

Damage Assessment and Residual Capacity of Fire-Exposed RC Columns: A Multi-Parameter 3D FEA Study

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ABSTRACT

Fire is one of the most critical hazards to reinforced concrete (RC) structures, particularly threatening the load-carrying capacity of columns, the primary structural members. The primary objective of this study is to evaluate the residual axial load-carrying capacity and damage behaviour of RC columns under standard fire exposure by simultaneously investigating the combined effects of concrete cover thickness, fire exposure configuration, longitudinal reinforcement ratio, and fire duration. To achieve this objective, a three-dimensional nonlinear finite element model is developed using ABAQUS, incorporating the Concrete Damage Plasticity (CDP) constitutive model within a sequential thermal-mechanical analysis framework. Fire loading is applied according to the ISO 834 standard fire curve at 60, 120, and 180 minutes, representing short, moderate, and prolonged fire exposure conditions, respectively. The model is validated against the experimental data of Abdurraheem and Kadhum for concentrically loaded fire-exposed RC columns, achieving less than 5% error in peak load prediction and a maximum rebar-level temperature deviation of 8.3%, confirming model fidelity for parametric application. Results demonstrate that the 50 mm cover column maintains rebar temperatures approximately 120°C lower than those of the 37.5 mm cover column throughout all fire durations, while retaining 12% higher axial capacity at 180 minutes. Compression damage consistently concentrates in the inner core, while tension damage manifests at the outer surface; both modes are more severe at lower cover thickness. Four-sided fire exposure results in approximately 10% greater capacity loss than two-sided exposure at 180 minutes, confirming that the column enclosure provides quantifiable structural fire protection. A reinforcement ratio of 1.5% is identified as a practical fire-resilience threshold, beyond which additional reinforcement yields negligible capacity improvement. The validated parametric results provide a reliable computational basis for post-fire structural assessment of RC columns.

Keywords: Reinforced concrete column, Fire exposure, Finite element analysis, Concrete damage plasticity, Residual capacity

1. INTRODUCTION

Fire represents one of the most severe structural hazards to RC buildings, with RC columns being particularly critical because their failure under fire loading can trigger progressive collapse of the entire structure [1,2]. Exposure to elevated temperatures leads to simultaneous degradation of both concrete compressive strength and steel yield strength, significantly reducing the load-carrying capacity of columns [3,4].

Extensive numerical and experimental investigations have been conducted on the fire behavior of RC members. Early studies by [5] and [6] examined fire-exposed RC columns using simplified analytical models. [7] established fundamental principles of structural fire engineering design. The role of concrete cover in protecting steel reinforcement from heat penetration has been documented as a decisive factor [8]. More recently, [9] conducted a parametric ABAQUS-based study on residual compressive strength of RC columns,

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[10] investigated post-fire behavior and repair of fire-damaged RC columns, [11] demonstrated the reliability of sequential thermal-mechanical FEA for fire-exposed RC members, and a recent review [12] confirmed that cover thickness and reinforcement ratio govern column fire resistance. Parametric studies of RC columns under Eurocode fire models have also been reported [13]. However, most existing studies address only one or two parameters at a time and typically assume uniform four-sided fire exposure. This condition does not represent corner or wall-adjacent columns in real buildings.

Despite these contributions, three notable gaps remain in the literature. First, most numerical studies on RC column fire resistance assume uniform four-sided fire exposure. This does not represent the realistic condition of corner or wall-adjacent columns that face fire from only two sides. Second, no previous study has simultaneously evaluated the combined effects of concrete cover thickness, fire exposure configuration, and longitudinal reinforcement ratio within a single validated three-dimensional CDP-based FEA framework for RC columns. Third, residual capacity values in the literature are rarely presented as normalized capacity ratios directly usable for post-fire structural assessment. The primary objective of this study is therefore to evaluate the residual axial load-carrying capacity and damage behavior of RC columns under ISO 834 standard fire exposure by simultaneously investigating concrete cover thickness (37.5 mm and 50 mm), fire exposure configuration (two-sided and four-sided), longitudinal reinforcement ratio (1%, 1.5%, and 2%), and fire duration (60, 120, and 180 minutes) within a single validated three-dimensional FEA framework incorporating the CDP model.

2. METHODOLOGY

A three-dimensional finite element model of an RC column was developed using ABAQUS to investigate the structural behavior under fire exposure. The numerical procedure was divided into model development, thermal analysis, validation, and parametric investigation stages.

The concrete component was modeled using the Concrete Damage Plasticity (CDP) constitutive model, which is widely used for nonlinear simulations of concrete cracking and crushing behavior. The CDP parameters were selected according to the procedure proposed by [7]. All material properties were defined as temperature-dependent functions. Thermal properties of concrete, including specific heat capacity and thermal conductivity as functions of temperature, were adopted from Eurocode

1 (EN 1991-1-2, 2002). The mechanical properties of concrete, compressive strength and modulus of elasticity as functions of temperature, and all thermal and mechanical properties of reinforcing steel were defined according to Eurocode 2 (EN 1992-1-2, 2004). Concrete density was taken as 2400 kg/m³ and steel density as 7850 kg/m³, both assumed constant with temperature. Reinforcing steel bars were embedded within the concrete body to represent composite action between concrete and steel.

Table I: Material Properties and Model Parameters

Parameter	Value	Reference
Column length	3048 mm	This study
Cross-section	508 mm × 508 mm	This study
Cross-section (reference)	254 mm × 254 mm	[7]
CDP dilation angle	30°	[7]
CDP eccentricity	0.1	Default (ABAQUS)
CDP viscosity	0.00025	[7]
Concrete tensile strength	10% of f_{ck}	[7]

The geometry of the reference column was defined with a square cross-section of 254 mm × 254 mm and a total height of 3048 mm. Appropriate material properties, section details, and reinforcement arrangements were assigned before meshing and analysis. The assembled model was discretized into finite elements suitable for coupled thermal and structural simulation. Table I: Material Properties and CDP Model Parameters Used in FEA.

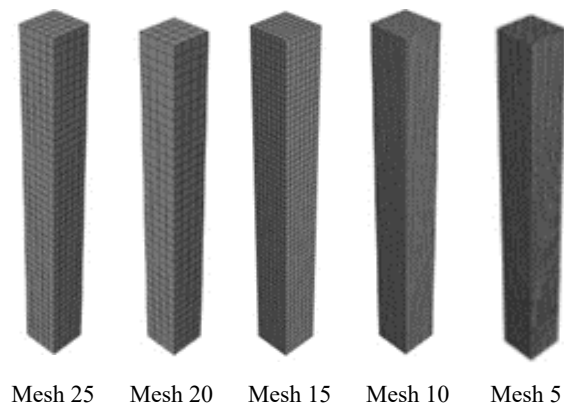


Fig. 1: Five mesh configurations (element sizes 25, 20, 15, 10, and 5 mm) evaluated in the mesh sensitivity study

The mesh convergence study showed that axial displacement varied by less than 1% between the 10 mm and 5 mm element sizes, confirming that the 10 mm mesh provides a mesh-independent solution. This mesh was therefore adopted for the validation model, while a 25 mm mesh was used for the parametric study columns to balance computational efficiency with accuracy. During the thermal analysis phase, the exposed surfaces of the column were subjected to elevated temperatures following the ISO 834 standard fire curve. Heat transfer was simulated by conduction within the solid body and convection-radiation at the exposed boundaries, with a convective heat transfer coefficient of 25 W/m²K on exposed surfaces and 9 W/m²K on unexposed surfaces. Emissivity values of 1.0 (fire) and 0.8 (concrete surface) were adopted per Eurocode 1. Fire exposure durations of 60, 120, and 180 minutes were applied.

After completion of the thermal step, the obtained temperature field was transferred to the structural analysis stage. Axial loading was then applied to determine the residual load-carrying capacity, deformation response, and damage pattern of the heated column. Compression damage, tensile cracking, and load-deflection behavior were extracted from the numerical results.

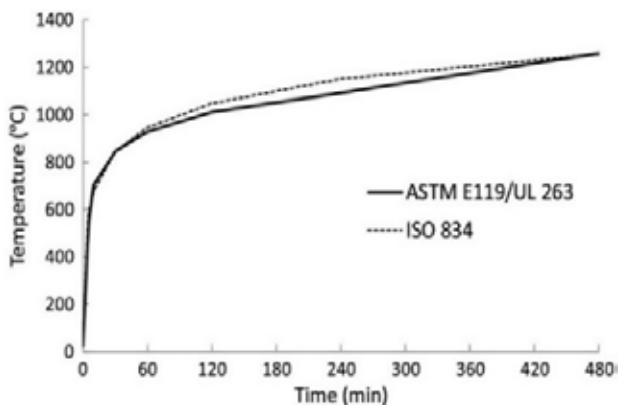


Fig. 2: ISO 834 and ASTM E119/UL 263 standard fire time-temperature curves, ISO 834 adopted in this study for all fire exposure simulations

The developed model was validated against the experimental results reported by [1] for concentrically loaded fire-exposed reinforced concrete columns. The FEA-predicted peak axial load differed from the experimental value by less than 5%, and temperature predictions at the rebar level showed a maximum deviation of 8.3% relative to the published thermocouple readings. These quantitative metrics confirm that the developed model accurately represents the thermo-mechanical behavior of fire-exposed RC columns and is suitable for parametric application.

Following validation, a parametric study was conducted by systematically varying key design parameters of the reference column. The investigated parameters included cross-sectional size, reinforcement ratio, clear cover thickness, and fire exposure condition. Two-sided and four-sided fire exposures were considered to represent practical fire scenarios. It should be noted that concrete spalling was not explicitly modelled in this study. This simplification is conservative, as it assumes the concrete section remains intact throughout the fire duration. The effect of spalling on thermal penetration and structural capacity is acknowledged as a limitation and is recommended for inclusion in future extensions of this work.

3. RESULTS

3.1 Effect of Concrete Cover on Thermal and Structural Response

Figures 3 and 4 present the temperature distribution contours across the column cross-section at 60, 120, and 180 minutes for the 37.5 mm and 50 mm clear cover columns, respectively. In both cases, the outer surface of the column achieves the highest temperature immediately upon fire exposure. The heat progressively penetrates the cross-section as fire duration increases. The ABAQUS NT11 nodal temperature output shows a clearly progressive, non-uniform thermal gradient developing from the outer surface toward the inner core with increasing fire duration for both covers. For the 37.5 mm cover column, the temperature contours show a thermal front toward the rebar zone by 120 minutes, with the innermost concrete layer showing temperatures approximately 400°C at 180 minutes. In contrast, the 50 mm cover column demonstrates a measurably less thermal gradient across the cover layer, with the rebar zone temperature remaining below that of the 37.5 mm cover column. This visual comparison confirms that the additional 12.5 mm of concrete in the 50 mm cover column provides a meaningful supplementary thermal barrier.

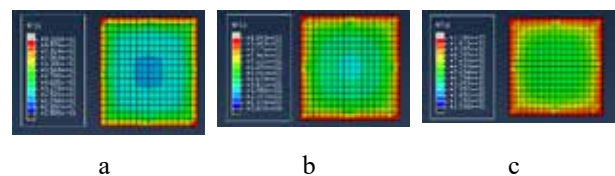


Fig. 3: ABAQUS NT11 nodal temperature contours (°C) for 37.5 mm clear cover column under four-sided ISO 834 fire exposure at (a) 60 min, (b) 120 min, and (c) 180 min

Figure 5 quantifies this difference by plotting rebar temperature vs fire duration for both cover configurations. The rebar temperature in the 37.5 mm cover column rises from approximately 350°C at 60 minutes to nearly 780°C at 180 minutes, following a progressive non-linear growth trajectory

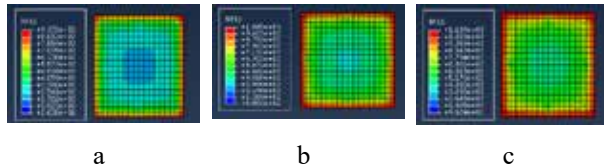


Fig. 4: ABAQUS NT11 nodal temperature contours (°C) for 50 mm clear cover column under four-sided ISO 834 fire exposure at (a) 60 min, (b) 120 min, and (c) 180 min.

Figure 5 shows rebar temperature for 37.5 mm and 50 mm cover. In the 50 mm cover column, rebar temperatures are approximately 120°C lower throughout the entire fire duration, reaching approximately 660°C at 180 minutes. This indicates that the 37.5 mm cover column enters the critical steel degradation regime earlier and sustains more severe strength loss over the same fire duration. Figure 6 presents the load-carrying capacity ratio (P/P_o) for both cover columns at 60, 120, and 180 minutes. At 60 minutes, both columns retain comparable capacity, with the 37.5 mm and 50 mm cover columns achieving P/P_o values of approximately 0.70 and 0.71, respectively. As fire duration increases, the capacity gap between the two cover configurations widens progressively. At 120 minutes, the 37.5 mm and 50 mm cover columns retain approximately 52% and 55%, respectively. At 180 minutes, the divergence is most pronounced. The 37.5 mm cover column capacity falls to approximately 41% while the 50 mm cover column retains approximately 46%, representing a P/P_o advantage of 0.05 for the thicker cover, equivalent to approximately 12% higher retained capacity relative to the 37.5 mm cover column. This escalating benefit with fire duration confirms that concrete cover thickness becomes increasingly decisive as a structural fire protection parameter beyond 60 minutes of exposure.

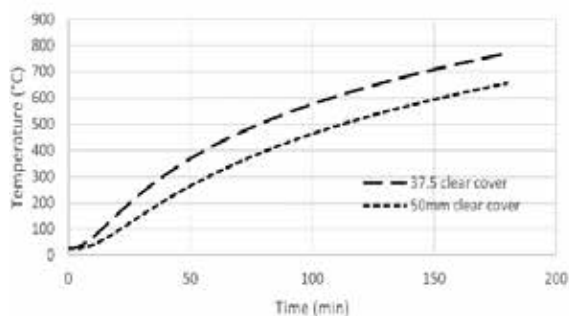


Fig. 5: Rebar temperature development vs fire duration for 37.5 mm and 50 mm clear cover columns

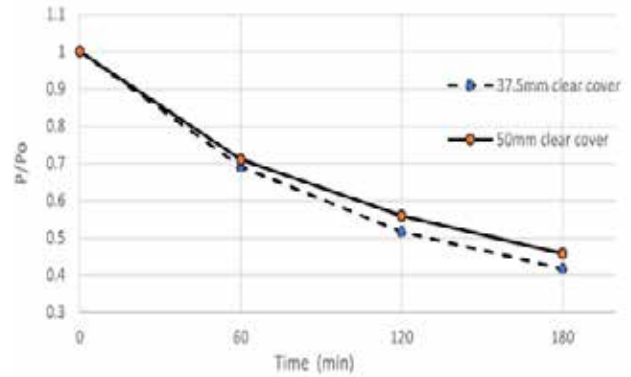


Fig. 6: Load-carrying capacity ratio (P/P_o) vs fire duration for 37.5 mm and 50 mm clear cover columns

3.2 Compression and Tension Damage Patterns

DAMAGET and DAMAGEC contour values displayed in Figures 7 through 10 represent element-level output averaged at shared nodes following extrapolation from integration points, as performed by ABAQUS. Values marginally exceeding 1.0 in some contour plots result from this nodal averaging extrapolation procedure and do not indicate violation of the physical 0-to-1 bound of the scalar damage variable within the constitutive model. Figures 7 through 10 present the compression (DAMAGEC) and tension (DAMAGET) damage contours from the CDP model for both cover configurations at 60, 120, and 180 minutes of fire exposure.

For the 37.5 mm clear cover column at 60, 120, and 180 minutes, compression damage consistently concentrates in Figure 7 within the inner core at mid-height, with maximum DAMAGEC indices of 0.8228, 0.4182, and 0.2735, respectively, for 3 different time durations. The damaged core zone expands progressively along the full column height, reflecting a redistribution of compressive stress across a wider volume. Tension damage in Figure 8 exhibits fundamentally different characteristics, manifesting exclusively at the outer surface of the column rather than the core. Maximum DAMAGET values of 1.5010, 1.6300, and 1.1410 for 3 different time durations confirm progressive surface cracking driven by thermal strain incompatibility between the hot exposed surface and the cooler interior.

For the 50 mm clear cover column, compression damage (Figure 9) follows the same inner core concentration pattern observed in Figure 7, with maximum DAMAGEC indices of 0.7459, 0.5236, and 0.2148 consistently lower than those for the 37.5 mm

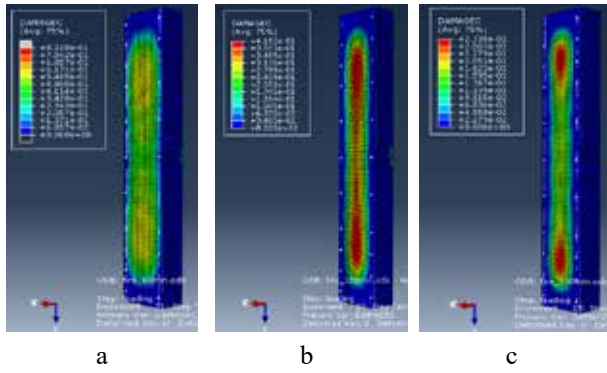


Fig. 7: ABAQUS DAMAGEC contours for 37.5 mm clear cover column under four-sided ISO 834 fire at (a) 60 min (max 0.8228), (b) 120 min (max 0.4182), and (c) 180 min (max 0.2735)

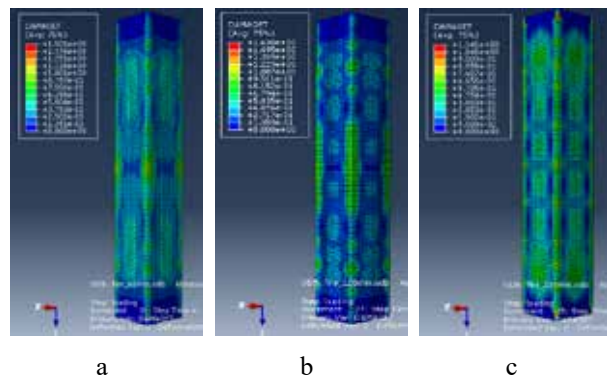


Fig. 8: ABAQUS DAMAGET contours for 37.5 mm clear cover column under four-sided ISO 834 fire at (a) 60 min (max 1.5010), (b) 120 min (max 1.6300), and (c) 180 min (max 1.1410)

cover, directly reflecting the lower core temperatures and delayed thermal penetration afforded by the thicker cover. Similarly, tension damage in Figure 10 shows reduced severity, with maximum DAMAGET values of 1.8390, 1.4370, and 1.5340, rather than the continuous cracking pattern observed in Figure 8, indicating that the gentler thermal gradient across the 50 mm cover layer produces significantly lower surface cracking intensity across all fire durations.

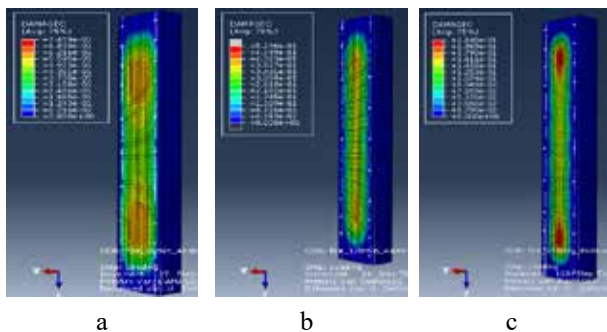


Fig. 9: ABAQUS DAMAGEC contours for 50 mm clear cover column at (a) 60 min (max 0.7459), (b) 120 min (max 0.5236), and (c) 180 min (max 0.2148)

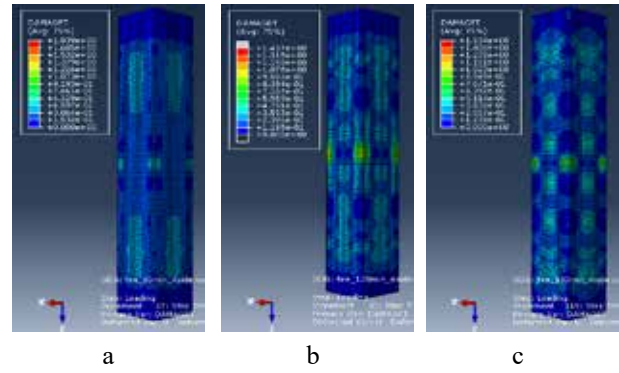


Fig. 10: ABAQUS DAMAGET contours for 50 mm clear cover column at (a) 60 min (max 1.8390), (b) 120 min (max 1.4370), and (c) 180 min (max 1.5340)

3.3 Effect of Fire Exposure Type Configuration

Figure 11 compares the cross-sectional temperature distribution under two-sided and four-sided fire exposure for the 50 mm clear cover column. Under two-sided exposure (Fig. 11a), the maximum surface temperature reaches 937°C at the two directly exposed faces, while the unexposed faces remain significantly cooler with 78°C

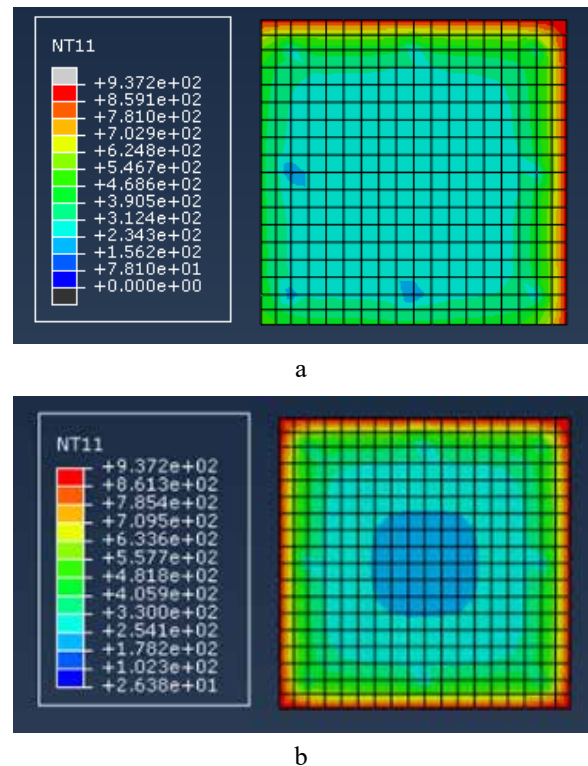


Fig. 11: ABAQUS NT11 cross-sectional temperature contours (°C) comparing (a) two adjacent side fire exposure and (b) four-sided fire exposure

temperatures. This strongly non-uniform thermal field results in differential rebar heating. Under four-sided

exposure (Fig. 11b), all faces simultaneously reach the maximum surface temperature of 937°C. The absence of any thermally protected rebar zone eliminates the compensating mechanism observed in the two-sided case.

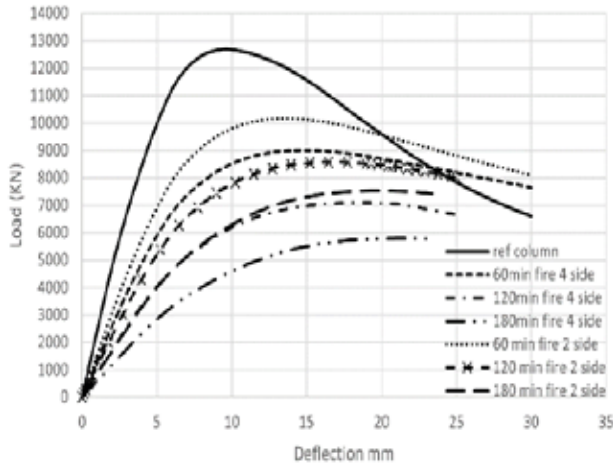


Fig. 12: Load vs deflection curves for 50 mm clear cover column under two-sided and four-sided ISO 834 fire exposure at 60, 120, and 180 minutes

Figure 12 quantifies the structural consequence of this thermal difference through load-deflection curves for both exposure configurations at 60, 120, and 180 minutes. The reference column (50 mm cover column) achieves a peak load of approximately 12,800 kN. Under two-sided exposure, peak loads of approximately 10,800 kN, 9,400 kN, and 7,800 kN are recorded at 60, 120, and 180 minutes, respectively. These represent capacity retentions of approximately 84%, 73%, and 61%, respectively. Under four-sided exposure, the corresponding peak loads of approximately 10,200 kN, 8,200 kN, and 6,500 kN yield capacity retentions of approximately 80%, 64%, and 51% at 180 minutes.

The four-sided exposure column, therefore, loses approximately 10% more capacity than the two-sided exposure column. The post-peak descent is also notably steeper for four-sided exposure at longer durations, indicating more brittle post-fire failure behavior as a consequence of uniform reinforcement degradation across the entire section.

3.4 Effect of Reinforcement Ratio

Three reinforcement ratios were investigated using a 508 mm × 508 mm (20"×20") column cross-section of 3048 mm height. Table II describes the three configurations as C21, C22, and C23, all sharing identical geometry and fire exposure conditions to isolate the effect of steel content on post-fire structural performance.

Table II: Different Reinforcement Ratios

Identification	Size	Reinforcement Ratio	Rebars
C21	20"×20"	2%	8 - #9
C22	20"×20"	1.5%	8 - #8
C23	20"×20"	1%	8 - #7

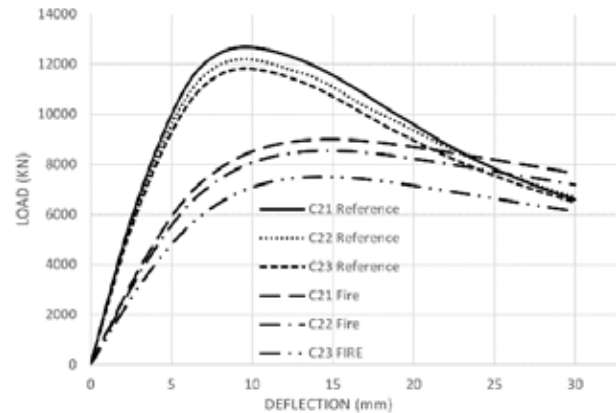


Fig. 13: Load vs deflection curves for columns C21 (2%), C22 (1.5%), and C23 (1%) under ambient reference and post-fire loading conditions

Figure 13 presents the load-deflection response for all three columns under both reference and post-fire loading conditions. Under ambient conditions, reference peak loads of approximately 12,700 kN, 12,200 kN, and 11,600 kN are recorded for C21, C22, and C23, respectively. After fire exposure, all three columns exhibit substantial capacity reduction, with post-fire peak loads of approximately 8,900 kN, 7,600 kN, and 6,500 kN for C21, C22, and C23, respectively. The post-peak descent becomes progressively steeper with decreasing reinforcement ratio, indicating more brittle post-fire failure behavior in columns.

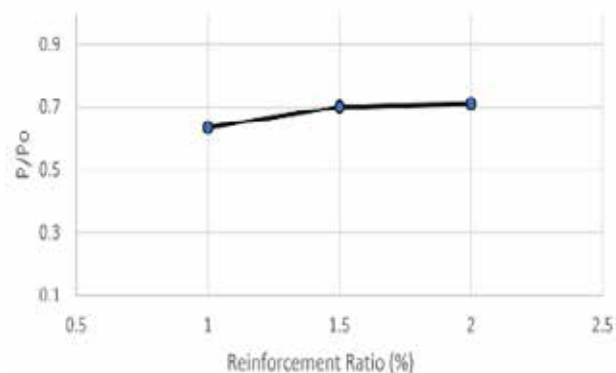


Fig. 14: Capacity Ratio for different reinforcement ratios

4. CONCLUSION

This study presented a validated three-dimensional sequential thermo-mechanical FEA model developed in ABAQUS incorporating the Concrete Damage Plasticity (CDP) model to investigate the thermal response, damage behavior, and residual axial capacity of RC columns under ISO 834 standard fire exposure. The following conclusions are drawn:

1. The validated FEA model accurately predicted both temperature distribution and load-deflection response, with less than 5% error in peak load and a maximum rebar-level temperature deviation of 8.3% compared to experimental data [1], confirming its reliability for parametric application.
2. Column residual capacity degrades progressively with fire duration, with the rate of degradation accelerating significantly beyond 60 minutes of fire exposure.
3. The 50 mm cover column maintains rebar temperatures approximately 120°C lower than the 37.5 mm cover column, resulting in a P/Po advantage growing from 0.01 at 60 minutes to 0.05 at 180 minutes, equivalent to approximately 12% higher retained capacity.
4. Four-sided fire exposure causes approximately 10% greater capacity loss than two-sided exposure at 180 minutes, confirming that wall enclosure of columns provides quantifiable structural fire protection.
5. Compression damage concentrates in the inner core while tension damage manifests at the outer surface; both modes are more severe at lower cover thickness and longer fire duration.
6. A reinforcement ratio of 1.5% is identified as a practical fire-resilience threshold, beyond which additional reinforcement yields negligible capacity improvement.

Future work should incorporate concrete spalling, the post-fire cooling phase, and eccentric loading of the model.

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