Research Article

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Residual strength and strengthening capacity of reinforced concrete columns subjected to fire exposure by numerical analysis

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Abstract: This study is a numerical investigation of the performance of reinforced concrete (RC) columns after fire exposure. This study aims to investigate the effect of introducing lateral ties and using the RC jacket on improving post-fire behavior of these columns, the effect of the duration of the fire on ultimate load of columns. The analysis was performed through ABAQUS, a 3D – non-linear finite element program. 4 m tall lengthening square RC column with a cross- section of 0.4 m × 0.4 m was used as a test specimen. The RC column was reinforced by 4Ø28 mm longitudinal bars bonded by steel tie bars of Ø10 mm spaced at 400 mm. The firing temperature was increased to 600°C for 60 min. The results indicated that as the fire load duration increases, the load's capacity diminishes. The effect of increasing the steel ratio of the RC column resulted in a decrease in the percentage of load reduction following combustion. It was concluded that strengthening columns by RC jackets is efficient and that the axial load capacity of reinforced columns was enhanced.

Keywords: fire period, reinforced concrete, post-fire strength, residual capacity, strengthening, ABAQUS, numerical analysis

1 Introduction

A column is any member that has a ratio of length to a minimum lateral dimensions equal to or greater than three and used primarily to support axial compressive load [1]. Tall building designers have focused on making buildings that are safer and have more usable floor space. Because of the limited strength and ductility of concrete, columns in tall buildings can be damaged by overloading and natural calamities such as earthquakes and fires [2,3]. There have been several experimental investigations on the impact of fire on RC columns [4,5]; however, there have been few works on numerical modeling [6]. As a result, numerical studies are required to examine the influence of fire duration on RC columns.

The purpose of structural fire engineering is to assess how well buildings and other structures will behave in the case of an unintentional fire. Engineers use numerical analysis extensively in their work. The numerical analysis of a structure exposed to fire is usually divided into three parts: first, the evolution of the gas temperature in the compartment; second, the thermal analysis to determine the temperature distribution in the structural member sections; and third, the mechanical analysis to assess the behavior of the structure exposed to high temperatures. Concrete has an excellent intrinsic behavior when exposed to fire. It does not burn, making it non-combustible, and it has a high thermal massivity, which slows heat transport through concrete pieces significantly. In truth, most typical fires only cause damage to the exterior layer of concrete, which is around 30–50 mm thick [7]. The installation of suitable fire safety measures for structural members is a major safety need in building design because fire is one of the most severe environmental situations to which structures may be exposed. The justification for this requirement can be traced to the fact that structural integrity is the last line of defense for building occupants and emergency workers when other fire-fighting measures fail [8]. The material properties of concrete and reinforcing steel vary as the temperature rises when concrete columns are exposed to fire. The overall strength of the column is reduced as yield strength and elasticity modulus decrease. The column will fail by crushing or flexural buckling whenever its strength falls below that of the imposed load. A column

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is placed in a furnace and subjected to a controlled fire while being loaded with a predetermined force in structural fire performance testing. The fire resistance rating of a column is the time it takes from the start of fire exposure to failure. Thermal gradients are form and occur in the concrete cross-section when it is subjected to fire [9]. Moisture in concrete turns to vapor as temperature rise, causing pore pressure due to volumetric expansion. The spalling phenomena happens when these stresses, induced by mechanical and hydraulic causes, exceed the tensile strength of concrete. Because the porosity of the cement paste expanded rapidly to develop microcracks [10] due to the drying of various cement paste compounds, most researchers believed that the damages in the concrete mixture began at a temperature above 300°C. Beyond 500°C, the release of water from the dissociation of $Ca(OH)_2$ in the range of 450–550°C and the release of CO_2 from the decomposition of $CaCO_3$ above 600°C causes a further increase in porosity [10]. Robston [11] discovered that up to 100°C, the thermal expansion of steel is essentially identical to that of regular concrete, and that above this temperature, the expansion of the steel will increase, while the concrete would shrink owing to cement drying, resulting in loosening. Cracks will emerge and grow as the link between concrete and steel weakens. Harada *et al.* [12] discovered that the residual bond strength between the concrete and the reinforcement was 44% of the control specimens at 300°C and rapidly declined to 10% at 450°C, while the residual compressive strength was 60% at the same temperature.

Although severe fires can cause significant damage to the reinforced concrete (RC) structures, the collapse of RC structures rarely occurs as a result of fire damage. As a result, if a fire breaks out during construction, structural engineers must examine the remaining strengths of younger RC structural components to determine the building's integrity and post-fire repairability [13]. AL-taai *et al.* [14] used ANSYS software to simulate the effects of fire on RC columns. He studied the influence of fiber reinforced polymer (FRP) on RC column load capacity. Using carbon fiber reinforced polymer (CFRP) improves ultimate strength range. For the fire simulation, Franssen *et al.* [15] used SAFIR non-linear finite element software. He used thermal parameters, such as the fire's position, to study the structural behavior. He demonstrated the software's ability to model and simulate structural parts under fire conditions.

The purpose of this research is to establish how fire affects the behavior of axially loaded RC columns that have been exposed to fire, as well as to calculate the percentage loss of column compressive strength when the following factors are considered:

1. Percentage ratio of the steel reinforcement.
2. The effect of burning on the load-bearing capability of a column with varying tie spacing.
3. Fire duration.
4. Strengthening of RC column after being exposed to fire by RC jacket.

2 Research significance

The primary goal of fire safety design is to protect people and property from the dangers posed by fires. If a fire spreads and becomes impossible to control using the building's facilities, the structure should be designed to remain stable for a certain period of time throughout the fire's duration. It should be possible to safely evacuate the building's occupants and put out the fire within this timeframe.

Most of the structural studies give the columns special importance compared to the rest of the structural elements because of the danger they pose, as the collapse of one of them can collapse an entire structure. The exposure of the column to a fire may be one of the reasons leading to that collapse, hence the importance of searching for developed fundamental information about the behavior of RC columns after and during the exposure periods to fire flame and predicting the damage by fire.

3 Work methodology

In order to achieve the abovementioned objectives, a numerical program was carried out using concrete column specimens of 4,000 mm length and a square cross-section of (400 mm × 400 mm), and confined by ties at different spacings of 100, 200 and 400 mm and no ties thereto, with depth of concrete cover of 40 mm. The tests also included axial loading, column specimens were exposed to burning by fire flame at a temperature level of 600°C for different exposure periods of 15, 30, 60, 90 and 120 min).

This research work is to develop a finite element procedure to analyze the RC column specimens under fire flame and strengthened by RC jacket to obtain the history of temperature distribution and structural behavior of column specimens utilizing ABAQUS/Standard 2017 computer program.

4 Finite element model (FEA)

The post-fire behavior of RC columns was simulated using a three-dimensional non-linear numerical analysis in this work. The numerical simulations were carried out with the ABAQUS/Standard 2017 computer program. The program is based on the finite element method and allows the non-linear thermo-mechanical analysis of concrete, steel and composite steel and concrete structures subjected to fire. The purpose of this work is to conduct research on the behavior of reinforced-concrete columns in fire conditions and the factors that influence column performance, as well as to provide a tool for further research and education in the field of reinforced-concrete column analysis. As stated in [16], the numerical models are classified as advanced calculation methods and they must allow the calculation of the temperature evolution in the structural members and also the evaluation of their mechanical behavior due to the fire.

For the numerical study dealing with fire scenario, the test specimen was square RC column of 4,000 mm tall and cross-section of 400 mm × 400 mm. The longitudinal bars' diameter was 4Ø28 mm and the ties were distributed as Ø10 mm @400 mm. The longitudinal reinforcement had a clear cover of 40 mm. Figure 1 shows the cross-section of RC column. The yield stress of the reinforcement was 415 MPa and the concrete compressive strength was 30 MPa. The structural analysis was completed in one step for the control (non-exposed) specimens (static general analysis step). To simulate the test specimens in the fire state, the column was pinned at the bottom end and restrained at the top end (zero displacement in the X and Z axes), with free movement in the direction of the column's longitudinal axis (Y-axis). Two phases of coupled temperature-displacement (Transient) analysis are used in the numerical model provided here. The first stage was utilized to simulate a fire. The second phase was to assess the structural response of concrete columns after being exposed to flames, with the same boundary and loading conditions used in the control columns. Furthermore, the requisite concrete characteristics are computed using the concrete damage plasticity (CDP) model. Figure 2 displays the plastic stress–strain relationships of CDP model for 30 MPa compressive strength. In CDP, the stress–strain values of both compression and tension for each material (concrete and steel) has to be input [16,17]. The damage parameters for the damaged plasticity constitutive concrete model are defined in Table 1 for characterizing the non-linear behavior of the normal strength concrete material. This amplitude is used to input the time-temperature curve of the entire fire development process so as to simulate a real fire exposed in the test and cooling of the columns before the start of the second stage, Figure 3 [18]. The specimens of the columns were tested under equal static incremental loads ranging from zero to failure. Because the numerical program's outputs are large amounts of data, a computer program is needed to organize the data and analyze the results. The specimens in each column were separated into finite element models. The exposed RC columns were modeled with three-dimensional solid elements in the concrete with an eight-node thermally connected brick, trilinear displacement and temperature (C3D8T) and a three-dimensional thermally coupled two-node truss element for reinforcement (T3D2T). All models are subjected to the same displacement analysis (20 mm). The mechanical and thermal properties, at elevated temperature of concrete and steel, are measured [16,17] as a source of data for a finite element program. Concrete, with a density of 2,400 kg/m^3, was chosen. In the analysis, column samples that exposed to fire from all sides

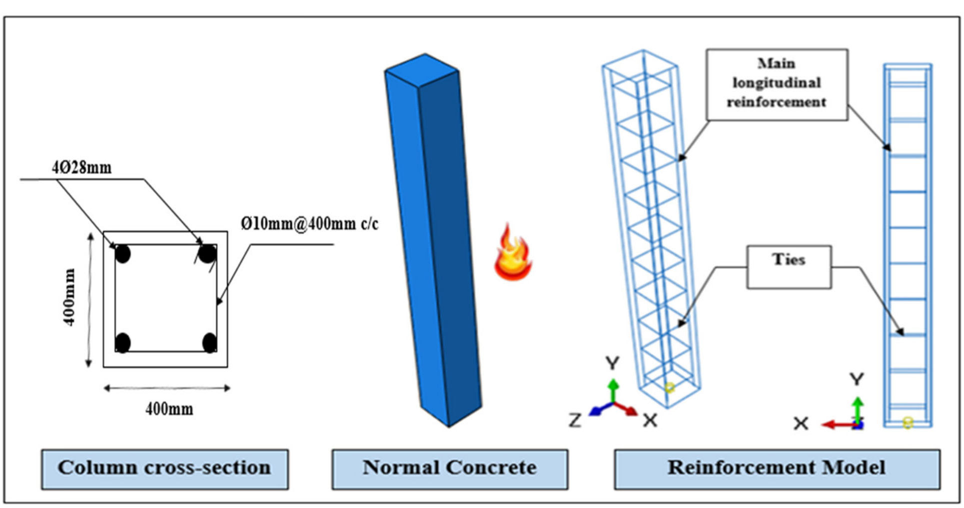

Figure 1: Details of dimensions and reinforcement of concrete column specimen.

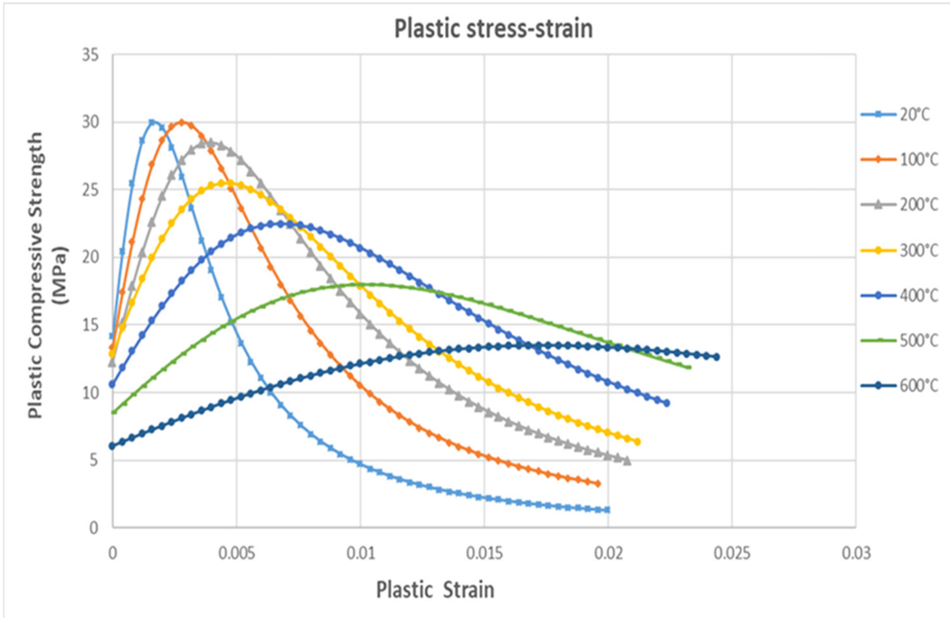

Figure 2: CDP stress–strain curves of the concrete at different temperatures [16].

Table 1: Damage parameters for concrete with a normal strength [19]

Dilation angle	Eccentricity	f_{bo}/f_{co}	K	Viscosity parameter
30°	0.1	1.16	0.667	0.001

to a temperature level of 600°C for a period of 1 h were used. The concrete's initial boundary condition is room temperature (20°C), which is referred to as Heat Sink. In the ABAQUS program, this parameter is called Surface Film Condition. The conductivity of the concrete was selected to be 1.951 W/(m °C) (at 20°C) and 0.915 W/(m °C) (at 600°C). Furthermore, the concrete specific heat is estimated to be 1,100 J/(Kg °C) at 600°C. The Embedded Region, it should be emphasized, is used to model the interaction between steel bars and concrete. At room temperature (20°C), the surface film coefficient of 0.3 is used. Furthermore, the maximum permitted

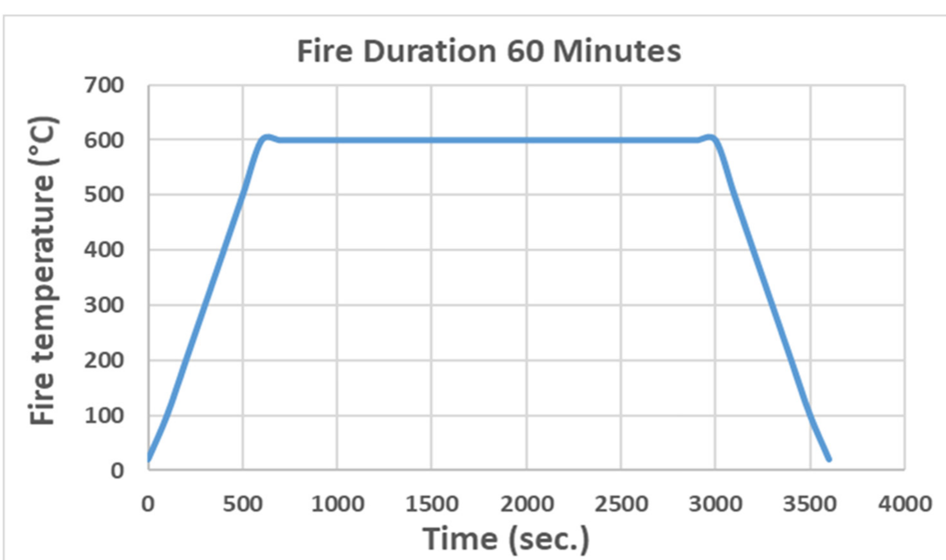

Figure 3: Curve of temperature-time for exposed columns at 600°C and throughout the course of a 1 h fire [18].

Table 2: Stress–strain expression [16]

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

temperature change every increment is set at 20°C. The concrete was modeled using the non-linear stress–strain relationships proposed in [16] which can be obtained from the following expression as shown in Table 2.

Two parameters are used to define this mathematical model: the compressive strength ($f_{c,\theta}$) for a given temperature, and the strain corresponding to the peak stress ($\varepsilon_{c1,\theta}$). The reduction factors in Table 3 of EC2 are used to obtained the values of these parameters at each temperature.

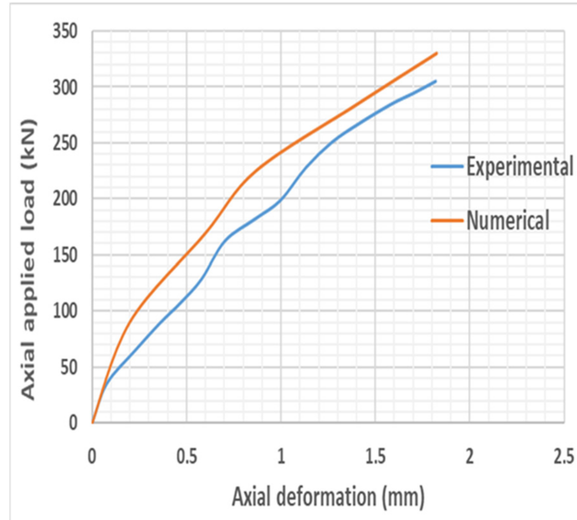

Figure 4: Load-vertical displacement without fire C1 (reference column) [20].

4.1 Verification of FE model

The numerical model is validated in this part by comparing the results of the FE model with Izzat's experimental results for the RC columns [20]. Izzat investigated the effect of fire flame (high-temperature) on specimens of short columns manufactured using self-compacted concrete (SCC) with the dimension of 100 mm × 100 mm × 700 mm. The longitudinal bars' diameter was 4Ø10 mm and the ties were distributed as Ø3 mm @100 mm. Two models [20] were numerically studied, C1 (reference column) and C2 (burned with a fire flame at 300°C) and comparison was made between the ultimate loads (load at failure) and axial displacement as shown in Figures 4 and 5. Comparing the experimental test results by Izzat with the numerical outcomes revealed an overestimation to the ultimate loads by 8.2% approximately for the reference column and by 9.5% for columns subjected to fire flame at 300°C, demonstrates that the proposed model is consistent and may be used with confidence. These differences between experimental results and FE results are attributed to many reasons, the most important of which is that the concrete was considered to be a homogenous material by the FEM; however, it is really a heterogeneous material and the modeling of bonds between steel bars and concrete. In ABAQUS, the concrete-steel interaction was modeled using an embedded region constraint that a perfect bond. Since

Table 3: The numerical results of analysis of RC column with different reinforcement ratios

Fire temp. (°C)	Steel ratio (ρ)	Load carrying capacity Pu (kN) FEM	Axial deformation Δu (mm) at ultimate load	Deformation due to thermal expansion ΔExpansion (mm)	Reduction in load capacity due to fire %
20	1%	5434.6	11.61	—	Ref.
	1.5%	5438.9	11.36	—	Ref.
	2%	5515.1	10.11	—	Ref.
	3%	6110.6	9.86	—	Ref.
	4.5%	7227.3	9.89	—	Ref.
600	1%	2886.4	15.98	+15.12	46.9
	1.5%	2917.3	14.97	+12.16	46.4
	2%	3059.2	14.49	+9.53	44.5
	3%	3480.1	12.72	+6.62	43.0
	4.5%	4130.1	10.06	+4.43	42.8

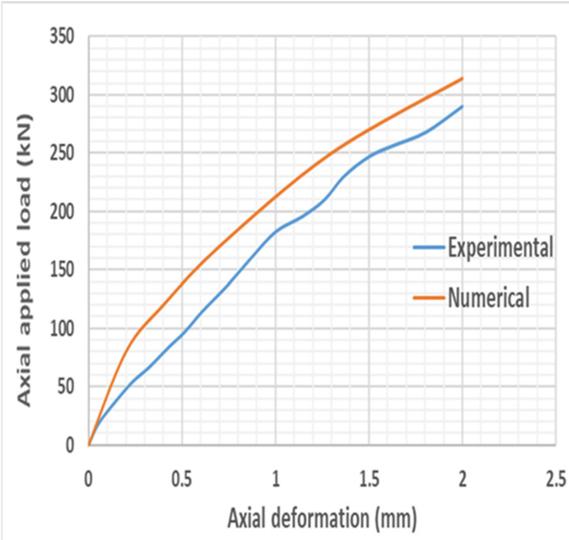

Figure 5: Load-vertical displacement C2 (burned with a fire flame of 300°C) [20].

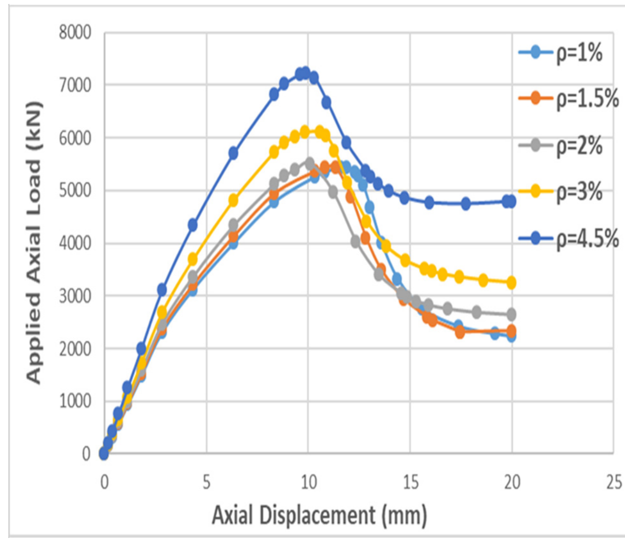

Figure 7: Numerical load-axial deformation of RC column with different reinforcement ratios at 20°C.

the actual bond is not perfect in reality, this idealization may also contribute to the spurious initial higher stiffness in the numerical model [21].

5 Results and discussion

5.1 Percentage ratio of steel reinforcement

One of the most important factors affecting all structural members is the ratio of reinforcement in terms of load carrying capacity, such as in the columns. Different percentages of steel reinforcement ratio ρ (1, 1.5, 2, 3 and 4.5%) are studied, which were exposed to 60 min fire. Table 3 includes the results of numerical analysis of the models referred above. The column failure load grew as the percentage reinforcement increased, according to numerical analysis. The effect of the fire on the columns by increasing the percentage of reinforcement ratio is imperceptible, according to the findings, but in general the reduction in the load capacity decrease with the increase in the reinforcement ratio as shown in Figure 6. Figures 7 and 8 show the load-deflection response of the

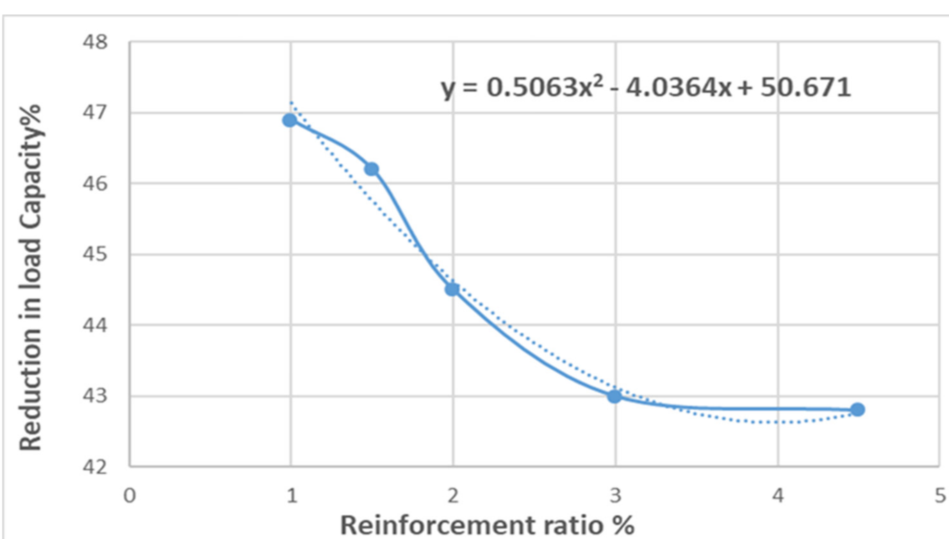

Figure 6: Reduction in load capacity–reinforcement ratio relationship.

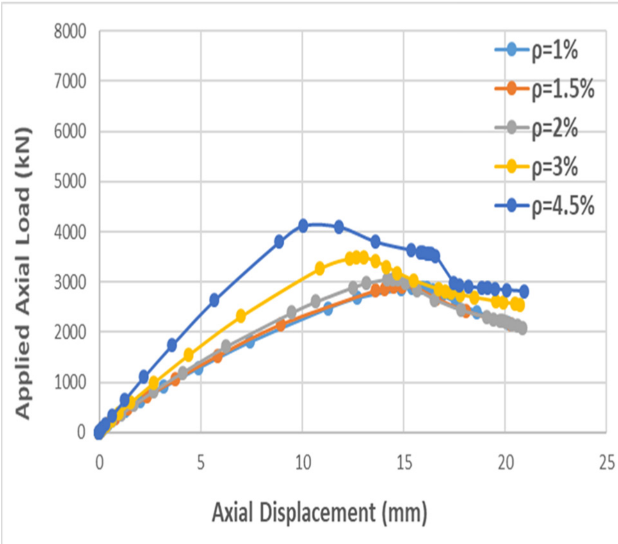

Figure 8: Numerical load-axial deformation of RC column with different reinforcement ratios after heating at 600°C.

specimen at 20 and 600°C with different reinforcement ratios of the FE analysis. The ratio 0.045 has more stiffness than other ratios.

5.2 The effect of burning on bearing ability of a column with varying tie spacings

The spacing and number of ties have an effect on the column sample's load bearing capacity, with lowering lateral tie spacing improving load bearing capacity. The core principle of the ACI-318/08 building code for lateral confinement is that the improvement in a column's load bearing capacity owing to lateral steel reinforcement should counteract or compensate for the loss of strength due to spalling of the concrete column cover. At burning temperature (600°C) and for duration of 1 h, the results reveal that reducing tie spacing from 400 to 200 mm increased the load carrying capacity by 6.4% for concentric loaded column specimen at 600°C. On the other hand, reducing the tie spacing from 400 to 200 mm increased the load carrying capacity by 4.5% for concentric loaded column specimen at 20°C, as shown in Table 4. The concentric loaded column sample's loading capacity improves almost marginally with the spacing and number of ties, according to these results. The load and displacement relationship of the fire-exposed column is shown in Figures 9–12.

Table 4: The numerical results of analysis of RC column with different spacings of ties

Fire temp. (°C)	Transverse reinforcement (ties)	Load carrying capacity Pu (kN) FEM	Axial deformation Δ*u* (mm) at ultimate load	Deformation due to thermal expansion ΔExpansion (mm)	Reduction in load capacity due to fire %
20	**Without**	5293.0	10.86	–	Ref.
	Ø10 mm @400 mm	5438.9	11.36	–	Ref.
	Ø10 mm @2 mm 00 mm	5685.4	12.11	–	Ref.
	Ø10 mm @100 mm	5933.5	12.86	–	Ref.
600	**Without**	2750.0	14.80	+11.26	48.0
	Ø10 mm @400 mm	2917.3	14.97	+12.16	46.4
	Ø10 mm @200 mm	3103.6	14.94	+13.40	45.4
	Ø10 mm @100 mm	3257.5	13.00	+15.15	45.1

Table 5: Numerical results of exposed RC columns at 600°C and various fire exposure durations

Fire duration (min)	Load carrying capacity Pu (kN) FEM	Axial deformation Δ*u* (mm) at ultimate load	Deformation due to thermal expansion ΔExpansion (mm)	Reduction in load capacity due to fire %
Without fire (reference column) ρ = 1.5%	5438.9	11.36	–	Ref.
15	3559.0	12.88	+6.73	34.5
30	3170.0	13.66	+8.49	41.7
60	2917.3	14.97	+12.16	46.3
90	2773.4	17.84	+15.28	49.0
120	2642.8	20.71	+18.98	51.4

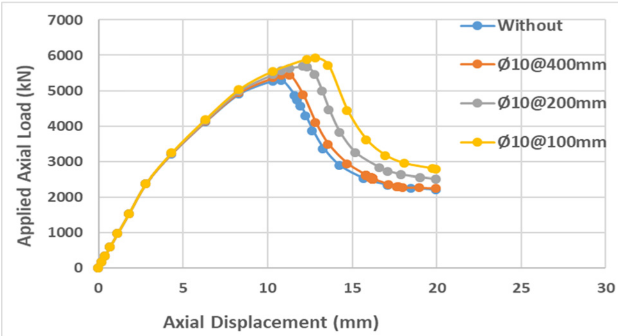

Figure 9: Numerical load-vertical displacement of RC column with different spacings of ties at 20°C.

5.3 Fire duration

The effect of time duration of exposing fire flame on ultimate load capacity and load-deflection response was studied for burned columns. Five duration periods were chosen (15, 30, 60, 90 and 120 min. The results of the parametric analysis indicate that there is a significant change related to the variation in the duration of exposure to fire on the load carrying capacity of all specimens at the same temperature level of 600°C. FEA revealed that with increase in the period of fire (15, 30 and 60 min.), the column failure load compared with column without fire (reference column) decreased by nearly 34.56, 41.716 and 46.36%, respectively, and the numerical analysis showed that as the period of fire increases, the column vertical displacement increased as shown in Table 5. Figure 13 depicts the load capacity decrease obtained from FEA. The models with fire loads are compared with model reference column in this figure. As shown, the longer the time of fire, the lower the load capacity becomes. In this approach, the load capacity of a column exposed to a 120 min fire load is reduced further (about 51.41%). The

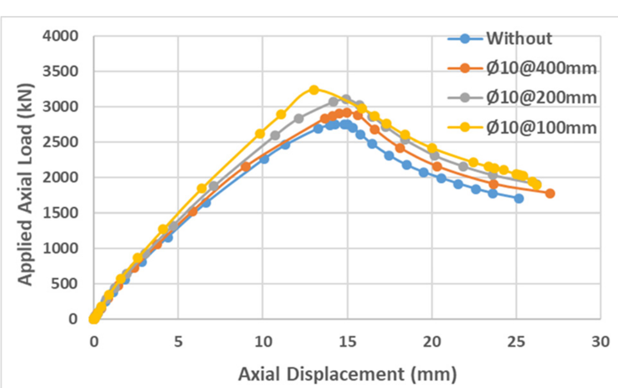

Figure 10: Numerical load-vertical displacement of RC column with different spacings of ties at 600°C.

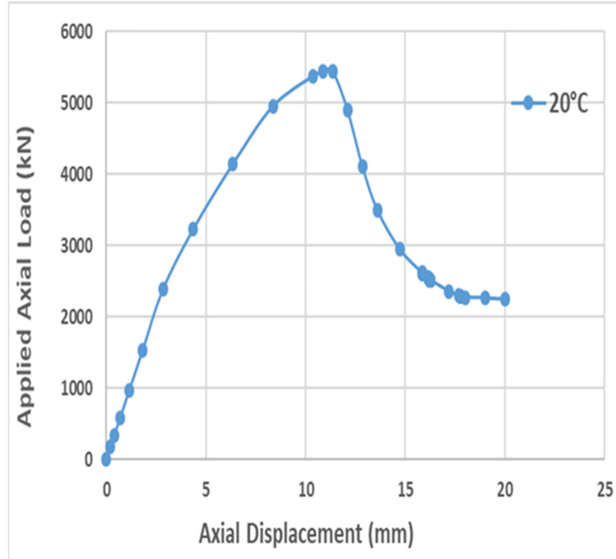

Figure 11: Load-vertical displacement without fire (reference column).

plastic equivalent strain (PEEQ) is depicted in Figure 14. As can be observed, as the time period of fire load rises, so does the distribution of plastic strain. Figure 15 shows the analytical temperature profiles of typical column cross-sections after 120 min of exposure to fire on four sides, and it is evident that as the exposure duration increases, the temperature rises towards the center of the RC columns. Furthermore, as seen in Figure 15, the temperature of the column cross-section changes at various depths, which is owing to the concrete's limited heat conductivity.

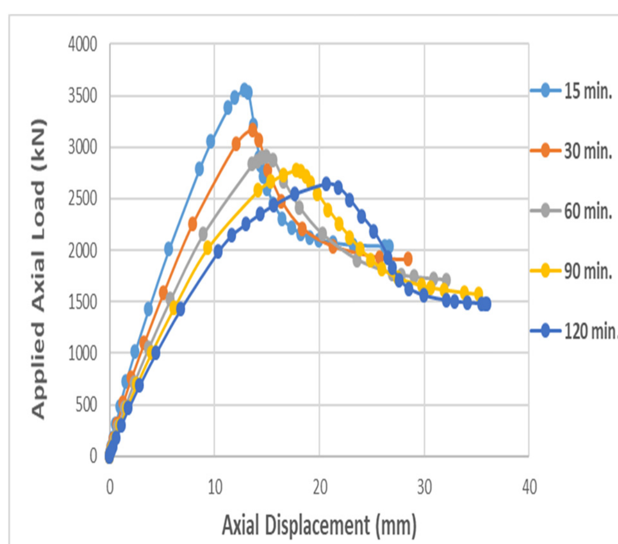

Figure 12: Effect of duration of exposing fire on load-vertical displacement behavior for column.

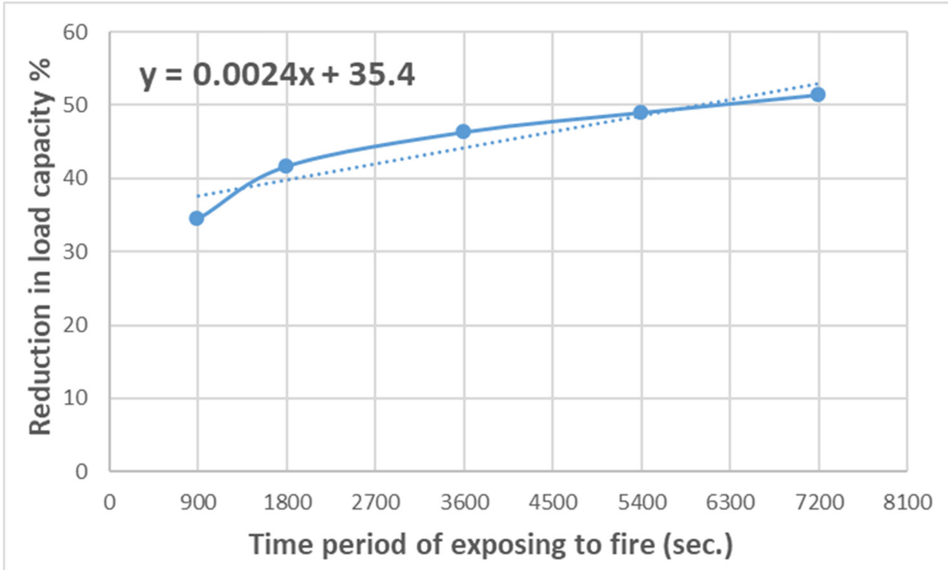

Figure 13: Reduction in load capacity due to fire.

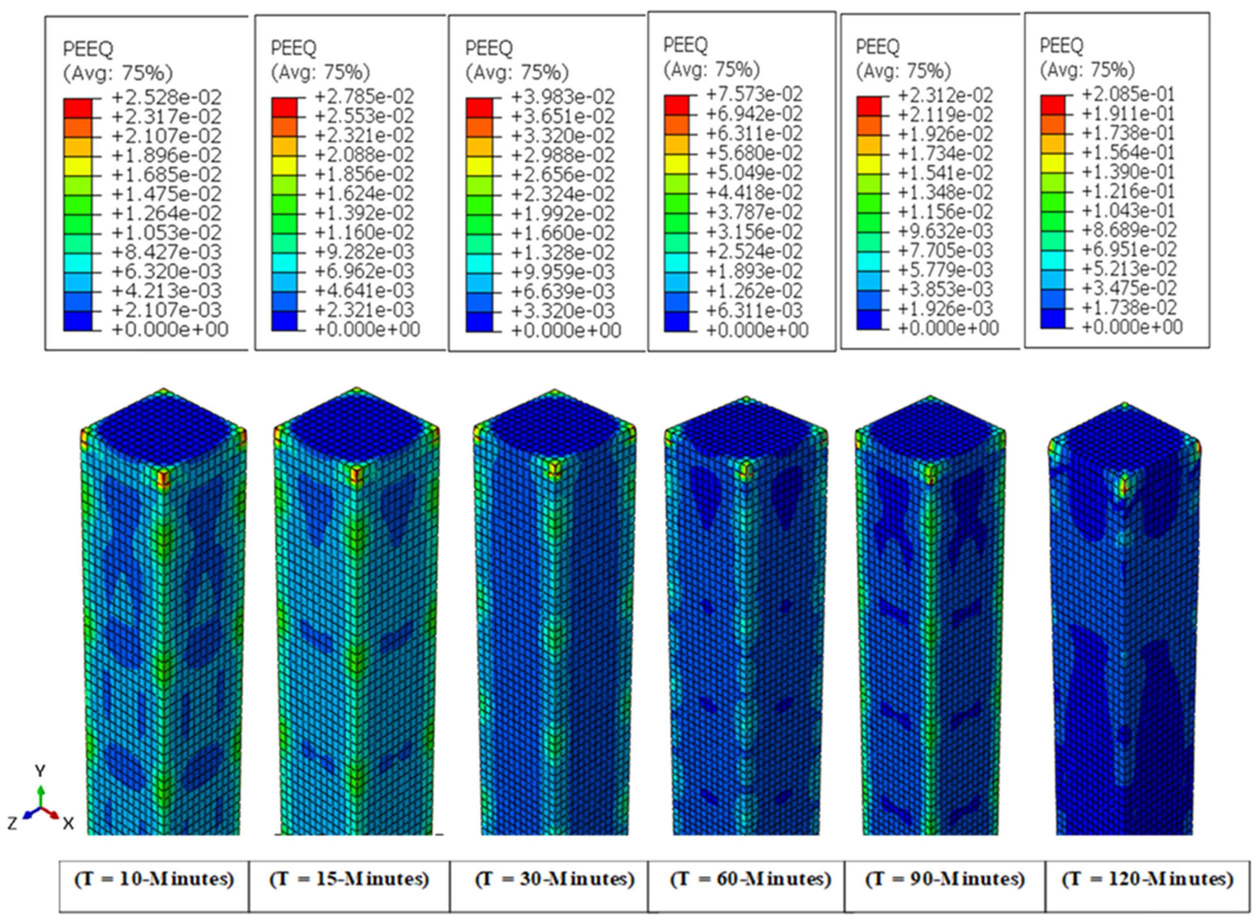

Figure 14: Equivalent plastic strain.

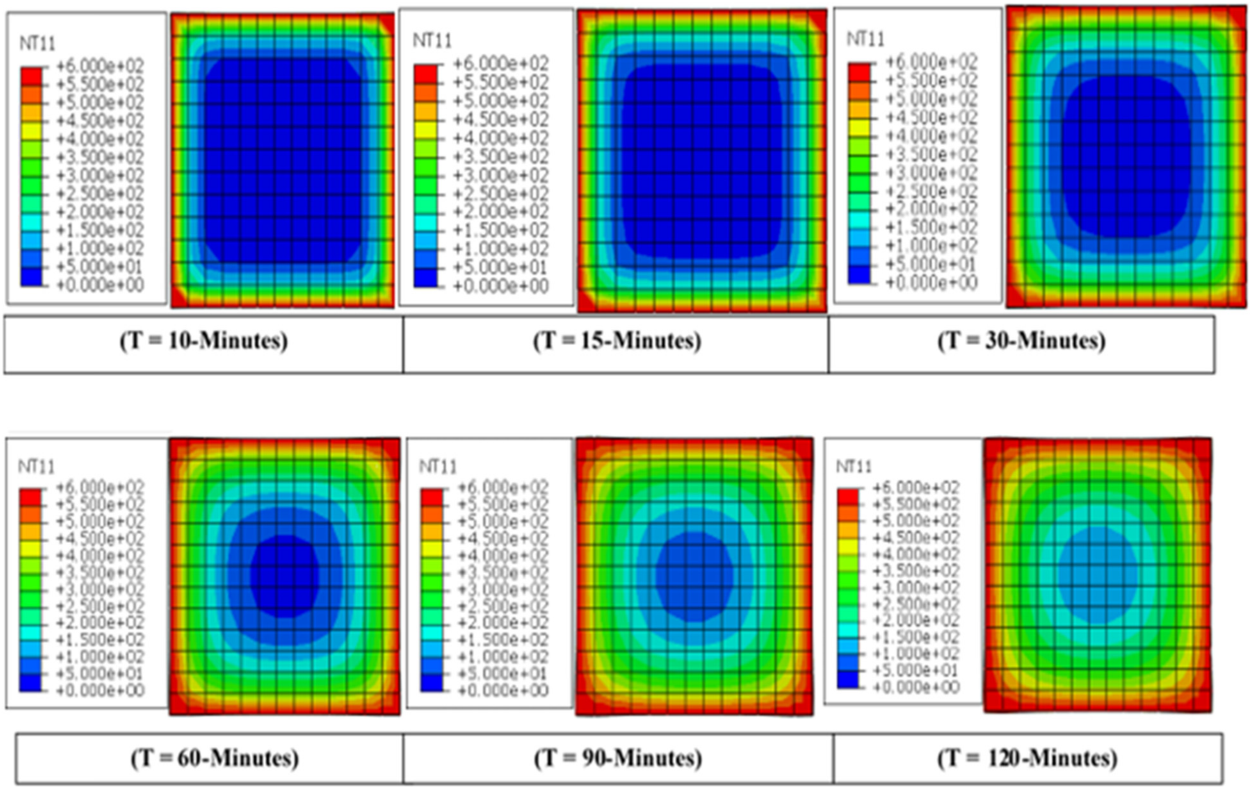

Figure 15: Heat transfer study over time for a column specimen following a 120 min burn at 600°C.

$$\text{Reduction in load capacity Pu \%} = \frac{\text{Load carrying capacity Pu } (T = 20°\text{C}) - \text{load carrying capacity Pu } (T = 600°\text{C})}{\text{Load carrying capacity Pu } (T = 20°\text{C})} \times 100.$$

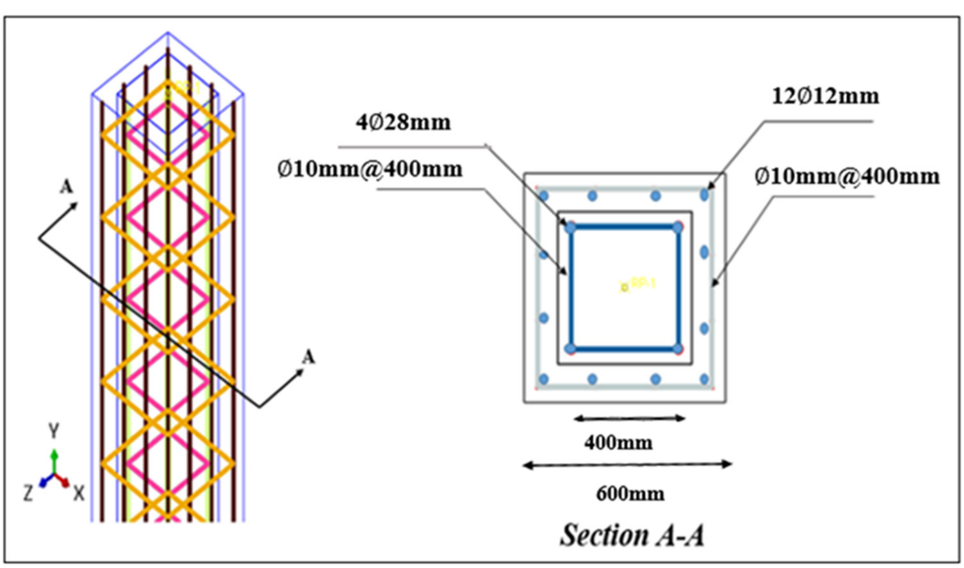

Figure 16: Details of strengthened RC column.

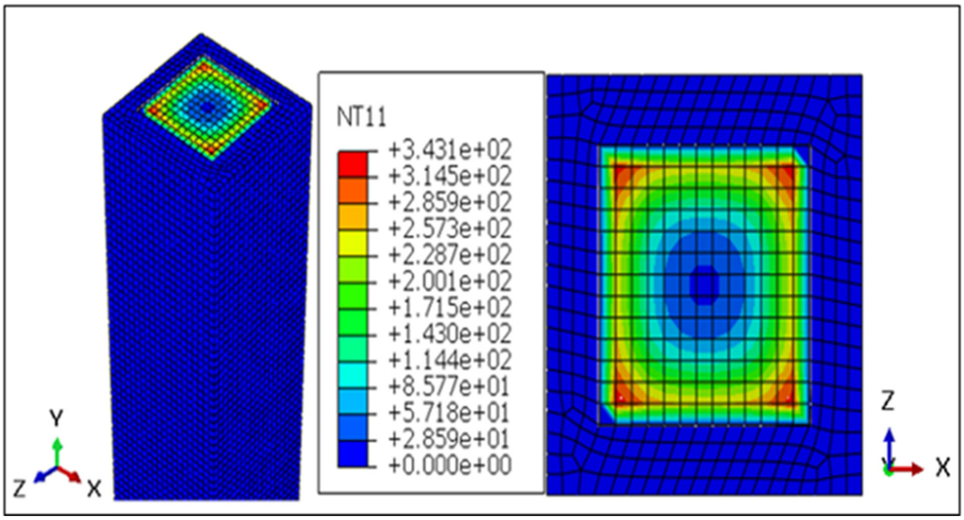

Figure 17: The temperature distribution after full time of burning for strengthened RC column.

5.4 Strengthening of RC column after exposure to fire by RC jacket

Elements of the structure during the period of their investigation are exposed to different working conditions and may differ from the design considerations for these elements, which leads to affecting their strength and durability, and makes them need to re-evaluate their durability. The phenomenon of fire and the resulting rise in temperatures are phenomena that must be given special importance due to their impact on the reliability and durability of concrete structures. In order to properly assess the durability of concrete structures, the specifications of building materials must be determined after being exposed to high temperatures, since there is a change in the physical and mechanical properties of concrete and steel reinforcement depending on its type, the temperatures to which it is exposed and the duration of its impact. The mechanical and thermal properties at elevated temperature of concrete and steel, are measured [16,17] as a source of data for a finite element program for

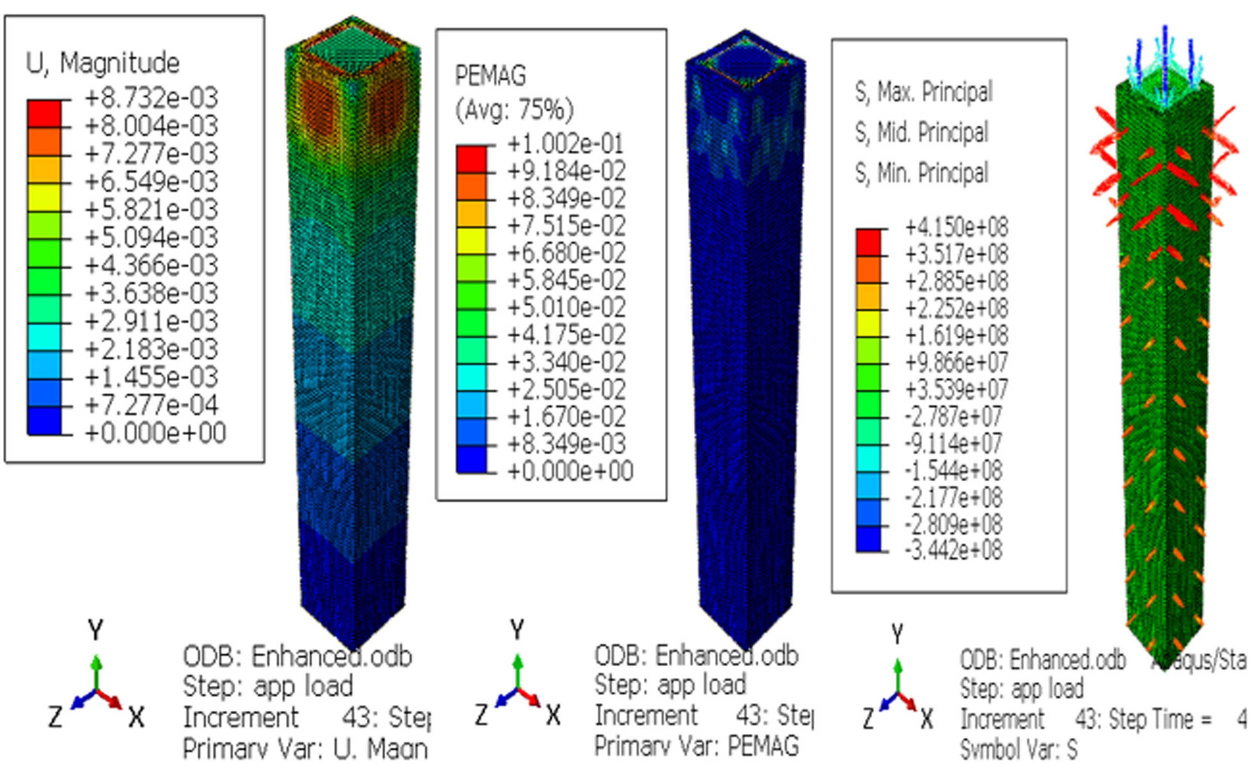

Figure 18: Changes in displacement, stress and plastic strain (PEMAG) for strengthened RC column.

RC jacket and fired column. From both a resource conservation and a cost perspective, rehabilitation and/or strengthening is a more sustainable approach than just dismantling and reconstructing the complete structure (*e.g.*, time, cost, materials, *etc.*). RC jacket was used to strengthen and repair the damaged RC column, which was burnt on 4 sides for 1 h. The RC jacket is a concrete body that is wrapped around the old concrete to strengthen the burnt column. The compressive strength of RC jacket and burnt column is 30 MPa and steel yield stress is 415 MPa, the longitudinal bars diameter was 12Ø12 mm and the ties were distributed as Ø10 mm @400 mm for jacket. The burnt column has a length of 4,000 mm and cross-section of 400 mm × 400 mm, the longitudinal bars' diameter was 4Ø28mm and the ties were distributed as Ø10 mm @400 mm as shown in Figure 16. It is worth noting that the tie is applied to simulate the constraints between the new section and the original section. Furthermore, it causes a change in the column's cross-sectional area, affecting the structure's mass and stiffness. The primary goal of this research is to know the effect of strengthening by RC jacket and its degree of improvement of the load-bearing capability of fire exposed column at a temperature of 600°C and for a duration of 1 h. The use of an RC jacket increased the load capacity by about 95.7% for the burnt column. The results showed that the column recovered its ultimate strength so it is possible to say that the method of strengthening is considered successful. Strengthening column by RC jacket reduces the axial deformation of column exposed to fire from 4 sides (from 14.97 to 7.4 mm) (Figures 17 and 18).

6 Conclusion

Based on the data presented and discussed herein, the following conclusions can be drawn in this study using numerical analysis for column exposed to fire under axial loads:

1. According to the load-displacement relationships, all the numerical models exhibit stiffer behavior than the experimental relationships.
2. The results show that increasing the reinforcement ratio, while using the same amount of steel bars increases load carrying capacity.
3. From zero loads through 70% of the failure load, numerical analysis indicated a linear behavior, which subsequently shifted to a non-linear behavior until the failure load occurred.
4. The longer fire period will lower the load capacity, the difference of load capacity reduction between 15 and 30 min is more than 11%, and the residual deflection increases.
5. The PEEQ visual distribution of RC columns demonstrates that increasing the period of fire load leads concrete to fail, with increased fire time, the mechanical properties of the materials at elevated temperatures degraded gradually, leading to stress redistribution within the cross-section.
6. The presence of transverse reinforcement with greater quantities (reducing tie spacing) showed an improvement in the post-fire behavior of the RC column, the load-carrying capacity was increased for specimen after fire exposure by 6.4% when reducing tie spacing from 400 to 200 mm.
7. For the reference columns, it is shown that concrete columns with a larger transverse reinforcement quantity exhibit more axial displacement than concrete columns without transverse reinforcement.
8. It is very effective to use RC jacket to strengthen burnt concrete columns, and the gain in the axial load capacity of the strengthened columns was quite promising. The axial load increase is 95.7% of those of the unstrengthened burnt column.

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