural soil shows a high failure strain, with an average value of 28%. The quantitative analysis indicates that the addition of various binder amounts significantly reduces failure strain, particularly at lower binder contents (1%, 2%, and 4%). Further increases in binder content up to 7% have less impact on reducing the failure strain. The reduction in failure strain becomes less pronounced at 7% binder content compared to 4%. Moreover, the failure strain decreases over time.

The reduction in failure strain can be attributed to the increased formation of cementing products. In general, for soil treated with 7% binder content, the average failure strain at 90 days is lower than that at 7 days. The decrease in failure strain leads to more brittle behaviour in specimens treated with 7% binder content compared to those with lower binder contents, which exhibit more ductile behavior. Consequently, the mode of failure gradually transitions from ductile to brittle as the binder content and curing time increase. The general trends of reduction in failure strain due to the addition of different binder types and over time for stabilizing different soil types are consistent with previous studies [7–15, 30, 48].

Table 7. Summarizes the strain at versus binder content and curing time for two different curing conditions

Binder content %	Curing time, Days	Atmospheric Pressure Specimens		Overburden Pressure Specimens		Effect of Applying Overburden Stress on reduction in failure strain, %
		Strain at failure %				
		Mean value	Failure strain ratio	Mean value	Failure strain ratio	
0	7	28.15	1	24.87	0.88	11.6
	14	27.90	1	23.48	0.83	15.8
	28	27.62	1	23.60	0.84	14.6
	60	28.67	1	22.50	0.80	21.5
	90	27.52	1	22.01	0.78	20.0
1	7	17.00	0.61	16.00	0.57	5.9
	14	14.00	0.50	13.00	0.46	7.1
	28	12.37	0.44	11.29	0.40	8.8
	60	11.78	0.42	11.19	0.40	5.1
	98	11.24	0.40	10.01	0.36	10.9

Binder content %	Curing time, Days	Atmospheric Pressure Specimens		Overburden Pressure Specimens		Effect of Applying Overburden Stress on reduction in failure strain, %
		Strain at failure %				
		Mean value	Failure strain ratio	Mean value	Failure strain ratio	
2	98	11.24	0.40	10.01	0.36	10.9
	7	9.13	0.33	9.02	0.32	1.2
	14	8.58	0.31	8.51	0.30	0.7
	28	8.65	0.31	8.61	0.31	0.5
	60	8.05	0.29	7.50	0.27	6.8
	98	7.46	0.27	7.07	0.25	5.2
4	7	4.72	0.17	4.52	0.16	4.3
	14	4.56	0.16	4.40	0.16	3.5
	28	4.25	0.15	4.04	0.14	5.0
	60	3.93	0.14	3.67	0.13	6.7
	90	3.61	0.13	3.32	0.12	8.1
	7	4.54	0.16	4.54	0.16	0.0
7	14	4.12	0.15	4.01	0.14	2.6
	28	4.01	0.14	3.87	0.14	3.4
	60	3.97	0.14	3.57	0.13	10.0
	90	2.66	0.10	2.39	0.09	10.0

3.2.3 Water Content and Density

For the natural soil, Fig. 6 shows the variation in soil water content with curing time for specimens cured under two different conditions. The water content of the specimens was measured for the natural soil after specimen preparation (before curing) and after curing (after UCS testing). For specimens cured under atmospheric pressure, almost no change in soil water content was observed for before and after curing (Fig. 6A).

For stabilized soil specimens cured under two different curing conditions, Figs. 7, 8, 9 and 10 show the variation in soil water content with curing time for specimens stabilized with 1%, 2%, 4% and 7% binder content, respectively. The soil water content was measured before treatment (prior to binder addition), after specimen preparation (before curing), and after curing (after UCS testing). Table 8 summarizes the impact of binder amounts and curing time on the soil water content under different curing conditions. The addition of various binder contents of 1%, 2%, 4%, and 7% immediately reduces the soil water content to 2%, 3.3%, 4.5%, and 6%, respectively. This effect was measured one hour after treatment. The percentage reduction in soil water content over time was calculated as the difference between the water content before and after curing, divided by the water content before curing. For specimens cured under atmospheric pressure, the results indicate that there is no significant reduction in soil water content over time (the percentage reduction less than 1%) as illustrated in Table 8. Since all prepared specimens were cured in a moist environment, the small variation in the water content of the prepared specimens before and after curing can be attributed to small evaporation that occurred during the UCS testing. The water content in the soil-binder mixture is primarily derived from the natural soil. Therefore, the initial reduction in soil water content can be attributed to the addition of dry binder, as well as hydration reactions between the binder and water [11-15].

The density of stabilized soil gradually increased with increasing binder content from 1% to 4%, with a slight increase observed when the binder content increased to 7% as shown in Fig. 11. The consumption of water during hydration and pozzolanic reactions results in the formation of a significant number of solid products within the soil matrix, thereby increasing the soil density after treatment. This densification is primarily attributed to the formation and deposition of CSH (calcium silicate hydrate) and CAH (calcium aluminate hydrate) gels generated during these reactions. These hydration products fill the pore voids, resulting in a denser soil structure. A similar trend of initial reduction in soil moisture content and increase in density for stabilized soils with cement-based stabilizer has been reported in previous studies [29-31].

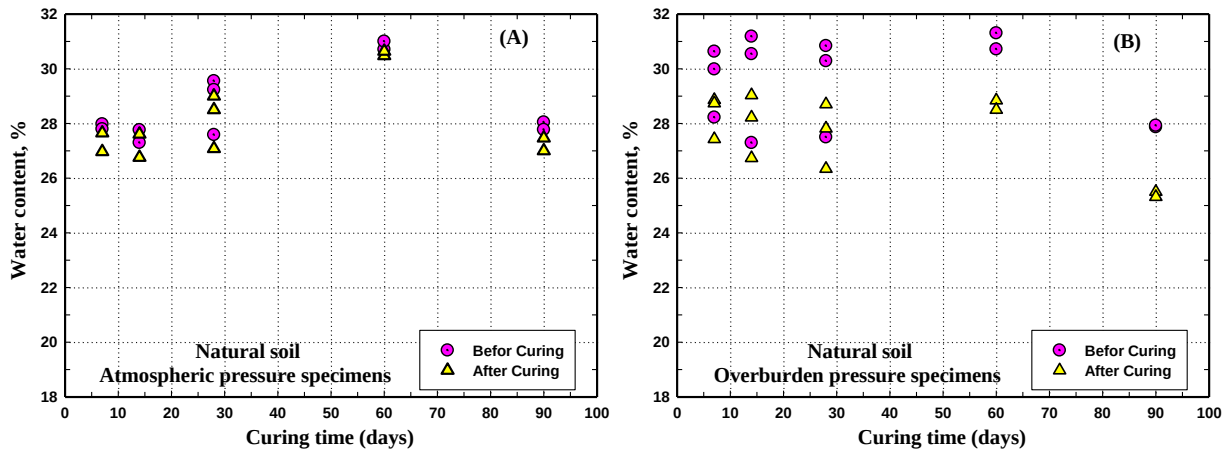


Fig. 6. Water content versus curing time for natural soil specimens under two different curing conditions

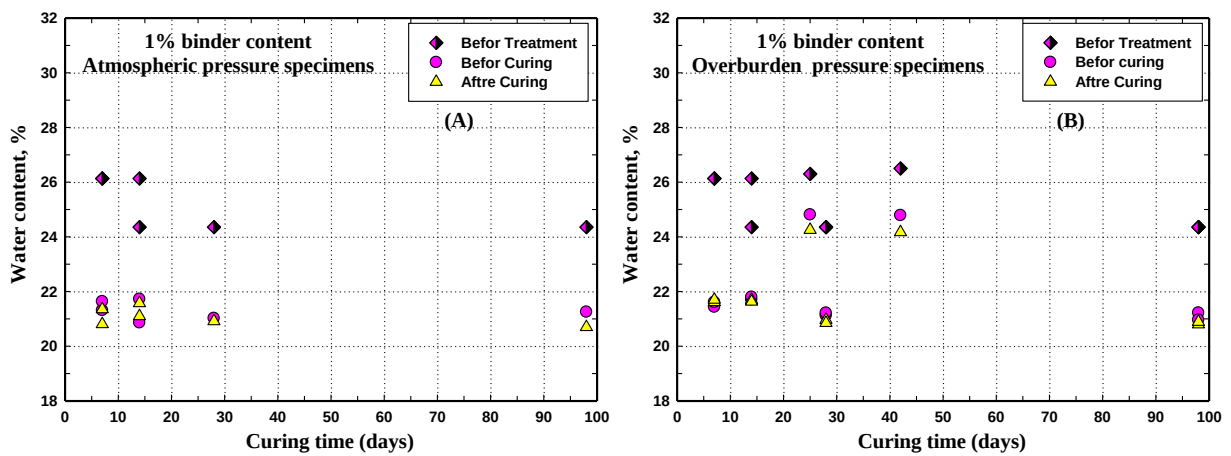


Fig. 7. Water content versus curing time for stabilized soil specimens treated with 1% binder content under two different curing conditions

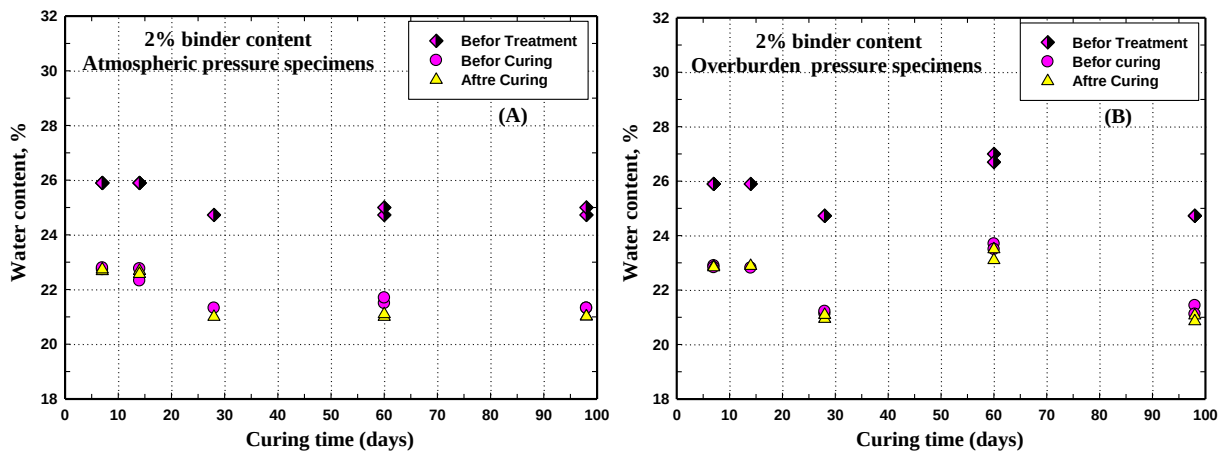


Fig. 8. Water content versus curing time for stabilized soil specimens treated with 2% binder content under two different curing conditions

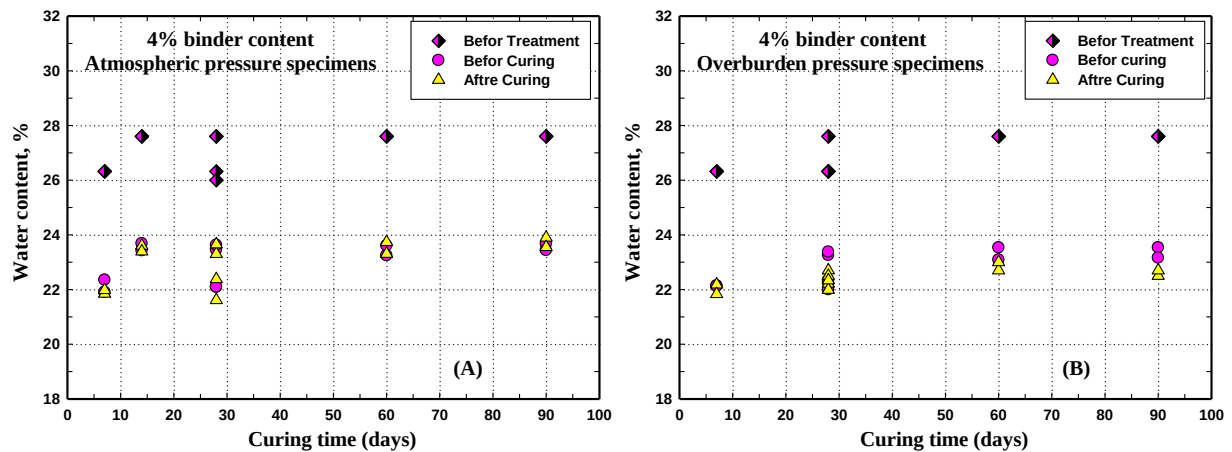


Fig. 9. Water content versus curing time for stabilized soil specimens treated with 4% binder content under two different curing conditions

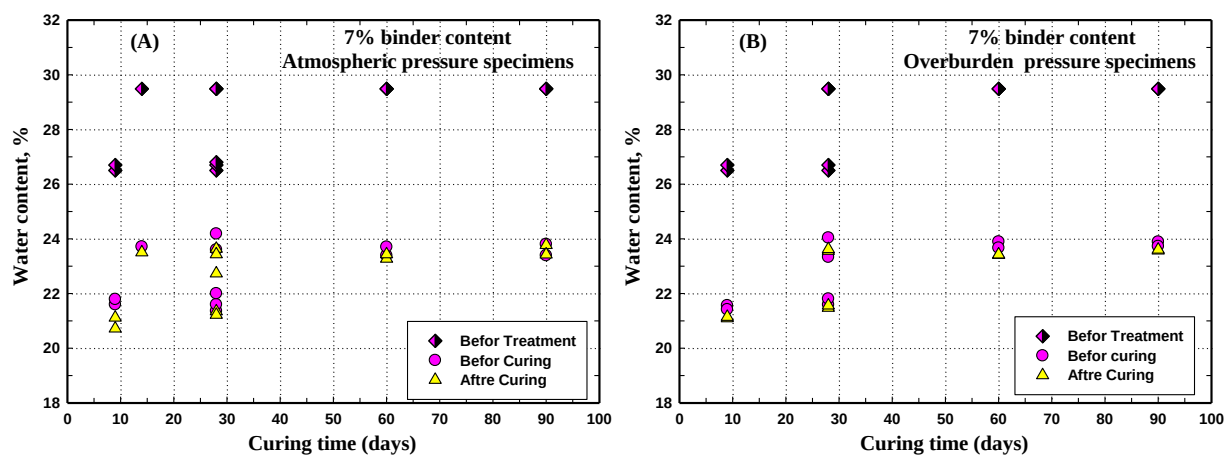


Fig. 10. Water content versus curing time for stabilized soil specimens treated with 7% binder content under two different curing conditions.

Table 8. Summarizes the impact of binder amount and curing time on soil water content for two curing conditions

Binder content, %	Curing time, Days	Atmospheric Pressure Specimens				Overburden Pressure Specimens				Effect of Binder content
		Water content, %								
		Mean value				Mean value				Initial reduction in Soil Water content, %
		Natural soil, ω_n	Before curing	After curing	Percent reduction in Water content	Natural soil, ω_n	Before curing	After curing	Percent reduction in Water content	
0	7	27.9	27.9	27.8	0.4	29.6	29.6	28.3	4	0
	14	27.5	27.5	27.4	0.5	29.7	29.7	28.0	6	
	28	28.8	28.8	28.5	1.0	29.5	29.5	27.6	6	
	60	30.9	30.9	30.6	1.0	31.0	31.0	28.7	8	
	90	27.9	27.8	27.5	1.0	27.9	27.9	25.4	9	
1	7	23.4	21.3	21.1	0.9	21.5	21.8	21.7	1	2
	14	23.7	21.5	21.3	0.7	21.8	21.9	21.7	1	
	28	23.8	21.0	20.9	0.5	23.0	21.2	20.9	1	
	98	23.8	21.1	20.9	0.9	20.9	21.1	20.8	1	

Binder content, %	Curing time, Days	Atmospheric Pressure Specimens				Overburden Pressure Specimens				Effect of Binder content	
		Water content, %									
		Mean value				Mean value				Initial reduction in Soil Water content, %	
		Natural soil, ω_n	Before curing	After curing	Percent reduction in Water content	Natural soil, ω_n	Before curing	After curing	Percent reduction in Water content		
2	7	25.9	22.8	22.7	0.4	25.9	23.0	22.9	1	3.3	
	14	25.9	22.6	22.5	0.4	25.3	23.2	22.9	1		
	28	24.7	21.3	21.0	1.5	24.7	21.4	21.0	2		
	60	24.9	21.2	21.1	0.7	26.8	23.6	23.3	1		
	98	24.9	21.4	21.2	0.9	24.7	21.3	21.0	1		
4	7	26.3	22.1	21.9	1.0	26.3	22.3	22.0	1	4.5	
	14	27.6	23.7	23.5	0.6	27.6					
	28	26.6	22.9	22.7	0.7	26.5	22.6	22.3	1		
	60	27.7	23.8	23.5	1.1	27.6	23.3	22.9	2		
	90	27.8	24.0	23.7	1.1	27.6	23.3	22.6	3		
7	7	26.7	21.7	21.4	1.4	26.5	21.1	21.1	0	6	
	14	29.5	23.7	23.5	0.9	29.5					
	28	27.6	22.5	22.5	0.3	27.6	22.6	22.6	0		
	60	29.5	23.5	23.4	0.8	29.5	23.4	23.4	0		
	90	29.5	23.7	23.6	0.4	29.5	23.6	23.6	0		

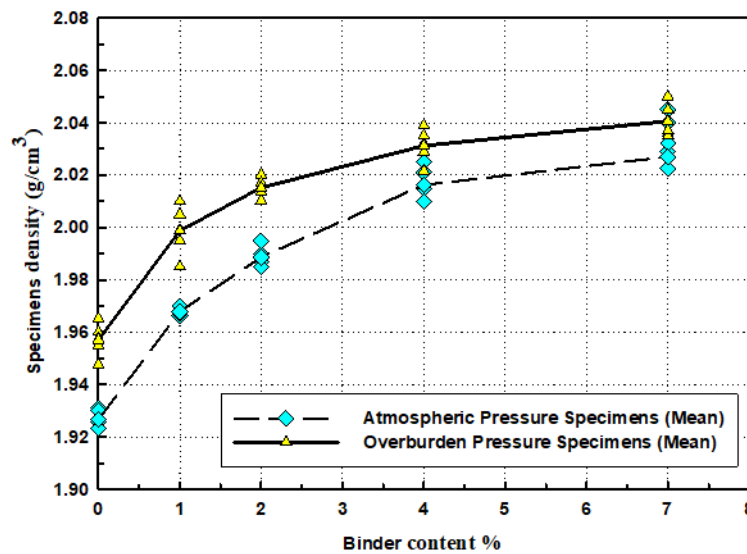


Fig. 11. Specimen density versus binder content for tow curing conditions.

3.3 Specimens Cured Under Overburden Pressure

3.3.1 Effect of Overburden Pressure on the UCS

The development of UCS over time for specimens cured under stress (10.5 kPa) shows a similar trend to that of specimens cured under atmospheric pressure (see Figs. 3, 4 and 5). The percentage increase in soil strength due to the effect of overburden stress was calculated as the ratio of the strength difference between specimens cured under overburden stress and those cured under atmospheric pressure to the strength of specimens cured under atmospheric pressure.

For natural soil, the quantitative analysis shows that the overburden stress results in gradual increases in soil strength over time, from 12% to 33% after 7 and 90 days, respectively. This improvement is attributed to the applied stress compressing the soil particles and allowing pore

water to drain from the upper side of the specimen. A similar observation of increased soil strength due to the application of overburden stress was reported by [22].

For stabilized soil, a similar trend of enhanced soil strength during the first 28 days of curing was observed for specimens cured under stress. The UCS ratio of overburden-cured specimens is slightly higher compared to that of specimens cured under atmospheric pressure, as illustrated in Table 6. The increase in soil strength due to the application of overburden stress gradually increased with time. For instance, the addition of 1% binder content improved soil strength from 1% to 7% after 7 and 98 days of curing, respectively. The same observation was noted when the binder content increased to 2%, where the improvement in soil strength increased from 4% to 5% after 7 and 98 days of curing, respectively. In contrast, the improvement in soil strength ranged between 6% and 7% after 7 and 90 days when the binder amount increased to 4% and 7%. Similar observations were noted for stabilized soil specimens treated with different binder contents, where soil strength increased due to the application of overburden stress and longer curing times [1, 23–25].

Regarding the effect of overburden stress applied during the curing period, further increases in soil strength are mainly related to the applied stress. The applied stress causes compression within the stabilized soil matrix by reducing the spacing between binder grains and soil particles. This densification facilitates the buildup of bonds generated from binder hydration reactions [20]. The general trend for soil improvement under overburden stress is consistent with previous studies that used cement kiln dust and cement to stabilize different soil types [1, 19–25, 31].

3.3.2 Strain at Failure

The reduction in failure strain for specimens cured under an overburden stress of 10.5 kPa follows a trend similar to that observed for specimens cured under atmospheric pressure. However, the quantitative analysis shows that the failure strain ratio is consistently lower compared to the specimens cured under atmospheric pressure as illustrated in Table 7. To illustrate the effect of overburden stress on failure strain, the reduction in failure strain (in percentage) was calculated as the ratio of the difference in failure strain between specimens cured under overburden stress and those cured under atmospheric pressure to the failure strain of specimens cured under atmospheric pressure.

For natural soil, the failure strain decreased further over time due to the effect of overburden stress, with a reduction ranging from 11.8% to 20% after 7 and 90 days of curing, respectively. For stabilized soil with 1% binder content, the failure strain decreased over time from 5.9% to 10.9% after 7 and 98 days, respectively. When the binder content increased to 2%, the reduction ranged from 1.2% to 5.2% over the same curing times. Similar trends were observed for stabilized soils treated with 4% and 7% binder content, with a more pronounced effect compared to 1% and 2% binder content. However, the percent reduction in failure strain is more pronounced in natural soil specimens compared to stabilized soil specimens with various binder contents.

Generally, the overburden stress has an effect on further decreasing the failure strain compared to the specimens cured under atmospheric pressure. A similar observation has been found in previous studies that used cement kiln dust and cement to stabilize different soil types [1, 19–25].

3.3.3 Water content and Density

For the natural soil specimens, the quantitative analysis shows that the applied stress caused a gradual reduction in soil water content from 4% to 9% after 7 and 90 days of curing, respectively, as shown in Table 8. This reduction in soil water content was attributed to the effect of overburden stress applied over the curing time, which facilitates drainage during curing (Fig. 6B).

The applied stress caused a slight reduction in water content of stabilized soil treated with 1%, 2% and 4% binder content. In contrast, the applied stress had a negligible effect on water content when the binder content increased to 7% (Table 7). The results further demonstrate that the applied stress has a more significant influence on the water content of natural soil compared to stabilized soil specimens.

Hydration reactions of the binder caused an initial reduction in soil water content, while the further reduction over time was mainly attributed to the pozzolanic reactions occurring between the binder and the soil for specimens cured under atmospheric pressure. In contrast, for specimens cured under stress, the further reduction over time was attributed to both the pozzolanic reactions between the binder and the soil and the overburden stress which caused drainage during curing. This finding is consistent with other studies [12, 19, 24].

The density of stabilized soil cured under an overburden pressure of 10.5 kPa shows a similar increasing trend with binder content; however, the measured densities are higher than those of specimens cured without applied pressure, as illustrated in Fig. 11. The applied stress contributes to an increase in the density of both natural and stabilized soil specimens by reducing the pore spaces between soil particles. Hydration and pozzolanic reactions of the binder consume pore water and generate cementitious products that fill the pore spaces within the soil matrix, resulting in increased density after treatment. This general trend of increased soil density is consistent with previous studies [12, 13].

3.4 Relations Between UCS - Water Content/Plastic Limit Ratio and Stiffness (E_{50})

3.4.1 Relation Between UCS And Water Content/ Plastic Limit Ratio

The ratio of water content (ω) to the plastic limit (PL) of soil was used to explain correlation between water content and the unconfined compression strength after treatment for specimens subjected to two curing conditions (Fig. 12). The general trends show an increase in UCS accompanied with decreasing water-to-plastic limit ratio for various binder contents. A scattered trend is evident, particularly when the water-to-plastic limit ratio ranges between 1.1 and 1.2 for specimens cured under atmospheric pressure. In contrast, a similar scattered trend is noted when the water-to-plastic limit ratio is between 1 and 1.1 for specimens cured under stress (water contents become more closer to soil plastic limit). Previous studies have demonstrated that natural soils gain strength as their water content decreases from the liquid limit toward the plastic limit. Treated soils are considered to be within the plastic range when their shear strength is around 100 kPa [12].

Reference [33] reported that the remolded clay exhibits a shear strength around 200 kPa when the soil water content becomes near the plastic limit of the soil, and decreases to 2 kPa when the soil water content becomes closer to the liquid limit. Moreover, previous studies mentioned that soil strength varies from 110 to 170 kPa at the soil plastic limit. Therefore, stabilized soil is regarded as being within the plastic range (ω/PL more than 1) when its shear strength is around 100 kPa. A similar trend is consistent with previous studies for soil treated with various binder types [12, 34].

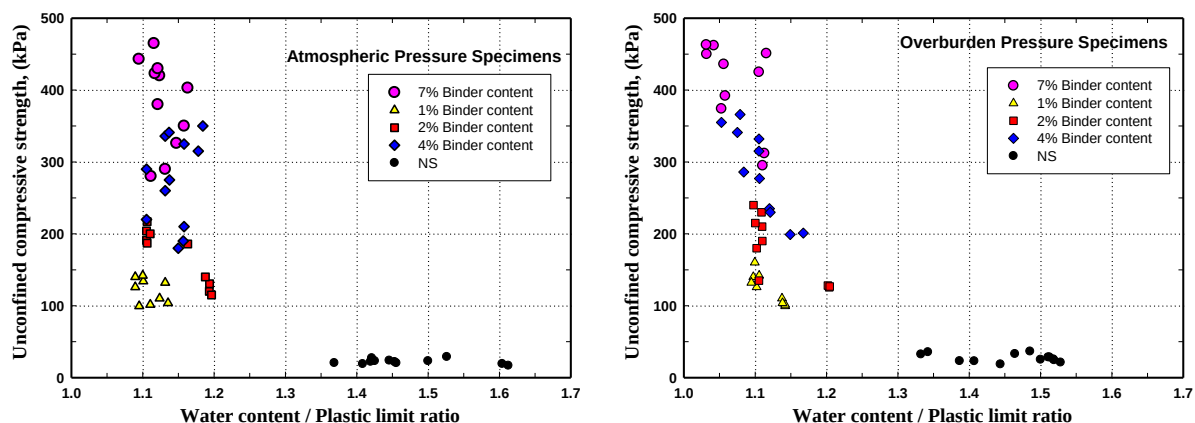


Fig. 12. Unconfined compressive strength versus water content/ plastic limit ratio for natural and stabilized soil specimens for two different curing times

3.3.2 Relations Between UCS And Strain at Failure

Fig. 13 shows the correlation between unconfined compressive strength (UCS) and failure strain for specimens cured under two different curing conditions. For specimens cured under atmospheric pressure, the natural soil shows low shear strength of about 23 kPa, with a high failure strain of about 28%. The addition of different binder contents (1%, 2%, 4%, and 7%) results in a reduction in failure strain to 13.3%, 8.5%, 4.2%, and 3.9%, respectively, accompanied by corresponding increases in soil strength of up to 120, 170, 265, and 400 kPa, respectively. Moreover, it was observed that further increases in unconfined compressive strength due to the addition of 7% binder content have less impact on reducing failure strain.

The correlation between unconfined compressive strength and failure strain has been reported in previous studies. Reference [12] found a significant reduction in failure strain occurs when soil strength is around 200 kPa or lower, whereas the reduction becomes less pronounced at higher strength levels for stabilized Swedish clay treated with various binders. For marine sediments treated with cement, lime, and fly ash, [35] proposed a correlation between unconfined compressive strength and failure strain as $q_u = (20-130)$ times the failure strain. Reference [37] showed that failure strain decreases with increasing unconfined compressive strength, and the relationship can be expressed as a power function between UCS and failure strain (failure strain = $1.06 q_u^{-0.62}$) for cement-treated zinc-contaminated clay. This observation is also consistent with previous studies on stabilized soil treated with various binder types [34, 36].

In contrast, the application of overburden stress reduces the failure strain to 23%, 12%, 8.1%, 4%, and 3.7% due to addition of 0%, 1%, 2%, 4% and 7% binder content respectively. This reduction is accompanied by increases in UCS of up to 33, 130, 180, 300, and 410 kPa, respectively. As discussed earlier, the application of overburden stress during curing further reduces failure strain, this reduction is accompanied by an increase in unconfined compressive strength. Generally, similar trends have been reported in previous studies for specimens cured under overburden stress compared to unstressed specimens [1, 19, 21-24].

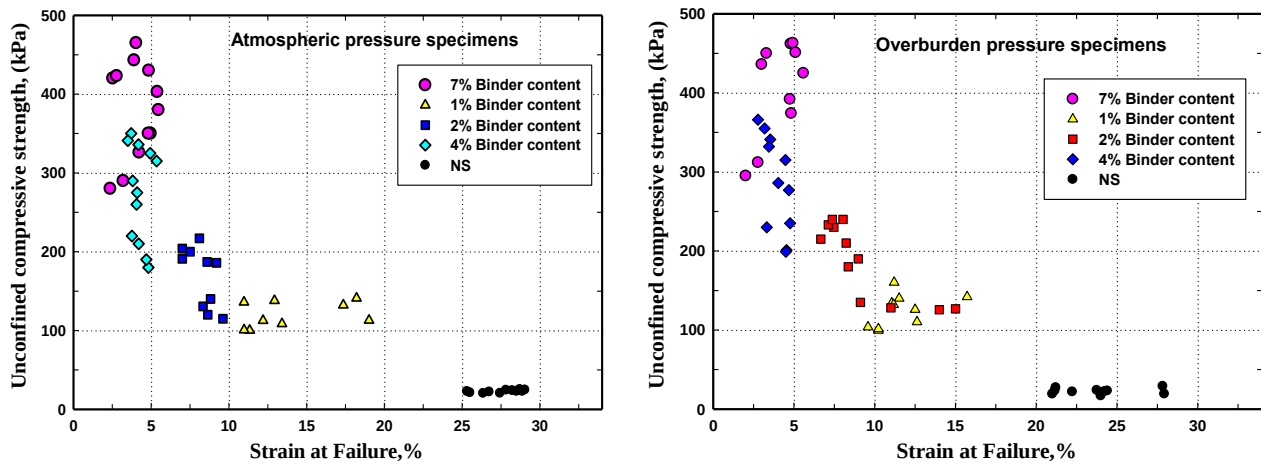


Fig. 13. Unconfined compressive strength versus failure strain for natural and stabilized soil specimens cured under different conditions

3.3.3 Relations Between UCS And the Stiffness

The stiffness of natural and stabilized soil was defined as the secant modulus of elasticity (E_{50}), which is determined from the stress-strain curve at half of the peak UCS value. Fig. 14 shows the correlation between the modulus of elasticity and the unconfined compressive strength for natural soil specimens cured under two curing conditions. Fig. 15 and 16 show the correlation between the modulus of elasticity and the unconfined compressive strength for stabilized soil cured under atmospheric and overburden stress, respectively. The relation between the secant modulus of elasticity and the UCS can be expressed by using the Eq (1):

$$E_{50} = k q_u \quad (1)$$

Where E_{50} : Secant modulus of elasticity, k : Strength coefficient, q_u : Unconfined compressive strength.

From Figs. 14, 15, and 16, it can be seen that there is a linear correlation between E_{50} and UCS. The upper and lower limits for the strength coefficient of natural and stabilized soil cured under different curing conditions are presented in Table 9. For specimens cured under atmospheric pressure (Fig. 15), the results show that soil stiffness increases with soil strength and is further enhanced by increasing binder content and curing time. This can be attributed to the formation of cementitious compounds, including C-S-H and C-A-H, generated through hydration and pozzolanic reactions. Higher binder contents lead to the production of more cementitious compounds. The increased formation and deposition of cementitious compounds fill the pore spaces, resulting in a denser soil and, consequently, increased stiffness. This trend is consistent with previous studies [9–14].

In contrast, the applied stress leads to a further enhancement in soil stiffness for both natural and stabilized soil specimens (Figs. 14 and 16). The enhancement in soil strength and stiffness has been reported for specimens cured under overburden stress compared to those cured under atmospheric pressure [19, 29, 47]. These effects are commonly attributed to compression during the formation of cementitious products. As discussed earlier, the applied stress decreases the pore space between soil particles, leading to a denser soil structure. The reduction in pore space improves contact points and enhances cementitious bonding. In addition, the applied stress can influence the distribution and orientation of hydration products, resulting in a more uniform and interconnected bonding system. Therefore, overburden stress during curing is a non-negligible factor in predicting the mechanical properties of stabilized soils. The general trend of soil improvement under overburden stress is consistent with previous studies that used cement-based stabilizers to improve different soil types [1, 19–25].

Table 9. Upper and lower value for the strength coefficient (k) for different curing condition

	Atmospheric Pressure Specimens		Overburden Pressure Specimens	
	k values			
	Upper limit	Lower limit	Upper limit	Lower limit
	Natural soil	6	9	7
Stabilized soil	13	64	13	74

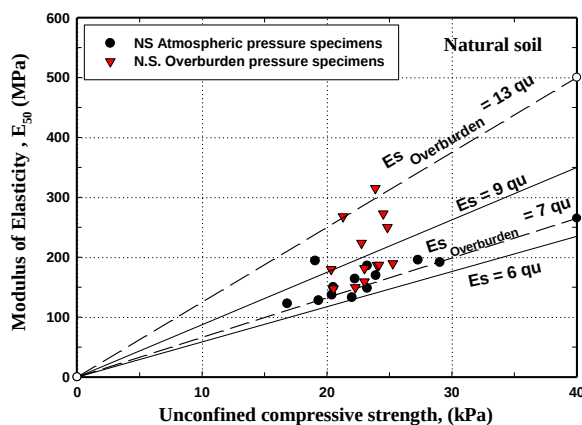


Fig. 14. Soil stiffness versus unconfined compressive strength for natural soil specimens cured under two conditions.

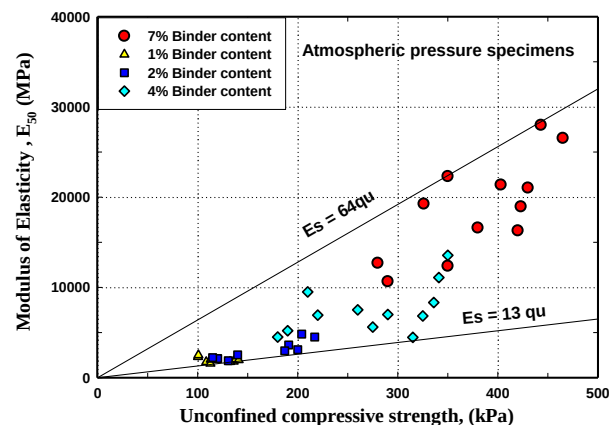


Fig. 15. Soil stiffness versus unconfined compressive strength for stabilized soil specimens cured under atmospheric pressure for all curing times.

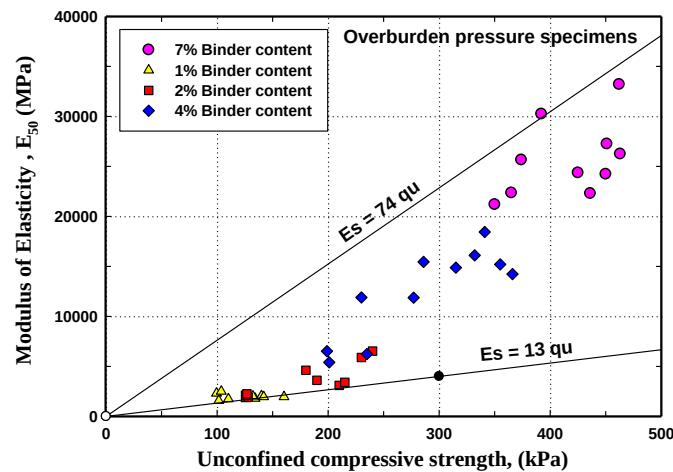


Fig. 16. Soil stiffness versus unconfined compressive strength for stabilized soil specimens cured under overburden pressure for all curing times

Many previous studies have reported varying conclusions regarding the correlation between the secant modulus of elasticity (E_{50}) and the unconfined compressive strength [34, 38-47]. A summary of previous studies is presented in Table 10. Overall, the findings of this study are generally consistent with those reported in previous research on cement-treated soils, and are particularly close to the findings reported by [44, 47].

Table 10. Upper and lower value for the strength coefficient (k) from pervious study

Author(s)	Type of Soil	Binder type	Binder content, %	Curing condition	k values	
					Upper limit	Lower limit
[34]	Silt, silty clay, and laterite soils	Cement	7% - 13%	Atmospheric Pressure	100	226
[38]	Marine clay	Cement	10% - 30%	Atmospheric Pressure	60	140
[39]	Soft clay	Cement	5% - 20%	Atmospheric Pressure	115	150
[40]	Very soft clay	Cement	5% - 35%	Atmospheric Pressure	96	129
[41]	Clay in Finland	Cement	3% - 37%	Atmospheric Pressure	100	200
[42]	Marine clay	Cement	10% - 20%	Atmospheric Pressure	150	275
[43]	Soft clay	Cement and cement kiln dust	10%-13%,	Atmospheric Pressure	99	159
[44]	Clayey silt	Cement	1%-7%	Atmospheric Pressure	16	85
[45]	Marine clay	Cement	10%- 30%	Atmospheric Pressure	30	169
[46]	Organic Low Plasticity Silts/Clays.	Cement	Water-cement ratio of 0.55	Atmospheric Pressure	39.9	57.7
[47]	Hight plasticity clay	Cement	50-200 kg/m3	Atmospheric Pressure	19	75
				Cured under pressure of 200-400 kPa	18	86

5. Conclusions

The effects of overburden stress over time on the strength and deformation characteristics of natural and stabilized fine-grained soils are presented and discussed in this study. The results are limited to an overburden stress of 10.5 kPa applied during different curing periods, and to stabilized lean clay soil with binder contents ranging from 1% to 7%. In addition, the vertical strain

during curing and suction effect were not measured in this study. Based on the results, the following conclusions can be made:

- The findings confirm that the general trend of the unconfined compressive strength and stiffness of soil is enhanced by increasing binder dosage and longer curing time. For specimens cured under overburden pressure, the unconfined compressive strength exhibits a pronounced gain in soil strength within the first 28 days of curing, followed by more gradual increases thereafter. In contrast, the specimens cured under atmospheric pressure (without stress) show a rapid increase during the first 28 days of curing with negligible improvement in soil strength beyond this period.
- Soil density increased after treatment with increasing binder dosages. A further increase in soil density was observed when curing stress was applied to both natural and stabilized soils.
- The results show that the failure strain decreases with increasing binder content and curing time, which gradually changes the failure behavior of the soil from plastic to brittle behavior. These improvements are mainly attributed to the production of cementitious compounds formed from hydration and pozzolanic reactions. Further reduction in failure strain was observed due to the application of overburden stress.
- Applying stress during curing led to further increases in soil strength and stiffness over time for both natural and stabilized soils. Significant improvement was noticed in natural soil at longer curing time, where the strength increased from 12% to 33% after 7 and 90 days, respectively compared with natural soil specimens cured under atmospheric pressure. For stabilized soils, additional improvements in soil strength were observed over time due to applied stress. Compared with stabilized soil specimens cured under atmospheric pressure, the additional increases in strength were approximately 6%, 5%, 6%, and 7% due to the addition of 1%, 2%, 4%, and 7% binder contents, respectively.
- The enhancement in unconfined compressive strength and stiffness due to the application of overburden stress is mainly attributed to physical densification. The reduction in pore space allows pore water to drain and increases particle contact points, resulting in higher density and lower water content. The increase in particle contact points leads to an improvement in the effectiveness of cementitious products such as C-S-H and C-A-H generated from cement hydration and pozzolanic reaction, which bind soil particles together and further enhance soil strength and stiffness.
- These findings confirm that the application of overburden stress during curing is a non-negligible factor controlling the strength and deformation behavior of fine-grained soils and should be considered essential for laboratory testing to achieve maximum strength development in practical geotechnical engineering applications.
- Further research is needed to investigate the effect of increasing overburden stress beyond 10.5 kPa, as well as to incorporate microstructural and mineralogical analyses (e.g., SEM and XRD) to better understand the mechanisms governing strength development and cementitious bonding in stabilized soils. Additionally, future studies should evaluate field applicability and long-term performance under realistic conditions, including non-uniform and time-dependent stress states, as well as durability aspects such as cyclic loading, wetting-drying cycles, and creep behavior.

References

- [1] Pigeot L, Dufour N, Soive A, Calissano H, Batilliot L, Dermenonville F, Loukili A. Influence of curing under stress on the geotechnical behavior of a treated silty soil. In *Geotechnical Engineering Challenges to Meet Current and Emerging Needs of Society*. 2024 Sep 17 (pp:2538-2542). CRC Press.
- [2] Lorenzo GA, Bergado DT. Fundamental parameters of cement-admixed clay—New approach. *Journal of geotechnical and geoenvironmental engineering*. 2004;130(10):1042-50. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:10\(10\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:10(10)).
- [3] Chompoorat T, Maikhun T, Likitlersuang S. Cement-improved lake bed sedimentary soil for road construction. *Proceedings of the Institution of Civil Engineers-Ground Improvement*. 2019;172(3):192-201. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:10\(1042\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:10(1042)).

- [4] Eskisar T. Influence of cement treatment on unconfined compressive strength and compressibility of lean clay with medium plasticity. *Arabian Journal for Science and Engineering*. 2015; 40(3):763-72. <https://doi.org/10.1007/s13369-015-1579-z>.
- [5] Liu L, Zhou A, Deng Y, Cui Y, Yu Z, Yu C. Strength performance of cement/slag-based stabilized soft clays. *Construction and Building Materials*. 2019; 211 :909-18. <https://doi.org/10.1016/j.conbuildmat.2019.03.256>.
- [6] Sanijya G, Bala M R, Samatha C P. Sustainable stabilization of native subgrade soil for low-volume roads using copper slag and cementitious materials. *Research on Engineering Structures & Materials*. 2025; 11(4): 1683-1702.
- [7] Miller GA, Azad S. Influence of soil type on stabilization with cement kiln dust. *Construction and building materials*. 2000;14(2):89-97. [https://doi.org/10.1016/S0950-0618\(00\)00007-6](https://doi.org/10.1016/S0950-0618(00)00007-6).
- [8] Miller GA, Zaman M. Field and laboratory evaluation of cement kiln dust as a soil stabilizer. *Transportation Research Record: Journal of the Transportation Research Board*. 2000; 1714 (1): 25-32. <https://doi.org/10.3141/1714-04>.
- [9] Ebrahimi A, Edil TB, Son YH. Effectiveness of cement kiln dust in stabilizing recycled base materials. *Journal of materials in civil engineering*. 2012; 24 (8): 1059-66. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0000472](https://doi.org/10.1061/(ASCE)MT.1943-5533.0000472).
- [10] Ismail AI, Belal ZL. Use of cement kiln dust on the engineering modification of soil materials, Nile Delta, Egypt. *Geotechnical and Geological Engineering*. 2016; 34(2):463-469. <https://doi.org/10.1007/s10706-015-9957-6>.
- [11] Yoon IH, Moon DH, Kim KW, Lee KY, Lee JH, Kim MG. Mechanism for the stabilization/solidification of arsenic-contaminated soils with Portland cement and cement kiln dust. *Journal of Environmental management*. 2010; 91(11):2322-8. <https://doi.org/10.1016/j.jenvman.2010.06.018>.
- [12] Åhnberg H, Johansson SE, Pihl H, Carlsson T. Stabilising effects of different binders in some Swedish soils. *Proceedings of the Institution of Civil Engineers-Ground Improvement*. 2003 Jan;7(1):9-23. <https://doi.org/10.1680/grim.2003.7.1.9>.
- [13] Chew SH, Kamruzzaman AH, Lee FH. Physicochemical and engineering behavior of cement treated clays. *Journal of geotechnical and geoenvironmental engineering*. 2004; 130(7):696-706. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:7\(696\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:7(696)).
- [14] Ranaivomanana H, Razakamanantsoa A, Amiri O. Effects of cement treatment on microstructural, hydraulic, and mechanical properties of compacted soils: Characterization and modeling. *International Journal of Geomechanics*. 2018; 18 (9) :04018106. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0001248](https://doi.org/10.1061/(ASCE)GM.1943-5622.0001248).
- [15] Pakbaz MS, Alipour R. Influence of cement addition on the geotechnical properties of an Iranian clay. *Applied Clay Science*. 2012; 67:1-4. <https://doi.org/10.1016/j.clay.2012.07.006>.
- [16] Rouhanifar S, Afrazi M, Fakhimi A, Yazdani M. Strength and deformation behaviour of sand-rubber mixture. *International Journal of Geotechnical Engineering*. 2021;15(9):1078-1092. <https://doi.org/10.1080/19386362.2020.1812193>.
- [17] Afrazi M, Yazdani M. Determination of the effect of soil particle size distribution on the shear behavior of sand. *Journal of Advanced Engineering and Computation*. 2021;5(2):125-134. DOI: <http://dx.doi.org/10.25073/jaec.202152.331>.
- [18] Al-Jabban W, Laue J, Knutsson S, Al-Ansari N. Briefing: Common laboratory procedures to prepare and cure stabilised soil specimens: a short review. *Geotechnical Research*. 2020; 7 (1): 3–10, doi: <https://doi.org/10.1680/jgere.19.00035>.
- [19] Åhnberg H, Bengtsson PE, Holm G. Effect of initial loading on the strength of stabilised peat. *Proceedings of the Institution of Civil Engineers-Ground Improvement*. 2001; 5(1):35-40. <https://doi.org/10.1680/grim.2001.5.1.35>.
- [20] Åhnberg H. Strength of stabilised soil-a laboratory study on clays and organic soils stabilised with different types of binder. Ph.D. thesis, Lund University, Lund, Sweden, 2006.
- [21] Zhao Y, Taheri A, Karakus M, Deng A, Guo L. The Effect of Curing under Applied Stress on the Mechanical Performance of Cement Paste Backfill. *Minerals*. 2021; 11(10):1107. <https://doi.org/10.3390/min11101107>.
- [22] He X, Chen Y, Wan Y, Liu L, Xue Q. Effect of curing stress on compression behavior of cement-treated dredged sediment. *International Journal of Geomechanics*. 2020; 20(11):04020204. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0001857](https://doi.org/10.1061/(ASCE)GM.1943-5622.0001857).
- [23] Åhnberg H. On yield stresses and the influence of curing stresses on stress paths and strength measured in triaxial testing of stabilized soils. *Canadian Geotechnical Journal*. 2007; 44(1):54-66. <https://doi.org/10.1139/t06-096>.

- [24] Shibi T, Ohtsuka Y. Influence of applying overburden stress during curing on the unconfined compressive strength of cement-stabilized clay. *Soils and Foundations*. 2021; 61(4):1123-1131. <https://doi.org/10.1016/j.sandf.2021.03.007>.
- [25] Haritwal DK, Agarwal BK, Reddy AV, Kumar S. Influence of Curing Stress and Curing Time on Unconfined Compressive Strength Behaviour of Cemented Clay. *Proceedings of the Indian Geotechnical Conference 2019: IGC-2019 Volume III* (pp. 87-97), Springer, Singapore, 25 April, 2021. https://doi.org/10.1007/978-981-33-6444-8_8.
- [26] ASTM D2487-11. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM International, West Conshohocken, PA, USA. 2018.
- [27] Al-Jabban W, Laue J, Knutsson S, Al-Ansari N. Effect of Disintegration Times of the Homogeneity of Soil prior to Treatment. *Applied Sciences*. 2019; 9(22):4791. <https://doi.org/10.3390/app9224791>.
- [28] ASTM D2166/D2166M-16. Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. ASTM International: West Conshohocken, PA, USA. 2016.
- [29] Rothhämel M, Al-Jabban W, Laue J. Influence of low temperature curing and freeze-thaw-cycles on the stiffness of silt stabilised with hydraulic binder. *Proceedings of the Deutsche Gesellschaft für Geotechnik (DGGT)* (pp. 432-437), Fachsektionstage Geotechnik, Würzburg, Germany, October 29-30, 2019.
- [30] Ziagharib A, Maurice C, Macsik J, Jia Q, Laue J. Performance of Excavated Sulfur-Rich Soil Stabilized with Binder: A Field Study of Mixing Efficiency. *Advances in Civil Engineering*. 2023;2023(1):5446332. <https://doi.org/10.1155/2023/5446332>.
- [31] Consoli NC, Rotta GV, Prietto PDM. Yielding–compressibility–strength relationship for an artificially cemented soil cured under stress. *Geotechnique*. 2006; 56 (1):69–72. doi: <https://doi.org/10.1680/geot.2006.56.1.69>.
- [32] Sariosseiri F, Muhunthan B. Effect of cement treatment on geotechnical properties of some Washington State soils. *Engineering geology*. 2009;104(1-2):119-25.
- [33] Wood DM. *Soil behaviour and critical state soil mechanics*. Cambridge university press; 1990.
- [34] Rashid AS, Kalatehjari R, Noor NM, Yaacob H, Moayed H, Sing LK. Relationship between liquidity index and stabilized strength of local subgrade materials in a tropical area. *Measurement*. 2014; 55:231-237. <https://doi.org/10.1016/j.measurement.2014.05.018>.
- [35] Wang D, Abriak NE, Zentar R. Strength and deformation properties of Dunkirk marine sediments solidified with cement, lime and fly ash. *Engineering Geology*. 2013; 166:90-99. <https://doi.org/10.1016/j.enggeo.2013.09.007>.
- [36] Ignat R. Field and laboratory tests of laterally loaded rows of lime-cement columns. PhD thesis, Royal Institute of Technology (KTH), Stockholm, Sweden, 2015.
- [37] Du YJ, Wei ML, Jin F, Liu ZB. Stress–strain relation and strength characteristics of cement treated zinc-contaminated clay. *Engineering Geology*. 2013; 167:20-26. <https://doi.org/10.1016/j.enggeo.2013.10.005>.
- [38] Lee FH, Lee Y, Chew SH, Yong KY. Strength and modulus of marine clay-cement mixes. *Journal of geotechnical and geoenvironmental engineering*. 2005;131(2):178-86. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2005\)131:2\(178\)](https://doi.org/10.1061/(ASCE)1090-0241(2005)131:2(178)).
- [39] Lorenzo GA, Bergado DT. Fundamental characteristics of cement-admixed clay in deep mixing. *Journal of materials in civil engineering*. 2006;18(2):161-74. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2006\)18:2\(16\)](https://doi.org/10.1061/(ASCE)0899-1561(2006)18:2(16)).
- [40] Jongpradist P, Jumlongrach N, Youwai S, Chucheeprakul S. Influence of fly ash on unconfined compressive strength of cement-admixed clay at high water content. *Journal of Materials in Civil Engineering*. 2010;22(1):49-58. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2010\)22:1\(49\)](https://doi.org/10.1061/(ASCE)0899-1561(2010)22:1(49)).
- [41] Hassan M. Engineering characteristics of cement stabilized soft Finnish clay–A laboratory study. Licentiate’s thesis, Helsinki University of Technology, Helsinki, Finland. 2009.
- [42] Dingwen Z, Libin F, Songyu L, Yongfeng D. Experimental investigation of unconfined compression strength and stiffness of cement treated salt-rich clay. *Marine Georesources & Geotechnology*. 2013;31(4):360-74. <https://doi.org/10.1080/1064119X.2012.690826>.
- [43] Yoobanpot N, Jamsawang P, Horpibulsuk S. Strength behavior and microstructural characteristics of soft clay stabilized with cement kiln dust and fly ash residue. *Applied Clay Science*. 2017; 141:146-156. <https://doi.org/10.1016/j.clay.2017.02.028>.
- [44] Al-Jabban W, Laue J, Knutsson S, Al-Ansari N. A comparative evaluation of cement and by-product petrit T in soil stabilization. *Applied Sciences*. 2019;9(23):5238. <https://doi.org/10.3390/app9235238>.
- [45] Kang GO, Tsuchida T, Kim YS. Strength and stiffness of cement-treated marine dredged clay at various curing stages. *Construction and Building Materials*. 2017; 132:71-84. <https://doi.org/10.1016/j.conbuildmat.2016.11.124>.
- [46] Cai H, Chen C, Wei S, Zhu S. Strength development of cemented soil cured in water-air conditions at varied temperatures: Experimental investigation and model characterization. *Journal of Materials in Civil Engineering*. 2023; 35(3):04022466. [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.00046](https://doi.org/10.1061/(ASCE)MT.1943-5533.00046).

- [47] Qiu C, Li Y, Li X, Xu G, Zhang D. Effects of High Curing Pressure on the Unconfined Compressive Strength of Cement-Stabilized Bottom Sediments with High Water Content. *Buildings*. 2025; 15(16):2869. <https://doi.org/10.3390/buildings15162869>.
- [48] Ziagharib A, Macsik J, Jia Q, Maurice C. Factors influencing the effectiveness of acid sulfate soil stabilization with cement-based binders. *Bulletin of Engineering Geology and the Environment*. 2026; 85(3):155. <https://doi.org/10.1007/s10064-026-04784-8>.

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Experimental evaluation of the mechanical performance and compressive stress–strain response of normal-strength concrete incorporating densified silica fume and local materials

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Abstract

In this study, the effect of densified silica fume on the fresh properties, mechanical performance, and ascending compressive stress-strain response of normal-strength concrete using local Iraqi materials was investigated. Densified Sika® Fume S92 D was used as the replacement for Portland cement at 10% and 15% by total binder mass. Three mixtures were studied: control concrete (NC), concrete containing 10% silica fume (SF10), and concrete containing 15% silica fume (SF15). The total binder content and water-to-binder ratio were kept at 350 kg/m³ and 0.55, respectively, without chemical admixtures. The slump increased from 130 mm for NC to 180 mm for SF10 and 160 mm for SF15. While this is in contrast to the commonly reported decrease in workability of silica fume, it could be due to the combination of saturated surface dry aggregates, high effective water content, very fine Zone 4 sand, and an incomplete deagglomeration of the densified silica fume. No visible bleeding or aggregate segregation was observed during the workability test. The compressive and splitting tensile strengths of SF15 increased by 22.6% and 24.0%, respectively, compared to NC. The peak compressive stress of the instrumented cylinders increased from 24.0 MPa for NC to 28.2 MPa for SF10 and 33.8 MPa for SF15, while the corresponding peak strain was increased from 0.00102 to 0.00116 and 0.00126. The secant modulus at 40% of peak stress also increased from 32.3 GPa for NC to 36.0 GPa for SF10 and 42.8 GPa for SF15. The descending branches of the SF10 and SF15 curves were not captured reliably, and therefore, no reliable conclusions could be drawn regarding ductility or post-peak behavior. In this range, SF15 had the best overall mechanical performance.

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1. Introduction

Rapid urban development, population growth, and expanding infrastructure are factors that are driving the global demand for concrete. Thus, there has been extensive research on the improved performance and material efficiency of concrete when additional cementitious ingredients and other minerals are added.

In the 1970s, silica fume became a key mineral to make denser and stronger cementitious compounds in concrete by virtue of its very fine particles, which can fill the voids between cement grains and improve particle packing. In the presence of calcium hydroxide, silica reacts with the cement paste and forms an additional C-S-H gel. These processes densify and refine the interfacial transition zone (ITZ) between the cement paste and aggregate [1-3]. This microstructural

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refinement can lead to stronger compressive and tensile strength and stiffness before peak. As the compressive stress-strain response is a fundamental input to structural analysis and constitutive modeling, the effect of silica fume on peak stress, peak strain, and pre-peak deformation should be examined in-depth [4,5].

Silica fume normally enhances the cohesiveness of the mixture and water consumption due to the relatively small size and high specific surface area of the particles. When the water content and chemical-admixture dosage do not differ, silica fume is generally used to reduce slump and slump retention. The effect is dependent on the silica-fume form, degree of deagglomeration, cement/aggregate properties, water-to-binder ratio, and the order of mixing. Therefore, high-range water-reducing admixtures are usually used to improve dispersion and workability of the material [1,6-8]. In hardened concrete, silica fume can improve tensile strength and resistance to transport-related deterioration by refining pores and strengthening ITZ. The C-S-H gel is produced by the pozzolanic reaction of reactive silica with calcium hydroxide, and it increases matrix density and mechanical resistance in a matrix [3,8-10].

However, the performance of silica-fume concrete depends on the cement, aggregates, silica-fume product, mixture proportions, dispersion process, and curing process. As a result, results obtained using one constituent-material system cannot be generalized to another. In the present work, two commonly used replacement levels of densified silica fume were investigated in normal-strength concrete made from locally available Iraqi materials with emphasis on fresh state behavior and strength development, as well as on the ascending compressive stress-strain response at a constant nominal water-to-binder ratio.

1.1 Research Gap and Contribution

Although silica-fume concrete has been investigated thoroughly, the fresh and mechanical response of normal-strength concrete is strongly dependent on cement, aggregates, water content, silica-fume form, and mixing process. The interaction of densified silica fume with locally available Iraqi constituent materials, particularly the Zone 4 fine aggregate used in the present work, has not been adequately demonstrated by a combined evaluation of workability, strength development, splitting tensile strength, and compressive stress-strain behavior. This study provides a controlled experimental dataset for normal-strength concrete produced with local materials and two practical silica-fume replacement levels.

The study examined the compatibility of densified silica fume with local cement and aggregates at a constant total cementitious material content (cement + silica fume) and water-to-binder ratio. The fresh-state slump, compressive-strength development, splitting tensile strength, ascending compressive stress-strain response, peak stress, peak strain, and apparent pre-peak stiffness were assessed. The post-peak branches of silica-fume mixtures were not adequately captured. The stress-strain results were only given for the increasing response and peak parameters.

1.2 Objective

- To evaluate the impact of 10% and 15% densified silica fume replacement on the measured slump of normal-strength concrete made of local materials;
- To determine the effects of silica fume on compressive-strength development at 7 and 28 days
- To evaluate the 28-day splitting tensile strength and its relationship with compressive strength.
- To investigate the ascending compressive stress-strain response, peak stress, peak strain, and apparent secant modulus of the mixtures; and assess the trade-off between fresh-state flowability and hardened mechanical properties within the considered replacement range.

2. Materials and Experimental Program

2.1 Constituent Materials

2.1.1 Cementitious Materials

Ordinary Portland Cement (OPC) manufactured by Saman Cement and supplied locally in Najaf, Iraq, was used as the main binder during the whole study. The cement conforms with IQS No. 5/2019 of the Iraqi standard. The manufacturer reported chemical composition of 21.0% SiO₂, 5.0% Al₂O₃, 3.8% Fe₂O₃, 63.4% CaO, 2.3% MgO, 2.5% SO₃, 0.22% Na₂O, 0.50% K₂O, and 1.6% loss on ignition (LOI). The silica fume (Sika® Fume S92 D) was used to replace cement at 10% and 15% in total binder mass. The product has a particle size below 1 µm, a specific gravity of 2.22, a bulk density of 130–430 kg/m³, and a specific surface area of 13,000–30,000 m²/kg. The physical and chemical properties of the cement and silica fume are summarized in Table 1.

Table 1. Physical and chemical properties of Ordinary Portland Cement and densified silica fume

Property	OPC	Silica fume (SF) (Sika® Fume S92 D)
Standard	IQS No. 5/2019	ASTM C1240
Cement type	Ordinary Portland Cement	Densified silica fume
Specific gravity	3.15*	2.22
Blaine fineness	317 m ² /kg	—
Particle size	—	<1 µm
Bulk density (kg/m ³)	—	130–430
Specific surface area (m ² /kg)	—	13,000–30,000
SiO ₂ (%)	21.0*	90–95
Al ₂ O ₃ (%)	5.0*	0.5–0.8
Fe ₂ O ₃ (%)	3.8*	0.07–1.9
CaO (%)	63.4*	0.50
MgO (%)	2.3*	0.04–0.90
Na ₂ O (%)	0.22*	0.30–0.42
K ₂ O (%)	0.50*	0.50–1.30
SO ₃ (%)	2.5*	—
LOI (%)	1.6*	≤6.0 **
Moisture content (%)	—	≤3.0 **

*Cement values are manufacturer-reported results for Saman Ordinary Portland Cement.

** Product-specific LOI and moisture values for Sika® Fume S92 D were not available; therefore, the ASTM C1240 acceptance limits are reported and are not presented as measured values.

2.1.2 Aggregate

The particle size distributions of the fine and coarse aggregates were determined by sieve analysis. As shown in Figures 1 and 2, the fine aggregate was within the grading limits of Zone 4, and the natural coarse aggregate was within the grading range of 5–14 mm according to Iraqi Standard Specification IQS No. 45/1984. The nominal maximum size of the coarse aggregate was 14 mm. The cylinder diameter-maximum aggregate size ratio was approximately 7.1 and 10.7 for the 100 and 150 mm diameter specimens, respectively. Both cylinder sizes provided dimensions well above the minimum commonly required relative to the nominal maximum aggregate size. The physical properties of the aggregates are summarized in Table 2.

Table 2. Physical properties of the fine and coarse aggregates.

Property	Fine aggregate	Coarse aggregate
Specific gravity	2.68	2.59
Water absorption (%)	0.64	1.03
Loose unit weight (kg/m ³)	1528	1396
Compacted unit weight (kg/m ³)	1689	1568

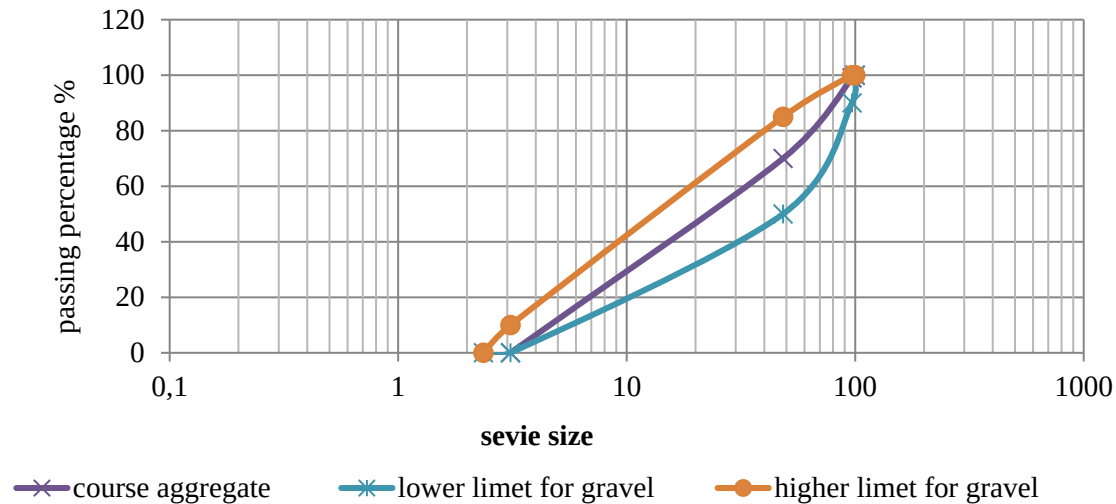


Fig. 1. Particle-size distribution of the coarse aggregate compared with the grading limits for Zone 5–14 according to IQS No. 45/1984

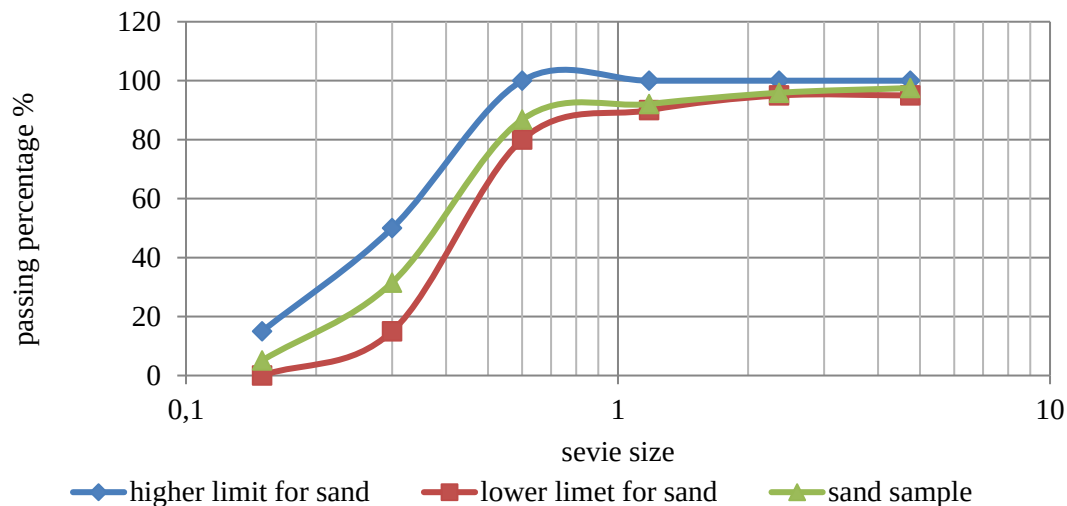


Fig. 2. Particle-size distribution of the fine aggregate compared with the grading limits for Zone 4 according to IQS No. 45/1984

2.2 Mixture Proportions

Three mixtures were considered:

- Control concrete with zero content of silica fume (NC);
- Concrete with 10% silica fume as a replacement of cement. (SF10).
- Concrete with 15% silica fume as a replacement of cementt (SF15).

The total binder content was kept at 350 kg/m³ for all mixtures. Therefore, silica fume was considered as a partial cement replacement and not as an additional powder. The water content was kept at 192.5 kg/m³ maintaining a constant water-to-binder ratio of 0.55. Before mixing, the fine and coarse aggregates were brought to the saturated surface-dry condition. Thus, no mixing water was needed for aggregate absorption, and the effective water-to-binder ratio was 0.55.

The water-to-binder ratio was maintained to study the effect of silica-fume replacement and particle packing without changing the nominal water content. The quantities in Table 3 are the actual batched proportions. Since cement was replaced with lower-density silica fume, the absolute

binder volume slightly increased. By using specific gravities of the model, the nominal absolute yields were around 1.036 m³ for NC, 1.041 m³ for SF10, and 1.043 m³ for SF15 for the nominal 1.0 m³ batch. The mass-based w/b ratio was therefore 0.55, but the uncorrected yield slightly decreased the total content of the constituent and weight per actual cubic meter. The effect was small and systematic, but it is acknowledged as a limitation when comparing mechanical properties.

Table 3. Concrete mixture proportions used in this study

Mixture	Cement (kg/m ³)	Silica fume (kg/m ³)	Fine aggregate *(kg/m ³)	Coarse aggregate (kg/m ³)*	Water (kg/m ³)	w/b
NC	350.0	0.0	670	1250	192.5	0.55
SF10	315.0	35.0	670	1250	192.5	0.55
SF15	297.5	52.5	670	1250	192.5	0.55

* The fine and coarse aggregates were used in the saturated surface-dry (SSD) condition. Hence, the reported water content is the effective mixing water, and the water-to-binder ratio was maintained at 0.55.

The replacement percentage was calculated as the mass of silica fume divided by the total binder mass, as shown in Eq. (1).

$$SF \text{ replacement } (\%) = [SF / (C + SF)] \times 100 \quad (1)$$

Where SF is the silica fume content in kg/m³, C is the cement content in kg/m³.

2.3 Mixing Procedure

The mixing procedure is shown in Figure 3. For each mixture the quantities were weighed before mixing to produce a nominal batch volume of 0.25 m³. Before batch mixing, the mixer drum was washed and wiped with a clean, dry towel to remove free water and maintain a uniformly damp internal surface. Cement and silica fume were first dry-blended to ensure a uniform distribution before being added to the total mixture.

2.4 Specimen Preparation and Curing

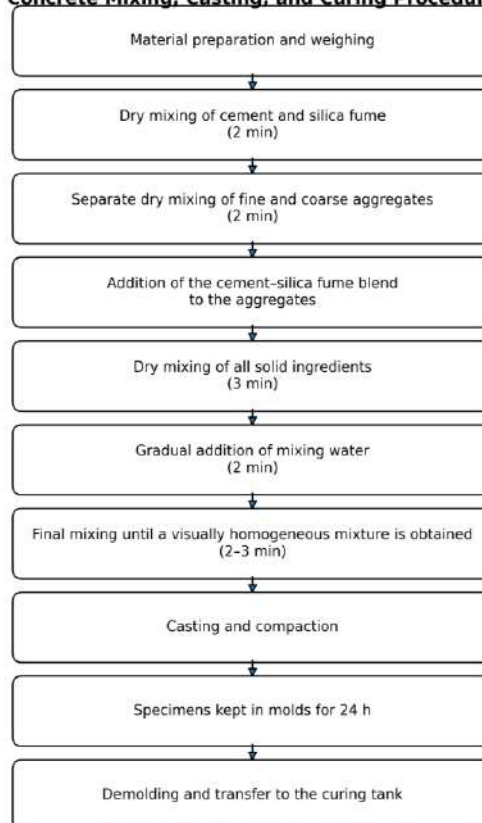
For each mixture, twelve cylinders were prepared. The steel molds were cleaned and lightly oiled before casting. After casting and compaction, the specimens were covered to minimize moisture loss and stored in the molds for 24 h. They were then demoulded and fully immersed in tap water maintained at 23 ± 2°C until the designated testing age. Because the specimens were submerged, the curing condition was treated as a saturated environment; ambient relative humidity was therefore not a controlling curing parameter. The specimens were removed immediately before testing, and excess surface water was wiped off. The preparation and curing sequence is shown in Figure 4.

2.5 Test Methods

- Workability was evaluated by slump test according to ASTM C143/C143M [11].
- The compressive strength testing was performed with cylinders measuring 150 × 300 mm (ASTM C39/C39M [12]), and the splitting-tensile strength testing with cylinders measuring 100 × 200 mm (ASTM C496/C496M [14] and compressive stress-strain response following the axial deformation measurement of ASTM C469/C469M [13]).

Mechanical tests were done with a compression testing machine had a maximum ultimate load of 2000 kN. The load was applied in continuous form at a stress rate of 0.25 ± 0.05 MPa/s, as per ASTM C39/C39 M. The splitting tensile load was applied continuously at the ASTM C496/C496M splitting-stress of 0.7-1.4 MPa/min rate of loading. The stress-strain cylinders were loaded under force control by the same continuous ASTM C39/C39M loading rate while axial deformation was measured.

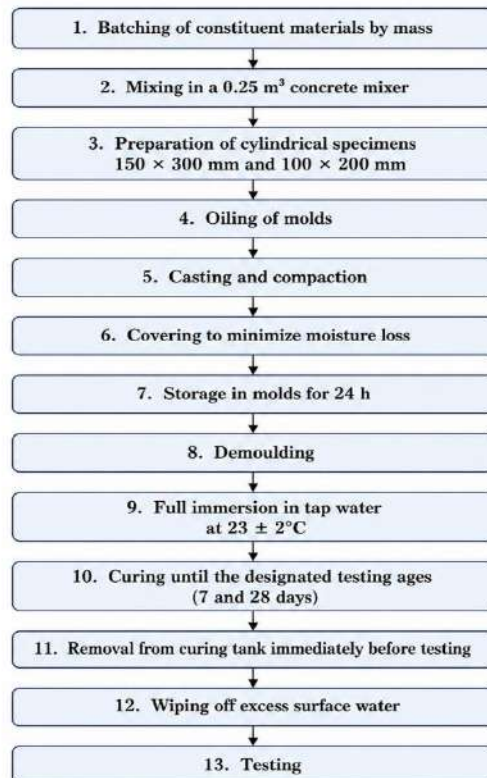
Concrete Mixing, Casting, and Curing Procedure



Note: Cement and silica fume were first blended together in the dry state before the cementitious blend was added to the aggregates.

Fig. 3. Mixing procedure of concrete

Specimen Preparation and Curing Procedure



Note: The specimens were fully submerged during curing; therefore, the curing condition was considered saturated.

Fig. 4. Specimen preparation and curing procedure

The numbers of testing specimens was three for each mixture and age. The specimens were tested for compressive and splitting tensile strengths being averages for the three specimens. Three separate 100 × 200 mm instrumented cylinders were used for the compressive stress-strain measurements. Axial deformation during the compressive stress-strain test was measured using an external yoke system with a laser displacement sensor. The sensor measured relative axial displacement between the two yokes during loading, and the axial strain was calculated by dividing the measured displacement by the gauge length. The setup is consistent with the general ASTM C469/C469M approach to measuring axial deformation; however, the resulting modulus values are observed in the form of apparent secant moduli rather than standard ASTM chord moduli. The percentage improvement to the control mixture was evaluated using Eq. (2).

$$\text{Improvement (\%)} = [(X_{SF} - X_{NC}) / X_{NC}] \times 100 \quad (2)$$

Where; X_{SF} is the measured property of a silica-fume mixture, X_{NC} is the corresponding value of the control mixture at the same age.

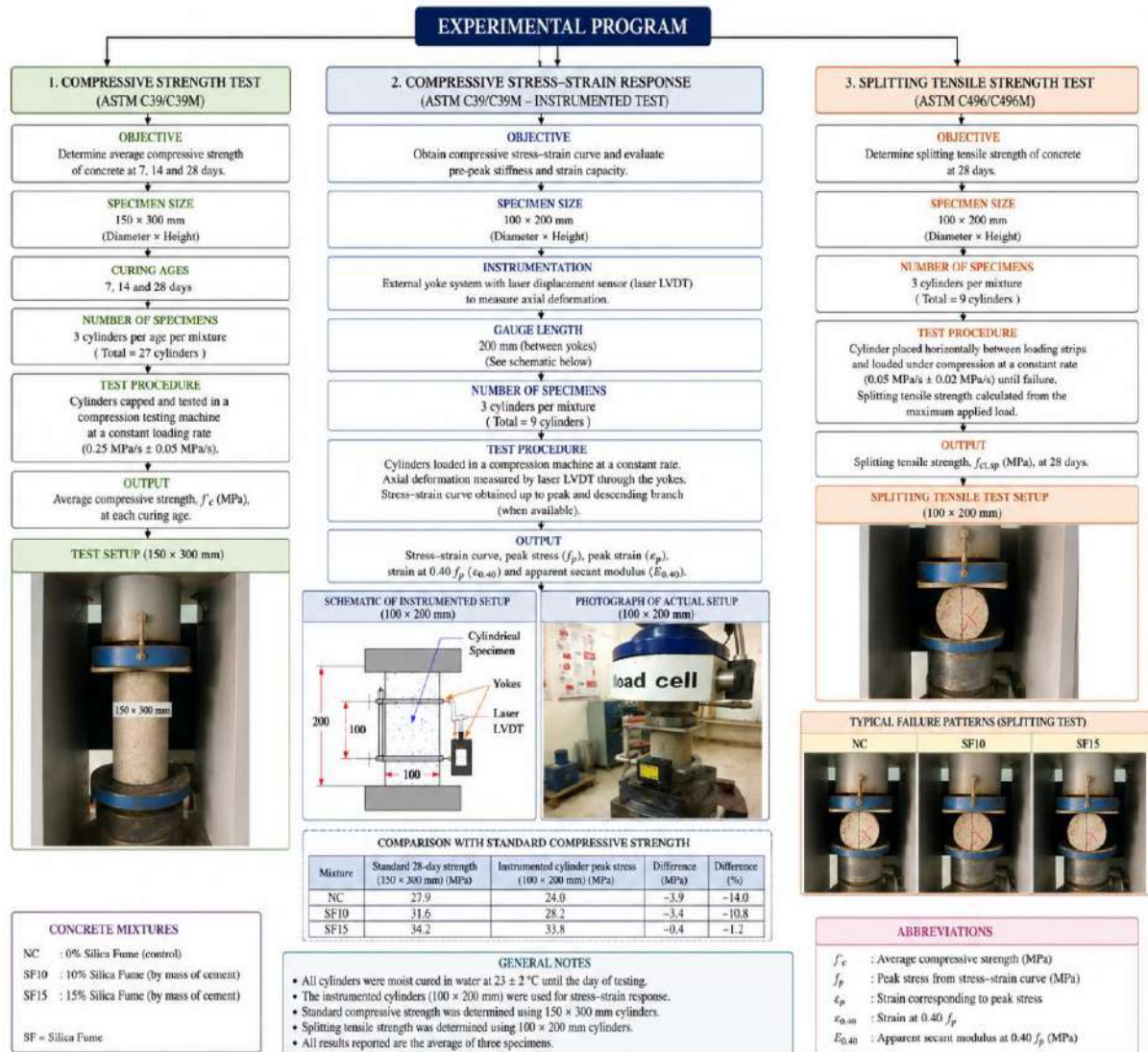


Fig. 5. The main stages of methodology program.

For splitting tensile strength, the standard expression is given by Eq. (3).

$$f_{ct,sp} = 2P / (\pi DL) \quad (3)$$

Where; $f_{ct,sp}$ is the splitting tensile strength in MPa, P is the maximum applied load in Newton, L is the cylinder length in mm, D is the cylinder diameter in mm.

To provide a concise visual summary of the experimental methodology adopted in this study, a process flow chart of the experimental program is presented in Figure 5.

3. Results and Discussion

3.1 Workability

The slump results are shown in Table 4 and Figure 6. NC had a slump of 130 mm, while SF10 and SF15 were at 180 and 160 mm, respectively, which is 38.5% and 23.1%. Thus, SF10 had a higher slump than SF15. The higher slump of the silica-fume mixtures was not expected, as the fully dispersed silica fume is responsible for the high water demand and cohesiveness of the mixture because of the very high specific surface area of the silica fume, but it has to be explained by more than one material and processing effect, such as:

- The aggregates were in the saturated surface-dry (SSD) condition, and so did not absorb a significant amount of 192.5 kg/m^3 mixing water in the aggregate skeleton.
- The high number of fine particles in Zone 4 sand may have increased the volume and continuity of the mortar phase and influenced the mobility and water distribution of the fresh mixture.
- Most importantly, densified silica fume needs to be efficiently deagglomerated. If there were no superplasticizer, some of the densified particles may have remained as larger agglomerates rather than as individual ultrafine particles. This would have diminished the effective surface area exposed to water during the short slump-test period and may temporarily lower the water demand.
- The Sika technical advice also provides that superplasticizers promote deflocculation and uniform distribution of Sika Fume S92 D [15].

Such mechanisms are physically plausible but not directly proven, as no particle dispersion, rheology, or pore-water distribution was observed. Thus, the increase should not be characterized as a general ball-bearing effect or a general effect of the silica-fume addition. Rather, it is a mixture-specific response to the combination of SSD conditions, high effective water content, fine aggregate grading, or incomplete deagglomeration of the densified product. Despite the 180 mm slump of SF10, there was no bleed-water accumulation, coarse aggregate settlement, or mortar separation during the slump test or handling. This was qualitative because no bleeding and segregation resistance tests were performed. By increasing the replacement level from 10% to 15%, the slump was reduced from 180 to 160 mm. This is consistent with the greater quantity and total surface area of ultrafine silica-fume particles in SF15, which would be expected to increase water demand and reduce the amount of free water available for flow.

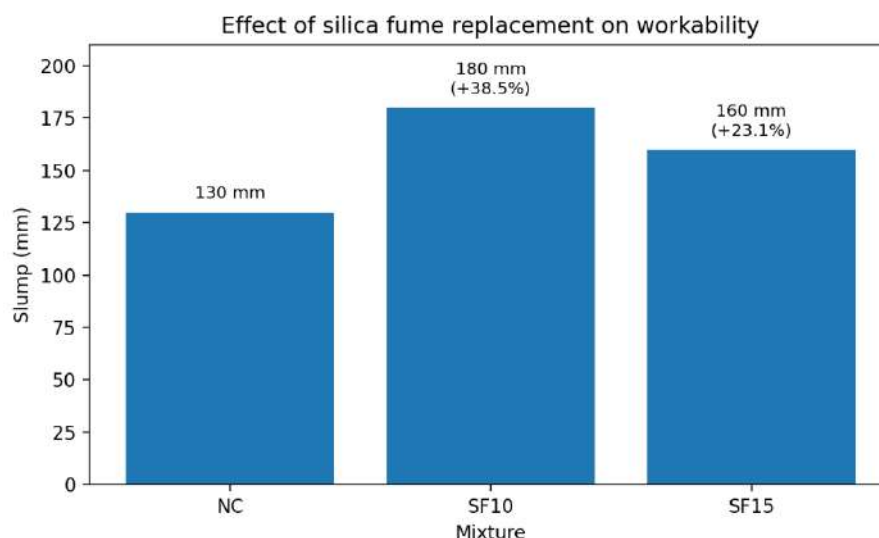


Fig. 6. Effect of silica fume replacement on concrete slump

Table 4. Slump results and percentage change relative to the control mixture.

Mixture	Slump (mm)	Change relative to NC (%)
NC	130	0
SF10	180	38.5
SF15	160	23.1

3.2 Compressive Strength Development

The strong compressive strength of silica fume was due to physical and chemical effects. At early ages, ultrafine particles can fill spaces between cement grains and provide nucleation sites for hydration products. This leads to denser particle packing and precipitation of hydration products. So the early-age strength is much stronger, even for the partial replacement of cement. At later ages, the pozzolanic reaction becomes more important. Silica fume reacts with calcium hydroxide released during cement hydration and forms a calcium-silicate-hydrate (C-S-H) gel. As a result, the secondary C-S-H gel reduces capillary porosity, improves the pore structure, and improves the ITZ between cement paste and aggregate. Combined filler, nucleation, and pozzolanic effects account for the higher 28-day compressive strength of silica-fume mixtures.

SF15 had the highest compressive strength at both testing ages. The 28-day strength was 6.3 MPa higher than NC and 2.6 MPa higher than SF10. Therefore, of the replacement levels studied, 15% silica fume had the highest compressive strength. This should be seen in the context of the present mixture and should not be considered as the universal optimum silica-fume content. The 28-day to 7-day strength ratios were 1.69 for NC, 1.74 for SF10, and 1.70 for SF15. The very close ratios indicate that all mixtures had relatively similar strength development patterns in the period in question. However, the strength increase of the silica-fume mixtures at 7 days despite the reduced cement content suggests that the early physical filler and nucleation effect was already considerable. The slightly higher ratio of SF10 might indicate a slightly greater relative contribution from the later-age pozzolanic activity, while the high early strength of SF15 resulted in a 28/7 ratio closer to the control mixture.

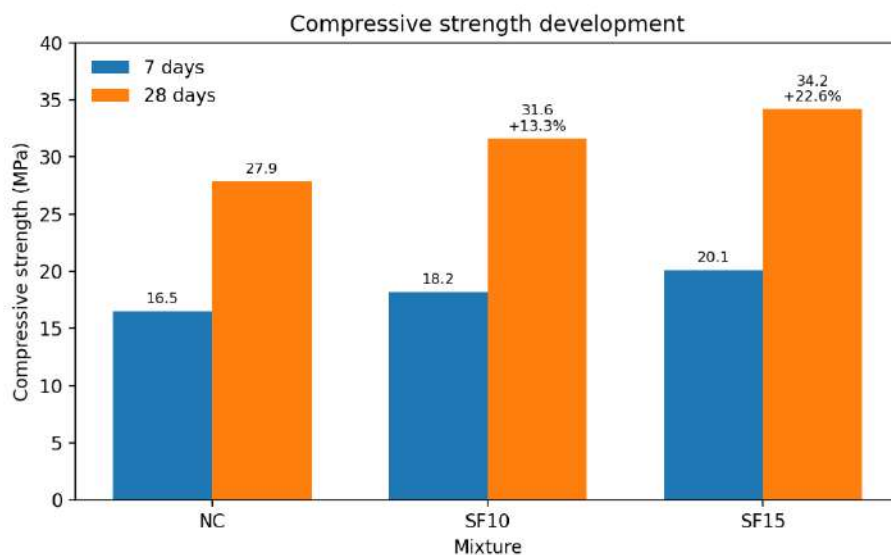


Fig. 7. Compressive strength development of control and silica-fume concrete mixtures

Since the aggregates were used in the saturated surface-dry condition, significant absorption of the designed mixing water was avoided. Therefore, the early-age strength increase should not be directly attributed to a significant reduction in the overall effective water-to-binder ratio. Nevertheless, local particle packing and water distribution in the paste may have influenced hydration and microstructural development, although these effects were not measured directly.

Table 5. Compressive strength results and calculated improvement indices.

Mixture	7-day f'_c (MPa)	Change relative to NC (%)	28-day f'_c (MPa)	Change relative to NC (%)	28/7 strength ratio
NC	16.5	0.0	27.9	0.0	1.69
SF10	18.2	10.3	31.6	13.3	1.74
SF15	20.1	21.8	34.2	22.6	1.70

3.3 Splitting Tensile Strength Development

The 28-day splitting tensile strength results are shown in Table 6 and Figure 8. NC achieved 2.5 MPa, while SF10 and SF15 achieved 2.9 and 3.1 MPa, respectively. This led to increases of 16.0% and 24.0%, respectively, compared to NC. The splitting tensile strength has been improved through the physical and chemical effects of silica fume on the hardened matrix and the ITZ.

The dense particles can fill micro voids and enhance the packing of aggregate surfaces, and the pozzolanic reaction gives rise to additional C-S-H gel and reduces weak porous areas in the ITZ. The experimental splitting-to-compressive-strength ratios were 8.96% for NC, 9.18% for SF10, and 9.06% for SF15. Their narrow range indicates that tensile strength increased in general in conjunction with compressive strength, and it is likely that ITZ refinement was part of the reason for the improvement. SF10 had the highest tensile-to-compressive-strength ratio, while SF15 had the highest absolute splitting tensile strength.

Table 6. Experimental splitting tensile strength.

Mixture	Experimental $f_{ct,sp}$ (MPa)	Change relative to NC (%)	$f_{ct,sp}/f'_c$ (%)
NC	2.5	0.0	8.96
SF10	2.9	16.0	9.18
SF15	3.1	24.0	9.06

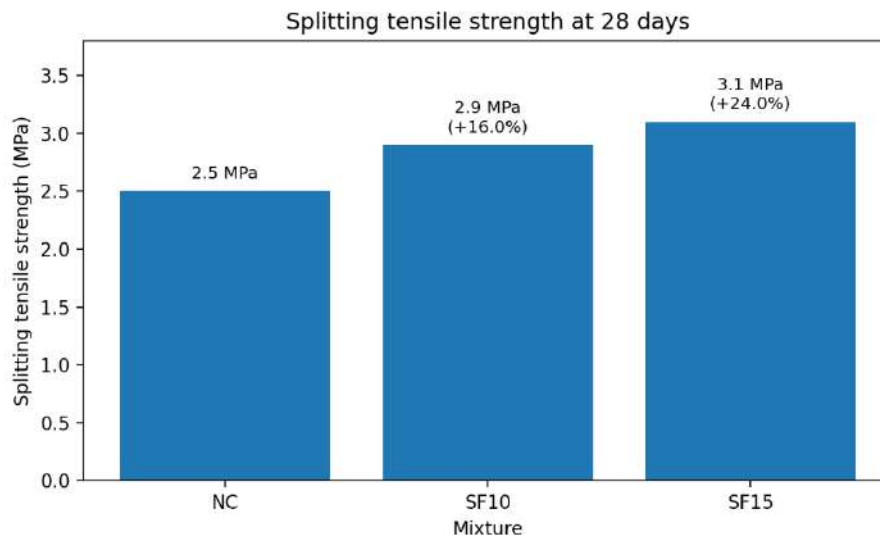


Fig. 8. Splitting tensile strength at 28 days

As shown in Figure 9, the fresh and hardened-state results exhibited different trends with increasing silica-fume replacement. The response was therefore not monotonic but independent of the replacement level. SF10 had the highest slump measured, whereas SF15 had the highest compressive and splitting tensile strengths.

The low slump of SF15 compared to SF10 is consistent with the higher quantity and total surface area of ultrafine silica-fume particles, which can increase water demand. In contrast, the higher compressive and tensile strength of SF15 indicates that greater contributions from particle

packing, matrix refinement, pozzolanic reaction, and the interfacial transition zone are observed. These results demonstrate the trade-off between fresh state flowability and hardened mechanical performance for the replacement range. SF10 exhibited the highest slump, and SF15 showed the strongest overall mechanical response. However, because the slump increases are unusual and no quantitative rheological or stability tests were performed, SF10 is not the best in terms of workability. It is more accurate to say that SF10 had the highest slump under the present experimental conditions. The results should not be taken as a universal optimum silica-fume content. The ideal replacement level is based on the combination of fresh-state handling, mechanical strength, durability, cost, and field placement.

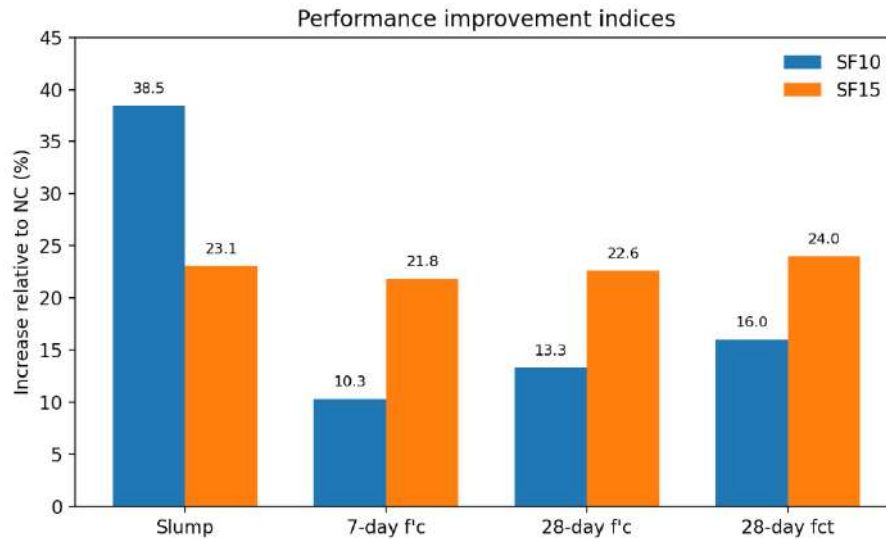


Fig. 9. Improvement indices relative to the control mixture

3.4 Stress-Strain Response

The compressive stress-strain curves of NC, SF10, and SF15 are shown in Figure 10. The curve for each mixture is the average response from separately cast 100 × 200 mm instrumented cylinders. The axial deformation was measured with an external yoke system with a laser displacement sensor, and the recorded displacement was divided by the gauge length to calculate the axial strain. The three mixtures showed the same behavior in the first ascending region, and the curves became more separated at higher stress levels. The rising response of silica-fume mixtures is stronger due to better particle packing, micro void filling, additional C-S-H gel growth and ITZ refinement. In fact, these mechanisms resulted in a denser, hardened matrix that can withstand a higher compressive stress before the peak.

The post-peak branch was only recorded for the NC mixture. The descending branches of SF10 and SF15 could not be captured reliably with the available testing setup. Thus, the analysis focused on the ascending response, peak stress, peak strain, and the apparent stiffness before peak. This is not an indication of ductility, toughness, energy absorption, or post-peak softening of silica-fume mixtures. The peak stresses produced by the instrumented cylinders were lower than the average 28-day compressive strengths obtained from the standard compression tests. The comparison can be interpreted with caution because the standard compressive-strength specimens were 150 × 300 mm cylinders, and the stress-strain specimens were 100 × 200 mm cylinders. In addition to the specimen sizes and strength, the dataset can be affected by the variability of the specimens, the attachment of instruments to the cylinders, the fit of the instruments, the load conditions, and the data. Because these factors were not studied separately. Since the descending branches of the SF10 and SF15 stress-strain curves were not reliably recorded, only the ascending response was considered. The apparent secant modulus was calculated at 40% of the peak stress using Eq. (4):

$$E_{0.40} = 0.40f_p / \varepsilon_{0.40} \quad (4)$$

Where; f_p is the peak stress obtained from the instrumented cylinder; and $\epsilon_{0.40}$ is the axial strain corresponding to 40% of the peak stress.

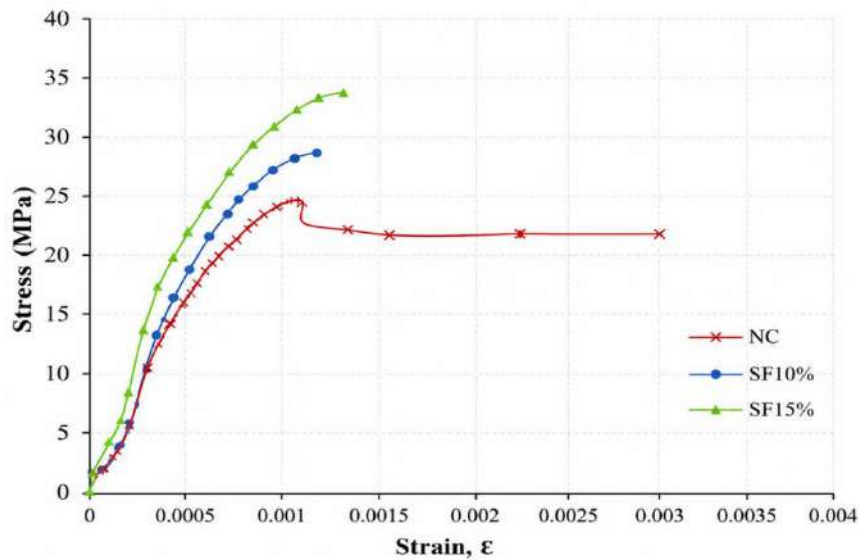


Fig. 10. Compressive stress–strain responses of NC, SF10, and SF15 obtained from instrumented cylinder tests

The increase is attributed to the combined filler effect, better particle packing, additional C-S-H gel formation, and ITZ refinement, which together resulted in a denser and stiffer hardened matrix. However, these values are apparent secant moduli calculated from the recorded stress-strain curves and should not be regarded as ASTM C469/C469M chord-modulus values.

Table 7. Peak parameters and apparent secant modulus at 40% of peak stress

Mixture	Mean values peak stress (MPa)	Mean values peak strain	40% peak stress (MPa)	Strain at 40% peak stress	$E_{0.40}$ (GPa)	Change relative to NC (%)
NC	24.0	0.00102	9.60	0.000297	32.3	—
SF10	28.2	0.00116	11.28	0.000313	36.0	11.5
SF15	33.8	0.00126	13.52	0.000316	42.8	32.5

3.5 Failure Patterns

The failure patterns for the splitting tensile tests are shown in Figure 11. In all mixtures, the main crack formed along the loaded vertical diameter, as expected from the splitting tensile failure. NC, SF10, and SF15 showed the same principal crack orientation, which indicates that the load is transferred along the expected diametric plane.

The three mixtures have similar splitting structures with a large vertical crack between loading lines. Cracking in the silica-fume mixtures appeared to be relatively localized along the splitting plane. As crack widths and propagation rates were not measured, this comparison is qualitative. The increased splitting strength of the SF10 and SF15 is consistent with a denser cementitious matrix and a better ITZ, but the photographs alone cannot tell whether cracks propagated mainly through the paste and along the aggregate-paste interface or if aggregate particles are involved.

The compressive cylinders cracked with their axial and inclined cracking, followed by local crushing, as shown in Figure 12. Failure was neither sudden nor explosive and was not accompanied by substantial ejection of fragments. The sequence is rather consistent with a columnar/shear failure rather than a perfect cone. No clear transition from gradual to brittle failure was seen with increasing silica-fume content. The higher peak stress and the apparent stiffness of

SF10 and SF15 should therefore not be interpreted as the result of a more brittle or explosive failure mode, as their post-peak branches were not captured reliably.

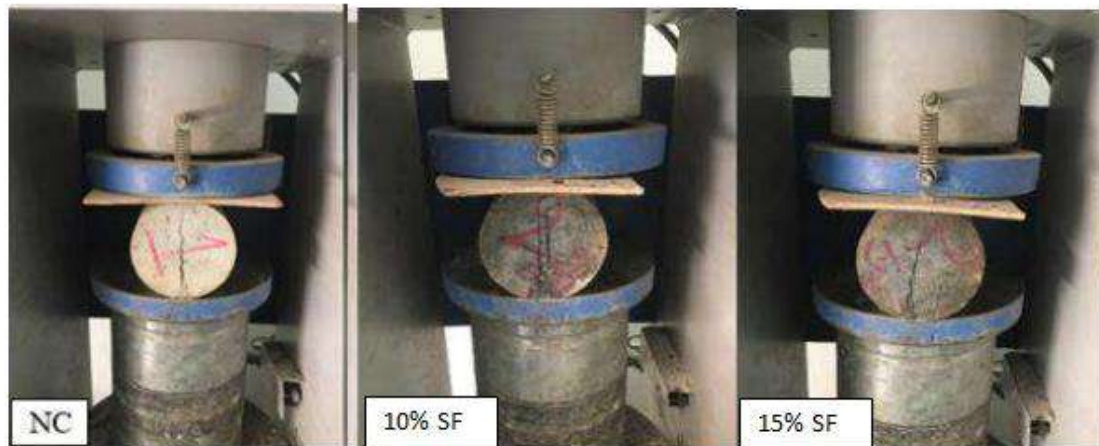


Fig. 11. Splitting tensile failure patterns of the cylinders: (a) NC, (b) SF10, and (c) SF15



Fig. 12. Progressive compressive failure observed during cylinder testing

3.6 Study Limitation

The economic feasibility of silica-fume concrete should be considered. Silica fume is often more expensive than ordinary Portland cement and may increase material cost. Higher strength, on the other hand, may lead to better member dimensions, less binder demand in new mixtures or less maintenance costs.

Due to the lack of a cost analysis, the economic feasibility of silica-fume incorporation could not be established. Although partial cement replacement and better strength may improve material efficiency, the net economic benefit depends on silica-fume cost, transportation, processing, mixture optimization, and potential reductions in structural dimensions or maintenance requirements. This might also lead to lower clinker consumption and hence even less embodied carbon in concrete production, but the environmental benefits will depend on the distance between silica-fume processing and the substitution plan in practice.

The study has several limitations. Only two silica-fume replacement levels, 10% and 15%, were studied. The mechanical properties were examined mainly at 7 and 28 days. Durability, microstructure, drying shrinkage, creep, long-term strength, and field performance were not investigated. In addition, the descending branches of the SF10 and SF15 stress-strain curves were not recorded, which prevented evaluation of post-peak softening, toughness, energy absorption, and ductility.

4. Conclusions

The effect of substituting 10% and 15% Portland cement with densified silica fume on fresh properties, mechanical performance, and the magnitude of compressive stress and strain rise of normal-strength concrete made of local materials was examined. In the replacement range, silica-fume incorporation showed a favorable effect on compressive and splitting tensile strengths at the peak compressive stress, peak strain, and apparent pre-peak stiffness of the sample compared to the control mixture. These benefits were due to the combined filler, particle-packing, nucleation, and pozzolanic properties of the silica fume, which promoted a stronger cementitious matrix that had fewer internal voids and a better interface between cement paste and aggregate. The main conclusions from this study are as follows:

- Among all mixtures studied, SF15 had the best overall mechanical performance at 7 and 28 days. It showed the highest compressive strength, splitting tensile strength, peak compressive stress, peak strain, and apparent secant modulus. SF10 had good mechanical performance, and also had the highest slump. While SF15 had a lower slump than SF10, its slump was still larger than that of the control mixture. The unusual slump response was considered mixture-specific and might have been due to the combined effects of SSD aggregates, high effective water content, fine-aggregate grading, and incomplete deagglomeration of the densified silica fume. No bleeding or coarse aggregate segregation was observed during mixing, slump testing, or handling. However, this observation was qualitative because no bleeding, segregation-resistance, or rheological tests were performed.
- The compressive stress-strain results showed that silica-fume incorporation enhanced the ascending response and the strain at peak stress as well. The apparent secant modulus also increased as the silica-fume content increased, suggesting a stiffer pre-peak response. However, the descending branches of the SF10 and SF15 curves could not be captured well. This left no reliable conclusions regarding ductility, toughness, post-peak softening, fracture energy, or energy-absorption capacity.
- From a practical point of view, the results suggest that densified silica fume can improve the mechanical efficiency of normal-strength concrete and partially reduce Portland cement content. However, the results should not be taken as a universal optimum replacement level. The performance of silica-fume concrete in terms of material properties, mixture proportions, dispersion efficiency, curing conditions, sample size, and testing procedures should be further investigated. These should include a more diverse range of replacement levels, controlled use of superplasticizers in the production process, rheological and stability measurements, durability tests, microstructural characterization, and displacement-controlled testing to obtain the full post-peak response.

5. Recommendations for Future Research

- Other studies can examine a wider range of silica-fume replacement levels and include mixtures containing superplasticizers to better understand how silica-fume content, dispersion, water demand, and fresh-concrete stability are related. More fresh-state tests (air content, segregation resistance, bleeding, rheological measurements) are recommended to verify the unusual slump behavior in this study.
- Future experiments should also include full displacement-controlled stress-strain tests to capture the descending branch and thus allow for reliable evaluation of ductility, toughness, fracture energy, and constitutive behavior. Microstructural characterization with scanning electron microscopy, X-ray diffraction, or mercury intrusion porosimetry (MIP) would also be useful to validate the proposed filler, pozzolanic, and ITZ-refinement mechanisms.
- Long-term strength, drying shrinkage, creep, permeability, chloride penetration, sulfate resistance, carbonation, and other durability indicators should also be studied. Finally, life cycle and cost analysis is necessary to determine if the mechanical and

environmental benefits of silica-fume replacement justify its additional material and processing costs in field applications.

References

- [1] ACI Committee 234. Guide for the Use of Silica Fume in Concrete (ACI PRC-234-06). Farmington Hills, MI: American Concrete Institute; 2006.
- [2] ASTM C1240-20. Standard Specification for Silica Fume Used in Cementitious Mixtures. West Conshohocken, PA: ASTM International; 2020.
- [3] Wang X, Zhang S, Wang C, Liu L, Wang J. Properties and microstructure of an interfacial transition zone enhanced by silica fume in concrete prepared with coal gangue as an aggregate. *ACS Omega*. 2023;8:48873–48884. <https://doi.org/10.1021/acsomega.3c08560>
- [4] Shah HA, Nehdi ML, Khan MI, Akmal U, Alabduljabbar H, Mohamed A, Sheraz M. Predicting compressive and splitting tensile strengths of silica fume concrete using M5P model tree algorithm. *Materials*. 2022;15(15):5436. <https://doi.org/10.3390/ma15155436>.
- [5] Luthfiana H, Wibowo, Safitri E. Study of stress-strain relationship of concrete with silica fume added as partial replacement for cement. *Jurnal Teknik Sipil*. 2024;24(1):789-799. <https://doi.org/10.26418/jts.v24i1.75323>.
- [6] Mazloom M, Ramezaniapour AA, Brooks JJ. Effect of silica fume on mechanical properties of high-strength concrete. *Cement and Concrete Composites*. 2004;26(4):347–357. [https://doi.org/10.1016/S0958-9465\(03\)00017-9](https://doi.org/10.1016/S0958-9465(03)00017-9).
- [7] Siddique R. Utilization of silica fume in concrete: Review of hardened properties. *Resources, Conservation and Recycling*. 2011;55(11):923–932. <https://doi.org/10.1016/j.resconrec.2011.06.012>.
- [8] Hamada HM, Abed F, Katman HYB, Humada AM, Al Jawahery MS, Majdi A. Effect of silica fume on the properties of sustainable cement concrete. *Journal of Materials Research and Technology*. 2023;24:8887–8908. <https://doi.org/10.1016/j.jmrt.2023.05.147>.
- [9] Zhao S, Sun W, Wu C, et al. Effect of silica fume in concrete on mechanical properties and dynamic behaviors under impact loading. *Materials*. 2019;12(19):3263.
- [10] Gaitero JJ, Campillo I, Guerrero A. Reduction of the calcium leaching rate of cement paste by addition of silica nanoparticles. *Cement and Concrete Research*. 2008;38(8-9):1112-1118.
- [11] ASTM C143/C143M-20. Standard Test Method for Slump of Hydraulic-Cement Concrete. West Conshohocken, PA: ASTM International; 2020.
- [12] ASTM C39/C39M-24. Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. West Conshohocken, PA: ASTM International; 2024.
- [13] ASTM C469/C469M-22. Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression. West Conshohocken, PA: ASTM International; 2022.
- [14] ASTM C496/C496M-17. Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens. West Conshohocken, PA: ASTM International; 2017.
- [15] Sika. Sika Fume S92 D technical guidance for shotcrete and concrete applications. Sika España; product information accessed 5 August 2026.

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Experimental investigation on the shear behavior of steel fibre reinforced concrete deep beams with strategic circular web openings

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Abstract

Building services often require openings in RCC deep beams, which can disrupt the internal load-transfer mechanism and adversely affect shear performance. This study experimentally and theoretically investigates the effect of circular web-opening location on the shear behavior and load-carrying capacity of RCC deep beams, with particular emphasis on openings positioned in the tension and compression zones. A total of 45 deep beam specimens representing 15 configurations, comprising solid deep beams (SDB), deep beams with openings in the compression zone (DB-C), and deep beams with openings in the tension zone (DB-T), incorporating crimped steel fibre (CSF) dosages of 0%, 0.25%, 0.50%, 0.75% and 1.00%, were tested under two-point loading. The results showed that compression-zone openings caused less reduction in load-carrying capacity than tension-zone openings. At 0% CSF, the capacity loss relative to SDB was 2.4% for DB-C and 7.3% for DB-T, while at 1% CSF, the corresponding losses were 2.8% and 6.7%, respectively. At 1% CSF, the first-crack load increased by 35.1-41.9% and the ultimate load 9.4-10.5% compared with the corresponding specimens without CSF. DB-C exhibited 9.1% higher first-crack load and approximately 4.2% higher ultimate load than DB-T at 1% CSF. The theoretical predictions agreed reasonably well with the experimental results, with variation of approximately 0.6-10.8%.

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1. Introduction

High-rise structures with long infrastructural spans utilize deep beams, acting as the fundamental backbone to absorb and distribute heavy load. To efficiently absorb immense structural demands, heavy duty buildings and bridges rely on deep profile beams to counteract vertical deflections and bending stress. A deep beam is a structural element whose depth is so significant relative to its length (span) that it behaves more like a solid wall or an upright tied arch than a traditional, slender beam. Openings in such beams with greater depth are commonly provided in modern construction scenarios like buildings, bridges etc., to accommodate essential services like electrical installations, AC ducts, drainage pipes etc., and sometimes for accessibility. Such openings are usually embedded in the beam web with required shape, size and strategic position relying on the service requirements. Two major opening shapes are commonly adopted (circular and rectangular) with circular opening^{**s**} demonstrating better performance in stress distribution and improved serviceability [11,19]. In such deep beams it is essential to assess the structural behavior along with ultimate strength since the presence of a circular opening disrupts the load transfer mechanism within the beam [6]. It is widely recognized that the classical elastic beam theory fails to accurately

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describe the behavior of these beams as their behavior is regulated by non-linear stress distribution and direct load transfer through compression struts instead of conventional flexure action in accordance with the load-transfer mechanism described by Strut-and-Tie Model (STM). This adverse effect primarily arises from improper load-path-oriented opening in the deep beam. The largest crack width at ultimate failure is encountered due to opening center positioned within the critical shear zone rather than any other portion in deep beams. The web openings in the deep beam should not be positioned closer to the vertical edges all around the beam since higher loading may reduce structural integrity causing premature failure. Also, the deflection in such beam with greater depth having opening is predominantly influenced by shear effect. Hence, positioning a web opening within the high-shear region interrupts the critical load path, significantly reducing beam stiffness and ultimate strength.

To locate optimum load-path-oriented position of circular web opening, a comprehensive load transfer mechanism is essential with following factors influencing the structural behavior pertaining to deep beam i.e., span to its depth ratio, cross section details, main and web reinforcements, properties of concrete, type and position of loading, type and shape of web opening etc. Web openings in beams of greater depth also interrupt the continuity of shear reinforcement and disrupt the formation of effective struts, which can cause a sudden drop in shear capacity and lead to premature shear failure, thereby adversely affecting the overall structural performance. Under such scenarios it is not feasible to place steel in the line of opening especially shear, main and side face reinforcement. Externally bonded steel sheets, or steel fiber infills effectively restore the continuity of shear transfer mechanism within the disturbed region by improving tensile stress transfer and bridging developing cracks [1,18].

Use of closely spaced and uniformly dispersed fibre in concrete reduces the effect of openings in the beams of greater depth. Fiber reinforcement, such as the use of CFRP or the, introduction of steel fibres is much effective for enhancing the shear performance of reinforced concrete beams of greater depth approximately by 35% at 0.75% fibre content when the openings are provided in compression zone and decreases the development of cracks which are formed in concrete primarily due to openings [2,14, 18]. Interface adhesion between steel fibre and concrete also enhance the stress transfer with optimum dosage of up to 1% [18]. Variety of such fibres are available with most widely manufactured and available is crimped type of steel fibre. The incorporated crimped type of steel fibre also features more in its presence in the concrete by restricting crack growth, improving tensile strength, and acting as shear reinforcement etc. Due to these advantages, CSF is an optimal additive for the concrete beams with larger depths. Key findings from previous studies are summarized below:

- Circular openings are more effective than rectangular, square, or other geometries [19].
- Increasing the steel fibre content up to 2% in the concrete leads to improvement in shear carrying capacity, reduced deflection, cracks with improvement against failure [15].
- Despite previous studies on deep beams with web openings and fibre-reinforced concrete, the combined influence of circular web openings and crimped steel fibres (CSF) on shear behavior remains insufficiently investigated. CSF was selected for its crimped geometry and enhanced mechanical anchorage, which promote crack bridging within the concrete matrix.
- Limited attention has been given to the combined influence of load-path-oriented circular opening placement and varying CSF dosages, particularly with respect to identifying an optimum opening position that minimizes disruption to the principal load-transfer path.

The present study explores the influence of load-path oriented circular web opening on the structural performance pertaining to deep beams, identifies the most favorable location of openings to satisfy service requirements while mitigating disruption to the natural load transfer path. This study also assesses the potential of crimped steel fibres in mitigating the adverse effect of circular web opening on the overall performance pertaining to deep beams. Therefore, the present study aims to experimentally investigate the influence of load-path-oriented circular web openings location in the compression and tension zones on the shear behavior and load-carrying capacity of RCC deep beams. The study further evaluates the effectiveness of CSF at different dosages in minimizing adverse effects of web openings and improving the ultimate load-carrying behavior reducing the cracks and deflection.

2. Methodology

To ascertain the influence of openings, a total of 45 deep beam specimens representing 15 configurations were fabricated and tested. The specimens had an overall length of 750 mm, a width of 75 mm and the depth of 325 mm, and were classified into three beam types: having openings at compression zone (DB-C) and tension zone (DB-T) along with solid specimen without openings (SDB) were fabricated and tested to failure under a gradually applied symmetrical two-point loading system using 1000 kN capacity universal testing machine (UTM). The tests were conducted under load-controlled conditions at a loading rate of 0.5 kN/s, with observations recorded at 10 kN load increments. Mid-span deflection was measured using a dial gauge with a least count of 0.01 mm.

Table 1. Summary of tested deep-beam specimens

Beam configuration	CSF volume fraction (%)	Replicates per CSF level	Total Specimens
SDB	0, 0.25, 0.50, 0.75, 1.00	3	15
DB-C	0, 0.25, 0.50, 0.75, 1.00	3	15
DB-T	0, 0.25, 0.50, 0.75, 1.00	3	15
			Total 45

The following experimental procedure is adopted:

- The physical and engineering properties of materials employed for concrete matrix preparation were evaluated in the laboratory as prescribed in respective Indian Standard (IS) code provision.
- Proportioning the concrete mix for M20 grade, designed in compliance with the recommended IS 10262:2019 by incorporating various volume fractions of CSF to cast the test specimens for compression and shear strength test.
- The deep beams were designed with longitudinal, vertical, and side-face reinforcement.
- Concrete beams of greater depth specimens were cast with the help of wooden formwork having specified measurements with provision of circular web opening of 75 mm diameter at designated position along with standard concrete cubes for compression test.
- To evaluate the compressive strength, a total of thirty concrete cubes were cast, comprising three specimens for each of the five CSF contents at each testing age of 7 and 28 days.
- After the specified curing period, laboratory tests were conducted for cast crimped steel fibre concrete cubes and beams of specified measurements for compressive strength using 2000 kN capacity digital compression testing machine, and shear carrying capacity pertaining to deep beam of circular web opening made in compression zone (DB-C), opening made in tension zone (DB-T) and SDB is conducted using 1000 kN capacity UTM.
- Results are analyzed and compared with theoretical model (Professor F. K. Kong's model) to validate the experimental work.

3. Material Properties

The following constituent materials, possessing the required physical and engineering properties are employed in the preparation of specified specimens:

3.1 Cement

53 Grade Ordinary Portland Cement (OPC) having specific gravity 3.11 is used for proportioning cement concrete of grade M20. Fine Aggregates: Manufactured sand as fine aggregates pertaining to zone II with specific gravity of 2.60 and a fineness modulus of 3.15. The measured silt content was 0% and moisture content was 4%. Coarse Aggregates: Considering the presence of CSF and limited beam width of 75 mm, 10 mm downsized coarse aggregate with a specific gravity of 2.72 and a bulk density of 1640 kg/m³ was used for preparing the concrete. Bore well water which is free from dust and dirt having pH 7.2 conforming to IS 456:2000 was used for mixing and curing.

A superplasticizer: RHEOBUILD SP1, manufactured by BASF Construction Chemicals (India) Pvt. Ltd., which is sulphonated naphthalene-formaldehyde polymer-based admixture with a specific gravity of 1.15 was included as high-range water-reducing superplasticizer to enhance the workability especially during mixing of concrete along with CSF which are more vulnerable to balling effect.

3.2 HYSD Steel Bars

For reinforcement 2 HYSD bars of 12 mm diameter, conforming to Fe 500 grade are provided as the main tension steel. In addition, 6 # 8 mm in 3 layers are provided as horizontal side face reinforcement. The physical properties of reinforcement bars obtained from tensile test are presented in the table 2.

Table 2. Physical test results of HYSD bars

Diameter (mm)	Sectional weight (kg/m)	Yield strength (N/mm ²)	Ultimate strength (N/mm ²)	Elongation (%)
12	0.863	515	590	16.05
8	0.373	510	588	16.55

CSF (crimped steel fibre) possessing an aspect ratio of 60 having a length of 60 mm and 1 mm diameter with a tensile strength of 1150 MPa with the density of 7850 kg/m³ is selected to achieve enhanced shear resistance and improved crack bridging action.

4. Concrete Mix Design

The concrete mix associated with M20 grade was proportioned in compliance with the guidelines of IS 10262:2019 for concreting in mild exposure conditions. To facilitate consistent fibre dispersion and to control balling effect, superplasticizer is added with nominal maximum size of aggregates was limiting to 10 mm, enhancing the cohesiveness and fresh state workability of fibre reinforced concrete.

CSF was incorporated as a discrete reinforcing phase at volume fractions ranging from 0% to 1%, in increments of 0.25%. The quantities of the concrete constituents presented in Table 3 are expressed in kg/m³. The water-cement ratio and superplasticizer dosage were maintained constant at 0.45 and 7.20 kg/m³, respectively, for all mixtures. The superplasticizer facilitated fibre dispersion and reduced the possibility of fibre balling. The measured 4% moisture content of the fine aggregate was accounted for when determining the net mixing water. The workability of the fresh concrete mixtures was evaluated using the slump test in accordance with IS 1199 (Part 2): 2018. The absolute-volume method was used to maintain a total concrete volume of 1 m³. Accordingly, the sum of the absolute volumes of cement, water, superplasticizer, CSF, fine aggregate and coarse aggregate was maintained as:

$$V_c + V_w + V_{sp} + V_{CSF} + V_{FA} + V_{CA} = 1.00 \text{ m}^3 \quad (1)$$

The absolute volume of each constituent was calculated using:

$$V_i = \frac{m_i}{SG_i \times 1000} \quad (2)$$

Where V_i , m_i and SG_i represent the absolute volume, mass and specific gravity respectively, of constituent i . As the CSF volume fraction increased, an equivalent volume was deducted from the combined aggregate volume, while the cement content, water content and superplasticizer dosage were maintained constant. The fine and coarse aggregate quantities were adjusted by maintaining approximately 39% and 61%, respectively, of the total aggregate volume. The resulting quantities of all constituents are presented in Table 3.

Table 3. Mix proportions and slump values of M20 concrete containing different CSF volume fractions

Crimped steel fibre (%)	Cement (kg/m ³)	Fine aggregates (kg/m ³)	Coarse aggregates (kg/m ³)	Water (kg/m ³)	Superplasticizer (kg/m ³)	CSF (kg/m ³)	Slump (mm)
0	380.88	709.90	1161.61	171.40	7.20	0	89
0.25	380.88	707.37	1157.46	171.40	7.20	19.63	81
0.50	380.88	704.83	1153.31	171.40	7.20	39.25	79
0.75	380.88	702.30	1149.16	171.40	7.20	58.88	74
1.00	380.88	699.76	1145.01	171.40	7.20	78.50	71

The slump decreased progressively from 89 mm for the control mixture to 71 mm at 1% CSF, corresponding to a reduction of approximately 20.2%. This reduction was attributed to the increased surface area, internal friction and interlocking effects introduced by the fibres. Nevertheless, all mixtures exhibited adequate workability for placement and compaction. A representative view of distribution of CSF in the fresh concrete mix is shown in Fig. 1, indicating adequate fibre dispersion without noticeable agglomeration.



Fig. 1. Distribution of CSF in the concrete mix

5. Design of RCC Deep Beam

Design data:

- Length of beam, $L = 750$ mm
- Depth of beam, $D = 325$ mm,
- Width of beam, $b = 75$ mm
- Support width = 100 mm
- Clear span = 550 mm
- Diameter of circular web opening = 75 mm

Effective depth, $d = 325 - 20 - 8 - (12/2) = 291$ mm, considering a clear cover of 20 mm, an 8 mm stirrup and 12 mm main bar. According to IS 456:2000, effective span of deep beam is smaller of:

- Centre-to-center distance between supports = $750 - 100 = 650$ mm
- 1.15 times clear span = $1.15 (550) = 632.5$ mm
- Consider smaller of above, effective span (l) = 632.5 mm

Effective span by overall depth ratio (l/D) = $632.5/325 = 1.94 < 2.0$; therefore, the specimen satisfied the IS 456:2000 criterion for a simply supported deep beam. For the symmetrical two-point loading system applied at one-third points of effective span, the shear span is,

- $a = 632.5/3 = 210.83$ mm

Therefore, shear span to effective depth ratio = $a/d = 0.72$. Reinforcement details:

- Main tension steel: 2#12 mm diameter HYSD bars.
- Horizontal Side-face reinforcement: #8mm @ 95mm c/c on both faces in 3 layers.

The circular opening interrupted one layer of side-face reinforcement in both DB-C and DB-T specimens. However, two-legged 8 mm diameter stirrups at 230 mm c/c were provided as shear reinforcement in all three beam configurations, namely SDB, DB-C and DB-T. The CSF provided supplementary crack-bridging resistance in the vicinity of the openings [19].

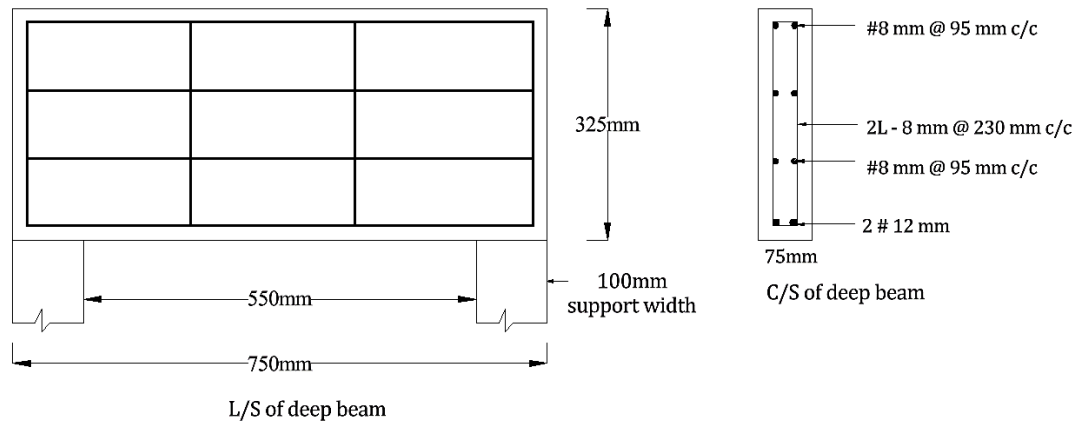


Fig. 2. Schematic illustration of longitudinal and cross-sectional reinforcement details of the deep beam (SDB)

Positioning (locating) of openings in the deep beam: While casting the concrete beams circular web openings were introduced at two different locations in the beam having larger depths: in compression zone (DB-C), in tension zone (DB-T) using PVC pipes of 75 mm outer diameter at the designated locations and the same is removed after the concrete had attained its initial set. The designated location of web openings was oriented based on load transfer mechanism in the deep beams. During the loading process, the load is transferred from source point to supports primarily through compression strut. To minimize the disruption of this load transfer mechanism, the openings are located outside the principal load path within the disturbed region. Accordingly, the openings in web are provided at compression and tension zone. The provision of openings and its size depends on the load-transfer zones and the corresponding geometrical parameters, as illustrated in Fig. 3 and 4.

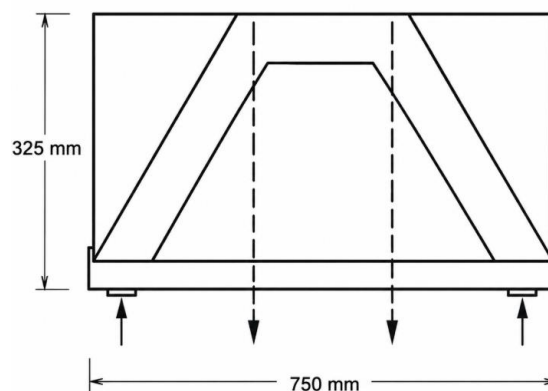


Fig. 3. Load transfer path in deep beam

The failure modes are categorized into two groups:

- Failure associated with compressing of inclined concrete member also called as inclined compression failure
- Bending (flexural) failure, which are represented by means of failure of horizontal concrete tie member.

As illustrated in fig. 4 load path with quadrants 1 and 3 signifies the primary load transfer regions, as 1 and 3 quadrants located adjacent to the loading and support zones. These regions transfer the principal compressive forces from the point of load application to the supports and hence play an

important role in structural performance pertaining to deep beam. In contrast, 2 and 4 quadrants are considered as unloading path or comparatively less stressed regions away from loading path of beam and where actually the openings are preferred.

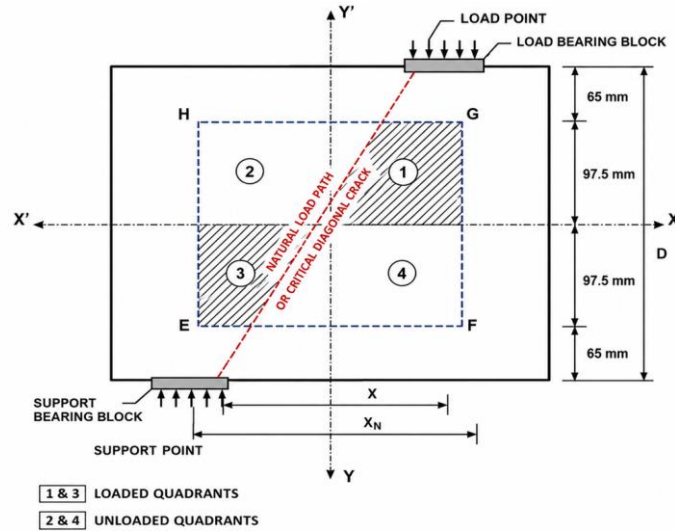


Fig. 4. Loading path with quadrants and geometrical parameters in deep beam

Considering the eccentricity in x and y direction (i.e., e_x and e_y) from center of opening and to the center of diagonal load path plays an important role in choosing the diameter of the opening in the deep beams. However, it is limited [19]:

- $e_x \leq X_N/4$
- $e_y \leq 0.6D/4$
- $X_{net} = (X_N - a_1 X)$ $e_x \leq X_N/4$
- $Y_{net} = (0.6D - a_2 D)$ $e_y \leq 0.6D/4$

Where: X_N = Distance from support point to right edge, X = Distance from support point to load point, D = Overall depth of the member, K_1 & K_2 = Coefficient for critical diagonal = 0.23 & 0.55 respectively. a_1 & a_2 = Opening location parameter = 0.60 & 0.23 respectively. e_x , e_y = Eccentricities of opening center from critical diagonal.

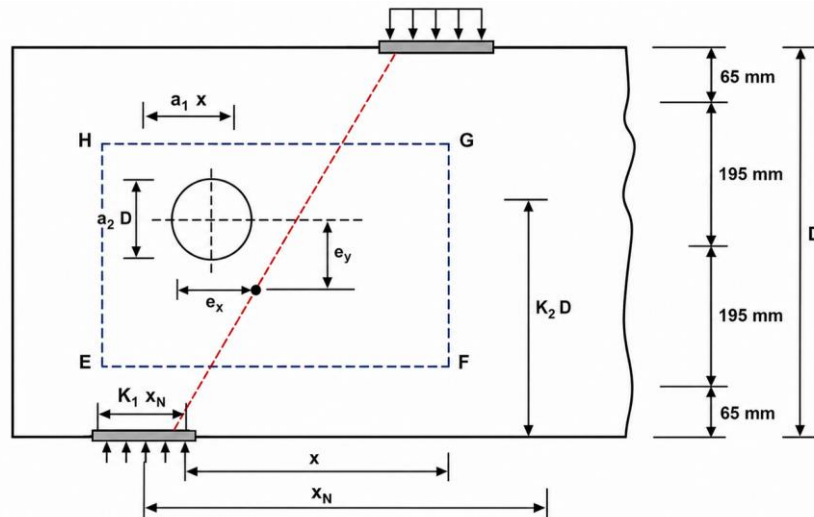


Fig. 5. Schematic representation of the opening location in the deep beam

6. Experimental Work

Standard concrete cubes (150 mm × 150 mm × 150 mm) containing 0%, 0.25%, 0.50%, 0.75%, and 1% CSF were tested using a 2000 kN capacity compression testing machine. The test was performed and reported as per the technical reference IS 516:2021 and IS 456:2000 for 7 days and 28 days curing period. The equivalent cylindrical compressive strength was estimated from the

cube compressive strength using a conversion factor of 0.80, i.e., $f_c = 0.80f_{ck}$, for comparison with cylindrical-based design provisions.

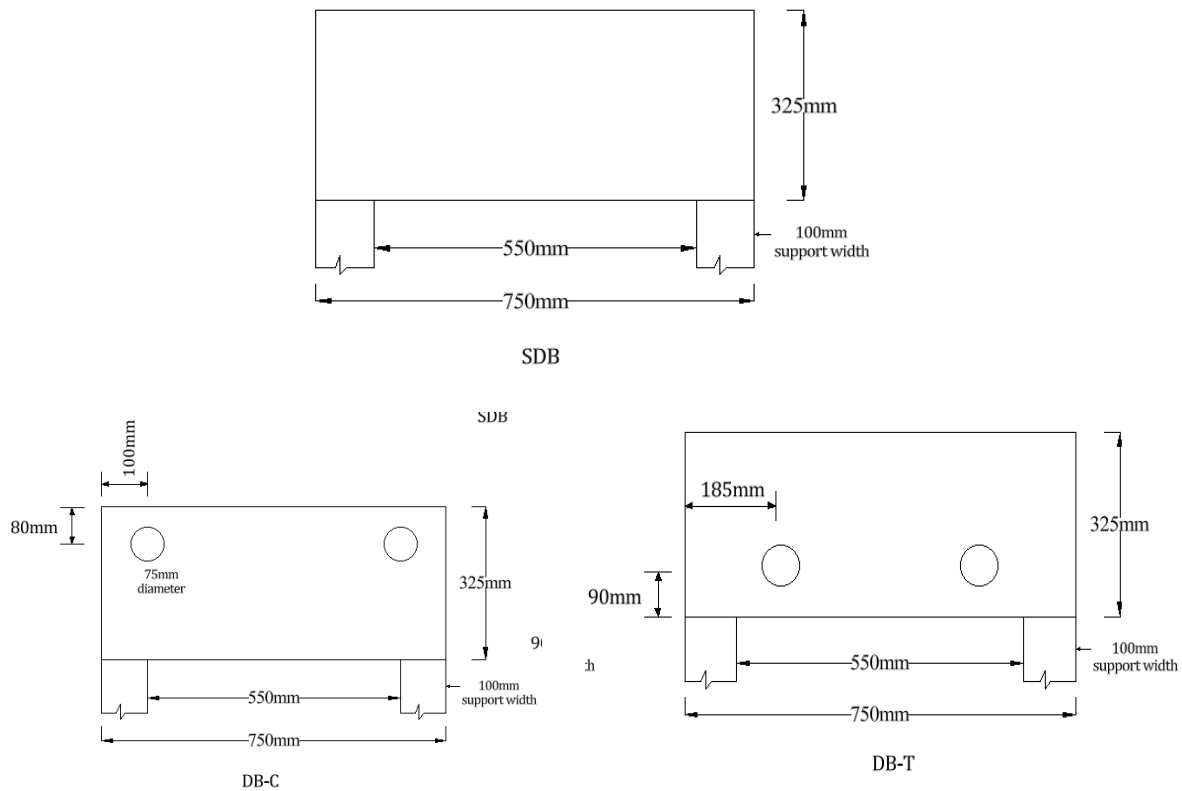


Fig. 6. Schematic representation of deep beam – SDB, DB-C and DB-T respectively

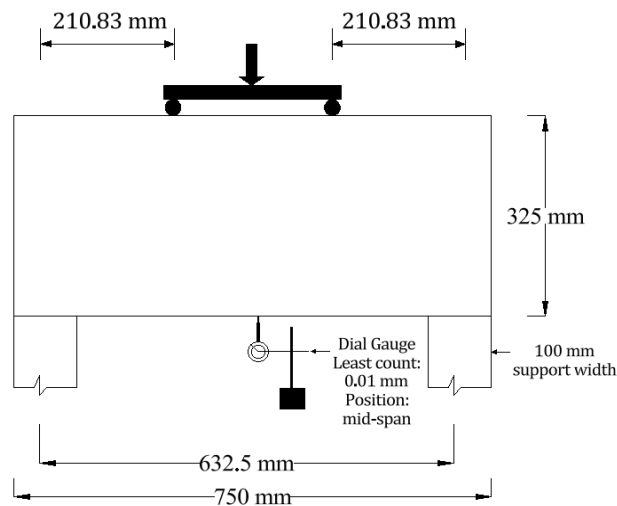


Fig. 7. Schematic representation of symmetric two-point loading setup and dial-gauge position at mid-span

The Shear behavior of the deep-beam specimens was evaluated under symmetric two-point loading applied at the one-third points of the effective span. All specimens were tested after 28 days of curing using a 1000 kN capacity universal testing machine in accordance with the applicable provisions of IS 516 (Part 1):2021.

The dimensions of all the specimens were measured prior to testing and were subjected to incremental loading until failure. During testing the first-crack load, ultimate load, crack initiation with its propagation and extension towards the openings were recorded. The maximum mid-span

deflection was measured using a dial gauge with a least count of 0.01 mm. The dial gauge was mounted on an independent rigid stand, with spindle positioned vertically in contact with the beam soffit at the exact mid-span, as shown in Fig. 7. Following type pertaining to deep beams with 0%, 0.25%, 0.50%, 0.75% and 1% steel fibre content are tested in the experiment work:

- Solid deep beam without openings (SDB)
- Deep beam with circular openings at compression zone (DB-C).
- Deep beam with circular openings at tension zone (DB-T).

Table 4. Measured cube and equivalent cylindrical compressive strengths of concrete with varying CSF content

Percentage of crimped steel fibres (%)	Average density of concrete (kN/m ³)	Average cube Compressive strength (N/mm ²) f_{ck}	Average cylindrical compressive strength (N/mm ²) $f_c = 0.80f_{ck}$
0	23.96	22.00	17.60
0.25	24.18	24.10	19.28
0.50	24.37	27.50	22.00
0.75	24.61	31.85	25.48
1.00	24.98	33.75	27.00

7. Results and Discussion

The laboratory investigation was performed to ascertain the influence of load-path oriented circular opening position on their load carrying behavior. A comparative assessment was performed for solid beam without opening (SDB), deep beam with circular opening in compression zone (DB-C) and deep beam with circular opening made in tension zone (DB-T). Each specimen responses for first crack load, the ultimate load, deflection along with crack propagation were analyzed. The numerical results represent the arithmetic means of three replicate specimens for each configuration and CSF volume fraction. Load values were rounded to the nearest 1 kN, while deflection values were reported to two decimal places.

7.1 Load Deflection Behavior of Solid Deep Beam (SDB)

Table 5 presents the experimental results indicating the addition of steel fibers enhance the structural response by delaying the first-crack load, increasing ultimate load capacity, and reducing maximum deflection. The addition of crimped steel fibres significantly enhances the behavior of solid deep beam (SDB).

Table 5. Load – Deflection of solid deep beam without opening for various CSF content

Percentage of crimped steel fibres (%)	Number of specimens	SDB		
		Mean First Crack Load (kN)	Mean Ultimate Load (kN)	Mean Maximum Deflection (mm)
0	3	74	164	2.50
0.25	3	80	168	2.46
0.50	3	85	170	2.41
0.75	3	98	178	2.36
1.00	3	100	180	1.80

The first crack load obtained for solid deep beam increased from 74 kN to 100 kN for the control specimen representing an improvement of approximately 35% at 1% CSF content. Similarly ultimate load also increases from 164 kN to 180 kN representing an improvement of approximately 9.8% at 1% CSF content. This higher improvement in first-crack load compared with ultimate load indicates that the primary contribution of the fibres was associated with crack initiation and crack-control mechanisms. CSF which are randomly distributed, bridge developing microcracks and

transfer tensile stresses across the cracked concrete matrix, thereby delaying the formation and propagation of visible cracks. At 1% steel fibre dosage, the maximum deflection was reduced by about 28% compared to control specimen

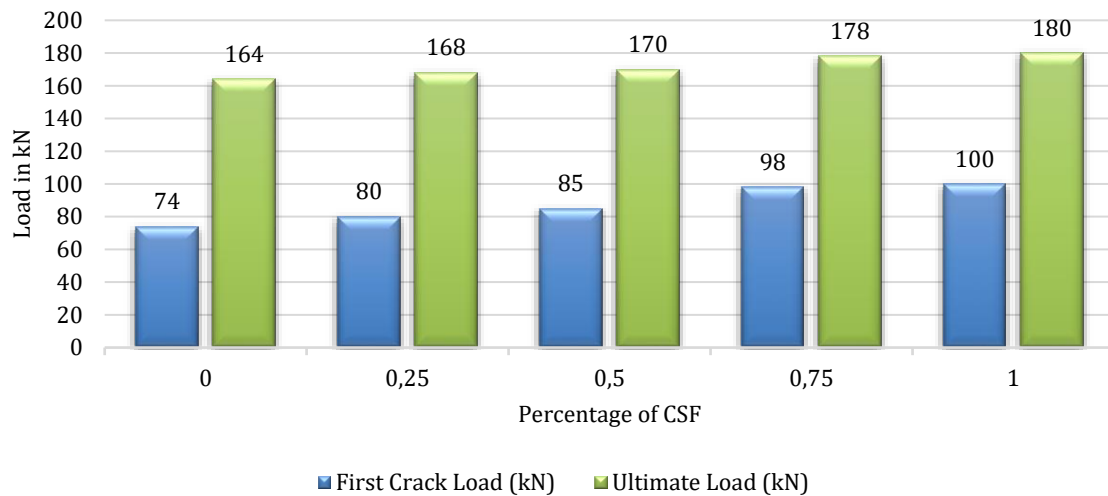


Fig. 8. First crack load and ultimate load for SDB

7.2 Load Deflection Behavior pertaining to deep beam with Circular Opening in Compression Zone (DB-C)

The behavior pertaining to deep beam with circular web opening in compression zone (DB-C) also demonstrated a progressive improvement with increase in crimped steel fibre content. Compared to the control specimen the first crack load increased from 68 kN to 96 kN representing an increase of about 41.2% for 1% fibre. Similarly, the ultimate load also increases from 160 kN to 175 kN with an improvement of 9.4% approximately compared to control specimen. The maximum deflection is reduced by 24% by adding up to 1% fibre content in an increment of 0.25%.

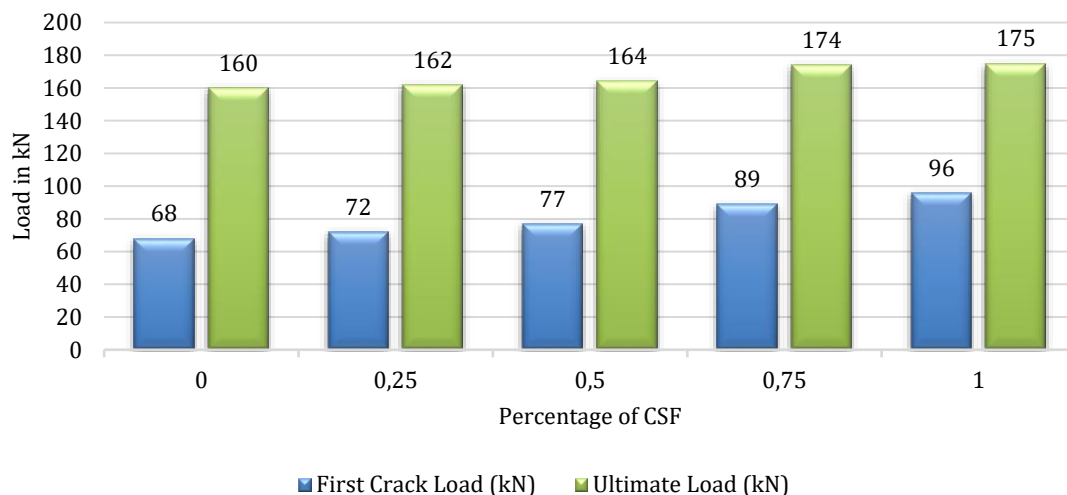


Fig. 9. First crack load and ultimate load for DB-C

Table 6. Load – Deflection pertaining to deep beam with opening in compression zone for various CSF content

Percentage of crimped steel fibres (%)	Number of specimens	DB-C		
		Mean First Crack Load (kN)	Mean Ultimate Load (kN)	Mean Maximum Deflection (mm)
0	3	68	160	2.58
0.25	3	72	162	2.52
0.50	3	77	164	2.29
0.75	3	89	174	2.14
1.00	3	96	175	1.96

7.3 Load Deflection Behavior pertaining to deep beam with Circular Opening in Tension Zone (DB-T)

The DB-T specimens also exhibited a comparable response for addition of crimped steel fibres. Approximately 41.9% of increase in first crack load is achieved with 10.5% of increase in ultimate load carrying capacity pertaining to deep beam at 1% steel fibre content. The maximum deflection of the DB-T specimens did not exhibit a strictly monotonic trend with increasing CSF content. Intermediate fluctuations were observed; however, the maximum deflection decreased overall from 3.08 mm for the specimen without CSF to 2.70 mm at 1% CSF, corresponding to a reduction of 12.34%. The observed fluctuations may be associated with experimental variability arising from random fibre distribution and orientation, localized stress concentration around the tension-zone openings, variation in diagonal crack paths and differences in fibre-bridging efficiency.

Table 7. Load – Deflection pertaining to deep beam with opening in tension zone for various CSF content

Percentage of crimped steel fibres (%)	Number of specimens	DB-T		
		Mean First Crack Load (kN)	Mean Ultimate Load (kN)	Mean Maximum Deflection (mm)
0	3	62	152	3.08
0.25	3	67	156	3.00
0.50	3	72	158	3.14
0.75	3	82	166	3.34
1.00	3	88	168	2.70

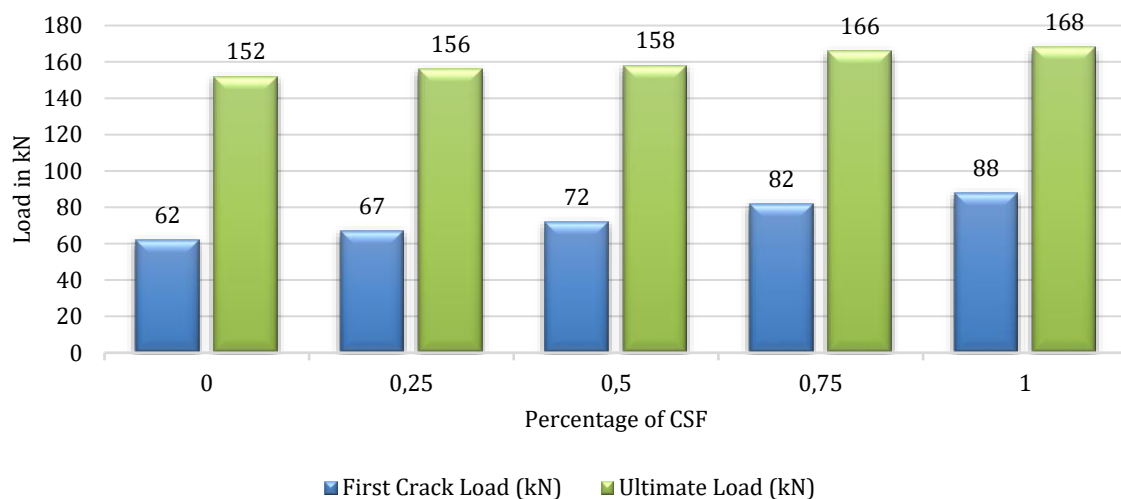


Fig. 10. First crack load and ultimate load for DB-T

The relative assessment of all three types of beam configuration demonstrates that the solid deep beam (SDB) exhibits the highest structural performance, followed by beam having circular web opening made in compression zone (DB-C), while the deep beam with circular web opening in tension zone (DB-T) recorded the lowest load resisting capacity. The placement of circular opening in the compression zone (DB-C) results smaller reduction in first crack load of about 4% and ultimate load of 2.8% while placing the opening in tension zone (DB-T) results in higher reduction of first crack load of 12% and ultimate load of 6.7% when compared with solid beam without opening (SDB)

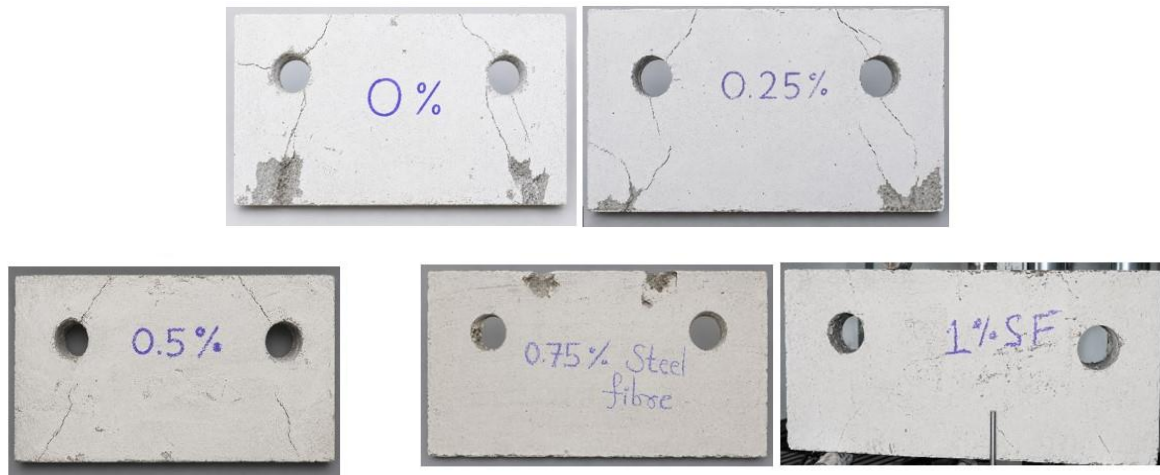


Fig. 11. Failure patterns of DB-C specimens with different CSF contents at ultimate load

The above figure 11 presents the representative post-failure crack pattern of DB-C specimens with different CSF contents, ranging from 0% to 1%. The figure 11 is provided to illustrate the development and propagation of cracks around the circular web openings and the influence of increasing CSF content on the failure behavior. Similar observations were recorded for the SDB and DB-T specimens.

7.4 Theoretical Formula to Find Ultimate Shear Strength of the Deep Beams

According to Professor F. K. Kong [19], the expression for the factored shear-resisting capacity pertaining to deep beam is given by equation below:

$$Q_u/bD = (P_u/2bD) = \xi_1[0.1 f_c (\lambda_1) (\lambda_2) (\lambda_3) + 0.0085 \Psi_s p_s f_{sy} + 0.01 \Psi_w K_w r_w f_{wy}] \quad (3)$$

Where; Q_u = ultimate shear bearing with single concentrated load, $P_u/2$ considered for two-point loading system. b = breadth pertaining to deep beam = 75 mm, D = Overall depth of beam = 325 mm, ξ_1 = performance factor or safety factor for beam failing in shear = 0.75, f_{sy} , f_{wy} = yield strength of longitudinal bar and web bar respectively (N/mm^2) for 12 mm diameter steel = 515 N/mm^2 and for 8 mm diameter steel = 510 N/mm^2 . f_c = cylindrical (150 mm dia.×300 mm height) compressive strength of concrete, Ψ_s = empirical co-efficient = 0.65, Ψ_w = empirical co-efficient = 0.50. p_s = (A_s/bD) * 100, expressed in percentage = 0.928, r_w = (A_w/bD) * 100, expressed in percentage = 0.412, K_w = 0.50. λ_1 , λ_2 , λ_3 = empirical co-efficient of deep beam with and without openings based on opening parameters and mentioned in the table 8 below.

Table 8. Empirical co-efficient used for Professor F. K. Kong's formula for ultimate shear strength prediction

Parameters	SDB	DB-C	DB-T
λ_1	1	0.88	0.67
λ_2	1	1	1
λ_3	0.72	0.55	0.52

Two-legged 8 mm diameter stirrups at 230 mm c/c were provided in SDB, DB-C and DB-T. Therefore, area of the two stirrups legs was $A_w = 100.53 \text{ mm}^2$. Accordingly, $r_w = (A_w/bD) \times 100 = 0.412\%$. Hence, $r_w = 0.412\%$ was used for SDB, DB-C, DB-T. The configurations were differentiated through their respective λ_1, λ_2 and λ_3 coefficients. For example, at 0% CSF, using $f_c = 17.60 \text{ N/mm}^2$, $p_s = 0.928\%$, $r_w = 0.412\%$, $\Psi_s = 0.65$, $\Psi_w = 0.50$ and $K_w = 0.50$, the theoretical ultimate loads were 162.08 kN for SDB, 146.90 kN for DB-C and 138.17 kN for DB-T. These were rounded to 162, 147 and 138 kN respectively, in Table 9.

Since the original Kong formulation was developed for conventional reinforced concrete deep beams and does not specifically account for the steel fibre volume fraction, the effect of increasing CSF content was considered indirectly through the fibre-modified concrete properties. In the present study, the experimentally determined cube compressive strengths for each CSF content were converted to equivalent cylindrical compressive strengths and incorporated into the theoretical calculation. Consequently, the theoretical model was used as a comparative approach to evaluate the influence of changes in fibre-modified concrete strength rather than a fibre-specified predictive model

Table 9. Theoretical ultimate load of deep beams with varying CSF contents

Percentage of crimped steel fibres (%)	SDB	DB-C	DB-T
	Ultimate Load (kN)	Ultimate Load (kN)	Ultimate Load (kN)
0	162	147	138
0.25	167	150	140
0.50	174	155	144
0.75	183	161	148
1.00	187	164	150

The theoretical prediction obtained using developed analytical model provides good agreement with experimental results of crimped steel fibre reinforced concrete deep beam with and without circular web openings. A variation of 0.6% to 10.8% between the theoretical and experimental values indicates satisfactory agreement between two sets of results. The analytical model (theoretical values) provides a reasonably accurate prediction of the experimental ultimate load of steel fibre-reinforced concrete deep beams, including SDB, DB-C and DB-T. The maximum variation of results comparable with experimental work and theoretical values are about 10.8% and minimum variation is about 0.60%.

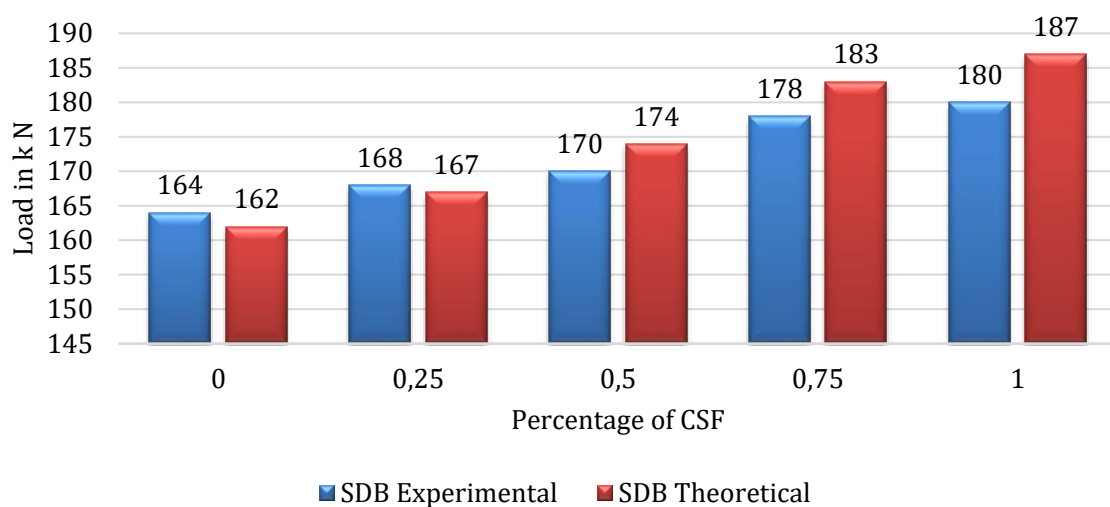


Fig. 12. Comparison of experimental and theoretical ultimate loads of SDB with varying CSF

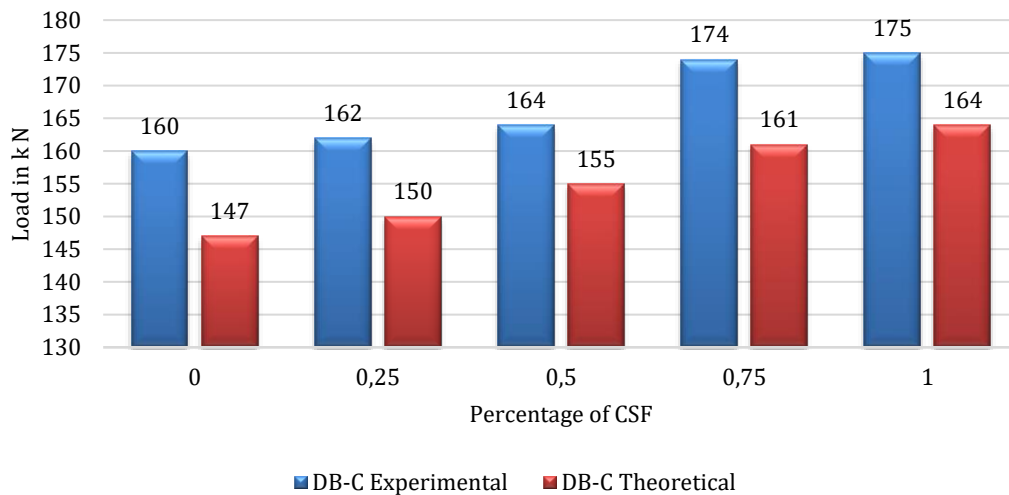


Fig. 13. Comparison of experimental and theoretical ultimate loads of DB-C with varying CSF

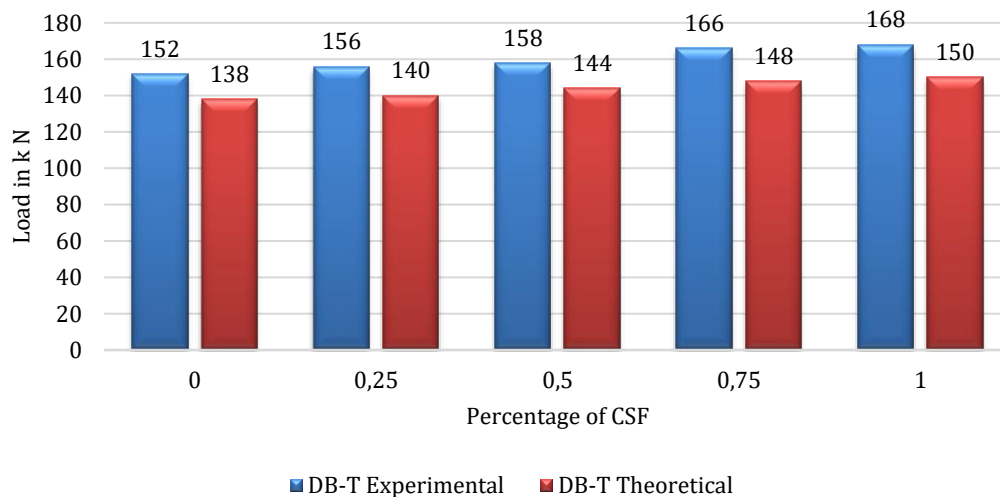


Fig. 14. Comparison of experimental and theoretical ultimate loads of DB-T with varying CSF

8. Conclusion

The introduction of circular web openings disrupted the natural load-transfer mechanism of the deep beams. Compared with SDB, DB-T exhibited a maximum reduction of 16.33% in the first-crack load at 0.75% CSF, decreasing from 98 kN to 82 kN. The maximum reduction in ultimate load-carrying capacity was 7.32% at 0% CSF, decreasing from 164 kN for SDB to 152 kN for DB-T. Across the investigated CSF contents, both DB-C and DB-T specimens exhibited crack development around the circular openings, with cracks extending towards the loading and support regions. These observations demonstrate the influence of opening location on the load-transfer mechanism and failure behavior of deep beams. The position of circular web opening relative to load path substantially influenced the structural behavior of specimens. Averaged over the five CSF contents, DB-C exhibited 8.34% and 4.38% higher first-crack and ultimate loads, respectively, than DB-T. At 1% CSF, DB-C also exhibited 9.09% and 4.17% higher first-crack and ultimate loads, respectively, than DB-T. These results indicate that positioning the circular opening in the compression zone was less detrimental than positioning it in the tension zone. The incorporation of CSF at volume fractions ranging from 0.25% to 1%, in increments of 0.25%, improved the first-crack and ultimate loads compared with the corresponding specimens without CSF. At 1% CSF, SDB exhibited increases of 35.14% (26 kN) in the first-crack load and 9.76% (16 kN) in the ultimate load. The corresponding increases for DB-C were 41.18% (28 kN) and 9.38% (15 kN), respectively, while DB-

T exhibited increases of 41.94% (26 kN) and 10.53% (16 kN), respectively. The improved response was attributed to the crack-bridging action of CSF, which restrained crack propagation and enhanced load transfer after cracking. Among the investigated CSF fibre volume fractions, 1% CSF provided the greatest improvement in load-carrying performance. The theoretical ultimate-load predictions showed reasonable agreement with the experimental results, with variations ranging from 0.6% to 10.8%. This agreement supports the applicability of Kong's model for estimating the ultimate shear-carrying capacity of the tested CSF-reinforced concrete deep beams with circular web openings. However, its applicability should be limited to the investigated material properties, specimen dimensions, opening geometry and loading conditions until further experimental validation is available.

9. Limitations and Future Scope

- The width of the designed reinforced members (75 mm) used in this study may introduce scale effects, and the laboratory-scale specimens may not be fully representing full-scale behavior.
- The use of CSF can potentially reduce conventional shear reinforcement, thereby simplifying reinforcement fabrication and reducing labor and construction time. However, a detailed cost assessment is required to quantify the economic benefits.
- Future studies should investigate different opening positions, size and shapes such as square and rectangular openings.
- The influence of alternative fibre types, such as polypropylene, glass, basalt and synthetic fibres can be investigated in future studies

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References

- [1] Ghalla M, Elsamak G, El-Zohairy A, Youssf O, Khairy S, & Badr el-din A. Experimental investigation on the structural performance of strengthened reinforced concrete deep beams using steel-based and SHCC techniques. *Structures*, 2026, 85: 111074. <https://doi.org/10.1016/j.istruc.2026.111074>
- [2] Sogut K, Ercan B. Behaviour of unstrengthened and DE CFRP-strengthened high-strength concrete deep beams with web openings. *Frontiers in Materials*, 2025, 12: pp. 1-15. <https://doi.org/10.3389/fmats.2025.1661180>
- [3] Kannan H, Veerappan S K and Sivasubramanian M V R. Experimental and numerical investigation of shear performance of RC deep beams strengthened with engineered cementitious composites. *Construction Materials*, 2025, 5(3), pp. 1-16. <https://doi.org/10.3390/constrmater5030051>
- [4] Khalaf D, Chassib S M. Shear Behaviour of RC Deep Beams Made with Recycled Bricks: Comparative Study. *Civil and environmental engineering*, 2025, 21(2): pp. 788-811. <https://doi.org/10.2478/cee-2025-0057>
- [5] Shallan M E, Khalil M, Mahmoud A and El-Diasity M H. Shear performance of continuous deep beams reinforced with steel, GFRP bars, stirrups, and novel grating system: an experimental study. *Materials and Structures*, 2025, 58(7): 231. <https://doi.org/10.1617/s11527-025-02751-5>
- [6] Shabanlou M, et al. Experimental and analytical study of the residual performance of reinforced concrete deep beams with circular web openings. *Results in Engineering*, 2025, 25: 104229. <https://doi.org/10.1016/j.rineng.2025.104229>
- [7] Abbas H, Al-Mahbashi M, Elsanadedy H, Abadel A, Almusallam T and Al-Salloum Y. Strengthening of high-strength RC continuous deep beams having rectangular web openings using bolted steel plates. *Engineering Structures*, 2025, 337: 120519. <https://doi.org/10.1016/j.engstruct.2025.120519>
- [8] Daudi S A. Shear performance of deep concrete beams with openings using waste tyre steel fibres: FEM and ANN analysis. *Civil Engineering Journal*, 2024, 10(8): pp. 2422-2449. <https://doi.org/10.28991/CEJ-2024-010-08-02>
- [9] Hasan AF, Al Amlin AS. Shear strengthened of R.C hollow deep beams with large opening by plates: A Review. *Al-Rafidain Journal of Engineering Sciences*, 2025, 3(1):89-112. <https://doi.org/10.61268/mcvakq81>

- [10] Mona S, Mohammad A, Mohamed Zakaria. Shear capacity prediction for reinforced concrete deep beams with web openings using artificial intelligence methods. *Engineering Structures*, 2023, 280: 115675. <https://doi.org/10.1016/j.engstruct.2023.115675>
- [11] Farouk M A, Moubarak A M, Ibrahim A and Elwardany H. New alternative techniques for strengthening deep beams with circular and rectangular openings. *Case Studies in Construction Materials*, 2023, 19: e02288. <https://doi.org/10.1016/j.cscm.2023.e02288>
- [12] Nancy K, Tamer E, Hilal E, Bilal E. Shear response of recycled aggregates concrete deep beams containing steel fibres and web openings. *Sustainability*, 2022, 14(2): 945. <https://doi.org/10.3390/su14020945>
- [13] Waleed A J, Abbas A and Nazar K A. Effect of size and location of square web openings on the entire behavior of reinforced concrete deep beams. *Civil Engineering Journal*, 2019, 5(1): pp. 209-226. <https://doi.org/10.28991/cej-2019-03091239>
- [14] Moradi M, Reza M. Application of the strut-and-tie method for steel fiber reinforced concrete deep beams. *Construction and Building Materials*, 2016, 131:423-437. <https://doi.org/10.1016/j.conbuildmat.2016.11.042>
- [15] Azharuddin N H, Pankaj B. Experimental Evaluation Shear Strength of Steel Fibre Reinforced Concrete Deep Beam. *International Journal of Engineering Research & Technology*, 2016, 5(5): pp. 97-106.
- [16] Ahmed S, Ihsan A S, Sabih Z. Effect of steel fiber on the behavior pertaining to deep beams with and without web opening. *Engineering and Technology Journal*, 2011, 29(1): pp. 1-19.
- [17] Naik U, Kute S. Span-to-depth ratio effect on shear strength of steel fiber reinforced high-strength concrete deep beams using ANN model. *International Journal of Advanced Structural Engineering (IJASE)*, 2013, 5(1): 29
- [18] Vengatachalapathy V, Ilangovan R. A Study on Steel Fibre Reinforced Concrete Deep Beams With and without Openings. *International Journal of Civil and Structural Engineering*, 2010, 1(3): pp. 509-517.
- [19] Kong F K. Reinforced concrete deep beams. CRC Press, 1990.
- [20] Shetty M S, Jain A K. Concrete Technology: Theory and Practice. 8th edition New Delhi, India, S Chand Publishing, 2019.

Compressibility and strength enhancement of clayey soil using magnesium chloride solution

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Article Info	Abstract
Article History: Received 02 June 2026 Accepted 13 June 2026 Keywords: Clayey soil; MgCl ₂ ; Compressibility; Unconfined compressive strength; Shear strength parameters	One of the most important problems of clayey soils are their susceptibility to compression when loads are applied due to weak bonds that allow water to enter and exit between soil particles. In this research, clay soil with a clay content of (72%) from Salah Al-Din governorate, Iraq was used. This soil was treated with magnesium chloride solutions at different concentration ratios (5, 10, and 15%). Compressibility and strength tests were performed on natural and treated soil, results showed that the compression decreased significantly with increasing magnesium chloride concentrations, the final strain after unloading was (1.8, 1.72, 1.0, and 0.95) % at concentrations of (0, 5, 10, and 15) % respectively. It is also noted a significant increase in the unconfined compression strength with enhancing ratio of (21, 27, 38) % at enhancing ratios of (5, 10, 15) % respectively. For direct shear test, cohesion of soil increased by (66, 94, 97) %, while the increase in value of soil's friction angle was (13, 5, 7) % at a solution content of (5, 10, 15) %. The results shows that the enhancement in soil strength is small and almost constant when the magnesium chloride content increases by more than 10% due to soil saturation.

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1. Introduction

Soil stabilization is the process employed to improve soil characteristics such as compressibility and shear strength. When the soil has a variety of engineering characteristics that are usually unfavorable makes it unsuitable for carrying structural loads, soil improvement is needed [1]. Most of the design practices in geotechnical engineering are based on settlement and strength criteria when the allowable bearing capacity is based on the minimum value of settlement or soil strength [2]. The total settlement distribution is based on characteristics of soil, clayey soil mostly exhibited greater settlement than sandy soil [3]. Therefore, it requires more attention in its improvement than granular soils.

Chemical stabilization which entails adding chemical materials such as lime, cement and chemical solution to improve clayey soils is widely used for many researchers. Physical and mechanical properties of clayey soil improved significantly by adding chemical stabilizers, which devolve the ability of using the chemical technique in soil stabilization to solve many geotechnical problems [4]. Deep soil mixing (DSM) is also one of the practical and effective methods for soil improvement that increases strength of soil and decreases its compressibility [5]. Depending on the amounts of chemical additives and the time interval of the test, physical and mechanical properties of the treated clayey soil could be considerably improved [6]. Major conclusions from experimental investigations on the effects of adding lime and cement on properties of expansive clay including Atterberg limits, swell, and linear shrinkage were presented by [7]. Hasan et al. also concluded that lime- silica fume additives improve both shear strength and unconfined compressive strength of soft clay soil [8]. Sodium silicate and Portland cement mostly act as active stabilizers because of their ability to alter

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soil properties, such as strength [9]. Class F fly ash can effectively stabilize the clay soil characteristics throughout improving plasticity and compaction behavior, unconfined compressive strength as well as shear strength [10]. Zhang et al. investigated the effect of presence of calcium chloride on the intermittent current efficiency for the dewatering and consolidation of silty clay. Solution of calcium chloride at 15% were used during electric treatment process with 0, 1, 2, 4, 8 h intervals time. Comparison between conventional electro osmosis and intermittent current were made. The results showed that calcium chloride enhance soil properties, it increased shear strength of soil as well as reduced the corrosion of anode. Power consumption caused by CaCl_2 injection gave more economically process throughout practical projects [11]. Effect of chloride salts on properties of expansive soil also investigated by El Kady et al., it is concluded that index properties and swelling behavior of soil was improved with increasing chloride salt [12].

Currently, magnesium chloride (MgCl_2) is mainly used in soil improvement; however, it has been effectively used for improving clayey soils. Previous studies in literature indicated that MgCl_2 is effectively used in improvement of soil as an antifreeze agent more than used in stabilization. MgCl_2 is also used in controlling humidity and dust as well as reducing the distraction that take place in coarse particles from surfaces of roads. Manikantha et al. stated the effectiveness of chemical stabilization using sodium hydroxide (NaOH) and magnesium chloride (MgCl_2) as soil stabilizers for improving soils characteristics. Unconfined compression was performed on black cotton soil treated with different concentrations of these chemical agents. 1.5% of NaOH and MgCl_2 with different molarities (2, 4, 6, 8, and 10) M was used to evaluate the improvement in mechanical strength and overall geotechnical behavior of the soil. Results indicate that increasing the salt concentration enhances the unconfined compressive strength (UCS) with significant improvement in soil stability. This shows the possibility of using chemical stabilization as a useful technique for improving expansive soils for engineering applications [13]. Habibbeyg and Nikraz investigate the effect of green stabilizer by the magnesium chloride (MgCl_2) on the geotechnical behavior of the treated clay soil by conducting several laboratory tests including the consistency limits, free swell strain as well as swell pressure of the treated samples. The results state that even a small percentage of MgCl_2 was used, it can stabilize expansive clay by improving the soil properties considerably and mitigate their swell potential effectively [14]. The results obtained by Teker et al. showed that the undrained shear strength of the kaolin clay increased as the percentages of MgCl_2 and MgO rose [15]. As well as Latifi et al. indicated that the strength of MgCl_2 -stabilized peat improved significantly when the degree of improvement reaches approximately six times than that for untreated peat [16]. Also, Sharma et al. concluded that most geotechnical properties of black cotton soil are enhanced at 8% MgCl_2 [17]. Sadiq et al. concluded that the compressibility and strength properties of clay soil improved greatly by using nano-magnesium oxides due to the ability of nano-magnesium to reduce pore size [18].

Rikhtehgar and Teymur using (MgCl_2) to improve the freeze-thaw properties of clay soils. The results showed that MgCl_2 reduces the consistency limits of soil as well as enhance durability and compressive strength. Using 14% of MgCl_2 solution keep volume of soil its strength under a temperature of -20°C , similar enhancing behavior was found for soil at -10°C temperature treated by 14% and 9% MgCl_2 solutions. Soil stability by MgCl_2 during initial stabilization was improved under the effect of freeze-thaw cycles, giving an effective solution for improving construction under cold conditions [19]. Tang et al. studied the effect of solution of MgCl_2 on the process of electrokinetic stabilization. Soil of marine clay from Zhuhai, China is subjected to six electrokinetic experiments. Results showed using 0.8% MgCl_2 solution with 20% concentration represent the effective percentage for the electrokinetic stabilization, greater concentration of MgCl_2 solution reduced the anode corrosion amount [20].

The effect of (MgCl_2) solution on the different engineering properties of clay soils was investigated by Turkoz et al. Swell pressure, percentage of swell, pinhole and crumb as well as shear strength by triaxial tests was studied at different curing intervals. Analysis by scanning electron microscopy (SEM) also used to observe the difference in sample microstructures. The results show that using solution of MgCl_2 effectively improved dispersive and expansive clay soils [21]. Hassan et al. stabilize organic soil with magnesium chloride (MgCl_2) to study its effect on strength properties and micro structural behavior of soil. Field-emission scanning electron microscopy (FESEM) as well as energy-dispersive X-ray spectrometry (EDAX) methods was used to observe changes in micro

structural whereas unconfined compression tests were performed to evaluate properties of shear strength. Results pointed that the unconfined compressive strength of organic soil improved by using MgCl_2 when the strength of soil is enhanced by about (3–5) times compared with that of unimproved soil at the first 7 days of curing. FESEM results gave a clear improvement of organic soil properties throughout enhance the soil porosity which filled by a cementation component material defined as magnesium silicate hydrate (M-S-H) [22].

Jahidin et al. studied the geotechnical properties of marine clay treated with MgCl_2 . Results indicated that the unconfined compressive strength increased with an addition of 6% MgCl_2 until the curing time of seven day and after that the value of strength begin to decrease. The increase in strength was attributed to the cementation products formation that led to enhance bonding between soil particles and also filling the spaces between particles of soils together with MgCl_2 . Increase applied surcharge cause soil particles to begin repel and consequently causing of soil strength reducing [23]. Kumar and Ganesh concluded that the chemical additives such as cement, MgCl_2 and fly ash based geopolymer showed a clear influence on the compaction, penetration and strength characteristics of expansive soil [24]. It is shown by Türköz et al. that the problematic clay soils can be sufficiently stabilized by using magnesium chloride solution, the effective percent of admixtures is 7 percent [25]. Mahdavian et al. examined the magnesium chloride solution effects of on clay soil properties. Different percentages of (MgCl_2) were used which are 3%, 5%, 7% and 10%. The experimental program included consistency limits, compaction characteristics, swelling behavior, unconfined compression test and scanning electron microscopy (SEM) tests. Results indicated that there is a clear reduction in swelling percentage compared to untreated soil with a percentage ratio of 4.95%, 3.98%, 2.8%, and 3.9% for samples treated with 3%, 5%, 7%, and 10% MgCl_2 respectively. Additionally, the improvement in the soil properties as conducted by SEM results was attributed to chemical reactions between the soil particles and MgCl_2 solution [26].

Although previous studies generally agree that magnesium chloride (MgCl_2) can improve the engineering behavior of clay soils, the reported optimum dosage and the dominant improvement mechanisms vary considerably. Some researchers reported maximum strength enhancement at relatively low MgCl_2 contents (about 6–8%), whereas others suggested that higher concentrations may be required depending on soil mineralogy and testing conditions. Likewise, while several studies attributed the improvement primarily to cementation and pore-filling mechanisms, others emphasized modifications in particle arrangement and reduction of diffuse double-layer thickness as the governing factors. These variations indicate that the effectiveness of MgCl_2 treatment is strongly dependent on soil type and environmental conditions. Furthermore, most previous investigations focused on consistency limits, swelling characteristics, or unconfined compressive strength, whereas comparatively less attention has been given to the combined evaluation of compressibility behavior and shear strength parameters of naturally occurring clay soils from Iraq. Therefore, additional studies are required to better understand the influence of MgCl_2 stabilization on both deformation and strength characteristics under different loading conditions. Therefore, this study aims to address this gap by providing a comprehensive assessment of the influence of different MgCl_2 concentrations on the geotechnical behavior of the investigated low plasticity clay soil (CL) obtained from Salah Al-Din Governorate, Iraq and determining the optimum treatment level for engineering applications.

2. Materials

2.1. Soil Properties

Soil samples were obtained from a site in Salah Al-Din governorate, Iraq from a depth of 1.0 m. The natural soil used in this study is characterized by a light brown color, a smooth texture, and a naturally aggregated (cloddy) structure, Fig.1 shows the location of the studied site. Standard Proctor tests were conducted to determine the compaction characteristics as shown in Fig. 2.

Physical properties of soil are found according to [27–30] and shown in Table 1. The relatively high maximum dry unit weight (18.68 kN/m^3) observed for the tested CL soil may be attributed to its mineralogical and chemical composition. In particular, the presence of iron oxide (Fe_2O_3) at a concentration of 4.81% contributes to an increase in the specific gravity of the soil solids. chemical

composition is shown in Table 2, this composition was conducted in chemical laboratory at university of Technology in Baghdad by XRD method. The soil samples were prepared and compacted at the maximum dry unit weight and the optimum moisture content containing 0%, 5%, 10%, and 15% by weight of magnesium chloride solution in a special mold (same specifications as the standard Proctor mold) and then saturated with same water containing the specified concentrations.

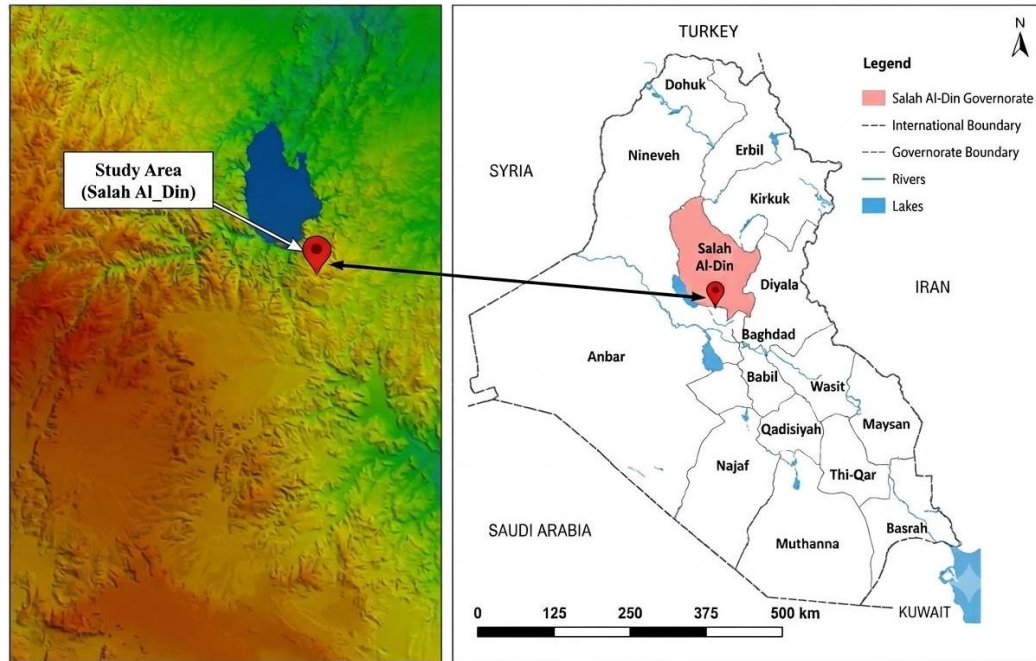


Fig. 1. Location of the study area in Salah Al- Din governorate, Iraq

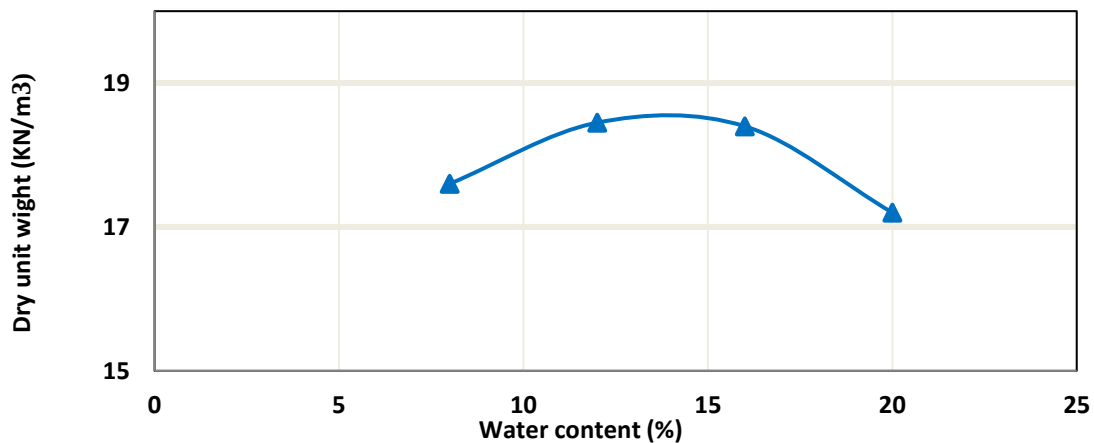


Fig. 2. Relation between dry unit weight and water content in compaction test for the used soil

The saturation process was carried out by gradually introducing the magnesium chloride solution at the top of the compacted specimen in mold while maintaining free drainage at the bottom. Saturation was considered complete when the solution began to emerge from the bottom of the specimen, indicating that the solution had permeated the entire soil sample. Due to the low permeability of the clay soil (containing 72% clay), the saturation process required approximately twenty days to ensure full penetration of the solution throughout the specimen. After the completion of saturation, the specimens were carefully extracted from the molds using a sample extraction jack and cut to prepare according to the requirements of the designated laboratory tests, including the consolidation test (ASTM D2435/D2435M-20), unconfined compression test (ASTM D2166/D2166M-16(2021)), and direct shear test (ASTM D3080/D3080M-23).

Sample preparation stages are shown in Fig. 3. During the specimen saturation period, no significant volume change or swelling was observed because the specimen is confined from top and bottom as shown in Fig 3 (c). Therefore, volumetric changes were not considered to affect the testing program or the interpretation of the results.

Table 1. Physical properties of clay soil

Properties of soil	Values	Specification
Specific Gravity (Gs)	2.885	ASTM D854-23
Liquid Limit (LL) %	48.52	ASTM D4318-17
Plastic Limit (PL) %	23.61	
Plasticity Index (PI)	24.91	
Soil classification according to USCS	CL	
Maximum dry unit weight (KN/m ³)	18.68	ASTM D698-12 (2021)
Optimum moisture content (%)	14.37	
Sand (%)	7	
Silt (%)	21	ASTM D7928-21
Clay (%)	72	

Table 2. Chemical composition of clay soil

Chemical composition	Values
SiO ₂	49.32
Al ₂ O ₃	25.67
Fe ₂ O ₃	4.81
K ₂ O	5.73
MgO	1.98
TiO ₂	2.06
SO ₃	2.64
CaO	3.41
Na ₂ O	4.12
losses	0.26



(a)



(b)



(c)

Fig. 3. Samples preparation stages (a) Compaction stage, (b) Saturation stage, and (c) Extraction stage

2.2. Magnesium Chloride (MgCl_2)

Magnesium chloride in the form of pure white powder that manufactured in Chemical Reagent Co., Ltd. in China with a purity of 99% was used in this study as a solution form after dissolved in distilled water. The magnesium chloride hexahydrate ($\text{MgCl}_2 \cdot 6\text{H}_2\text{O}$) solution was prepared by dissolving a specified weight of magnesium chloride in distilled water according to the concentrations specified in the research.

3. Results and Discussion

After preparing the magnesium chloride solution and preparing the test specimens with maximum density and optimal moisture, the specimen was saturated with magnesium chloride solution at concentrations of (5, 10, 15) %. After removing the specimen from the mold, the specimen was cut to perform strength tests on the specimens before and after adding the magnesium chloride solution. The results were as follows:

3.1. Compressibility Test

By conducting a compressibility test on the samples before and after adding the concentrations of magnesium chloride solution, the results indicate a clear decrease in compressibility with increasing magnesium chloride concentration. Using magnesium chloride (MgCl_2) solutions can reduce the compressibility of clay soil by increasing its strength and density through chemical and physical changes. The stabilization mechanisms are based on ion exchange and flocculation, where magnesium (Mg^{2+}) bonds replace other cations (ion exchange) on the clay surface, altering the soil structure. This leads to flocculation, where clay particles clump together, forming a more stable and less compressible structure. The addition of MgCl_2 introduced Mg^{2+} ions into the pore fluid, which significantly reduced the thickness of the diffuse double layer surrounding clay particles. Due to their divalent charge, Mg^{2+} ions were more effective than monovalent cations in neutralizing the negative surface charges of clay minerals and compressing the diffuse ion cloud. This compression decreased the repulsive forces between adjacent clay particles and promoted particle flocculation and aggregation.

The percent of decrease in final strain compared to final strain of untreated soil (1.8%) was (4, 44, 47) % at MgCl_2 concentrations of (5, 10, 15) %. At concentrations of 10% and 15%, the results are approximately similar which indicate that a concentration of 10% is optimal in improving the compressibility properties of the clay soil and that increasing the concentration gave small different in results (Fig. 4). During the unloading phase, it is noted a clear decrease in the value of the change in final height after treatment (Fig. 5).

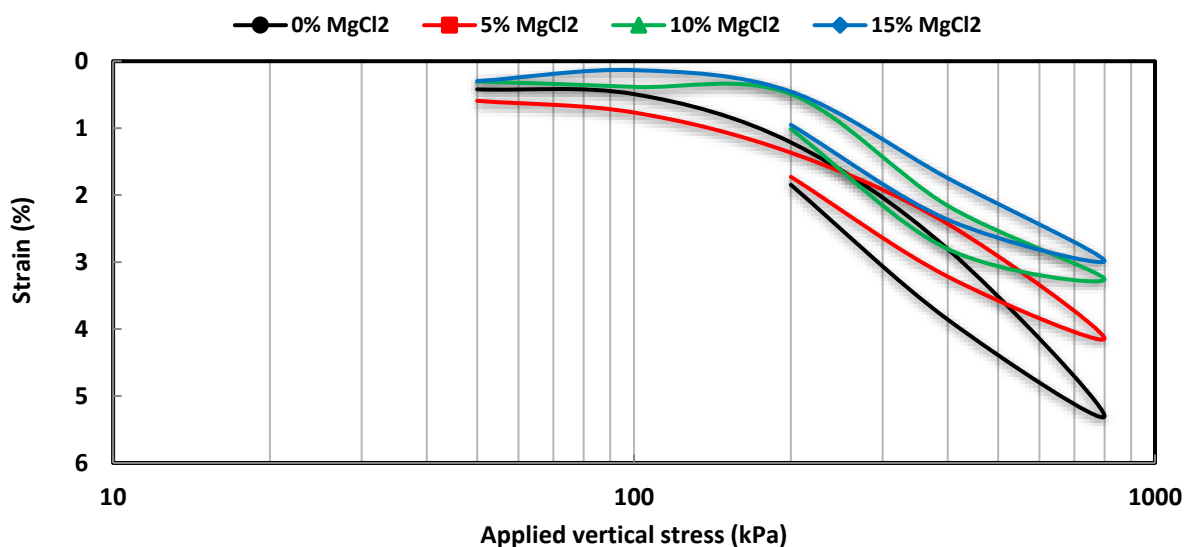


Fig. 4. Results of consolidation test for untreated and treated soil with different concentrations of MgCl_2

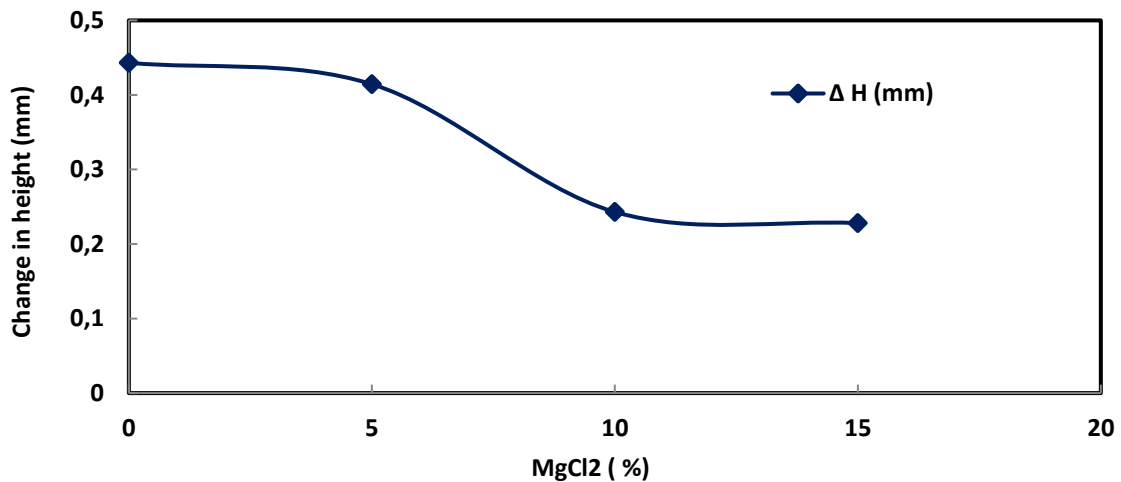


Fig. 5. The relationship between magnesium chloride concentration and final change of sample height

3.2. Unconfined Compression Test

Results of the unconfined compression test indicate an increase in the strength of the samples with increasing magnesium chloride concentration as shown in Fig. 6. The variation of unconfined compressive strength values with $MgCl_2$ concentration is shown in Fig. 7. The increase in UCS was (21, 27, 38) % at concentrations of (5, 10, 15) % respectively. Magnesium chloride ($MgCl_2$) can react with other soil components to form new crystalline cementations compounds such as magnesium silicate hydroxide (M-S-H), filling pores and binding particles which lead to increase soil strength and enhanced its properties.

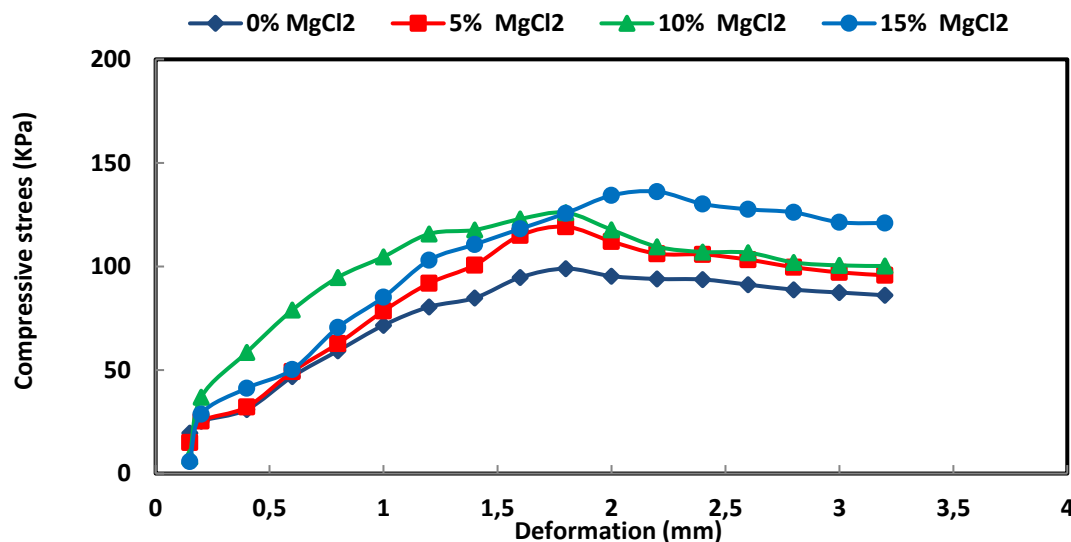


Fig. 6. The relationship between compression stress and deformation at different magnesium chloride concentration

Figure 8 noting the change in the shape of the failure, the sample failure shape before the addition of magnesium chloride solution was in the form of cracks at an angle of approximately 45 degrees at the edge of the sample (Fig. 8a), while the shape of the failure after the addition of magnesium chloride concentrations was in the form of cracks along the length of the sample (Fig. 8b- 8d). The failure mechanism of the untreated soil differed significantly from that of the $MgCl_2$ -treated specimens. The untreated clay exhibited an inclined shear failure surface at approximately 45°, which is characteristic of ductile behavior in cohesive soils. In this condition, the soil particles are

primarily held together by weak physico-chemical forces and interparticle friction, allowing considerable plastic deformation before failure. Consequently, the applied compressive stress is dissipated through the development of shear planes within the soil mass.

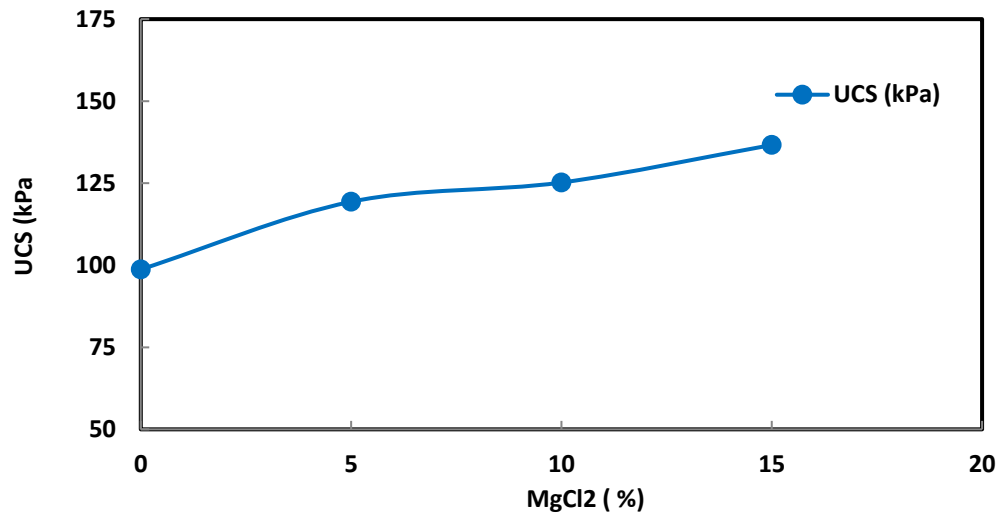


Fig. 7. The relationship between magnesium chloride concentration and unconfined compressive strength values

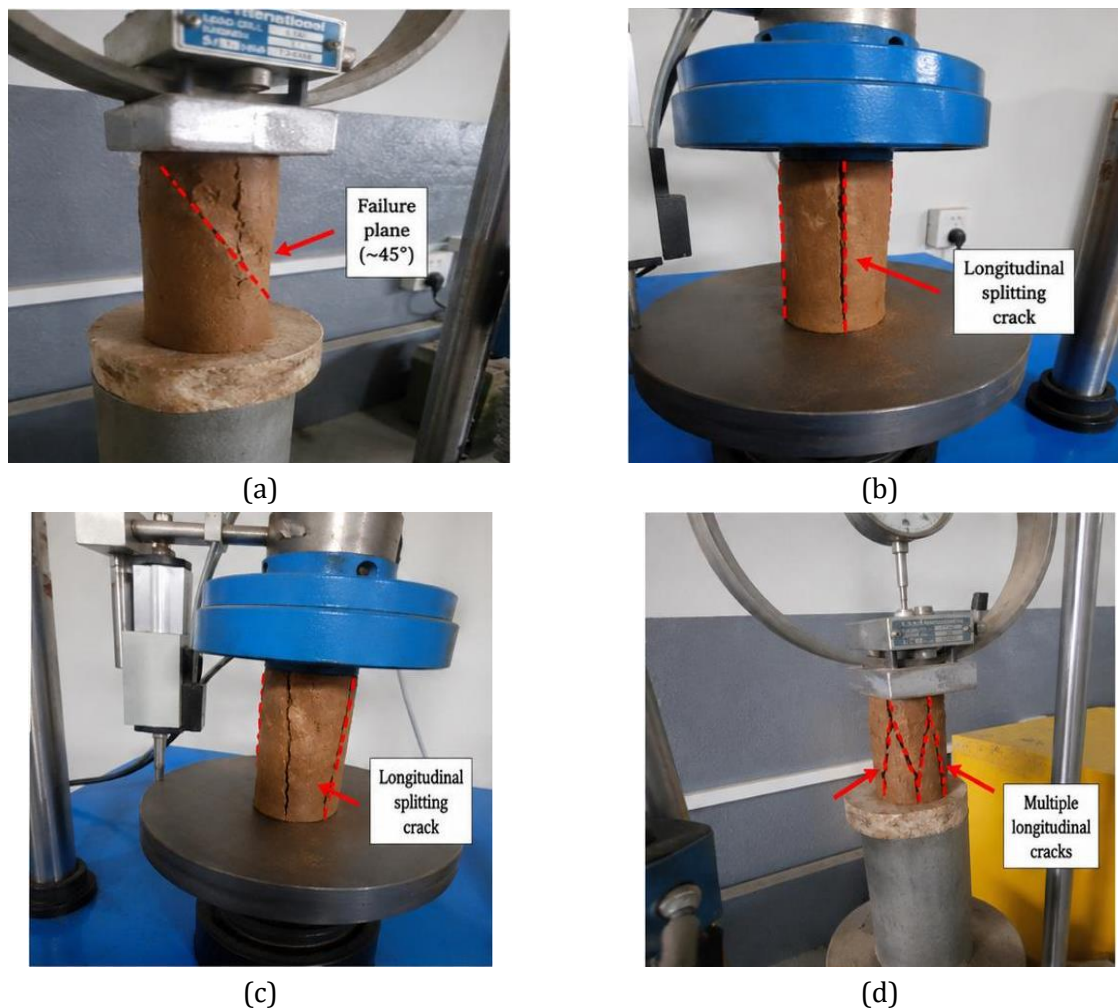


Fig. 8. The form of failure in unconfined compressive test (a) Before treatment, (b) Soil treatment with 5% MgCl₂, (c) Soil treatment with 10% MgCl₂, and (d) Soil treatment with 15% MgCl₂

In contrast, the MgCl_2 -treated specimens exhibited predominantly longitudinal splitting failure, indicating a transition from ductile to brittle behavior. This mechanical transformation can be attributed to the interaction of Mg^{2+} ions with the clay mineral surfaces, which reduced the thickness of the diffuse double layer and promoted stronger interparticle bonding. The resulting denser and more cemented soil structure increased stiffness and compressive strength while reducing the capacity for plastic deformation. Under axial loading, the strong cementations bonds restricted shear deformation; therefore, when the applied stress exceeded the tensile resistance of the bonded structure, cracks propagated rapidly along the specimen axis, leading to longitudinal splitting failure.

3.3. Shear Strength Parameters

The results of the direct shear test show a clear increase in the value of the shear strength in the soil. An increase of 66%, 94%, and 98% in cohesion was observed at magnesium chloride concentrations of 5%, 10%, and 15%, respectively, noting that at a concentration of 15% the increase was very small compared with that at 10%. The angle of internal friction of the soil was approximately not affected and its value was decreased at 10% and 15% MgCl_2 concentration (Figs. 9 and 10), this can be attributed to the lubricating effect due to the excess salt content which reduce the friction characteristics between soil particles.

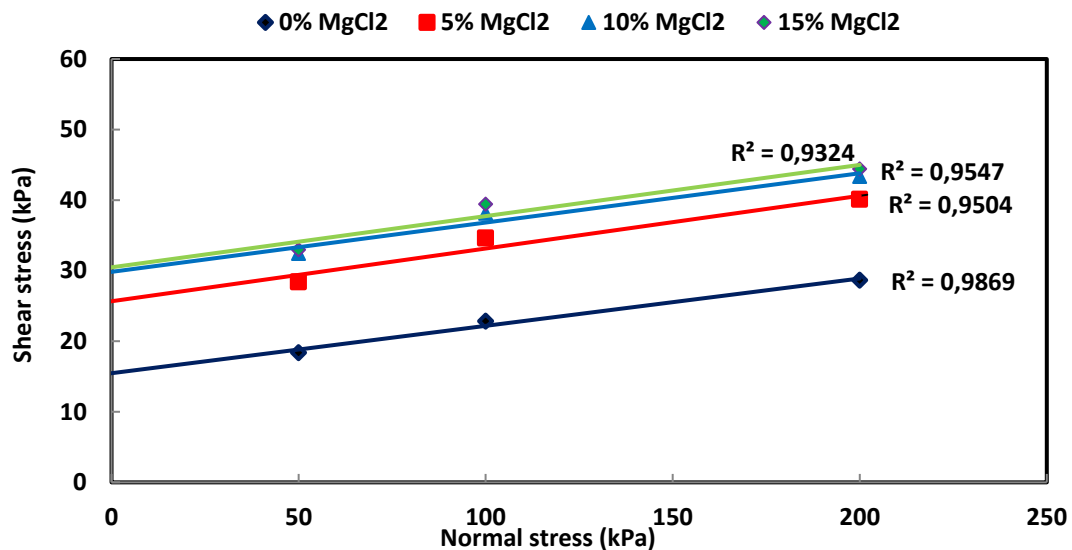


Fig. 9. Direct shear test results for untreated and treated soil with different percentages of magnesium chloride solution

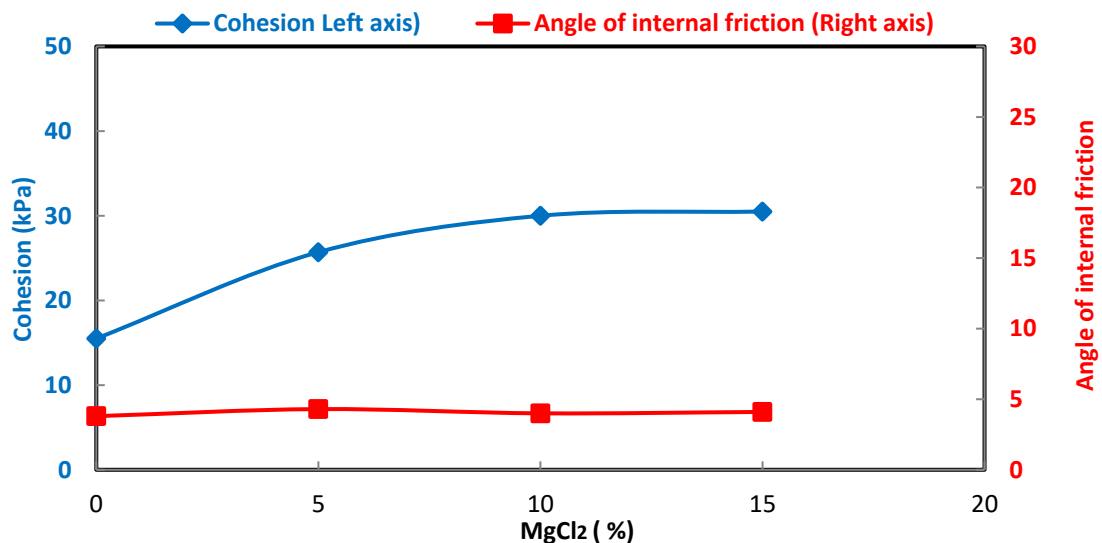


Fig. 10. Variation of cohesion and angle of internal friction under different condition of treatment with magnesium chloride solution

Table 3 shows the summary of the enhancement ratios for soil properties included compression, unconfined compression strength, and shear strength parameters with different magnesium chloride concentration.

Table 3. Variation of soil properties with magnesium chloride concentration

MgCl ₂ (%)	Final strain (%)	Enhancing (%)	UCS (kPa)	Enhancing (%)	c (kPa)	Enhancing (%)	Ø (o)	Enhancing (%)
0	1.8	0	98.73	0	15.5		3.8	0
5	1.72	4	119.35	21	25.7	66	4.3	13
10	1.0	44	125.21	27	30.0	94	4.0	5
15	0.95	47	136.65	38	30.5	97	4.1	8

4. Conclusions

- Magnesium chloride treatment proved to be an effective stabilization technique for the tested low plasticity clay soil, resulting in noticeable improvements in both strength and compressibility characteristics. The observed improvements are attributed to ion exchange, flocculation, and the formation of cementations compounds that enhanced particle bonding and reduced pore spaces within the soil structure.
- The findings indicate that the effectiveness of magnesium chloride treatment is strongly dependent on the interaction of Mg²⁺ ions with the major mineral constituents identified in the soil, particularly silica (SiO₂) and alumina (Al₂O₃). The interaction between magnesium chloride and the soil constituents plays a key role in determining the extent of improvement achieved.
- The compressibility of the soil decreased significantly with increasing MgCl₂ content. The improvement was particularly pronounced up to a concentration of 10%, indicating a substantial reduction in settlement potential and an increase in soil stiffness. Increasing the concentration beyond 10% provided only marginal additional benefits.
- The unconfined compressive strength increased with MgCl₂ addition, reflecting the enhanced interparticle bonding within the stabilized soil matrix. Furthermore, the mode of failure changed from an inclined shear-type failure in untreated specimens to predominantly longitudinal splitting in treated specimens, indicating a modification in soil behavior after stabilization.
- Magnesium chloride treatment contributed to a considerable increase in soil cohesion, while the internal friction angle exhibited only minor changes. This suggests that the improvement in shear strength was mainly associated with enhanced cohesion rather than frictional resistance.
- Among the investigated concentrations, 10% MgCl₂ provided the most balanced and efficient improvement in soil performance. Although a concentration of 15% produced slightly higher strength values, the additional improvement was relatively small compared with the increase in chemical content, making 10% the optimum treatment level for the tested soil.
- The findings indicate that MgCl₂ stabilization may be considered a practical alternative for improving clayey soils encountered in Salah Al-Din Governorate and similar regions with comparable mineralogical compositions. MgCl₂ treatment is benefit for soils used in road subgrades, embankments, and shallow foundation applications where excessive settlement and insufficient strength are major concerns.
- Result of this study are based on laboratory testing of a single clay soil type. Field performance may be affected by environmental conditions, groundwater chemistry. Therefore, further field-scale investigations are recommended to evaluate the long-term effectiveness, environmental impact, and cost-benefit performance of MgCl₂ stabilization under actual service conditions
- Future research should focus on evaluating the long-term durability and performance of MgCl₂-stabilized clay soils under various environmental conditions, including repeated

wetting–drying and freeze–thaw cycles. In addition, further investigations are recommended to assess the microstructural and mineralogical changes induced by MgCl_2 treatment using advanced techniques such as SEM, XRD, EDAX, and EDS analyses. Also, the effect of MgCl_2 on other soil properties such as cation exchange capacity, hydraulic conductivity, long term leaching is recommended.

Acknowledgement

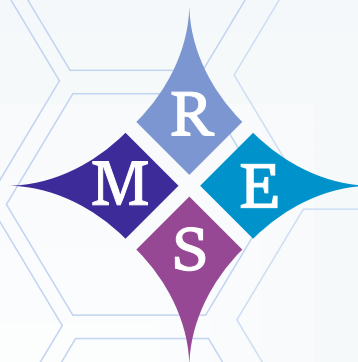
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References

- [1] Sachar A. An analysis of soil stability techniques. *International Journal of Innovative Research in Computer Science & Technology*. 2022; 10(2): 357-361.
- [2] Al-Atalla M, Al-Zubaydi A Al-Kiki I. Strength and consolidation characteristics of compacted clayey soil having a special case of standard compaction curve. *Eng. & Tech. Journal*. 2013; 31(9): 1791-1801. <https://doi.org/10.30684/etj.2013.82190>
- [3] Farichah H, Utama D, Alextrianto V, Satyanaga A, Ghifari F. Numerical investigation of consolidation settlement for runway construction on soft soil: A case study in Sumbawa, Indonesia. *Advance Sustainable Science, Engineering and Technology*. 2025; 7(3): 0250304-01-0250304-011. <https://doi.org/10.26877/s9q4fa83>
- [4] Marinković N, Zlatanović E, Bonić Z, Romić, Cvetković N, Stanković B. Comparison of the effects of different additives on the improvement of physical and mechanical properties of clayey soil. *proceedings of the 18th European Conference on Soil Mechanics and Geotechnical Engineering*. 2024; 2445-2450. <https://doi.org/10.1201/9781003431749-470>
- [5] Bakr R. Stabilization of soft clay soil by deep mixing with lime and cement in the presence of salt water. *Civil Engineering and Architecture*. 2024; 12(1): 78-96. <https://doi.org/10.13189/cea.2024.120107>
- [6] Marinković N, Zlatanović E, Bonić Z, Djordjević D, Romić N. Influence of traditional and alternative additives on the physical and mechanical properties of clayey soil. *Građevinar*. 2011; 76(6): 533-542. <https://doi.org/10.14256/ICE.4023.2024>
- [7] Mutaz E, Shamrani M, Puppala A, Dafalla. Evaluation of chemical stabilization of a highly expansive clayey soil. *Journal of the Transportation Research Board*. 2011; 2204(1): 148-157. <https://doi.org/10.3141/2204-19>
- [8] Hasan M, Alrubaye A, Seng L, Ideris M, Mart A. Undrained shear strength of soft clay mixed with different percentages of lime and silica fume. *Jurnal Teknologi*. 2016; 78(8-5): 23-30. <https://doi.org/10.11113/jt.v78.9606>
- [9] Al-Kinani A, Fattah M. Geotechnical characteristics of saline soft soils improved by chemical agents. *Magazine of Civil Engineering*. 2024; 17(7): Article 13105.
- [10] Striprabu S, Siti N, Norazzlina M, Fauziah A. Chemical stabilization of sarawak clay soil with class F fly ash. *Journal of Engineering Science and Technology*. 2018; 13(10): 3029-3042.
- [11] Zhang H, Ma Q, Su W, Hu H. On the dewatering of electroosmotic soil using intermittent current incorporated with calcium chloride. *Environmental Technology*. 2019; 42(3): 468-478. <https://doi.org/10.1080/09593330.2019.1634154>
- [12] El Kady A, Bakr M, Gad S . Evaluation of salt water effect on the physical properties and compaction characteristics of expansive soil. *International Journal of Civil Engineering and Technology*. 2020; 11(11): 49-65. <https://doi.org/10.34218/IJCET.11.11.2020.005>
- [13] Manikantha A, Eswara V, Bhargava G, Kumar I, Reddy C, Rao A. A Laboratory study on the effect of concentration-based magnesium chloride compounds on expansive soil for foundation applications. *Journal of Mechanical and Civil Engineering*. 2025; 22(2): 50-54.
- [14] Habibbeyg F, Nikraz H. Compression behaviour of highly expansive clays stabilized with a green stabilizer of magnesium chloride. *International Journal of Geomate*. 2018; 14(45): 144-155. <https://doi.org/10.21660/2018.45.10697>
- [15] Teker A, Akbulut S, Mutlue A. Investigation of the mechanical properties of magnesium chloride on magnesium oxide improved kaolin clay. *UNEC Journal of Engineering and Applied Sciences*. 2023; 3(2): 33-38. <https://doi.org/10.61640/ujeas.2023.1204>
- [16] Latifi N, Safuan A, Rashid A, Marto A, Tahir M. Effect of magnesium chloride solution on the physico-chemical characteristics of tropical peat. *Environmental Earth Science*. 2016; 75(220): 2-9. <https://doi.org/10.1007/s12665-015-4788-6>

- [17] Sharma S, Kumar V, Bindlish A. Enhancing the engineering properties of black cotton soil by using magnesium chloride. Proceedings of Indian Geotechnical Conference. 2020; 109-119. https://doi.org/10.1007/978-981-16-1831-4_8
- [18] Sadiq A, Fattah M, Aswad M. Enhancement of clay compressibility and strength using nano magnesium oxide. E3S Web of Conferences. 2023; 427: 1-8. <https://doi.org/10.1051/e3sconf/202342701013>
- [19] Rikhtehgar Y, Teymur B. Effect of magnesium chloride solution as an antifreeze agent in clay stabilization during freeze-thaw cycles. Applied Science. 2024; 14: 4140. <https://doi.org/10.3390/app14104140>
- [20] Tang Y, Wang N, Xu F, Teng D, Cui X. Electrokinetic stabilization of marine clayey soil using magnesium chloride solution: A green soil stabilizer. Marine Georesources and Geotechnology. 2021; 40(2): 248-254. <https://doi.org/10.1080/1064119X.2021.1895384>
- [21] Turkoz M, Savas H, Acaz A, Tosun H. The Effect of Magnesium Chloride Solution on the Engineering Properties of Clay Soil with Expansive and Dispersive Characteristics. Applied Clay Science. 2014; 101: 1-9. <https://doi.org/10.1016/j.clay.2014.08.007>
- [22] Hassan W, Hasmda W, Rashid A, Latifi N, Horpibulsuk S, Borhamdin S. Strength and morphological characteristics of organic soil stabilized with magnesium chloride. Quarterly Journal of Engineering Geology and Hydrogeology. 2017; 50(4): 454-459. <https://doi.org/10.1144/qjegh2016-124>
- [23] Jahidin R, Marto A, Pakir F, Jusoh S. unconfined compressive strength of compacted marine clay treated with magnesium chloride. AIP Conference Proceedings. 2021; 2355 (1): 030001. <https://doi.org/10.1063/5.0053471>
- [24] Kumar C, Ganesh B. Use of flyash based geopolymer and magnesium chloride on pavement system laid on expansive clays. International Journal for Science and Advance Research In Technology. 2025; 7(12): 61-67.
- [25] Türköz M, Savaş H, Tosun H. Stabilization of problematic clay soils with magnesium chloride solution. Journal of the Faculty of Engineering and Architecture of Gazi University. 2021; 26(1): 233-242.
- [26] Mahdavian S, Rashidi N, Estabragh A, Abdollahi J. Effect of magnesium chloride solution on the improvement of a clay soil. Journal of Engineering Geology. 2021; 19(1): 60-83.
- [27] ASTM D854-23. Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer. West Conshohocken, PA: ASTM International; 2023.
- [28] ASTM D4318-17. Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. West Conshohocken, PA: ASTM International; 2017.
- [29] ASTM D698-12(2021). Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft³ [600 kN-m/m³]). West Conshohocken, PA: ASTM International; 2021.
- [30] ASTM D7928-21. Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis. West Conshohocken, PA: ASTM International; 2021.
- [31] ASTM D2435/D2435M-20. Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading. West Conshohocken, PA: ASTM International; 2020.
- [32] ASTM D2166/D2166M-16(2021). Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. West Conshohocken, PA: ASTM International; 2021.
- [33] ASTM D3080/D3080M-23. Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. West Conshohocken, PA: ASTM International; 2023.

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