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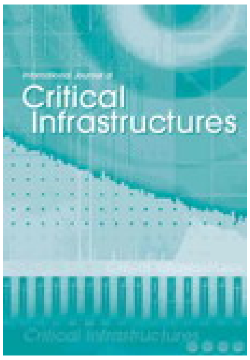

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Rehabilitation of pre-loaded reinforced concrete columns exposed to fire using advanced strengthening techniques

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Abstract: This study presents an experimental investigation on the rehabilitation of fire-damaged reinforced concrete (RC) columns using two strengthening techniques: full wrapping with carbon fibre reinforced polymer (CFRP) and jacketing with ultra-high-performance concrete (UHPC). Seven small-scale RC column specimens were tested under eccentric loading with an eccentricity of (e = 45 mm). The fire-exposed specimens were subjected to temperatures of (500 °C) and (700 °C) for (60 min) under a pre-applied axial load equal to (50%) of the ultimate load capacity, followed by sudden water quenching. An unexposed reference column was used for comparison. The results showed that fire exposure reduced the ultimate load-carrying capacity by (29.69%) and (51.26%) at (500°C) and (700°C), respectively. After rehabilitation, columns strengthened with CFRP showed capacity increases of (35.86%) and (83.91%), whereas those strengthened with UHPC achieved higher increases of (93.62%) and (157.47%), respectively.

Keywords: fire-damaged reinforced concrete columns; carbon fibre reinforced polymer; CFRP; ultra-high-performance concrete; UHPC; eccentric loading.

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1 Introduction

Investigating the behaviour of reinforced concrete structural elements under fire exposure is a critical topic in the field of experimental structural mechanics. Such studies primarily aim to experimentally evaluate the response of key structural components, particularly reinforced concrete columns, under service loads during and after fire events. Numerous research efforts have been conducted to examine the behaviour of concrete and reinforced concrete structures when subjected to fire or elevated temperatures (Eskandani et al., 2019). Fire resistance requirements are among the most important safety criteria in building design, as fire represents one of the most severe environmental hazards that structures may encounter (Shubbar and Alwash, 2020; Raut and Kodur, 2011; Jawad and Ali, 2020). This significance arises from the fact that the structural integrity of a building constitutes the final line of defence for protecting occupants and emergency responders when all other fire containment measures fail. Fire resistance is defined as the duration during which a structural element maintains its stability, integrity, and thermal insulation when exposed to standard fire conditions, and it is widely used to assess the thermal performance of structural components. In general, the fire resistance of any structural member depends on its design configuration, material properties, applied load intensity, and the characteristics of the fire itself (Al-Khafaji and Falah, 2020; Alkafaji, 2015). From a technical standpoint, fire resistance is defined as the time period during which a structural element resists heat transfer while preserving structural stability and integrity under elevated temperatures (Shah and Sharma, 2017). Several factors influence the fire resistance of reinforced concrete members, including concrete cover thickness, moisture content, reinforcement detailing, fire type and exposure duration, load intensity, and concrete strength grade (Balaji et al., 2016; Kodur et al., 2017). Due to the significant variability in fire sources, spread, and severity, standardised fire tests are essential to conservatively evaluate the performance of structural elements. Among the available standards, ASTM E-119 is the most widely adopted internationally for assessing fire resistance. However, most previous studies have primarily focused on the behaviour of concrete columns during fire exposure, whereas post-fire performance is equally important. In practical scenarios, many reinforced concrete structures subjected to fire do not experience complete collapse and can be rehabilitated using appropriate repair techniques (Hibner, 2017). Nevertheless, rehabilitating fire-damaged structures remains a complex engineering task. According to Yaqub and Bailey (2011), repairing and reusing fire-affected reinforced concrete structures is often more economical than demolition and reconstruction, as damage is typically limited to superficial cracking of the concrete cover without significant deterioration of the reinforcement or stirrups. Experimental results reported by Kodur et al. (2013) demonstrated that reinforced concrete columns retain a considerable portion of their load-carrying capacity even after severe fire exposure. Previous studies have proposed various rehabilitation techniques depending on the structural type and extent of damage, with strengthening being one of the most common and effective approaches for restoring the capacity of damaged columns. Depending on the level of deterioration, techniques such as injection, removal and replacement, and jacketing may be employed.

To prevent structural failure or collapse under elevated temperatures, it is essential to investigate effective rehabilitation techniques for fire-damaged elements, particularly through the use of advanced construction materials. One prominent approach involves fibre-reinforced polymer (FRP) systems, especially carbon fibre reinforced polymer (CFRP) composites (Sarsam et al., 2018; Ahmed and Hasan, 2019; Al-Bayati et al., 2021). Al-Kamaki et al. (2014) demonstrated that confinement using FRP composites can significantly enhance the strength and ductility of fire-damaged concrete. Their experimental program involved exposing reinforced concrete columns to elevated temperatures while sustaining 30% of their ultimate load capacity, followed by cooling, CFRP confinement, and subsequent axial compression testing. The results confirmed that CFRP wrapping is an effective technique for restoring and improving the capacity of thermally damaged columns. Furthermore, Alhassnawi and Alfatlawi (2018) reported that full CFRP confinement of fire-exposed columns resulted in a 37.55% increase in load-carrying capacity compared with burned specimens.

An alternative rehabilitation approach involves the use of ultra-high-performance concrete (UHPC), which represents a modern and efficient technique for extending the service life of reinforced concrete structures in a cost-effective manner (Alasmari, 2023). UHPC is characterised by its exceptional load-bearing capacity, even in the presence of cracking. It typically consists of cement, fine sand, a high content of silica fume, water, superplasticisers, and steel fibres, resulting in superior strength and ductility. Abbas et al. (2016) indicated that UHPC constitutes a new class of cementitious materials due to the homogeneous distribution of steel fibres within the matrix, which significantly enhances compressive and tensile strength, toughness, and durability. Similarly, Brühwiler and Denarié (2013) highlighted that UHPC is particularly suitable for repair applications owing to its ease of placement in localised damaged regions. In addition, Meda et al. (2008) demonstrated that jacketing short reinforced concrete columns with high-performance fibre-reinforced concrete exhibiting compressive strength up to 170 MPa markedly improves durability, contributing to the development of more sustainable and corrosion-resistant structures. Rabehi et al. (2014) conducted an experimental study to investigate the stress-strain behaviour of damaged reinforced concrete columns rehabilitated using section enlargement and FRP jacket strengthening. The study employed both carbon and glass fibre-reinforced polymer jackets, alongside normal-strength concrete and ultra-high-performance fibre-reinforced concrete (UHPFRC) for section enlargement. The results demonstrated that the applied repair techniques significantly enhanced the columns' compressive strength and ductility, with the overall performance influenced by the type of strengthening material, the original concrete strength, and the concrete type used for the enlargement.

At present, achieving repair solutions that are both cost-effective and time-efficient has become a key requirement in modern structural rehabilitation practice. Accordingly, the present study focuses on simulating realistic fire scenarios to investigate the behaviour of short reinforced concrete columns under load during fire exposure, followed by rapid water cooling. Subsequently, the rehabilitation performance of the columns using UHPC and CFRP jacketing is evaluated. The primary objective of this study is to assess the effectiveness of UHPC and CFRP in restoring and enhancing the load-carrying capacity of fire-damaged short columns that have experienced strength degradation without undergoing complete structural failure.

2 Materials and mix proportions

2.1 Material

Materials utilised in this study include:

2.1.1 Normal strength concrete

All specimens were cast using normal-strength concrete with a compressive strength of 32 MPa. The mix proportions are presented in Table 1.

Table 1 Mix proportions of the concrete used in this study

Materials	*Amount*
Cement (kg/m^3)	390
Sand (kg/m^3)	685
Gravel (kg/m^3)	1,075
w/cement ratio	0.47
f' c (28 days) MPa	32

2.1.1.1 Steel

Three types of steel reinforcing bars were used in the experimental program. Deformed bars with a nominal diameter of Ø10 mm (Al-Mass production) were employed as longitudinal reinforcement in all column specimens. Deformed bars with a diameter of Ø12 mm (Al-Mass production) were used to reinforce all corbels, while Ø6 mm deformed bars (Turkish production) served as transverse ties in both the corbels and the column specimens.

2.1.2 Carbon fibre reinforced polymer

This study used Sika Wrap Hex-230C, a type of CFRP sheet with a width of 50 cm.

2.1.3 Ultra-high-performance concrete

a Cement: type V Portland cement was selected based on trial mix evaluations to obtain an optimal balance between compressive strength and workability.

b Fine sand: locally sourced natural sand, sieved through a 0.6 mm mesh, was used as the fine aggregate for UHPC in accordance with established recommendations in the literature.

c Silica fume (SF): silica fume, a by-product of silicon and ferrosilicon alloy production, consists of particles significantly finer than cement. It reacts with calcium hydroxide to form additional calcium silicate hydrate (C-S-H) and fills microvoids, thereby reducing porosity and enhancing the strength and durability of the concrete.

d Superplasticiser (SP): a high-range water-reducing admixture based on carboxylic ether polymers, MasterGlenium 54 (BASF), was used to achieve the required workability of UHPC. Based on trial mix results, a dosage of 3.5% by weight of cementitious materials was adopted.

e Steel fibre: copper-coated straight micro steel fibres were incorporated into the UHPC at a volume fraction of 2%, as determined from trial mixes and supported by previous studies. The physical properties of the fibres comply with the requirements of ASTM A820.

2.1.3.1 Concrete mix design

The mixing procedure and material proportions of the UHPC were developed and optimised 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. The dry constituents, including cement, sand, and silica fume, were first combined and mixed for 1–2 minutes, followed by the gradual addition of water and superplasticiser, and mixed for an additional 12–15 minutes until a homogeneous and flowable paste was obtained. Steel fibres 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 optimised trial mix achieved a notional 28-day compressive strength of 135.7 MPa. The constituent materials and their corresponding weight proportions are presented in Table 2.

Table 2 Mix proportions of UHPC

Constituents	*The proportion by weight (kg/m^3)*
Cement	950
Fine sand	1,050
Silica fume	190[a]
Water	195[b]
Superplasticiser	40[c]
Micro steel fibre	157[d]

Notes: [a]silica fume = 20% of cement weight, [b]w/b = 17.1%, [c]sp/b = 3.5% and [d]micro steel fibre = 2% of total volume

3 Specimens details

All specimens were identical in terms of external geometry and overall configuration. The columns had constant dimensions with a square cross section. Each column had a total height of 1,400 mm and a cross-sectional area of 150 × 150 mm. The centre-to-centre distance between the corbels was 800 mm, and each corbel had dimensions of 150 × 250 × 300 mm. The corbels were constructed to facilitate the application of an eccentric load. Each column was provided with a clear concrete cover of 20 mm and reinforced with four longitudinal deformed steel bars with a reinforcement ratio of ρ = 0.0140 and a nominal diameter of 10 mm. In addition, steel transverse ties with a diameter of 6 mm were installed at a spacing of 100 mm. All columns were fabricated in accordance with the provisions of ACI 318-19. Details of the column and corbel reinforcement are shown in Figure 1. As summarised in Table 1, the present experimental program investigated seven reinforced concrete column specimens made with normal-strength concrete. This included one control column (C_1) that was not subjected to preloading or fire exposure, and six columns that were exposed to fire followed by sudden water cooling under a sustained preloading level of up to 50% of the ultimate load (Pu).

Columns C_2 to C_4 were exposed to fire at a temperature of 500°C for 60 minutes, followed by sudden cooling using water. After fire exposure, column C_3 was rehabilitated by removing the damaged concrete cover and replacing it with normal concrete, followed by full confinement using CFRP wrapping, whereas column C_4 was repaired by replacing the damaged concrete cover with UHPC. Similarly, columns C_5 to C_7 were exposed to fire at a temperature of 700°C for 60 minutes and subsequently subjected to sudden water cooling. Columns C_6 and C_7 were rehabilitated using the same techniques adopted for columns C_3 and C_4, respectively. It should be noted that all six fire-exposed columns were subjected to a constant preloading level equal to 50% (178.5 kN) of the ultimate load (Pu) during both fire exposure and cooling, with a fixed load eccentricity of e = 45 mm (e/h = 0.3) for all specimens.

Figure 1 Layout of reinforcement in the tested columns (see online version for colours)

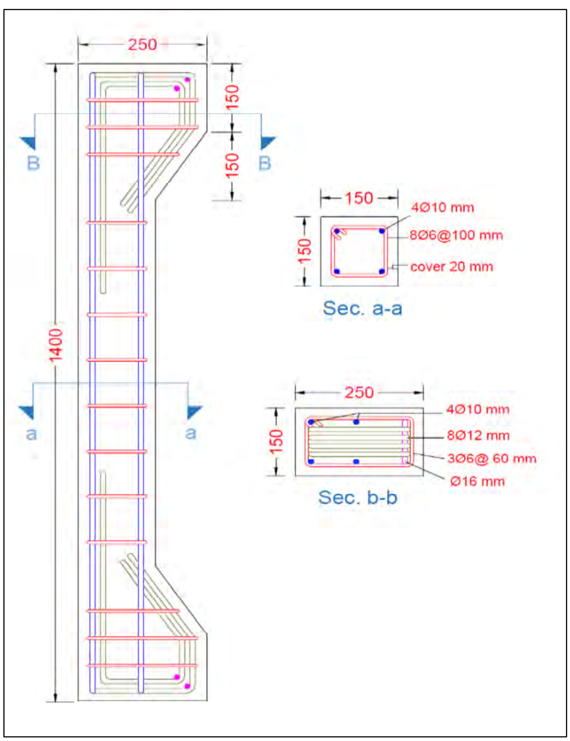


Table 3 Details of the examined specimens

Specimen symbol (Ci)	*Fire exposure (Ti) (°C)*	*Repair of fire damaged specimen (R)*
C_1	-	-
C_2 T500 W	500	-
C_3 $T_{500}WR_1$	500	R_1
C_4 $T_{500}WR_2$	500	R_2
C_5 $T_{700}W$	700	-
C_6 $T_{700}WR_1$	700	R_1
C_7 $T_{700}WR_2$	700	R_2

Notes: C = concrete column; T = fire temperature; W = water cooling; $R_{1,2}$ = repair of fire damaged (R_1; CFRP, R_2; UHPC).

4 Casting procedures (NC)

Concrete mixing was carried out using a central batching mixer with a capacity of 10 m³, supplied by Al-Mustaqbal Ready-Mix Concrete Company. Prior to casting, the internal surfaces of the cube and cylinder moulds were thoroughly cleaned and lubricated to prevent adhesion to hardened concrete. Each steel reinforcement cage for the columns was then placed horizontally within the wooden formwork and properly fixed in position. Concrete was cast into the moulds in a single layer, compacted, and subsequently vibrated for approximately two minutes using a built-in vibrator. The standard procedures for casting and compaction of conventional concrete in cube and cylinder moulds, including the number of layers and rodding, were followed, as illustrated in Figure 2.

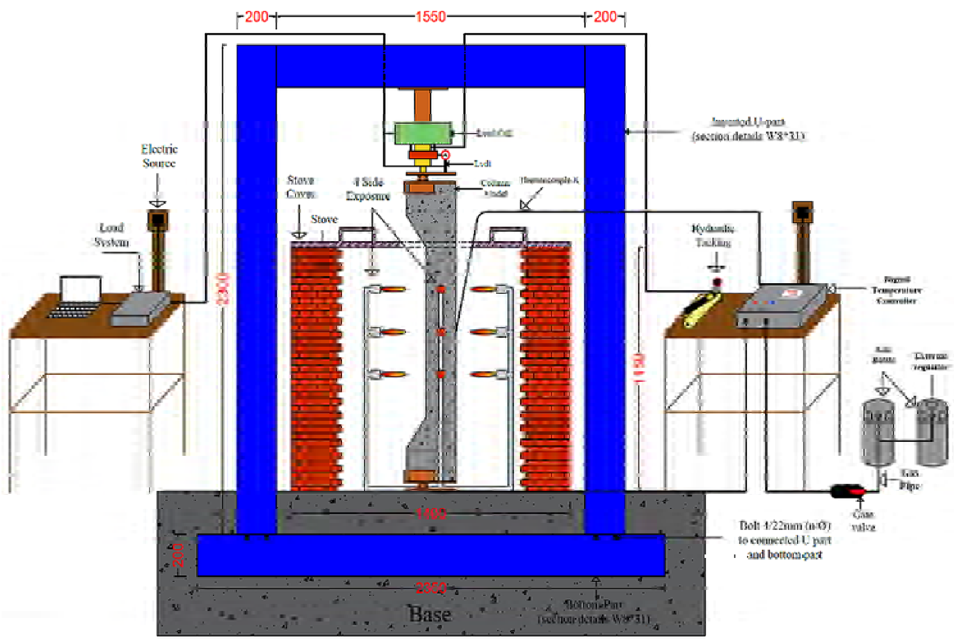
Figure 2 Stages of the casting process, (a) assembly of molds with reinforcement mesh (b) concrete vibration (c) columns after casting (d) specimens after mold removal (see online version for colours)

(a) (b) (c) (d)

5 Fire test

A brick furnace with internal dimensions of 1,400 × 1,400 × 1,150 mm was employed for the fire exposure phase, as illustrated in Figure 3. Fire exposure duration of 60 minutes was adopted, during which the maximum furnace temperatures reached approximately 500°C and 700°C. The furnace temperature was regulated using a digital controller connected to a gas regulator, which automatically adjusted the fuel supply based on real-time sensor feedback once the target temperature was specified. Furnace temperatures were monitored using 4 mm K-type thermocouples. Eccentric axial loading was applied to the specimens using a hydraulic jack to reach the designated service load. The testing protocol was based on applying the service load prior to fire exposure. Accordingly, the specimens were axially loaded at a rate of 5 kN/s until the target load was achieved, corresponding to 50% of the ultimate load at ambient temperature. During the heating phase, the applied load was maintained constant and continuously monitored to detect any fluctuations before and during fire exposure. A 2 MN load cell was installed between the hydraulic jack piston and a solid steel rod to ensure accurate load transfer. These loading levels were selected to realistically simulate the in-service mechanical conditions experienced by columns in actual building structures.

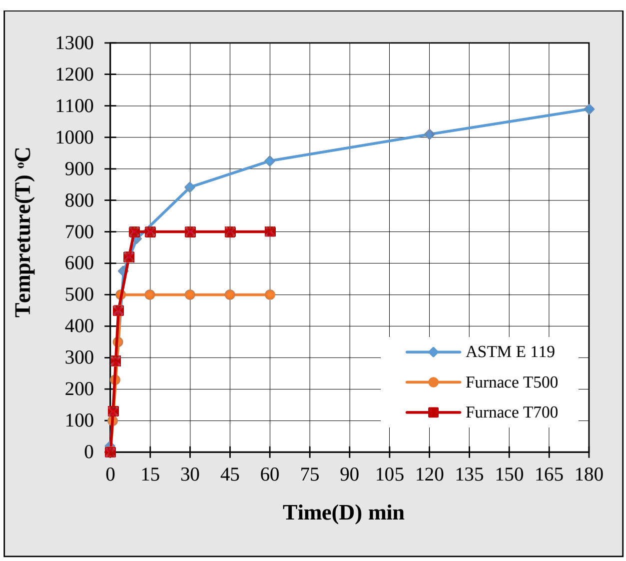
Figure 3 Description of the loading frame (see online version for colours)



Column C_1 was not exposed to fire and served as the control specimen. In contrast, six columns were exposed to fire at the age of three months under two maximum temperature levels, namely 500°C and 700°C, for an exposure duration of 60 minutes after reaching the target temperature. Upon completion of heating, the flames were extinguished, the furnace enclosure was removed, and the cooling system specifically designed for this study was activated. The columns were then rapidly cooled by water spraying until reaching room temperature while maintaining the pre-applied load, in order to simulate realistic firefighting conditions in buildings. Comprehensive details of the furnace, testing setup, and preloading system are presented in Figure 4. During the fire tests, the heating regime was precisely controlled so that the average furnace temperature closely followed the standard ASTM E119-20 time-temperature curve, as shown in Figure 5. Temperature distributions within specimens C_2 T_{500}W and C_5 T_{700}W were recorded using K-type thermocouples installed at a single cross-section located at mid-height of the column, as depicted in Figure 6. Four thermocouples were distributed across the section: one at the mid-depth of the concrete cover (T1), one attached to the longitudinal reinforcement (T2), one located at the mid-distance between the longitudinal reinforcement and the column core (T3), and one at the column core (T4). Temperature data were recorded using an eight-channel data acquisition system, and the resulting fire curves are presented in Figure 7.

Figure 4 Specimens subjected to fire exposure under pre-loading curve (see online version for colours)

Figure 5 Standard fire exposure according to ASTM E119-20 (see online version for colours)



Based on the thermocouple measurements, the specimen exposed to 500°C for 60 minutes exhibited a gradual reduction in temperature toward the column core. The recorded temperatures corresponded to 26.8%, 23.4%, 21.4%, and 20% of the furnace temperature at locations T1, T2, T3, and T4, respectively. Rapid water cooling reduced the cooling duration to 45 minutes to reach room temperature. In contrast, the specimen exposed to 700°C for 60 minutes experienced a more pronounced and faster heat transfer. The temperatures measured at locations T1, T2, T3, and T4 were 31.9%, 29.6%, 27.1%, and 23.4% of the furnace temperature, respectively. In this case, water cooling required approximately 60 minutes to return the specimen to ambient temperature.

Figure 6 Column details and thermocouple positions through cross-sections (see online version for colours)

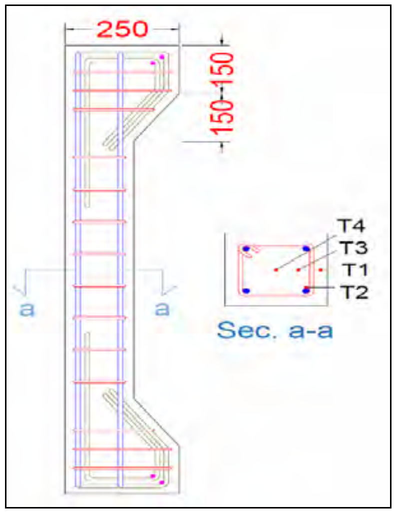
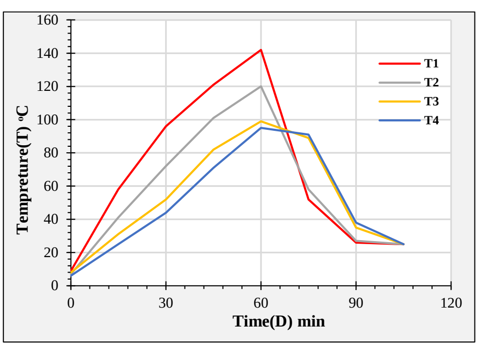
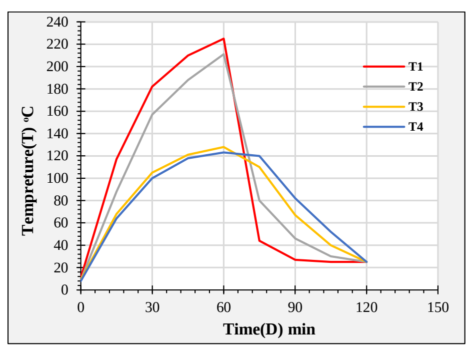


Figure 7 Temperature evolution at the cross-section specimens, (a) the relationship between temperature and time for the specimen C_2 T_{500}W (b) the relationship between temperature and time for the specimen C_5 T_{700}W (see online version for colours)



(a)



(b)

6 Repair of fire-damaged column specimens

The rehabilitation of fire-exposed columns (C_3, C_4, C_6, and C_7) was carried out through the following main procedures:

a *Removal of damaged concrete:* the deteriorated outer concrete layer was carefully removed manually until the longitudinal reinforcement was fully exposed. Mechanical methods were avoided to prevent dynamic vibrations that could weaken the structural elements.

b *Installation of shear connectors:* shear connectors with a diameter of 4 mm were horizontally welded on all four sides of the stirrups to ensure reliable bonding between the existing concrete core and the newly added outer concrete layer.

c *Epoxy preparation and application:* the epoxy resin (Sikadur®-32 LP) was prepared by separately mixing the base and hardener, followed by thorough mixing using a low-speed drill for two minutes until a uniform colour was achieved, in accordance with the manufacturer's instructions. The cleaned surface of the existing concrete was then coated with a layer of epoxy to enhance the bond between the old and new concrete layers.

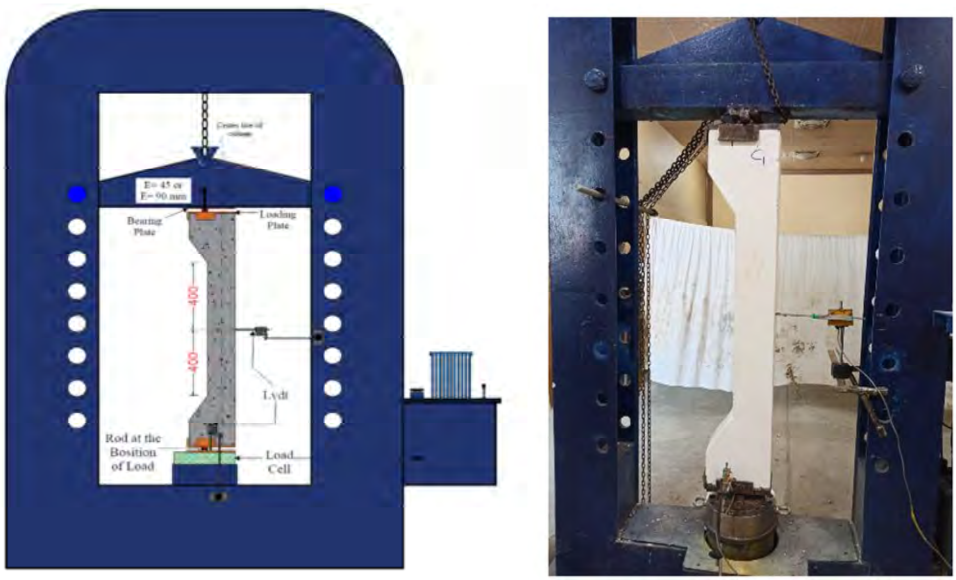
d *Casting of new concrete:* two types of concrete were used for strengthening. Normal concrete was applied for the first strengthening stage (R_1) with a compressive strength of 29.4 MPa, while UHPC was used for the second strengthening stage (R_2). Both concrete types were water cured for 28 days. All procedures are illustrated in Figure 8.

Figure 8 Sequential steps of RC column repair (see online version for colours)

For the application of CFRP sheets on the fire-exposed columns, the following steps were performed:

1. *Cutting of CFRP sheets:* the CFRP sheets were cut to the required lengths, and the concrete surface was thoroughly cleaned to remove any contaminants.
2. *Epoxy preparation:* the two components of the epoxy adhesive (Sikadur-330, components A and B) were mixed at a ratio of 4:1 until a homogeneous colour was obtained.
3. *Epoxy application:* an epoxy layer with an approximate thickness of 1.5 mm was applied to both the column surface and the CFRP sheets.
4. *Installation of CFRP sheets:* a single layer of CFRP sheets were carefully placed onto the epoxy-coated column surface and pressed using a rubber roller to ensure full contact and to expel excess epoxy from both sides of the sheets. Any remaining epoxy at the sheet edges was removed, as shown in Figure 8.

Figure 9 Test setup with instrumentation details (see online version for colours)



7 Test setup and procedure

The specimens were tested under monotonic loading using a hydraulic universal testing machine with a maximum capacity of 2,500 kN. The applied load was measured through a load cell installed at the base of the testing machine and connected to a data acquisition system. The load was gradually increased until failure, while the corresponding load-deflection response was continuously recorded. Axial and lateral displacements of the eccentrically loaded columns were monitored using linear variable differential transformers (LVDTs) installed along the column height. Two LVDTs were placed at the base of the machine along the axial direction to measure column shortening at each loading level, while an additional LVDT was installed at the mid-height of the column to record lateral deflection. This procedure was repeated for each loading stage. During testing, appropriate safety measures were implemented while monitoring crack initiation and propagation. A detailed evaluation of the failure modes, crack patterns, and ultimate load-carrying capacity was subsequently performed, as illustrated in Figure 9.

8 Results and discussion

The test results of each column were systematically compared with those of the other specimens to assess the effects of fire exposure duration and intensity, while maintaining constant load stability and eccentricity. The evaluated parameters comprised the failure mode, ultimate load-carrying capacity, first cracking load, and axial deformation of the column specimens, as summarised in Table 4.

Table 4 Results of laboratory testing on column specimens

Specimen symbol (Ci)	*Ultimate load capacity kN*	*Percentage decreasing load carrying capacity (%)*	*Ultimate axial deformation (mm)*	*Ultimate mid-height lateral deflection (mm)*
C_1	357	0	10.33	10.63
C_2 $T_{500}W$	251	–29.69	8.56	9.94
C_3 $T_{500}WR_1$	341	–4.48	10.96	20.93
C_4 $T_{500}WR_2$	486	+36.13	9.83	11.86
C_5 $T_{700}W$	174	–51.26	8.62	13.86
C_6 $T_{700}WR_1$	320	–10.36	9.51	21.35
C_7 $T_{700}WR_2$	448	25.49	9.33	10.09

8.1 Load carrying capacity

The maximum load applied and determined from the experimental tests represents the ultimate capacity of the column specimens. Beyond this load level, a noticeable reduction in the recorded load was observed, accompanied by rapid and irreversible deformation, which is identified as structural failure. The behaviour of the columns tested under ambient conditions was adopted as a reference to evaluate and quantify the residual load-carrying capacity of columns after exposure to elevated temperatures. As previously noted, the results indicated that the unexposed reference column exhibited the highest ultimate load capacity among all tested unstrengthened specimens.

Compared with the unexposed reference column, the ultimate load-carrying capacity of columns C_2 $T_{500}W$ and C_5 $T_{700}W$, which were exposed to fire at temperatures of 500°C and 700°C, decreased by)29.69% and 51.26% (respectively, as illustrated in Figure 10 and Table 4. Subsequently, the fire damaged columns C_3 $T_{500}WR_1$ and C_6 $T_{700}WR_1$ were rehabilitated using a full CFRP wrapping system. The results revealed that this strengthening technique did not fully restore the original capacity of the columns, as slight reductions of (4.48% and 10.36%), respectively, were still observed relative to the reference column. This reduction is attributed to the very low axial compressive stiffness of the CFRP sheets due to their small thickness compared with the concrete cross-section, which led to epoxy matrix rupture. Consequently, the primary load-carrying component within the composite layer was the unidirectional CFRP fibres, which inherently lack the capacity to resist axial compressive forces, in agreement with the findings reported by Nicolae et al. (2008). Nevertheless, full CFRP confinement provided a notable improvement when compared with the corresponding fire-damaged columns, increasing the ultimate load capacity by (35.86% and 83.91%) relative to columns C_2 T_{500}W and C_5 T_{700}W, respectively.

Alternatively, UHPC was employed to strengthen and rehabilitate the fire-damaged columns C_4 $T_{500}WR_2$ and C_7 $T_{700}WR_2$ with the aim of enhancing and restoring their load-carrying capacity. During the rehabilitation process, UHPC completely reconstructed the deteriorated concrete cover. Compared with the unexposed reference specimen, this technique resulted in significant increases in ultimate load capacity of (36.13% and 25.49%), respectively. Furthermore, the strengthened specimens exhibited substantially higher improvements of (93.62% and 157.47%), respectively, when compared with their corresponding fire-damaged columns C_2 T_{500}W and C_5 T_{700}W, as illustrated in Figure 11. The superior enhancement in load-carrying capacity observed in severely fire damaged columns is mainly attributed to the mechanical degradation of the original concrete core. Thermal damage leads to a pronounced reduction in the stiffness and elastic modulus of the original concrete, causing a greater redistribution of stresses toward the high stiffness UHPC jacket. As a result, the UHPC layer carries a larger proportion of the applied load and provides effective confinement to the inner concrete core. In contrast, for columns subjected to moderate thermal damage, the contribution of UHPC to the overall structural performance is relatively limited, as a larger portion of the original concrete strength is retained, leading to lower enhancement ratios despite having the same jacket thickness.

The outstanding mechanical properties of UHPC, particularly its extremely high compressive strength, make it a highly effective material for strengthening and rehabilitating fire damaged columns. This characteristic is essential for cement-based repair materials used in structural members governed predominantly by compressive forces. However, compressive strength alone should not be considered the primary criterion for evaluating the effectiveness of UHPC in structural rehabilitation. Other mechanical properties, especially tensile strength, play a more critical role, as they contribute to reducing or preventing cracking and deformation induced by shrinkage (Alasmari, 2023).

Figure 10 Load capacity versus fire flame temperature (see online version for colours)

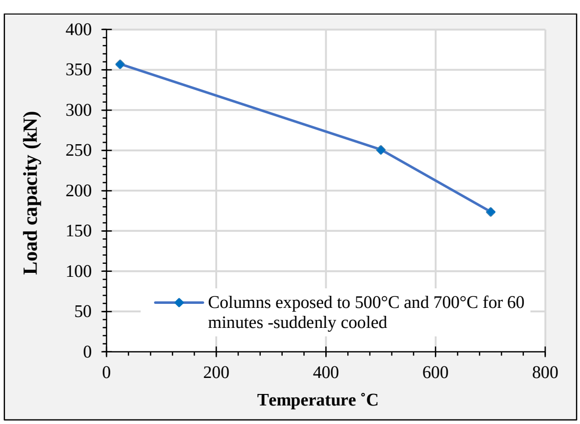
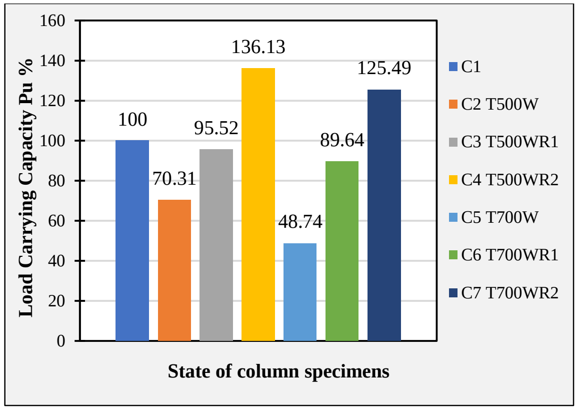


Figure 11 Percentage of load carrying capacity for the tested column specimens (see online version for colours)



8.2 Load-displacements relationship of reinforced concrete columns

A clear convergence was observed between the lateral and axial displacement values, with both indicating an increase in ultimate displacement as the fire exposure temperature increased. Strengthening with CFRP contributed to an improvement in the displacement behaviour of the columns under loading. Figures 12 and 13 present selected load-displacement curves illustrating this response.

Furthermore, the overall deformation behaviour of the fire-damaged columns under loading was significantly enhanced when strengthened using UHPC. This improvement is primarily attributed to the exceptionally high compressive strength and elastic modulus of UHPC, which increase the stiffness of the rehabilitated columns and effectively restrain crack propagation due to the presence of steel fibres.

Figure 12 Load-deflection of exposed and strengthened column (see online version for colours)

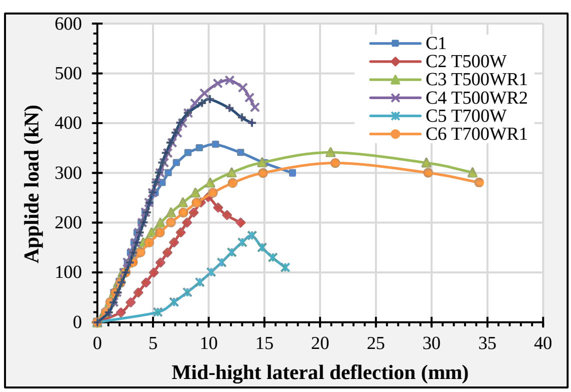
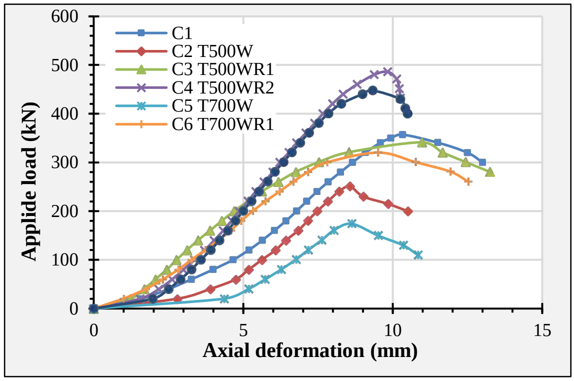


Figure 13 Load-axial deformation of exposed and strengthened column (see online version for colours)



8.3 First crack load

The reference reinforced concrete columns did not exhibit any signs of cracking during the early stages of loading. Crack widths were measured using a crack gauge, while the initiation of the first visible crack was identified through direct visual observation and the corresponding load was recorded. In eccentrically loaded specimens, transverse flexural cracks typically initiate in the tension zone and propagate toward the compression zone, whereas longitudinal cracks generally develop as shear cracks near the corbel region. Very fine cracks, commonly referred to as hairline cracks, were observed in the fire-exposed specimens, indicating that thermal effects had induced pre-cracking in these columns. The black-coloured cracks resulted from the influence of eccentric loading during mechanical testing, whereas the red-coloured cracks were attributed to axial loading effects after fire exposure. The maximum crack width values for both strengthened and unstrengthened columns under service load conditions and after fire exposure are summarised in Table 5.

Table 5 Crack width of the columns

Specimen symbol (Ci)	*Maximum crack width after exposure to fire (mm)*	*Crack width at service load (mm)*	*Location of crack*
C_1	-	0.24	In the middle of the column
C_2 $T_{500}W$	0.24	0.4	In the middle of the column
C_3 $T_{500}WR_1$	0.2	-	In the first quarter of the column
C_4 $T_{500}WR_2$	0.18	0.18	In the middle of the column
C_5 $T_{700}W$	0.4	0.62	In the middle of the column
C_6 $T_{700}WR_1$	0.46	-	In the middle of the column
C_7 $T_{700}WR_2$	0.48	0.06	In the last quarter of the column

The non-fire-exposed columns exhibited the smallest crack widths among the unstrengthened specimens, with an average value of 0.24 mm, as shown in Figure 14(a) and Table 5. In contrast, the fire-exposed and rapidly cooled columns showed increased average crack widths of (0.32 and 0.51) mm for crack width after fire exposure and crack width under service load, respectively, as illustrated in Figure 14(b). This increase is mainly attributed to the adverse effects of fire exposure on the concrete. For the fire-exposed columns subsequently rehabilitated using UHPC, the average crack widths were reduced to (0.33 and 0.12) mm for crack width after fire exposure and crack width under service load, respectively, as shown in Figure 14(c). This reduction is primarily attributed to the presence of steel fibres in the UHPC, which enhance tensile resistance and significantly improve crack control in the rehabilitated columns.

9 Failure's mode

In general, the columns were tested under an eccentric load with an eccentricity of (e = 45 mm). Columns subjected to this type of loading typically exhibited compression-controlled failure. The failure process developed gradually, with cracks first appearing on the tension side and then progressively propagating toward the compression side, extending across the remaining faces of the column. For the unexposed reference column, crack initiation and propagation were relatively slow and limited compared with the fire-exposed columns. In contrast, the fire-exposed columns contained pre-existing cracks induced by thermal damage; therefore, crack propagation and widening occurred more rapidly and under lower load levels. Furthermore, the ease of crack initiation increased with longer fire exposure durations. In general, failure occurred due to spalling of the concrete cover in the compression zone of the column. The cover subsequently began to crush and propagate both longitudinally and transversely until complete detachment occurred, accompanied by buckling of the longitudinal reinforcement, as illustrated in Figure 15.

For the columns strengthened with CFRP, failure was characterised by tearing of the CFRP sheets on the tension side and spalling of the concrete on the compression side, accompanied by significant column buckling and the development of a shear crack at the corbel region, as shown in Figure 15. This behaviour confirms that fire-damaged columns strengthened with CFRP exhibit a failure mode similar to that of unstrengthened columns. CFRP confinement contributed to restraining the concrete core, thereby enhancing its resistance to crushing under the same load level. In addition, the applied load was shared between the reinforced concrete column and the CFRP composite layer. In contrast, the columns strengthened with UHPC exhibited cracking on the tension side; however, the cracks were narrower and less widely distributed due to the presence of steel fibres, as shown in Figures 14 and 15. The UHPC jacket provided additional confinement, which increased the column's resistance to crushing under the same load level. Owing to the high compressive strength and high modulus of elasticity of UHPC, no crushing failure occurred in the compression zone at the same loading level. With further load increase, shear failure was observed at the corbel region, which had been adversely affected by fire exposure due to elevated concrete temperatures.

Figure 14 Spread of crack in specimens, (a) column specimens without fire (b) column specimens with fire exposure and c. Repaired column specimen (see online version for colours)

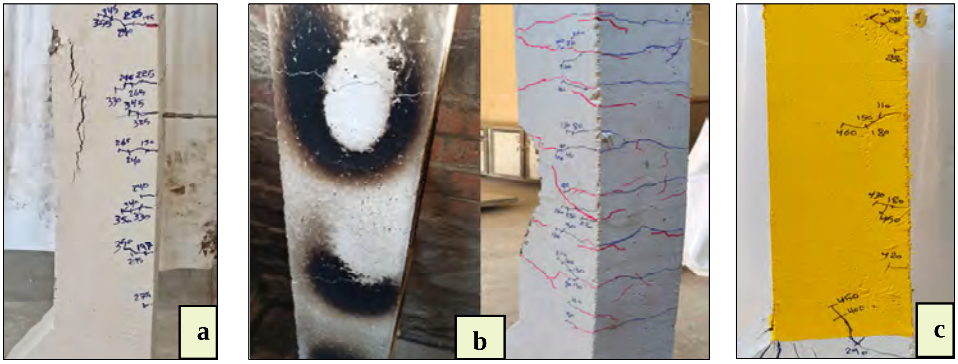
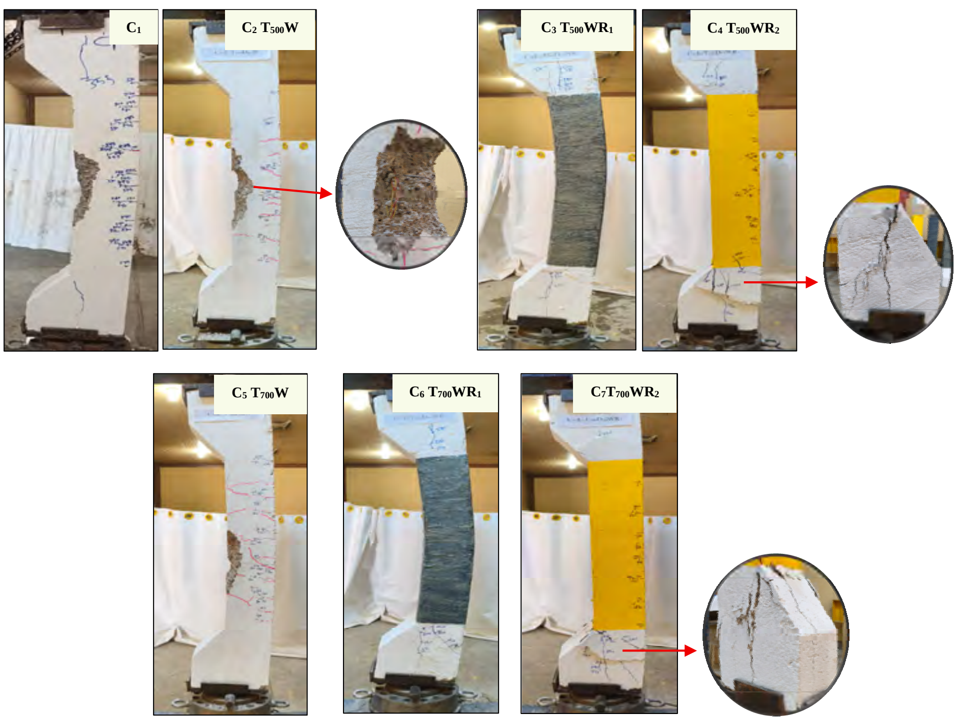


Figure 15 Failure modes of columns after testing (see online version for colours)

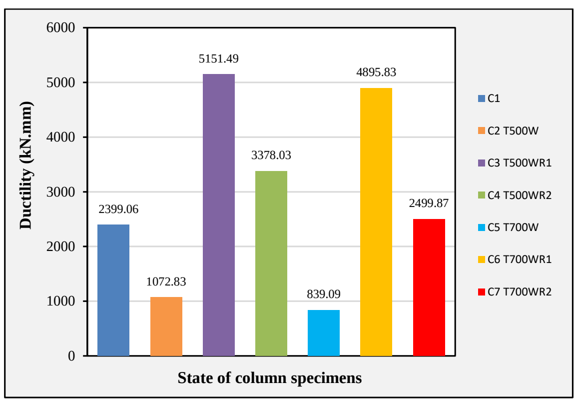


10 Ductility

In this study, the energy absorption capacity method was adopted to evaluate the ductility of reinforced concrete columns. The energy absorption capacity is defined as the area under the load displacement curve up to the maximum applied load. The calculated areas under the curves, which represent the ductility of the columns, are presented in Figure 16.

The results clearly demonstrate that strengthening with CFRP significantly enhances the ductility of fire exposed columns. Specifically, CFRP strengthening increased the ductility by 380.18 % and 483.47 % for columns exposed to fire temperatures of 500°C and 700°C, respectively. These findings highlight the effectiveness of CFRP in improving the post fire ductile behaviour of reinforced concrete columns by increasing their energy absorption capacity and enhancing resistance to sudden failure. These results are consistent with the findings reported by Toutanji (1999), who indicated that confining concrete with FRP sheets can substantially improve its strength, ductility, and energy absorption capacity. On the other hand, the obtained results also indicate that the ductility of fire exposed columns was markedly improved when strengthened with UHPC compared with the reference specimens. In particular, UHPC strengthening increased the ductility by 214.87 % and 197.93 % for columns exposed to fire temperatures of 500°C and 700°C, respectively. These results clearly confirm the efficiency of UHPC in enhancing the post fire ductile behaviour of reinforced concrete columns by increasing their capacity to absorb energy and improving their resistance to sudden failure.

Figure 16 The ductility of specimens (see online version for colours)

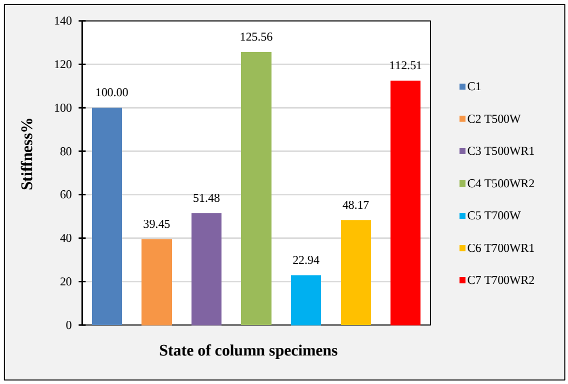


11 Stiffness parameter

Stiffness is defined as the amount of load required to induce a unit deformation in a structural member. A commonly adopted method for evaluating stiffness is the secant slope approach, in which the slope of the load-displacement curve at a point corresponding to 75% of the ultimate load is taken as the stiffness value (Muthuswamy and Thirugnanam, 2014). As illustrated in Figure 17, the stiffness of each column was calculated and compared with that of the reference column.

The results clearly indicate that column stiffness decreases significantly with increasing fire exposure temperature and duration. At exposure temperatures of 500°C and 700°C, reductions in stiffness of 60.55% and 77.06%, respectively, were observed, accompanied by a corresponding decrease in load-carrying capacity. In contrast, strengthening with UHPC resulted in a substantial enhancement in both load-carrying capacity and column stiffness. Increases in stiffness of 218.24% and 390.42% were recorded for columns exposed to 500°C and 700°C, respectively. Conversely, strengthening with CFRP led to comparatively smaller improvements in stiffness, amounting to 30.49% and 109.97% for columns exposed to 500°C and 700°C, respectively.

Figure 17 Stiffness of specimens (see online version for colours)



12 Conclusions

The following summarises the main conclusions that can be drawn regarding the behaviour of the investigated columns based on the experimental results:

1 After fire exposure, specimens subjected to 500°C exhibited higher stiffness than those exposed to 700°C. This behaviour is attributed to the elevated fire temperature, which caused greater thermal expansion in both the concrete and the reinforcing steel, resulting in increased internal stresses.

2 Increasing the fire exposure temperature combined with sudden cooling led to a significant reduction in the load-carrying capacity of the columns. Compared with the reference column, the load capacity decreased by 29.69% and 51.26% for the specimens exposed to 500°C and 700°C, respectively.

3 Observations of crack propagation in preloaded columns exposed to fire indicated that crack initiation, widening, and spread became more pronounced with increasing fire temperature and longer exposure durations.

4 Full confinement of fire-damaged columns using CFRP significantly improved structural performance, increasing the load-carrying capacity by 35.86% and 83.91% for the specimens exposed to 500°C and 700°C, respectively, compared with the unstrengthened columns. In contrast, UHPC-strengthened columns showed superior performance, with increases in load capacity of 93.62% and 157.47% for the 500°C and 700°C exposures, respectively. This substantial improvement is mainly attributed to the severity of thermal damage, which caused considerable thermal expansion in both concrete and reinforcement, leading to reductions in stiffness and strength. Moreover, UHPC strengthening altered the failure mode from axial compression failure to shear failure in the corbel region.

5 Full jacketing of fire-damaged columns with UHPC resulted in substantial improvements in both stiffness and ductility. Compared with the reference specimens, ductility increased by 206.4%, while stiffness improved by 304.33%. In comparison, full CFRP strengthening enhanced ductility by 431.83% relative to the fire-damaged columns, while stiffness increased by 70.23%.

6 Overall, both strengthening techniques proved effective in restoring the load-carrying capacity of fire-damaged columns. However, UHPC strengthening provided superior performance compared with CFRP strengthening. This is primarily because the columns failed under compression with load eccentricity, and UHPC possesses high compressive and tensile strengths, which prevented column failure and shifted the failure mode to the corbel region. In contrast, CFRP exhibits limited compressive capacity but high tensile strength; although it effectively restrained tensile cracking through confinement, it led to significant column buckling and rupture in the tension zone, along with crushing of the confined concrete in the compression zone due to the unidirectional nature of the fibres.

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Data availability statement

The data that support the findings of this study are available on request from the corresponding author.

Declarations

All authors declare that they have no conflicts of interest.

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