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Post-Fire Behavior and Repair of Fire-Damaged RC Columns Using Composite Jackets

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ABSTRACT

Reinforced concrete (RC) structures are frequently employed in construction owing to their versatility, strength, and durability. However, these structures can be vulnerable to fire incidents, which can significantly compromise their structural integrity and load-carrying capacity. In the aftermath of a fire, damaged RC columns often necessitate rehabilitation to restore their strength and functionality. The present study intends to carry out a numerical investigation of the behavior of reinforced concrete (RC) columns after their exposure to fire. As a first step, the study examined the effects of exposing the columns to fire for different periods (15, 30, 60 and 90 minutes) 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 techniques utilized by adding reinforcement and incorporating composite jackets, where each method used three external concrete compressive strength values, 25, 30, and 40 MPa, 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. Moreover, A simple equation with sufficient accuracy has been proposed to predict the bearing capacity of reinforced columns. Finally, it was revealed that the strengthening methods used herein allowed restoring the capacity of the columns exposed to fire, but the strengthening technique using a composite jacket with steel plates showed better results in terms of strength. Where this technique allowed, it restored the capacity of the columns exposed to fire for a period of one hour by up to 182%.

1. INTRODUCTION

1.1 Background

Concrete structures are among the most common types of structures in the world. So studying the behavior of these structures and their collapse is among the most important fields of research. Fires and earthquakes are the worst threats to buildings.

The fire resistance of concrete-reinforced (RC) buildings is essential for ensuring their combustibility and safety. This capacity is substantially impacted by thermal conductivity and the resistance of load-bearing components such as beams and columns. After fire exposure, the reinforced concrete structure can remain standing and be repaired and brought back into service due to the concrete's incombustibility, low thermal conductivity, and typical RC sections' high thermal massivity. In 2008, the Concrete Society issued a technical report concluding that fire or abrupt high-temperature disturbances rarely cause the collapse of concrete structures [1]. After a fire, the structure must be restored, strengthened, or destroyed and rebuilt. Assessing, quantifying, and comparing a concrete structure's residual load-bearing capability to safety criteria is essential for post-fire serviceability. Concrete structural elements can degrade (concrete and steel), distort owing to restraint effects, and redistribute structural loads during fires. The extent of these effects depends on the intensity and duration of the fire exposure [2, 3]. The post-fire performance evaluation will demand suitable repairs for fire-damaged structural elements based on their residual bearing capability. The strengthening process restored the structure's load-bearing capability, strengthened the concrete, and improved its serviceability for its remaining service life. Strengthening technologies that repair or increase fire resistance of heat-damaged reinforced concrete (RC) elements must also be developed and validated [4]. 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 [5]. However, most fires destroy the outer layers of concrete, causing spalling. Dislocated concrete was 30-50mm. Concrete spalls when mechanical and hydraulic loads exceed its tensile strength [6]. 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℃, the disintegration of $CaCO_3$ at

600℃ increases the material's porosity, while the dissociation of Ca $(OH)_2$ releases water at 450-550℃. Water in concrete begins to evaporate around 100℃, while structural water in cement paste escapes at 300℃, generating volumetric expansion and pore pressure. Concrete cracks are due to cement paste-aggregate incompatibilities [7, 8]. After fire, several researchers analyze residual concrete properties, notably mechanical ones like modulus of elasticity and compressive strength, which are used in concrete structure modeling. After cooling, residual characteristics are measured. Fire-induced mechanical property degradation studies by the study of [9, 10]. 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 [11, 12]. Chemical processes can expand micro-cracks, reduce compressive strength, and delay structural failure. Other experimental research was performed on the destructive and non-destructive testing of cold and high-temperature degraded concrete [13-15]. The mathematical models used to describe the mechanical behavior of concrete at high temperatures were subjected to a thorough evaluation [16]. Analytical models that predict the residual load-capacity of concrete structures after fire [17].

1.2 Residual load capacity of columns after fire exposure

The concrete-reinforced building seldom collapses after fire exposure. Resistance recovery improves the performance of reinforced concrete construction components including columns, walls, and beams following fire exposure. This is mostly due to concrete's residual bearing capacity and mechanical qualities as a function of fire duration. The cooling phase of a fire is more likely to cause building failure due to thermal inertia-induced load capacity variations and material mechanical property deterioration [18]. Various studies on RC columns were conducted and focused on the decrease in their load-bearing capacity following fire exposure. 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 [19]. 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 [20-23]. The axial load capacity, lateral/flexural strength, and rigidity of columns that have cooled to room temperature as a result of Stanford's fire tests were significantly diminished in other studies [24, 25]. Experimental investigation and the results of post-fire testing on the behavior of RC columns were presented in [26].

1.3 Effectiveness repairing technique on post-fire RC columns

Repairing structural parts improves performance. Steel and concrete jacketing and fiber-reinforced polymer (FRP) materials can enhance older reinforced concrete (RC) columns or those damaged by overloading. These confinements boost the columns' structural and load-bearing capabilities. Recent structural repairs have employed several technologies. External jacketing repairs structural members cheaply. Concrete and steel jackets stiffen and prevent bond failure in ductile construction. Steel jackets expand sectional area and discontinuities. FRP wraps outperform concrete jacketing for strengthening or repairing. No increase in structural self-weight, section area, or stiffness-important features for seismic rehabilitation-can be reported here. FRP wraps outperform concrete jacketing for strengthening or repairing. No increase in structural self-weight, section area, or stiffness-important features for seismic rehabilitation-can be reported here. In addition, due to their fire sensitivity, insulation should be applied to achieve satisfactory fire resistance [25]. Since the first steel-jacketing method as a seismic retrofitting technique was introduced, a series of studies have been conducted on jacketing repair methods to improve and establish the reliability of each [26-30].

Fire-exposed reinforced concrete (RC) columns are known to lose partial or complete bearing capacity, depending on the intensity and extent of the fire. A fair amount of compressive strength and stiffness can be lost. 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 [31]. It is known that the degradation of residual mechanical characteristics of concrete caused by fire exposure is largely recoverable with time [32].

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 Bailey studied the mechanical performance of fire-damaged RC columns [33]. They examined the influences of column cross-section geometry and repair materials on the effectiveness of axial compression and seismic performanc. In these studies, circular and square RC columns were heated to 500℃ and retrofitted with single-layer FRP jackets after heating. Moghtadernejad et al. [34] investigated the rehabilitation of short, rectangular, post-heated RC columns with FRP jackets. One or two layers of carbon FRP and glass FRP are applied to the fire-damaged columns. 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 et al. [35] and Al-Nimry and Ghanem [36], 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 [37-39]. Review used an anthor configuration to repair post-fire RC columns using hybrid FRPs in order to improve their load-bearing capacity and the reliability of the repair technique used. The hybrid method was a combined form of two or multiple FRP repair techniques [40-42].

FRPs restored compressive strength, not stiffness, in post-fire columns. Regardless of fire intensity and restoration, post-heated columns have half or less stiffness. Fire intensity doesn't diminish stiffness. Steel rebars reduce column stiffness, while concrete damage reduces strength. Fiber-reinforced polymer (FRP) reinforcing measures significantly enhance the ultimate strength of columns. This is mainly because the strength of steel rebars is typically regained through the use of FRP, and FRP sheets allow for a significantly higher strain capacity compared to traditional steel reinforcement [43]. 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 [44]. 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 [45].

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 effect of burning on the load-bearing capacity of the columns;
2. Fire duration;
3. Geometrical properties of the column;
4. The strengthening of reinforced concrete (RC) columns following fire exposure utilizing various jacketing methods. The strengthening of reinforced concrete (RC) columns following fire exposure utilizing various jacketing methods repaired using composite jacketing with steel plates was explored.

It is useful to remember that one of the main objectives of examine how fire duration affects axially loaded reinforced concrete (RC) columns following exposure to fire. Calculate the column's compressive strength loss, after that, 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, a numerical program was carried out using concrete columns. It was deemed interesting first to use RC columns with various cross-sections, i.e., (0.3×0.3) m², (0.4×0.4) m² and (0.6×0.6) m², with a cover of 30mm, and various heights, i.e., 3m, 4m and 5m. In order to numerically measure the reduction in the bearing capacity of columns after their exposure to a parametric fire of different durations (15, 30, 60 and 90 minutes). The amount of burn exposure is measured by Eq. (1).

$$T = 20 + 345 \times \mathrm{LOG}\,(8 \times t + 1) \qquad (1)$$

These samples were tested under axial loading. Then, one of these columns, namely the one with a cross-section (0.3×0.3) m² and a height of 3m height. The longitudinal the diameter of the bars was 8Ø12mm, and the spacing between links was 10mm. was subjected to Post-fire Behavior of RC columns repaired using composite jacketing with steel plates given below:

- Reinforced concrete jacketing;
- Composite jacketing with steel shells;
- Composite jacketing with steel plates.

Composite jacketing is a commonly used technique for enhancing the load-carrying capacity and stability of fire-damaged structures, particularly steel and concrete elements. It involves wrapping the damaged structural members with layers of composite materials, such as steel plate (SP), fiber-reinforced polymers (FRP), to provide additional strength and confinement. The SP jacketing was comprised of four steel plates were 3mm, each of which covered one of the specimen's four contiguous faces and was longitudinally bonded to the heat-damaged surface. The steel slabs were attached to the column's four sides. Indeed, it is important to note that this study considered two different jacket thicknesses (50mm and 100mm) and various concrete compressive strength values (25 MPa, 30 MPa, and 40 MPa) as part of its investigation. These variations in jacket thickness and concrete strength allowed for a comprehensive analysis of their effects on the performance of the reinforced concrete columns after fire exposure. By examining different jacket thicknesses, the study could assess the influence of the external confinement on the post-fire behavior of the columns. Thicker jackets are expected to provide more effective confinement, potentially leading to improved structural performance and higher load-carrying capacity after fire exposure. Similarly, investigating different concrete compressive strength values allowed the study to explore how the strength of the concrete material affects the structural response of the columns in fire scenarios. Higher concrete compressive strengths might result in enhanced fire resistance and structural integrity of the columns. Considering these various parameters in the study contributed to a more comprehensive understanding of the behavior of reinforced concrete columns under fire conditions and provided valuable insights into the factors that influence their post-fire performance. Such data can be valuable for developing practical design guidelines and repair strategies to enhance the fire resistance of concrete structures and ensure their structural safety. Structural and thermal analysis using SAFIR computer program [46] was used in this research in order to estimate the structural behavior of the columns under study. SAFIR is a computer program developed by Franssen and Gernay [12] at the University of Liège in Belgium for the analysis of structures under ambient and elevated temperature conditions. The program, which is founded on the Finite Element Method (FEM), can be utilized to analyze the behavior of one-, two-, and three-dimensional structures composed of concrete, steel, reinforced concrete, wood, composites, etc. As a finite element program, SAFIR can use distinct elements for various idealizations, calculation methods, and material models in order to account for stress-strain behavior. The elements include the 2-D SOLID elements, 3-D SOLID elements, BEAM elements, SHELL elements, and TRUSS elements. Several stages may be involved in analyzing a structure that has been subjected to fire. The first stage, thermal analysis, entails predicting the temperature distribution within the structural members. The' structural analysis', the final step of the analysis, is performed primarily to determine the response of the structure to static and thermal loading The analysis consists of two parts: a thermal analysis to evaluate the history of fire temperature distribution within the columns, and a structural analysis to evaluate the structure's structural response, as depicted in Figure 1. Utilizing the SAFIR computer program, an analysis was conducted. Four sides of the models were subjected to parametric fire.

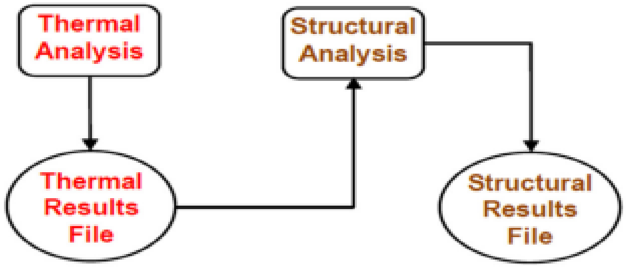

Figure 1. Analysis methodology

3. FINITE ELEMENT MODEL DEVELOPMENT

In this study, a 2D non-linear numerical analysis was performed to simulate the post-fire behavior of RC columns. Using the SAFIR computer program [46], the numerical calculations were performed. This program enables the nonlinear thermo. In this study, a 2D non-linear numerical analysis was performed to simulate the post-fire behavior of RC columns. This research's primary objective is to investigate the behavior of reinforced concrete columns under fire conditions and the factors that influence their performance. The study aims to gain insights into how these columns respond to high-temperature exposure and understand the influence of various factors, such as column design, material properties, fire duration, and applied loads. As mentioned in design of concrete structures [47], numerical models used for analyzing structural behavior during fires are considered advanced calculation methods. These models must possess the capability to calculate the temperature evolution within the structural members and assess their mechanical behavior under fire conditions. Heat is transferred in the material from one atom to another while the atoms remain in their places, i.e.,, thermal energy is transferred by conduction, Eq. (2).

$$\mathrm{H} = \lambda \times A \text{ contact} \times \frac{\Delta T}{L} \tag{2}$$

where,

λ: Thermal Conductivity Coefficient W/m°C;

ΔT: The difference between the temperatures of the two surfaces in contact with a contact area is A contact.

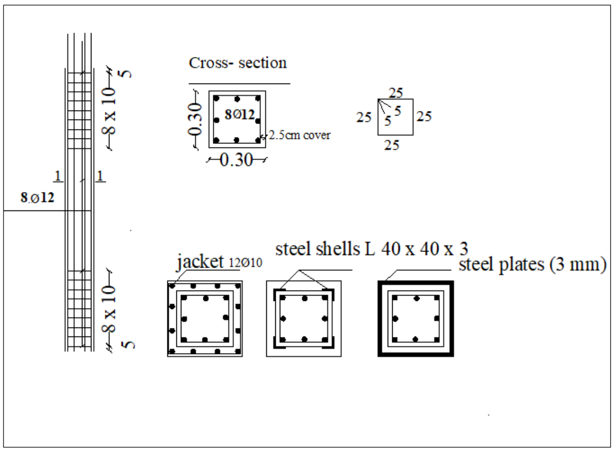

Figure 2. Geometry and reinforcement details

The numerical study focused on a fire scenario involving a square reinforced concrete (RC) column with a height of 3 meters and a cross-section measuring 0.3×0.3m^2. The longitudinal bars used had a diameter of 8Ø12mm, while the ties were distributed with a diameter of Ø 10mm with a (E-modulus Young 2.1e11; poisson ration 0.3; Yield strength 2.45e8 N/m^2.) The longitudinal reinforcement had a clear cover of 0.03 meters. Figure 2 depicts the cross-section of the RC column and geometry and reinforcement details. In the numerical study, the longitudinal reinforcement used in the RC column had a yield stress of 415 MPa, while the concrete compressive strength was 30 MPa. The structural analysis of the control specimens was conducted in a single step, which means that all the relevant calculations and simulations for the specimens were performed together as a comprehensive analysis, without intermediate stages or iterations. This approach allowed for an efficient and complete evaluation of the behavior of the control specimens under the specified conditions, considering both the concrete and reinforcement properties, and provided valuable data for further analysis and comparison with other scenarios, such as fire-exposed conditions.

To replicate the fire state of the test specimens, the column was fastened at the bottom and constrained at the top to prohibit movement along the boundary conditions X and Z axes (zero translation along the Z and X axes). However, the column was allowed to experience free movement along its longitudinal axis, represented by the Y-axis (free of mobility along the Y-axis of the column). The numerical model utilized a two-phase coupled temperature-displacement (Transient) analysis. In the first stage, the analysis was conducted to simulate the fire exposure. During this phase, the temperature distribution within the column was calculated over time to mimic the actual fire scenario. The heat transfer and thermal effects on the column's material properties were considered in this stage. In the second phase of the analysis, the structural response of the concrete columns was evaluated after they were exposed to flames. The boundary and loading conditions used for the analysis were the same as those applied to the control columns. This phase aimed to study the behavior of the columns under the post-fire conditions and assess how the fire exposure affected their mechanical properties and structural integrity. Additionally, the necessary concrete characteristics are calculated using the concrete damage plasticity model. This model is utilized to evaluate the behavior of concrete under various loading conditions, taking into account its nonlinear and time-dependent response, as well as the potential damage and degradation it may experience during loading and unloading cycles. By employing the concrete damage plasticity model, in the numerical analysis, the stress-strain values for both compression and tension must be provided as input for each material, i.e.,, concrete and steel. These stress-strain values are essential to accurately model the mechanical behavior of the materials under different loading conditions, including the effects of fire exposure. For the finite element program, mechanical and thermal properties of concrete and steel at elevated temperatures were measured and utilized as data inputs. The analysis selected a concrete with a density of 2,400kg/m^3; convection coeffe hot 35; and thermal conductivity 0.7. The study's column samples were exposed to 1000°C flames for 15, 30, 60 and 90 minutes show Figure 3. The initial boundary condition of the concrete was set to room temperature (20°C), serving as a heat sink. In the SAFIR program, this parameter is referred to as the "Surface Film Condition". The "Embedded Region" in the model was used to represent the interaction between the steel bars and concrete. At room temperature (20°C), a surface film coefficient of 0.3 was applied. In the analysis, the concrete material was represented using non-linear stress-strain relationships proposed in reference [47]. These relationships can be obtained from the following expression, as presented in Table 1. The mathematical model requires two parameters to define the behavior of concrete under elevated temperatures: the compressive strength (fc, θ) at a specific temperature and the strain corresponding to the peak stress (ε_{c1}, θ).

In order to ascertain the values of these variables at all temperatures, the reduction coefficients from Table 1 standard were utilized. The reduction factors account for the change in concrete properties with increasing temperature. By applying these factors to the compressive strength and peak strain at room temperature, the corresponding values at elevated temperatures were obtained.

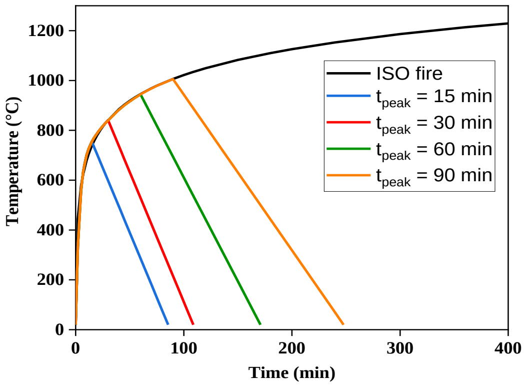

Figure 3. Parametric and standard temperature-time curves used

Table 1. The stress-strain relationships, Eurocode 2 [47]

Strain Range	Stress σ(θ)
$\varepsilon_{c,\theta} \leq \varepsilon_{c1,\theta}$	$\sigma_{c,\theta} = \dfrac{3.\,\varepsilon_{c,\theta} \cdot f_{c,\theta}}{\varepsilon_{c1,\theta}\left[2 + \left(\dfrac{\varepsilon_{c,\theta}}{\varepsilon_{c1,\theta}}\right)^3\right]}$
$\varepsilon_{c1,\theta} \leq \varepsilon_{c,\theta} \leq \varepsilon_{cu1,\theta}$	A descending branch should be used for numerical purposes. Models that are linear or non-linear are both acceptable

4. VALIDATION OF THE FINITE ELEMENT MODEL

This section compares the results predicted by the FE model with the experimental ones given by Izzat in order to validate the numerical model [48] and also with those resulting from the numerical model developed in ABAQUS by Mohammed and Said [49]. The program is based on the finite element method and permits non-linear thermo-mechanical analyses of concrete, steel, composite steel, and concrete structures exposed to fire. In this respect, Izzat investigated how self-compacting concrete (SCC) short columns with dimensions of with overall length of 700mm and cross-sectional area of 100×100mm were affected by high-temperature flames, using furnace manufactured for this purpose. The links were placed 3mm apart, at 100mm from the transverse reinforcement, while the diameter of the longitudinal bars ranged from 4 to 10mm. Figures 4 and 5 show the comparison of the final loads (load on failure) as well as the axial displacements of column model C1 (the reference column not exposed to fire) and column model C2 (columns burned with fire flames at 300℃).

Furthermore, a simple comparison of the findings of the experimental tests with the numerical ones allowed for the conclusion that the ultimate loads were incorrectly estimated. Percentage differences of about 5.77% for the reference column and 8.54% for the columns subjected to fire flames at 300℃ were found. This indicates that the suggested model is reliable and consistent and can therefore be used. This discrepancy between the FE model results and the experimental ones may be attributed to a number of factors. The most significant one is that concrete was dealt with on the basis of the FEM and was considered a homogeneous substance, while in reality it is quite heterogeneous.

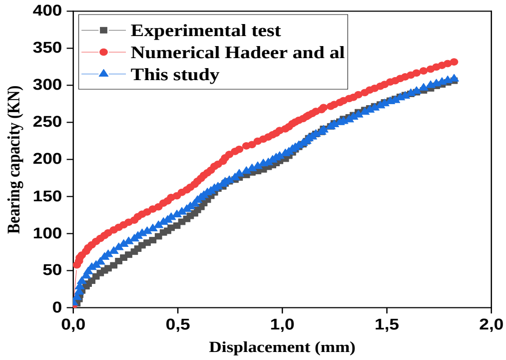

Figure 4. Without fire, load-vertical displacement, C1 (reference column) [47]

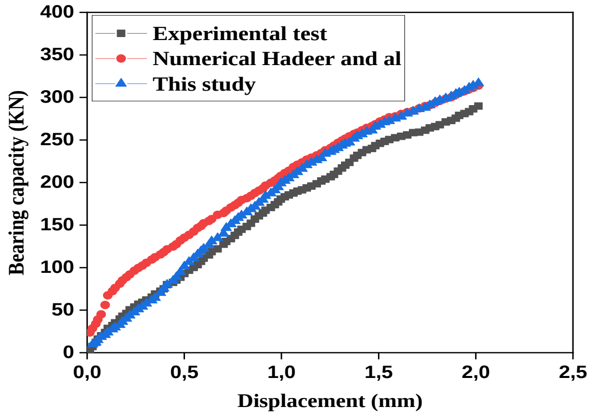

Figure 5. (Burned with a 300°C fire flame) load-vertical displacement C2 [47]

5. RESULTS AND DISCUSSION

5.1 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}=15min to t_{peak}=90min) on each of the column's four sides are compiled in Table 2. It is important to note in this regard that the cross-sectional area of (0.3×0.3) 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 2. Influence of the column's height, for different fire durations

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

5.2 Effect of the cross-sectional area

This time, the column height is kept constant at 3m, while the cross-sectional area takes the values (0.3×0.3), (0.4×0.4), and (0.6×0.6) m². Figure 6 depicts the distribution of maximum temperatures reached in the exposed portions of the column during the medium and long fires for the different cross-sectional areas under consideration. Table 3 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 Ncollapse of the cross-section (0.3×0.3) m² is reduced by 76.56% when this section is exposed to fire for 90min. However, this reduction is lower (32%) for the cross-section (0.6×0.6) m2 for the other cross-section sizes. This means that the effect of fire is lower when the cross-sectional area increases. Conversely, it is observed that the time of collapse (tcollapse) increases as the time of fire exposure goes up.

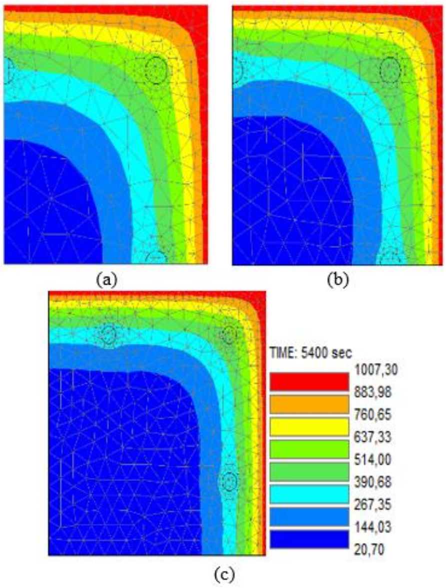

Figure 6. Distribution of the maximum expected temperatures after 90minutes in different cross-sections: (a) 0.3×0.3m^2, (b) 0.4×0.4m^2, (c) 0.6×0.6m^2 (A cross-sectional quarter with marked rebar positions is shown)

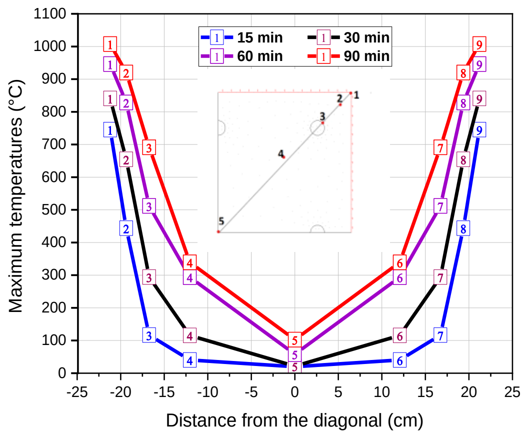

Figure 7. Temperature maximum at diagonal section (cross-section quarter with rebar placements)

Table 3. $N_{collapse}$ for different cross-sectional areas

Section (m^2)	N_{20}°C (kN)	t_{peak}=15min	t_{peak}=30min	t_{peak}=60min	t_{peak}=90min
		$N_{collapse}$ (kN)			
0.3×0.3	2445	1723	86	1425	69
0.4×0.4	4455	3640	615	3291	520
0.5×0.5	9808	8720	279	8221	350

Figure 7 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 (0.3×0.3)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.

5.3 Effect of fire duration

The impact of fire flame exposure duration on the ultimate load-bearing capacity and load-deflection reaction of charred columns was studied. Four duration periods were selected for this purpose: 15, 30, 60, and 90 minutes. A parametric analysis was performed, and the results obtained showed a significant change in the load-carrying capacity of all specimens. 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 29.53, 41.72, 61.96 and 76.56%, respectively, compared with the 3m high reference column (without fire), for the cross-sectional area of (0.3×0.3) m², as shown in Figure 9. As for the cross-section of area (0.4×0.4) m², the failure load dropped by about 18.29, 26.13, 39.68 and 51.72%, respectively. The minimum load reduction was recorded for the cross-section (0.6×0.6) m²; the reduction percentages were approximately 11.09, 16.18, 27.38 and 32.02%, 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 (0.3×0.3) 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 4.

Table 4. Results related to RC columns exposed to fire for various durations

Fire Duration (min)	Load Capacity P_u (KN)	Axial Deformation Δ_u (mm) at Ultimate Load	Load Capacity Reduction Due to Fire (%)
Ref-Column	2445	7.19	-
15	1723	8.83	29.53
30	1425	10.85	41.72
60	930	14.94	61.96
90	573	18.84	76.56

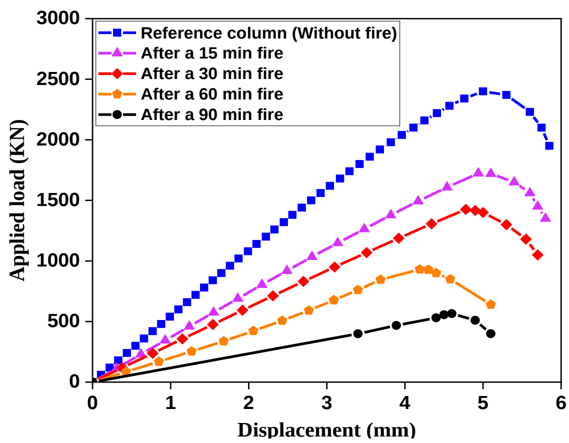

Figure 8. Effect of fire exposure time on the load-vertical displacement behavior of the column

Figure 8 represents the load-displacement relation of the column exposed to a fire. Figure 8 compares the model of the columns exposed to fire with the model of the reference column. It is observed that the load-bearing capacity diminishes as the duration of the fire increases.

Figure 9 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 Eq. (4). The function representing this variation is obtained by calculating the reduction in load-carrying capacity as a function time's t (min) as follows Eq. (3) [49].

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

where,

$P(\%)$ 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.

$$P(\%) = -0.0031t^2 + 0.9542t + 15.911 \qquad (4)$$

where, t time of exposure to fire in minutes.

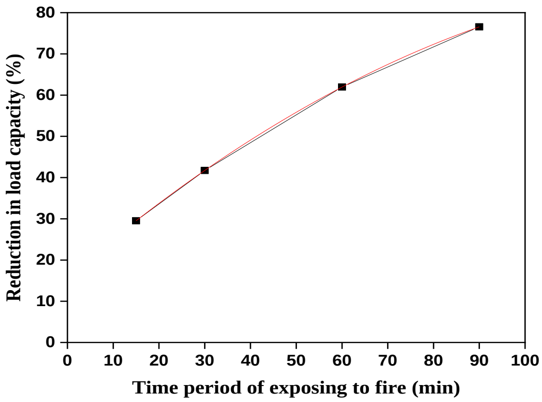

Figure 9. Reduction in load-bearing capacity of columns exposed to fire

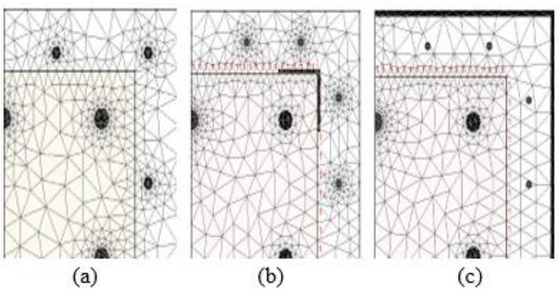

Figure 10. Details of the repaired RC column with different jacketing techniques

On the other side, the temperature distributions at various depths of a column with a cross-sectional area equal to (30×30) m^2 and a height of 3m, 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 the load-bearing capacity of the columns using the three rehabilitation techniques shown in Figure 10.

- Reinforced concrete with jacketing (a);
- Reinforced concrete with Steel shells and jackets (b);
- Reinforced concrete with steel plates and jackets (c).

Moreover, a parametric study was conducted 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, and 40 MPa, and the jacket thicknesses used were 50mm and 100mm. The strengthening efficiency can be calculated using the following Eq. (5).

$$P_{str}(\%) = \frac{P_{str} - P_U(T = 20°C)}{P_U(T = 20°C)} \times 100 \qquad (5)$$

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

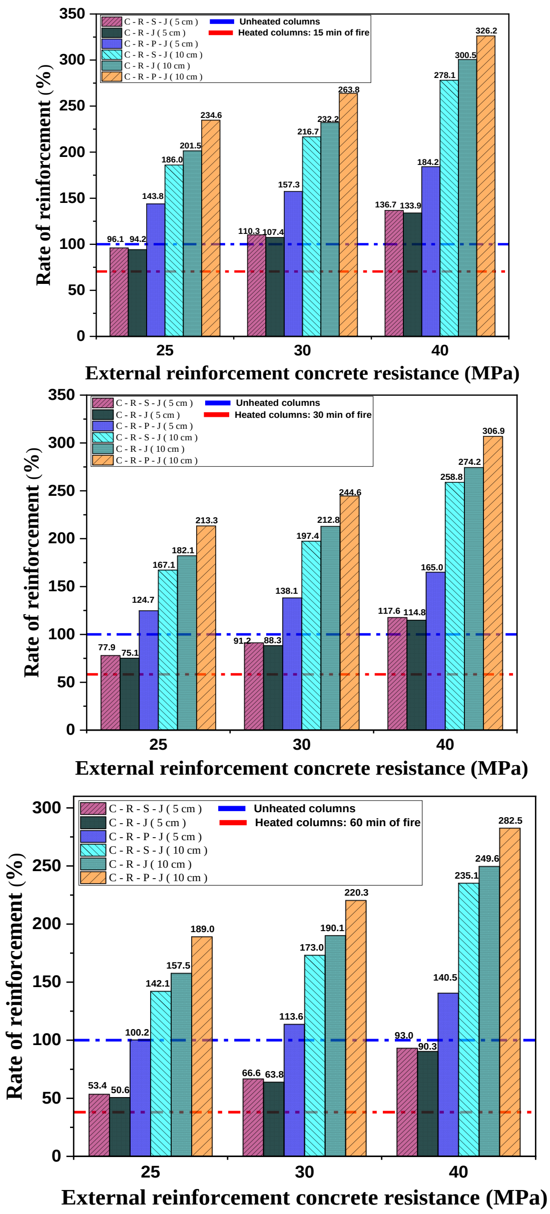

Figure 11. Repairing the efficacy of various post-fire column rehabilitation techniques after 15, 30, and 60 minutes of fire exposure

In order to better represent the data reported in Tables 5-7, it was decided to use Figure 11 which illustrate the comparison between the different strengthening techniques, according to the variation of external strengths (25, 30 and 40 MPa), for different fire durations. It was noticed that the load-bearing capacity of the column decreased progressively until collapse as the fire durations increased to 15 minutes, next to 30 minutes, then to 60 minutes and finally to 90 minutes. In addition, the strengthening rate using the steel plate composite jacket with a cover of 10cm as well as the steel shell composite jacket with a cover of 10cm was considered and recommended. However, the RC jacket with a cover of 5cm was not recommended. On the other side, it was noted that when the variation in the external strength was equal to 25, 30 and 40 MPa, the load-bearing capacity of the columns increased by 89.0%, 120.3% and 182.25%, for the fire duration of 60 minutes. It should also be noted that after 90 minutes, the column cannot be repaired because it has already lost 76.56% 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.

Table 5. Numerical results for technical external reinforcement using fc'=25 MPa

Fire Duration (min)	Strengthening Efficiency (%) of Columns for Concrete with 25 MPa					
	C-R-S-J*		C-R-J*		C-R-P-J*	
	5cm	10cm	5cm	10cm	5cm	10cm
15	25.68	115.50	23.68	131.00	73.33	162.17
30	19.63	108.34	16.81	123.85	66.42	155.05
60	15.33	104.01	12.52	119.47	62.13	150.67
90	13.74	101.31	10.89	117.79	60.50	149.08

***C-R-S-J**: column repaired with steel shells and jackets of different thicknesses (5cm, 10cm);
***C-R-J**: column repaired with jackets of different thicknesses (5cm, 10cm);
***C-R-P-J**: column repaired with steel plates and jackets of different thicknesses (5cm, 10cm).

Table 6. Numerical results for technical external reinforcement using fc'=30 MPa

Fire Duration (min)	Strengthening Efficiency (%) of Columns for Concrete with 30 MPa					
	C-R-S-J		C-R-J		C-R-P-J	
	5cm	10cm	5cm	10cm	5cm	10cm
15	39.80	146.18	36.97	161.68	86.83	193.37
30	32.90	139.10	30.06	154.52	79.84	186.34
60	28.59	134.72	25.77	152.02	75.54	182.25
90	26.91	133.05	24.17	148.47	100.86	180.20

The column repaired with steel plates and jackets (C-R-P-J) of various thicknesses (5cm, 10cm) of external reinforcement concrete resistance 40 MPa. Provided the best results (255.75%, 248.59%, and 242.62%) for the fire duration of 15, 30, and 60 minutes, respectively, compared to other methods of column repair with jackets (C-R-J) and column repair with steel shells and jackets (C-R-S-J).

Table 7. Numerical results for technical external reinforcement using fc'=40 MPa

Fire Duration (min)	Strengthening Efficiency (%) of Columns for Concrete with 40 MPa					
	C-R-S-J		C-R-J		C-R-P-J	
	5cm	10cm	5cm	10cm	5cm	10cm
15	66.26	207.61	63.44	230.07	113.74	255.75
30	59.35	200.53	56.52	215.95	106.75	248.59
60	54.97	197.07	52.23	211.57	102.45	244.21
90	53.37	194.48	50.63	209.90	100.86	242.62

These results demonstrate that the use of steel plates and jackets with varying thicknesses of external reinforcement concrete significantly enhanced the load-carrying capacity of the repaired columns under fire conditions. In comparison, other methods such as column repair with jackets (C-R-J) and column repair with steel shells and jackets (C-R-S-J) did not yield as substantial improvements in load-carrying capacity.

The success of the C-R-P-J method highlights its effectiveness in enhancing the fire resistance and overall performance of the repaired columns. The use of steel plates and jackets, in combination with external reinforcement concrete, contributed to the improved load-carrying capacity and structural integrity of the repaired columns, making this method a preferred choice for column repair in fire-exposed scenarios. The reason why C-R-P-J repair technique performs better than another is because of its effectiveness confinement on concrete columns.

6. PROPOSED SIMPLIFIED EQUATIONS

Based on the findings of this study, it was concluded that the best strengthening technique is the steel plate jacketing technique. The MATLAB software has been used for curve fitting method, where Curve Fitting is Polynomial, the Model: Polynomial functions and Error Metric: R-squared to form a relationship between the cover thickness C (cm), and the concrete compressive strength fc (KN/cm2) and column length L (cm), load-bearing capacity residual after fire Pres (KN) with the corresponding load-bearing capacity after strengthening PStr (KN) of the proposed SAFIR FEM. In case of load-bearing capacity after strengthening PStr their properties depend on C, fc, L and Pres.

A total of 48 data sets were generated using SAFIR FEM by varying the C, fc, L and P_{res}. After applying these values, created a simplified equation for calculated new load-bearing capacity after strengthening P_{Str} (KN) as a function of the residual axial load-bearing capacity P_{res}, C, fc and L. shows this equation in Eq. (6).

$$P_{str} = \frac{18.8}{30} \times (C\, f_c\, L) + P_{res} \tag{6}$$

In addition, Figure 12 shows the efficacy of various post-fire column rehabilitation techniques after 15, 30, and 60 minutes of fire exposure and a comparison of rehabilitation techniques. That the results given by Eq. (5) and the numerical ones resulting from the SAFIR software are close to each other. It was indeed found that the maximum-recorded error between them was 4.88%, which does not exceed 5% and the R_squared between them was 0,986.

The R_squared value can be calculated using the following Eq. (7).

$$R\_squared = 1 - (SSR/TSS) \tag{7}$$

where, Calculate the total sum of squares (TSS) and the sum of squared residuals (SSR) using the original data and the fitted values.

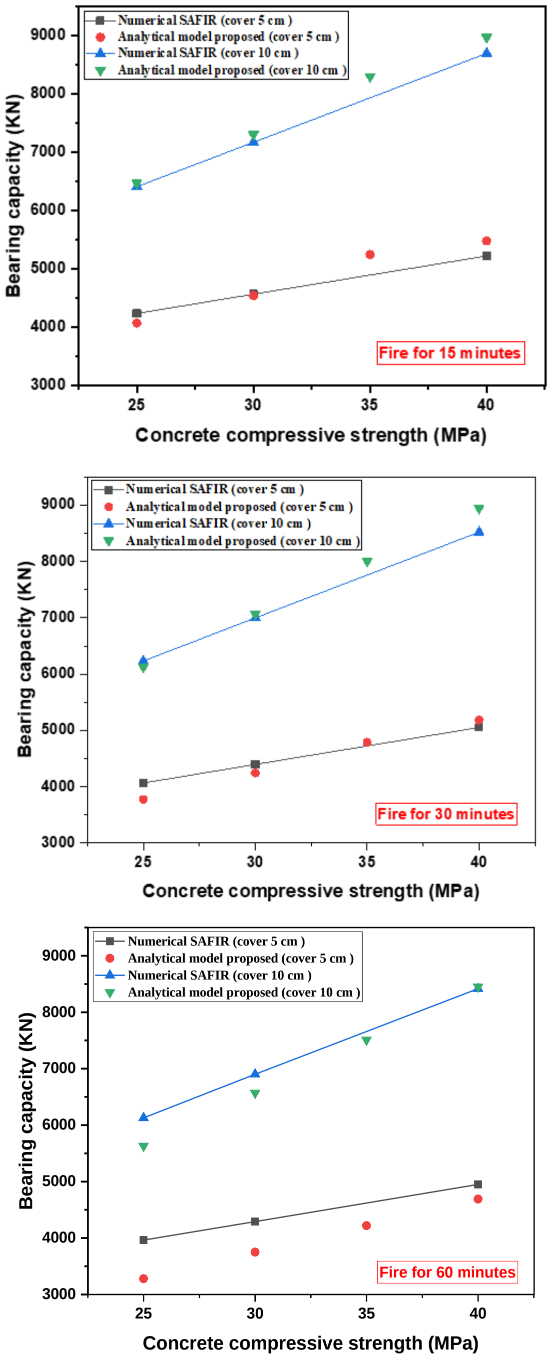

Figure 12. Validation of the proposed equation using model findings for a fire duration of 15, 30 and 60 minutes

7. CONCLUSIONS

This study aimed first to investigate the effect of different elevated temperature environments on the residual load-carrying capacity of structural columns while considering some decisive geometrical parameters such as the height and the cross-sectional area of the column. 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 [46], was used for the analysis of the post-fire behaviour of columns. Three technical methods, using RC jackets as well as steel shells and steel plate composite jackets, were utilized to repair and strengthen the fire-damaged columns while considering different jacket thicknesses and various strengths of concrete. This study underlines that post-fire restoration design requires numerical tools with proper models. These technologies will enable engineers and researchers to efficiently restore fire-damaged structures for future work. Based on the above, a number of conclusions could be drawn:

(1) The load-bearing capacity of the column decreased after its exposure to a fire temperature of 1000°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 29.53%, 41.72%, 61.90% and 76.56% 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 and 40 MPa, the load-bearing capacity of the column also grew, respectively, by 89.0%, 120.3% and 182.25%, 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) In addition, a simplified equation is proposed to express the new load-bearing capacity P_{str} (KN) as a function of the residual axial load-bearing capacity P_{res} (KN), cover thickness C (cm), concrete compressive strength fc (KN/cm^2) and column length L (cm) once the column has been strengthened. The results obtained with the proposed equation turned out to be very close to those obtained with the SAFIR software.

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

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NOMENCLATURE

t_{peak}	Time of exposure to fire, min
$N_{20°C}$	Load capacity in room temperature t=20℃, KN
$N_{collapse}$	Load capacity in t C, KN
Pu	Load capacity during collapse, KN
P (%)	Reduction in load capacity, (%)
Pres	Load-bearing capacity residual after fire, KN
Pstr	Load-bearing capacity after strengthening, KN
Δu	Axial deformation at ultimate load, mm
f_c	Concrete compressive strength, KN/cm^2
L	Column length, cm
C	Cover thickness, cm
Subscripts	
RC	Reinforced concrete
FRP	Fiber-Reinforced Polymer
FEA	Finite Element Analysis
C-R-S-J	Column Repaired with Steel shells and Jackets
C-R-J	Column Repaired with Jackets
C-R-P-J	Column Repaired with steel Plates and Jackets
SSR	Sum of Squared Residuals
TSS	Total Sum of Squares