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Post-fire behavior of RC columns repaired with square hollow section steel tube and RC concrete jackets

Comportamiento post-incendio de columnas CR reforzadas con tubo de acero de sección hueca cuadrada y camisas de concreto RC

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Resumen
Este artículo investiga columnas de hormigón armado (RC) expuestas numéricamente al fuego. Como primer paso, el estudio examinó los efectos de exponer las columnas al fuego durante 60 minutos del estándar ISO 834 sobre la capacidad de carga residual de la columna considerando algunos parámetros geométricos decisivos como la altura de la columna y su área de sección transversal. El segundo paso consistió en investigar la efectividad de la técnica de reforzamiento utilizada mediante la incorporación de jackets compuestas, considerando diferentes resistencias del hormigón, con el fin de mejorar el comportamiento post-incendio de estas columnas. Los resultados mostraron que cuanto más tiempo está expuesta la columna al fuego, menor es su capacidad portante. Sin embargo, también se encontró que aumentar el área de la sección transversal de la columna puede reducir el porcentaje de capacidad de carga. Finalmente, se reveló que el método de refuerzo aquí utilizado permitió restaurar la capacidad de las columnas expuestas al fuego por un período de una hora hasta en un 180%.
Palabras clave: Columna de hormigón armado; Daño por fuego; Fortalecimiento post-incendio; chaqueta compuesta.

Abstract
This paper investigates numerically fire-exposed reinforced concrete (RC) columns. As a first step, the study examined the effects of exposing the columns to fire for 60 minutes according to the ISO 834 fire standard on the column's residual load-bearing capacity by considering some decisive geometrical parameters such as the column height and its cross-sectional area. The second step consisted of investigating the effectiveness of the strengthening technique utilized by incorporating composite jackets, considering different strengths of concrete, in order to improve the post-fire behavior of these columns. The results showed that the longer the column is exposed to fire, the

lower its bearing capacity. However, it was also found that increasing the column cross-sectional area can reduce the percentage of load-bearing capacity. Finally, it was revealed that the strengthening method used herein allowed restoring the capacity of the columns exposed to fire for a period of one hour by up to 180%.
Keywords: Reinforced concrete column; Fire damage; Post-fire strengthening; Composite jacket.

1. Introduction

1.1 Background

Concrete buildings are among the most prevalent forms of structures worldwide. Consequently, the study of the behavior and collapse of these structures is one of the most significant areas of study. Fires and earthquakes pose the greatest danger to structures.

Fire resistance of reinforced concrete (RC) buildings is crucial for ensuring their combustibility and security. Thermal conductivity and resistance of beams and columns affect this capacity. Due to their inflammability, low thermal conductivity, and large thermal mass, typical RC sections can withstand fire exposure and be repaired and reinstated into service. In 2008, the Concrete Society issued a technical report concluding that fire or abrupt high-temperature disturbances rarely cause the collapse of concrete structures (Qiu *et al.*, 2021) . In the post-fire performance evaluation, it must be determined whether the structure should be repaired and reinforced or demolished and rebuilt from scratch. The post-fire serviceability of a concrete structure is contingent on its residual load-bearing capacity, which must be assessed, quantified, and compared to safety standards. Under fire conditions, concrete structural elements are susceptible to numerous alterations and damages, such as material degradation (concrete and steel), heat-induced deformation (due to restraint effects), and redistribution of structural loads. The intensity and duration of the fire dictate the severity of these consequences (Franssen & Kodur, 2001)(Bailey & Khoury, 2011)(Industry, 2008). During the post-fire performance evaluation, the heat-damaged structural components will necessitate adequate repairs, taking into account their residual bearing capacity. Repairing the structure restored its load-bearing capability, strengthened the concrete, and improved its serviceability for its remaining service life. Developing and verifying strengthening techniques to repair or increase the fire resistance of heat-damaged reinforced concrete (RC) components is critical (Melo *et al.*, 2022). The research on strengthening technique reliability is a task to be developed based on durability and technical advancements. Concrete has a high thermal massivity and is non-combustible; hence, it behaves well in fire (Zhaodong & Jie, 2018). However, most fires destroy the outer layers of concrete, causing spalling. Dislocated concrete was 30–50 mm. Concrete spalls when mechanical and hydraulic loads exceed its tensile strength (Denoël, 2007). The cement paste and aggregate undergo physical and chemical changes when concrete is fired due to heat gradients inside the concrete cross section. At temperatures exceeding 500°C, the disintegration of $CaCO3$ at 600°C increases the material's porosity, while the dissociation of $Ca\ (OH)_2$ releases water at 450–550°C. Water in concrete begins to evaporate around 100 °C, while structural water in cement paste escapes at 300 °C, generating volumetric expansion and pore pressure. Concrete cracks are due to cement paste-aggregate incompatibilities (Industry, 2008). After fire, numerous researchers study concrete's residual qualities, especially mechanical ones like modulus of elasticity and compressive strength, which are employed in concrete structure modeling. Residual properties are determined immediately after the concrete cools. Studies of the deterioration of mechanical properties after a fire procedure by (Mohammadhosseini *et al.*, 2018). All cooling tests show lower residual strength (RS) and residual modulus of elasticity (RME) than heated ones. Heat rising into the enormous cross section for a long time causes thermal strains and cracks during cooling, causing additional concrete damage (Industry, 2008) ,Chemical processes can expand micro-cracks, reduce compressive strength, and delay structural failure (Gernay & Franssen, 2015). Other experimental research was performed on the destructive and non-destructive testing of cold and high-temperature degraded concrete (Muthuswamy & Thirugnanam, 2014). The mathematical models used to describe the mechanical behavior of concrete at high temperatures were subjected to a thorough evaluation (L. Li & Purkiss, 2005). Analytical methods predict concrete structures' post-fire load-carrying capability. These models use mathematical formulae and computations depending on concrete type, fire severity, exposure length, and structural parameters. These characteristics let analytical models evaluate the concrete structure's residual strength and load-carrying capability after the fire. These projections help engineers and designers evaluate structural integrity, decide on post-fire repairs or retrofitting, and ensure structure safety and stability (Y.-H. Li & Franssen, 2011)(Chang *et al.*, 2006).

1.2 Fire-Exposed Residual Load Capacity Of Columns

Fire-exposed, reinforced concrete structures rarely collapse. Components of fire-exposed reinforced concrete construction, such as columns, walls, and beams, benefit from resistance recovery. This is primarily due to the retained mechanical properties and bearing capacity of concrete after fire duration. A literature review indicates that a structure's load-bearing capacity decreases during heating, reaches a low point, then rebounds following cold exposure (Gernay & Franssen, 2015). Structure collapse during conflagration settling, as opposed to during the high temperature phase, can be attributed to fluctuations in load capacity caused by thermal inertia and a further degradation of material mechanical properties(Franssen & Kodur, 2001)(Franssen & Kodur, 2001). Various studies on RC columns were conducted and focused on the decrease in their load-bearing capacity following fire exposure. The parameters considered are the geometric characteristics (thermal massivity and slenderness), support conditions, aggregate types, heating conditions, steel reinforcement ratio, and load levels under fire(W. Li *et al.*, 2019)(Franssen & Kodur, 2001). The research employed numerical analyses to determine how the duration and intensity of a fire affect the remaining load capacity and the incidence of delayed failure in concrete-reinforced columns (Gernay & Dimia, 2011). The axial load-capacity, lateral/flexural strength, and rigidity of columns that have attained ambient temperature as a result of Stanford's fire tests were significantly diminished in other studies (Mostafaei *et al.*, 2010)(Kodur *et al.*, 2017). Experimental investigation and the results of post-fire testing on the behavior of RC columns were presented in (Gernay & Franssen, 2015) and (Lie *et al.*, 1986)

1.3 *Effectiveness repairing technique on post-fire RC columns*

Fire-exposed columns made of reinforced concrete may lose 50% of their bearing capability. Compressive strength and stiffness might decrease significantly (Yaqub *et al.*, 2013). Therefore, the expert engineer may make two distinct judgments regarding post-fire concrete structures: to repair or demolish. The economic predilection for the repair option makes it preferable to the demolition option (Yaqub *et al.*, 2013). It is known that the degradation of the residual mechanical characteristics of concrete caused by fire exposure is largely recoverable with time (Mostafaei *et al.*, 2010). The effectiveness of the repair action depends on the degree of fire damage to structural elements and the restoration technique used. Consequently, post-fire rehabilitation effectiveness is determined by the recovered load-bearing capacity relative to structural member capacity. Numerous studies on repairing post-fire columns have taken into account the type of repair method and the intensity of the fire that structural members experienced. Several studies were performed on the effectiveness of FRP jacketing to rehabilitate the fire-damaged column elements.

Yaqub and investigated fire-damaged RC column mechanical performance. Column cross-section geometry and repair materials impacted seismic and axial compression (Yaqub & Bailey, 2011b) . After heating to 500 °C, circular and square RC columns were retrofitted with single-layer FRP casings in these experiments. Moghtadernejad *et al.* investigated the rehabilitation of short, rectangular, post-heated RC columns with FRP jackets (Moghtadernejad *et al.*, 2021). Fire-damaged columns are coated with one or two layers of carbon FRP and glass FRP. The experimental results indicate that the Post-heated columns repaired with two layers of CFRP had far higher bearing capacities than unheated columns. In recent studies undertaken by Bisby (Bisby *et al.*, 2011) and

Al-Nimry (Al-Nimry & Ghanem, 2017), they investigated the influence of FRP confinement on heat-damaged RC columns and strengthened fire-damaged concrete. Performed research on the strengthening of concrete-filled steel tubular columns considering FRP jackets after exposure to ISO fire and the effect of the number of FRP layers on repairing the post-fire columns.

Based on the consulting of the research above, FRPs restored compressive strength rather than stiffness in post-fire columns. The stiffness of post-heated columns after repair is only half or lower than the initial value, regardless of fire intensity and restoration procedures. No clear dependency between the stiffness reduction and the fire intensity can be found (Qiu *et al.*, 2021). The stiffness loss in columns can be attributed to the damage incurred by the concrete, while the strength loss is mitigated to some extent by the presence of steel rebars (Chen *et al.*, 2009). Fiber-reinforced polymer (FRP) reinforcement increases column strength. FRP sheets have a better strain capacity than steel reinforcement and regain the strength of steel rebars (Yaqub & Bailey, 2011a). Hence, hybrid repair methods have the potential to provide superior strength and stiffness. FRP jackets are more effective at confining RC columns with circular cross-sections than rectangular ones. The bulk of RC columns in contemporary buildings have square or rectangular cross-sections. (Yaqub *et al.*, 2013). The effectiveness of confinement is heavily influenced by the shape of the column cross-section (square, circular, or rectangular) and the level of confining pressure, which is determined by the number of layers of FRP sheets wrapped around the column (Lenwari *et al.*, 2016).

This paper contributes to the improvement and reliability of strengthening techniques for evaluating the fire-damaged RC column's strength performance when the following factors are considered:

1. The influence of fire on the load-carrying capacity of the columns;
2. Fire duration;
3. Geometrical properties of the column;
4. The strengthening of reinforced concrete (RC) columns following fire exposure using rehabilitation technique composite jacketing with steel plates (CRPJ).

The first section examines how fire duration affects axially loaded reinforced concrete (RC) columns following exposure to fire. Calculate the column's compressive strength loss. In the second portion, a numerical study examines how different jacketing methods affect fire-damaged RC column strength.

2. Methodology

For the purpose of achieving the objectives of this research, it was deemed interesting first to use RC columns with various cross-sections, i.e. (0.3× 0.3) m², (0.4 x 0.4) m² and (0.6 x 0.6) m², with a cover of 30 mm, and various heights, i.e. 3 m, 4 m and 5 m. In order to numerically measure the reduction in the load bearing capability of columns after their exposure to a parametric fire of different durations (15, 30, 60 and 90 minutes) show in Figure 2. These samples were tested under axial loading. Then, one of these columns, namely the one with a cross-section (0.3 x 0.3) m² and a height of 3 m height, was subjected to post-fire repair and strengthening using the technique given below composite jacketing with steel plates.

It is worth indicating jacket thicknesses 10 cm were considered in this study. Note also that the SAFIR computer program (Franssen, 2005) was used in this research in order to estimate the structural

3. Finite element model development

This research modeled the behavior of RC columns after a fire using a 2D nonlinear numerical analysis. Using the computer program SAFIR (Franssen, 2005), the numerical calculations were performed. This program enables the nonlinear thermo mechanical analysis of fire-vulnerable materials such as steel, concrete, and composite steel. Using the computer program SAFIR (Franssen, 2005), the numerical calculations were performed. This program enables the nonlinear thermo mechanical analysis of fire-vulnerable materials such as steel, concrete, and composite steel structures. The analysis technique is founded on the method of finite elements. The study's goal is to examine reinforced concrete columns' fire behavior and assess their performance characteristics. In contrast, a square RC column measuring 3 m tall and having a (0.3 x 0.3) m^2 cross-section was chosen as the test specimen for the numerical analysis addressing fire scenarios. Longitudinal bars with a diameter of 12Φ16 mm (Figure 1), in addition to a 0.03 m cover on the longitudinal reinforcement, were used. Moreover, the concrete's compressive strength was 25 MPa, whereas the reinforcement's yield stress was 400 MPa. Also, take note that the structural examination (static analysis) for the control (non-exposed) specimens was finished in a single phase. To replicate along with specimens undergoing fire testing the Y-axis, the research column was securely fixed at the bottom end and constrained at the top, allowing zero displacement along the X- and Z-axes while allowing freedom of travel in that direction.

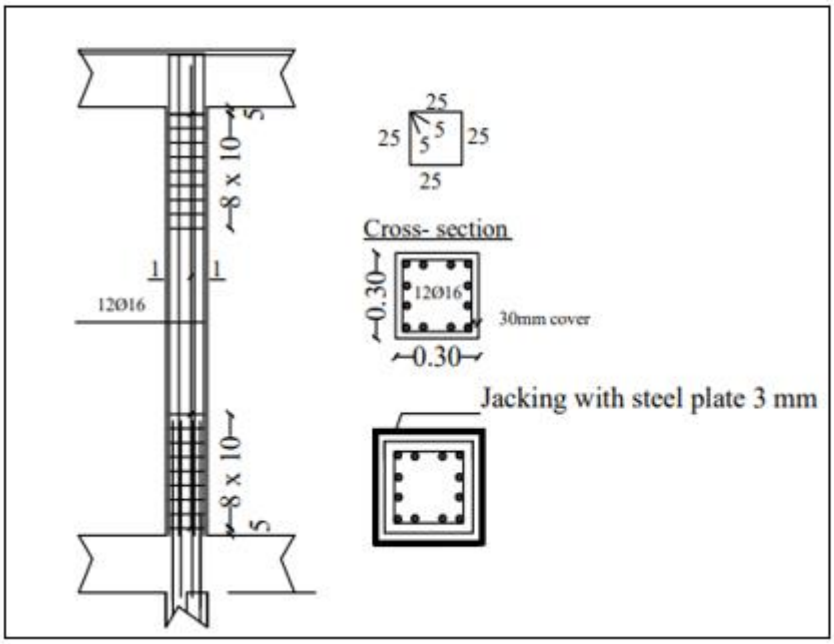


Figure 1 - Geometry and reinforcement details

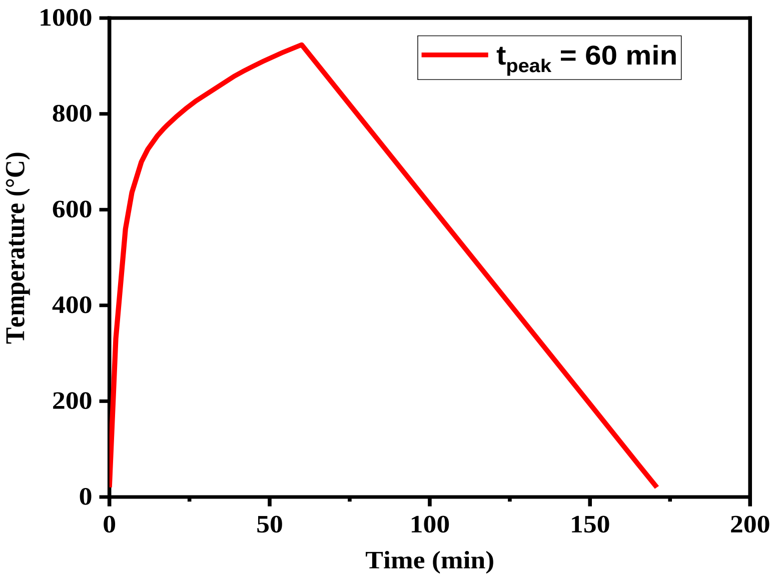


Figure 2 - Parametric and standard temperature-time curves used

4. Results and discussion

4.1 *Effect of effective column height*

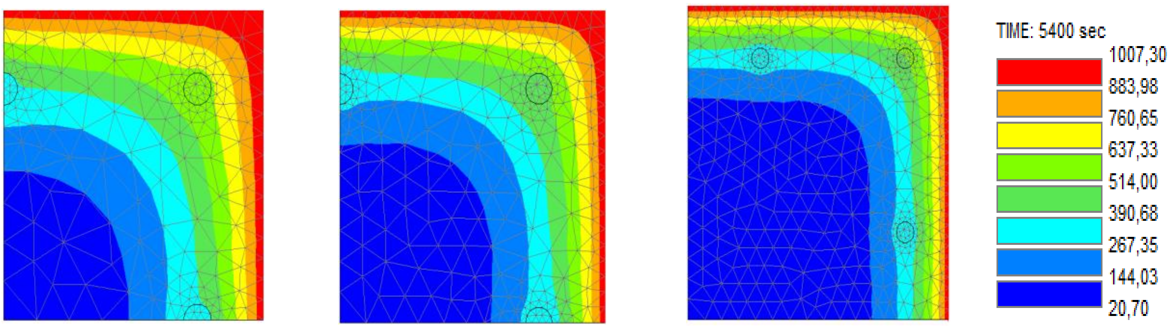
The findings for the load-bearing capacity of columns for a range of heights and fire durations (from t_{peak} = 60 min) show Figure 2 in on each of the column's four sides are compiled in Table 1. It is important to note in this regard that the cross-sectional area of (0.3 x 0.3) m^2 is regarded as constant. The evidence made it possible to draw the conclusion that there was fire present when the thin columns suffered significant burns.

Table 1 – Influence of the column's height, for different fire durations

Height (m)	**$N_{20°C}$ (KN)**	**t_{peak} = 15 min**	**t_{peak} = 30 min**	**t_{peak} = 60 min**	**t_{peak} = 90 min**
		$N_{collapse}$ (KN)			
3	2445	1723	1425	930	573
4	2221	1297	965	554	322
5	1970	915	640	348	206

4.2 *Effect of the cross-sectional area*

This time, the column height is kept constant at 3 m, while the cross-sectional area takes the values (0.3 x 0.3), (0.4 x 0.4), and (0.6 x 0.6) m^2. Figure 5 depicts the distribution of the highest temperatures attained in the exposed portions of the column during medium and lengthy fires for the various cross-sectional areas under consideration. Table 2 illustrates the bearing capacity of the column for each cross-section and for different fire durations. One can clearly see from that table that the bearing capacity decreases as the fire duration increases. In addition, the $N_{collapse}$ of the cross-section (0.3 x 0.3) m^2 is reduced by 76.56% when this section is exposed to fire for 90 min. However, this reduction is lower (32%) for the cross-section (0.6 x 0.6) m^2 for the other cross-section sizes. This means that the effect of fire is lower when the cross sectional area increases. Conversely, fire exposure enhances $t_{collapse}$.



(a) (b) (c)

Figure 5 - Distribution of the maximum expected temperatures after 90 minutes in different cross-sections: (a) 0.3×0.3 m^2, (b) 0.4×0.4 m^2, (c) 0.6×0.6 m^2 (A cross-sectional quarter with marked rebar positions is shown)

Table 2 - $N_{collapse}$ for different cross-sectional areas

Section (m²)	N_{20} °C (kN)	t_{peak} = 15 min	t_{peak} = 30 min	t_{peak} = 60 min	t_{peak} = 90 min
		N collapse (KN)			
0.3 x 0.3	2445	1723	86	1425	69
0.4 x 0.4	4455	3640	615	3291	520
0.6 x 0.6	9808	8720	279	8221	350

Figure 6 illustrates the use of the SAFIR FEA (Finite Element Analysis) program to estimate the anticipated maximum temperatures. Based on the results of a thermal transfer study conducted on the diagonal cross-section measuring (0.3 x 0.3) m^2 and extending from point 1 to point 5, these temperatures are calculated.They do not correlate to the same time point at every site because of the spreading thermal wave.

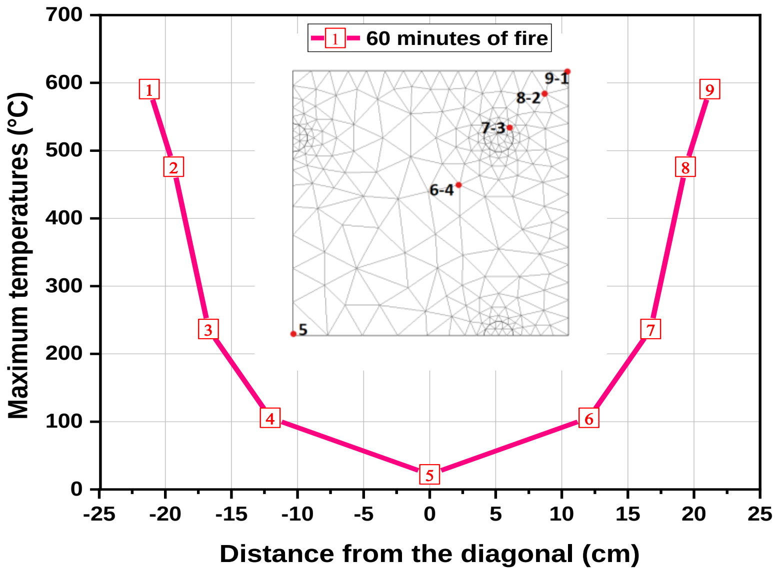


Figure 6 - Temperature maximum at diagonal section (Cross-section quarter with rebar placements).

4.3 *Effect of fire duration*

The influence of combustion duration exposure on the ultimate load-bearing capacity and load-deflection reaction of charred columns was investigated. Four duration periods were selected for this purpose: 15, 30, 60, and 90 minutes. The outcomes of the parametric study are presented. All specimens exhibited a substantial change in their load-bearing capacity. This was likely due to the variance in duration of fire exposure. Moreover, the FEA revealed that as the length of the fire (15, 30, 60, and 90 minutes) increased, so did the damage, the failure load of the fire-exposed column decreased by nearly 29.5, 41.7, 61.9 and 76.6%, respectively, compared with the 3 m high reference column (without fire), for the cross-sectional area of (0.3x0.3) m², as shown in Figure 5. As for the cross-section of area (0.4x0.4) m², the failure load dropped by about 18.3, 26.1, 39.7 and 51.7%, respectively. The minimum load reduction was recorded for the cross-section (0.6x0.6) m²; the reduction percentages were approximately 11.1, 16.2, 27.4 and 32.0%, respectively. Consequently, it was decided to study, in the following sections, the effect of strengthening the most damaged column with a cross-section of area (0.3x0.3) m² using different techniques.

Moreover, the numerical analysis revealed that the column's vertical displacement increased as the duration of fire exposure increased. As shown in Table 3.

Table 3 - Results related to RC columns exposed to fire for various durations

Fire duration (min)	Load bearing capability P_u (KN)	Axial deformation Δ_u (mm) at ultimate load	Load capacity reduction due to fire (%)
Ref-column	2444	7.19	-
15	1724	8.83	29.5
30	1426	10.85	41.7
60	930	14.94	61.9
90	573	18.84	76.6

The load-displacement relationship of the column exposed to a conflagration is depicted in Figure 7. This diagram contrasts the models of the columns that were exposed to fire with the model of the reference column. The load-bearing capacity is observed to decrease as the duration of the conflagration increases.

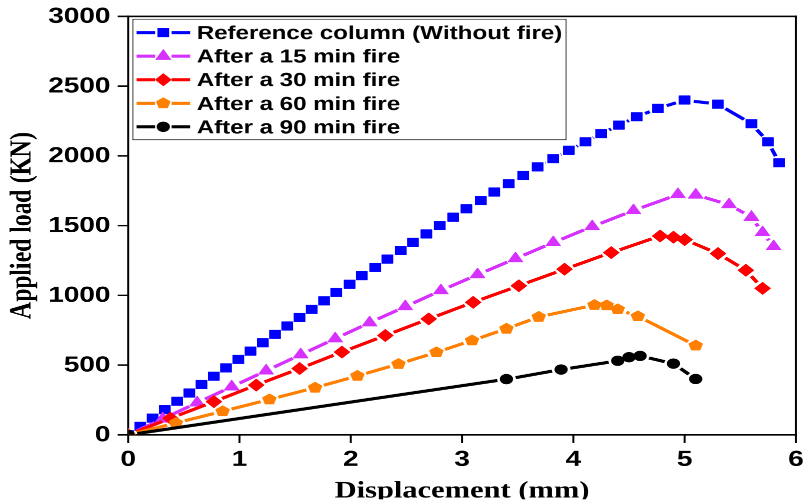


Figure 7 - Fire exposure period affects column load-vertical displacement.

Figure 8 shows the reduction in load-bearing capacity as a function of time. This phenomenon can be modeled by a nonlinear function that is represented by Equation 2. The function representing this variation is obtained by calculating the reduction in load-carrying capacity as a function time's t (min) as follows Equation 1:

$$P_U(\%) = \frac{P_U(T = 20\,°C) - P_U(T\,°C)}{P_U(T = 20\,°C)} \times 100 \tag{1}$$

Where $P_U(\%)$ Reduction in load capacity, $P_U(T = 20°C)$ Load carrying capacity in $T = 20°C$ and $P_U(T\,°C)$ Load carrying capacity in different temperature

$$P_U(\%) = -0.0031t^2 + 0.9542t + 15.911 \tag{2}$$

Where t time of exposure to fire in minutes

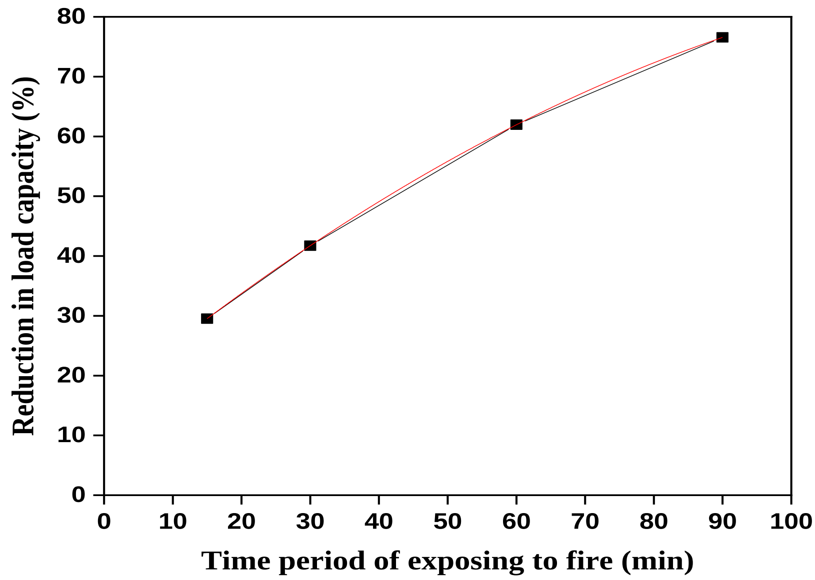


Figure 8 - Reduced load-bearing capacity of fire-exposed column.

5. Fire-exposed composite jacketing strengthens the RC column

On the other side, the temperature distributions at various depths of a column with a cross-sectional area equal to (30 x 30) m^2 and a height of 3 m, exposed to fire for durations ranging from 15 to 90 minutes, Strengthening of a reinforced concrete column after exposure to fire It is widely admitted that, in general, the durability of columns in fire-exposed buildings depends on the effectiveness of the techniques used to repair, strengthen, and treat these columns in order to rehabilitate them and achieve structural safety. The following tables show the percentages of increase in load-bearing the column's capacity using rehabilitation technique composite jacketing with steel plates (CRPJ).
***CRPJ**: column repaired with steel plates and jacket

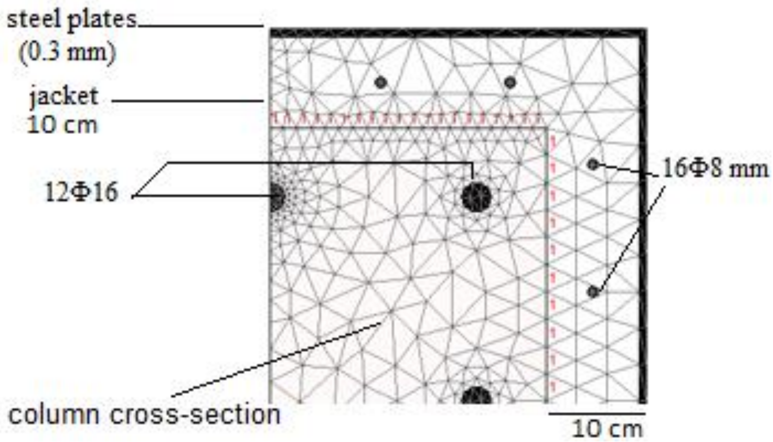


Figure 9 - Details of the repaired RC column after the fire
(A cross-sectional quarter with marked rebar positions is shown)

Moreover, a parametric research was performed to explore the effect of various design variables on the efficiency of a jacketed column that had previously been subjected to fire, for various durations (15, 30, 60 and 90 minutes). The compressive strengths found for concrete were equal to 25, 30, 35 and 40 MPa, and the jacket thicknesses used were 10 cm. The strengthening efficiency can be calculated using the following equation.

$$P_{str}(\%) = \frac{P_{str} - P_U(T\ = 20°C)}{P_U(T\ = 20°C)} \times 100 \qquad (3)$$

Where $P_{str}(\%)$ strengthening efficiency, P_{str} strengthening collapse load and $P_U\ (T = 20\ °C)$ Load-carrying capacity in $t = 20\ °C$.

On the other side, it was noted that when the variation in the external strength was equal to 25, 30, 35 and 40 MPa, the load-bearing capacity of the columns increased by 89.0%, 120.3%, 147.14 and 182.25%, for the fire duration of 60 minutes. It should also be noted that after 60 minutes, the column cannot be repaired because it has already lost 61.9 % of its original load-bearing capacity which corresponds to 2445 KN. Moreover, the models used in this study showed that the different structural strengthening techniques led to a significant increase in the resistance of columns to fire, which means that their load-carrying capacity was significantly improved.

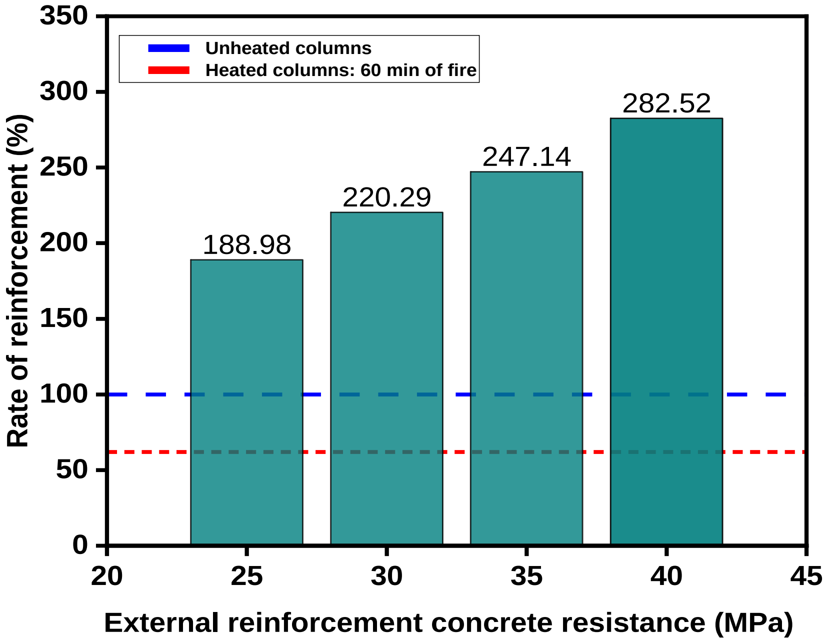


Figure 10 - Repairing the efficacy of various post-fire column rehabilitation techniques after 60 min of fire exposure.

6. Conclusions

This study's primary objective was to determine the effect of various elevated temperature environments on the residual load-carrying capacity of structural columns, taking into account important geometrical parameters such as the column's height and cross-sectional area. It also sought to study the impact of incorporating RC and composite jackets on improving the post-fire behaviour of these strengthened columns using various techniques. For this, the SAFIR software, a 3D nonlinear finite element program (Franssen, 2005), columns' post-fire behaviour was examined using this way. Taking into account different concrete strengths, steel plate composite jackets were employed to replace and reinforce fire-damaged columns. Based on the above, a number of conclusions could be drawn:

(1) The columns load-bearing capacity decreased after being exposed to a fire temperature of 1,000°C for 15, 30, 60, and 90 minutes. It also decreased as the height of the column went up.

(2) The load-bearing capacity of the column decreased respectively by 29.5%, 41.7%, 61.9% and 76.6% for the fire exposure times of 15, 30, 60 and 90 minutes in comparison with the reference column.

(3) When the concrete compressive strength increased, i.e. 25, 30, 35 and 40 MPa, the load-bearing capacity of the column also grew, respectively, by 89 %, 120 %. 147 % and 182 %, for a fire duration of 60 minutes.

(4) Similarly, when the fire duration was increased, i.e. 15, 30, 60 and 90 minutes, the load-bearing capacity of the column decreased significantly until failure.

(5) The strengthening of the concrete column is ineffective after an hour of exposure to fire at high temperatures.

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