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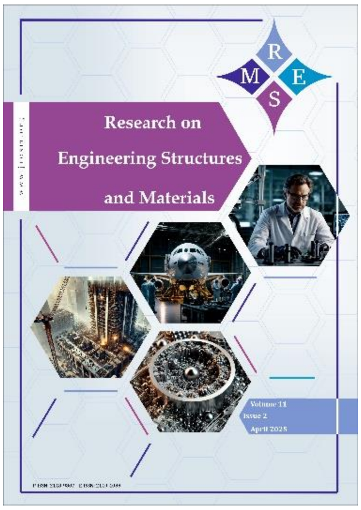

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Research Article

Strengthening of fire-damaged reinforced concrete columns under pre loading

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Abstract

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.

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/m^3 of cement, 685 kg/m^3 of natural sand, and 1075 kg/m^3 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 ρ = 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 (C_1) 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 (Pu). In the present experimental investigation, the specimens were categorized into two main groups, which are described in the following sections.

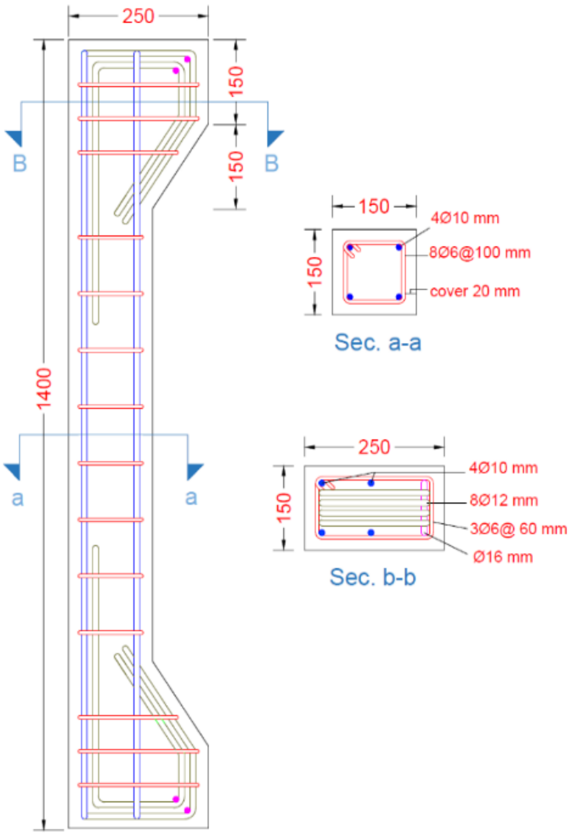


Fig. 1. Layout of reinforcement in the tested columns

The first group consists of four columns (C_2 to C_5). Columns C_2 and C_3 were exposed to fire at a temperature of 500 °C for 60 minutes. Column C_3 was subsequently rehabilitated after removing the damaged concrete cover and replacing it with Ultra-High-Performance Concrete (UHPC). Columns C_4 and C_5 were exposed to fire at a temperature of 500 °C for 120 minutes, and similarly, column C_5 was repaired in the same manner as column C_3. The second group also consists of four columns (C_6 to C_9), 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 (Pu) 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) (ºC)	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 m^3, 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.

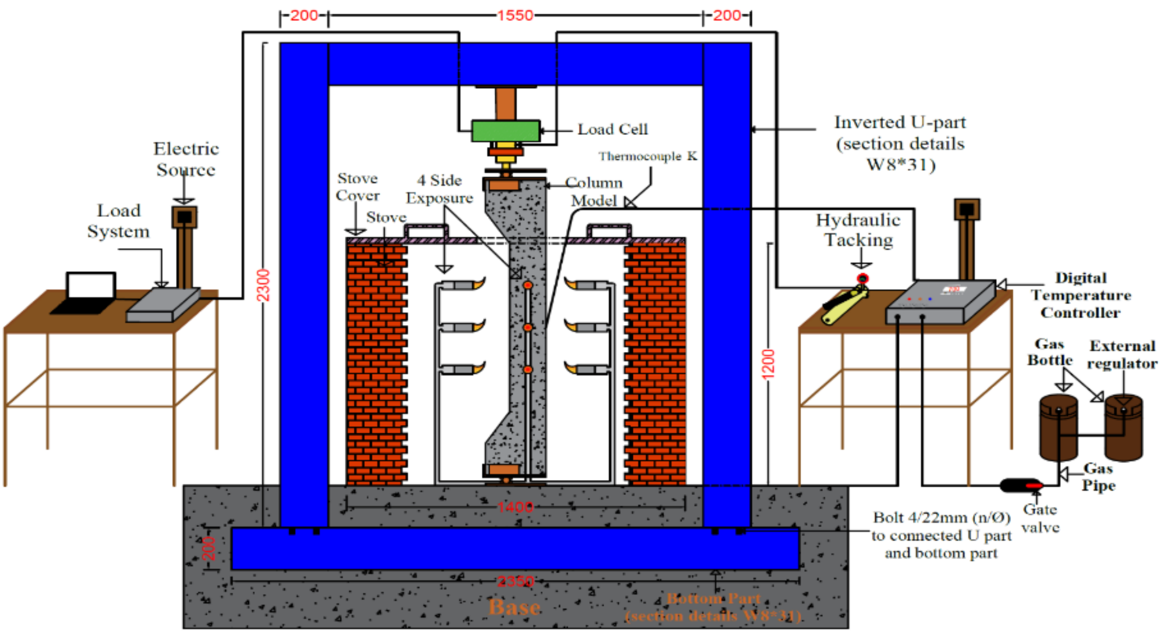


Fig. 3. Description of the loading frame

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.

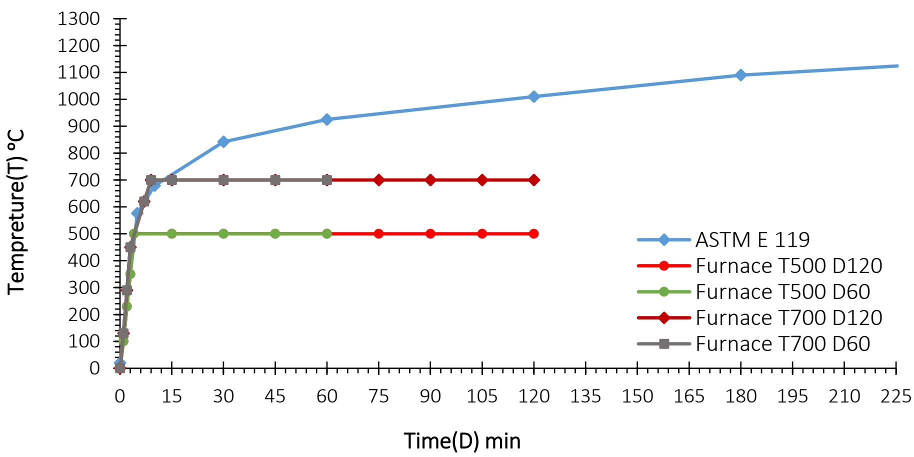


Fig. 4. Standard fire exposure according to ASTM

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 (C_3, C_5, C_7, and C_9) 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).

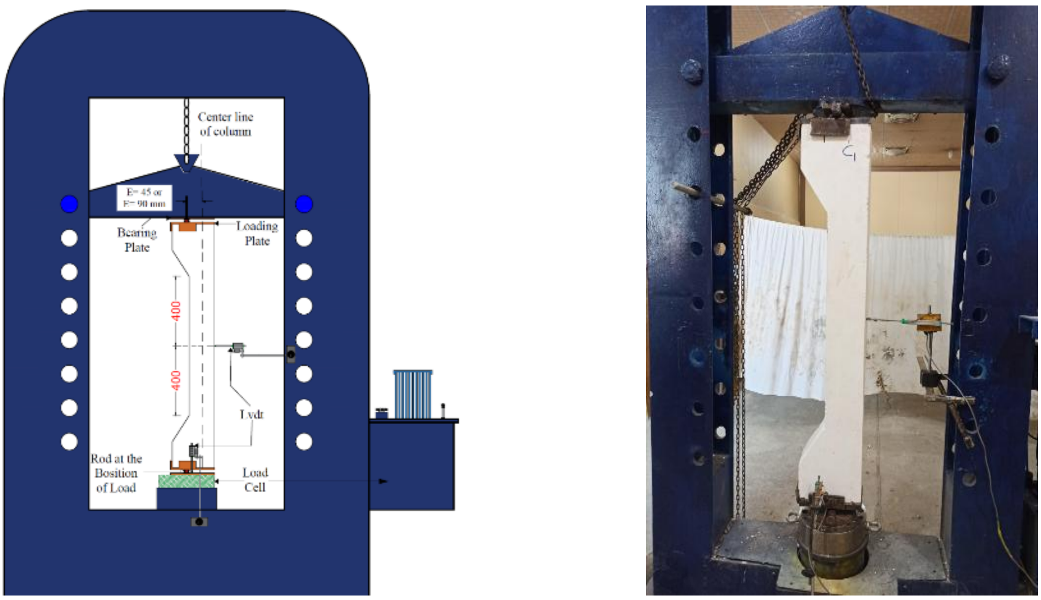


Fig. 7. Test setup with instrumentation details

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 (C_1) 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 (C_1), specimens $C_2T_{500}D_{60}$ and $C_4T_{500}D_{120}$, 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 $C_6T_{700}D_{60}$ and $C_8T_{700}D_{120}$ 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 $C_2T_{500}D_{60}$ and C4T500D120. Likewise, specimens $C_7T_{700}D_{60}R$ and $C_9T_{700}D_{120}R$ showed increases of (120.16% and 157.28%), respectively, relative to specimens $C_6T_{700}D_{60}$ and $C_8T_{700}D_{120}$, as shown in Fig. 8.

When compared with the unexposed control specimen (C_1), the rehabilitated columns demonstrated capacity enhancement rather than only recovery. The load-carrying capacities of specimens $C_3T_{500}D_{60}R$ and $C_5T_{500}D_{120}R$ exceeded that of C1 by (80.98% and 72.39%), respectively, while specimens $C_7T_{700}D_{60}R$ and $C_9T_{700}D_{120}R$ 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 $C_9T_{700}D_{120}R$, 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.

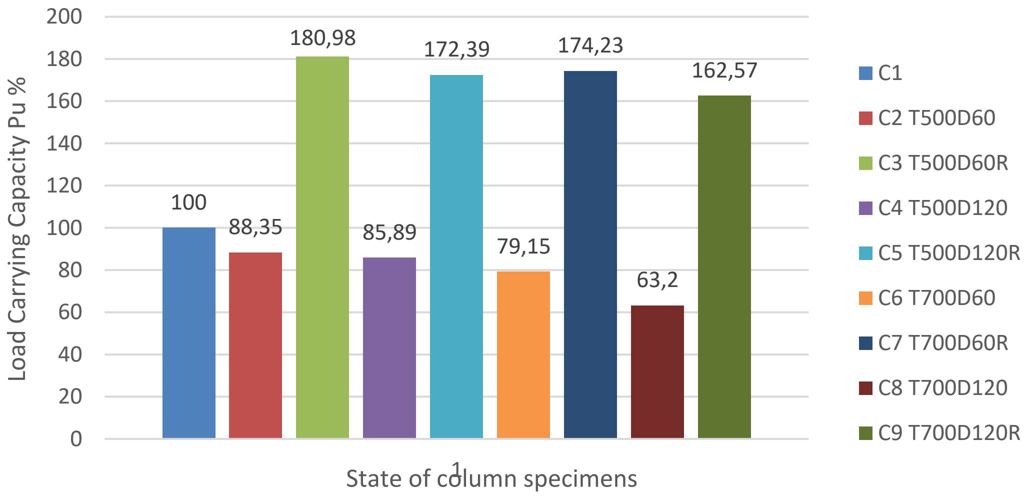
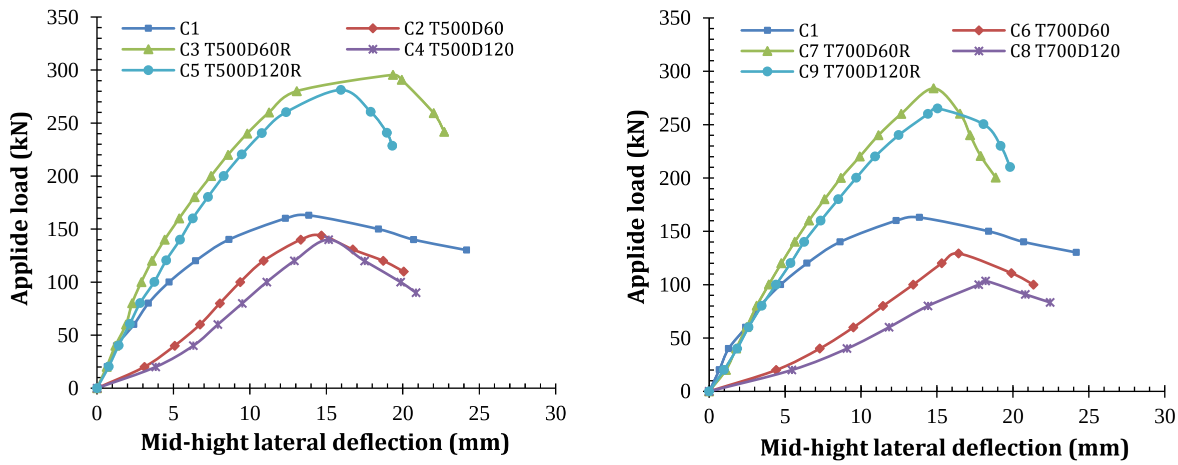


Fig. 8. Percentage of load carrying capacity for the tested column specimens

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.



(a) (b)

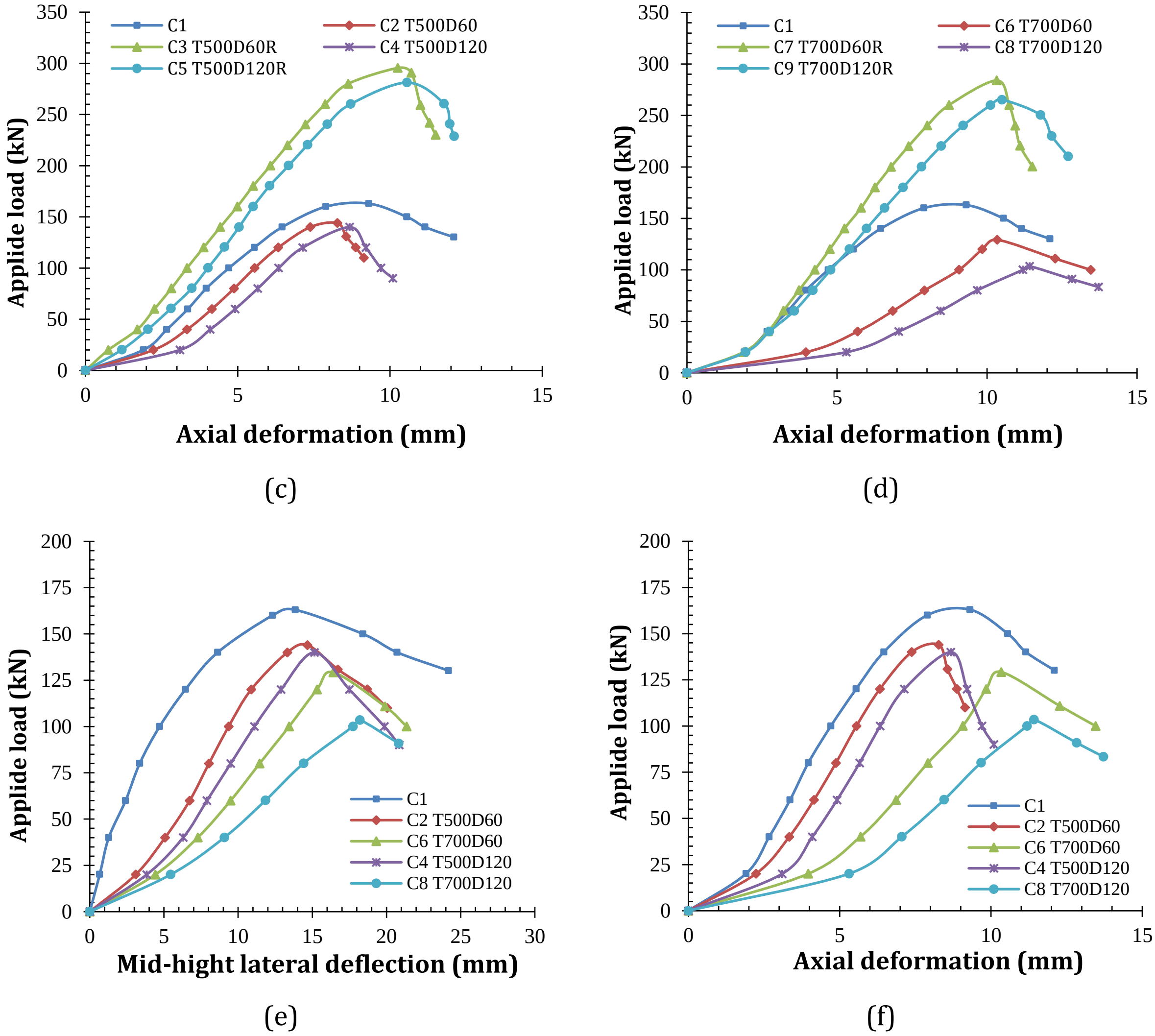


Fig. 9. **L**oad-displacement of Columns for each group

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 (C_1) 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.

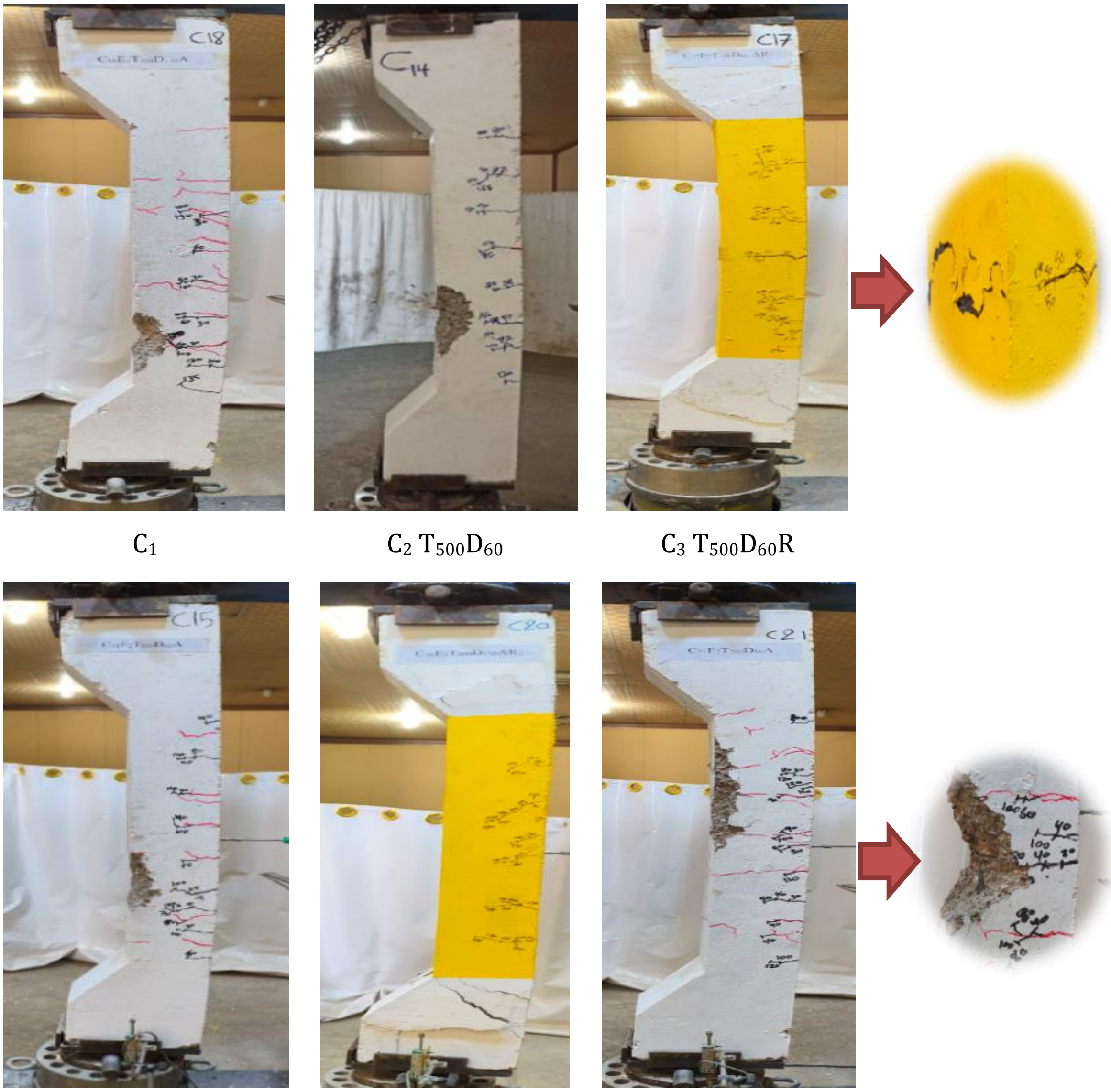
For the fire-exposed unstrengthened specimens ($C_2T_{500}D_{60}$, $C_4T_{500}D_{120}$, $C_6T_{700}D_{60}$, and $C_8T_{700}D_{120}$), pre-existing cracks induced by sustained eccentric loading during fire exposure were clearly visible before the mechanical test. Crack propagation started at lower load 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 ($C_3T_{500}D_{60}R$), 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.

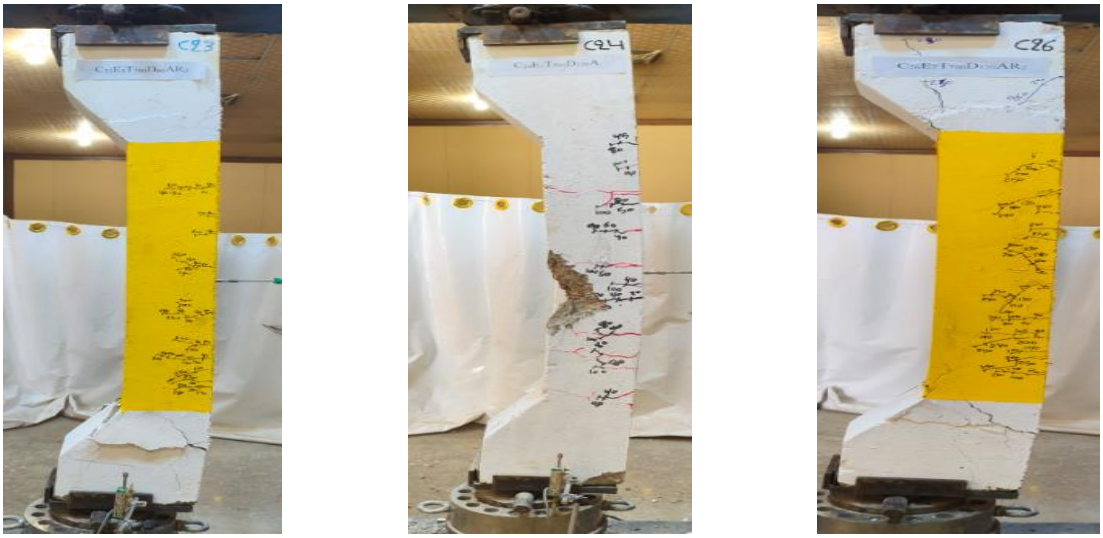
(a) (b) (c)

Fig. 10. Spread of crack in specimens, (a) Column specimens without fire, (b) Column specimens with fire exposure and (c) Repaired column specimen



C_1 C_2 $T_{500}D_{60}$ C_3 $T_{500}D_{60}R$

C_4 $T_{500}D_{120}$ C_5 $T_{500}D_{120}R$ C_6 $T_{700}D_{60}$



$C_7T_{700}D_{60}R$ C_8 $T_{700}D_{120}$ C_9 $T_{700}D_{120}R$

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.

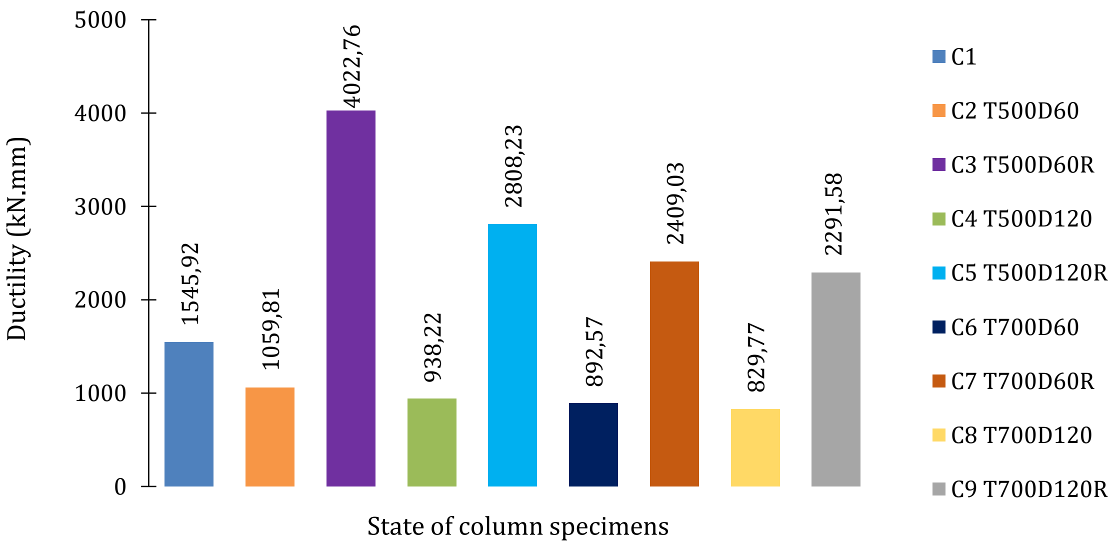


Fig. 12. The ductility of tested specimens

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.

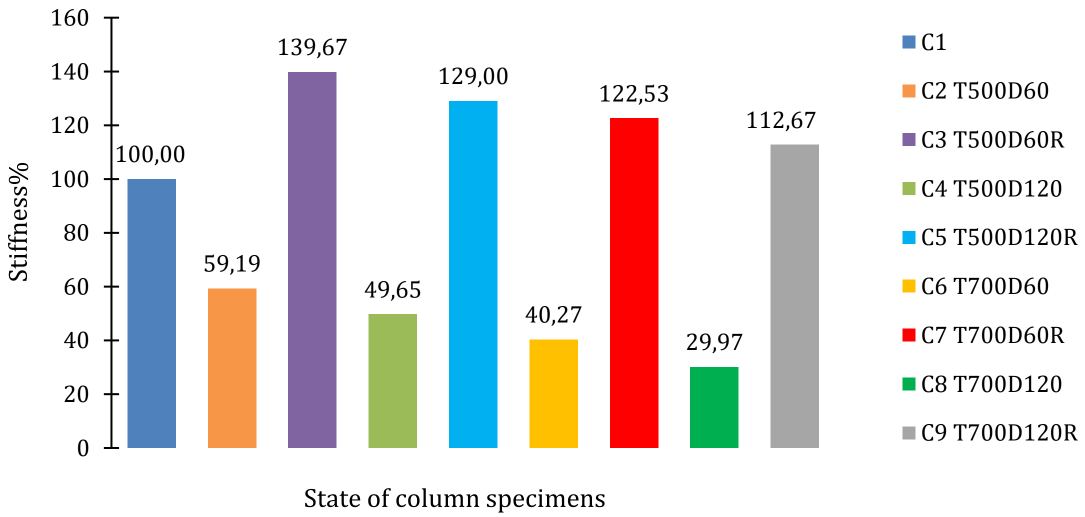


Fig. 13. Stiffness of tested specimens

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.