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A NUMERICAL STUDY ON FIRE RESISTANCE OF BLAST DAMAGED REINFORCED CONCRETE COLUMNS

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Abstract

An explosion or blast can trigger a subsequent fire in the building structure. In the case of reinforced concrete (RC) structures, blast loading can result in relatively large residual deformations and locked in stresses in the affected members prior to fire exposure. Such residual deformations and locked in stresses can have a detrimental effect on the fire performance of RC members. Accordingly, this paper presents results from a numerical study on fire resistance of blast damaged RC columns. A three dimensional numerical model of an RC column was developed using general-purpose finite element software ABAQUS and validated against data from the experimental fire resistance tests. The developed continuum-based finite element model of RC column was subjected to blast loads of distinct scaled distances simulating the far-field and near-field scenario. Further, the blast-damaged RC columns were exposed to a standard fire, and calculated the fire resistance period corresponding to each blast case. Results indicate that the blast damaged RC column show a significant reduction in fire resistance period compared to the undamaged RC column.

Keywords: RC Column, Blast, Post-blast Fire, Fire Resistance, Damage, ABAQUS.

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1 INTRODUCTION

Fire is a plausible secondary hazard following accidental or intentional blasts (explosions). In the case of reinforced concrete (RC) structures, blast damage can result in residual (permanent) deformation and locked in stresses. Such residual deformation and locked in stresses can have a detrimental effect on the fire performance of RC members subject to a post-blast fire (PBF) scenario. Therefore, it is of interest to quantify the reduction in the fire resistance of RC members under the PBF conditions.

Recognizing the catastrophic consequences of a PBF scenario on structural systems, it has received noticeable attention in the past. Early works by researchers [1, 2] comprised an integrated environment for conducting fire and blast (explosion) analysis of steel frames. The framework was verified for a number of examples, including an isolated column subject to explosion followed by fire. Similarly, the combined effects of blast and fire and the PBF scenario of steel frames and isolated steel columns were investigated in various studies [3-9]. In some other studies [10], the progressive failure of steel framed structures was studied due to the combined effects of blast and fire.

The literature review suggests that the PBF scenario has received some attention in the past. However, very limited understanding exists on the topic, especially in the case of concrete members. This can be attributed to the complexities of modeling the interactions between blast and fire. Furthermore, no studies exist on critical load-bearing elements such as RC columns when subject to a PBF scenario.

Accordingly, this paper presents results from a numerical study on an isolated RC column subject to a PBF scenario using the general-purpose finite element software ABAQUS [11]. The blast loading is calculated using Kingery-Bulmash [12] relations and assumed to be uniformly distributed across one face of the RC column. The fire analysis is conducted in a sequentially coupled manner wherein the temperature field is independent of the stress field, i.e., the stresses do not alter the calculation of temperatures in the member. Results from the analysis are used to highlight the difference in fire resistance periods of the undamaged and post-blast fire damaged RC column obtained for four different blast loading conditions representing far-field and near-field conditions.

2 APPROACH FOR SIMULATING PBF SCENARIO

In order to simulate the PBF scenario on an isolated RC column under axial load, a step-by-step procedure of three steps was adopted. This approach could explicitly account for the interaction between blast loading and subsequent fire exposure. Details of this approach are presented in this section.

2.1 General Procedure

For simulating the PBF scenario, a three-step procedure was considered. The dead load and live load acting on the RC column were simulated as part of the first step of the approach. Subsequently, in the second step, blast loading was applied on a single face of the RC column. The peak pressure and impulse (duration) of each blast load were calculated based on an assumed charge weight and standoff distance using the Kingery-Bulmash [12] equations. Furthermore, the blast loading distribution was assumed to be uniform which is a reasonable assumption made in a number of previous studies [13-15]. The analysis was conducted using a dynamic implicit procedure available within the framework of ABAQUS [11] software. The analysis was continued for almost 100 times the duration of the blast pulse to ensure that a steady state of vibration following the pulse loading was attained within the member.

Once the blast loading was applied in the second step, a sequentially coupled thermo-mechanical analysis was conducted in the third step. A separate heat transfer model with identical geometry was developed as part of this step. The results from the heat transfer analysis were imported as a predefined field condition into the thermo-mechanical model in the third step to simulate fire exposure. Appropriate failure criteria were applied in the second and third steps to determine failure during blast loading or fire exposure. The three steps of analysis for simulating the PBF scenario are illustrated through a flowchart in Figure 1.

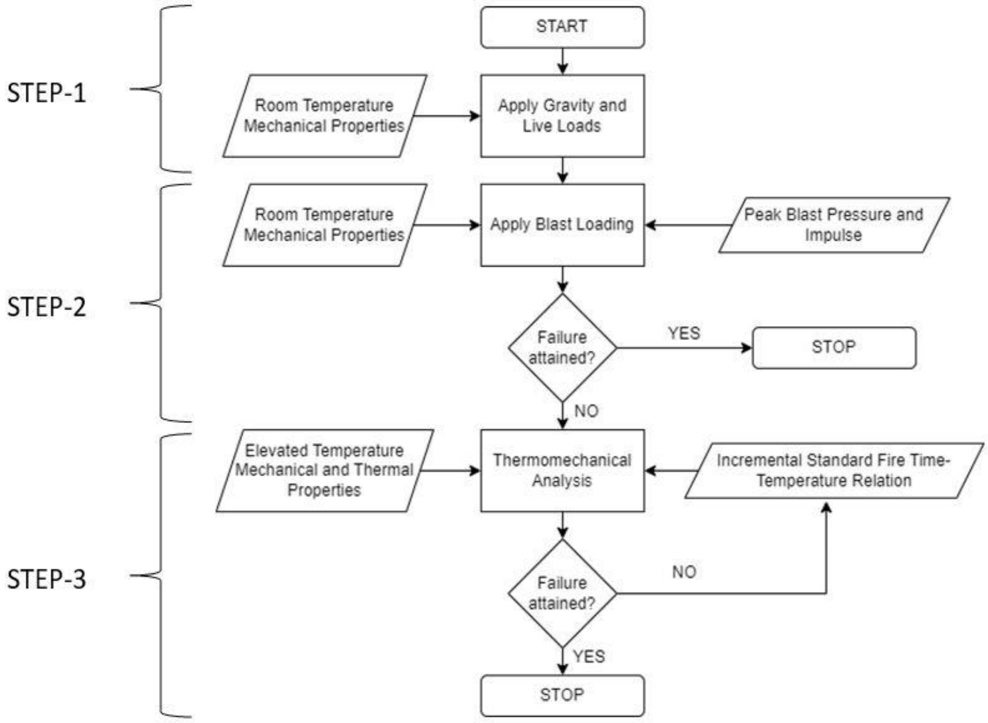

Figure 1: Flow chart describing the three steps in simulating the PBF scenario.

2.2 Properties of constituent materials

The properties of constituent materials, i.e., concrete and rebar, have a significant bearing on the PBF performance of the RC column. Since it is not possible to modify the material model, i.e., different suitable material models for blast (strain-rate dependent) and fire (temperature dependent), after initiating the analysis, the room temperature properties are utilized for the first two steps with the calculated average representative strain rate for each blast case, followed by temperature-dependent constituent models to be followed for the thermomechanical analysis during the fire.

The stress-strain relationship provided by Saenz (1964) [16] was modified to account for strain rate effects by employing the dynamic increase factors (DIFs) [17] to describe the dynamic behavior of concrete under uniaxial compression. Similarly, the tensile behavior of concrete under the strain rate effects was modeled by adopting the constitutive relation from the CEB-FIP Model code (2010) [18] with the inclusion of DIFs [19]. Furthermore, the elastic-plastic hardening models were used in modeling the reinforcement bars and updated the stress-strain relation with the DIFs to account for strain rate effects [20]. Numerical analysis was carried out for each blast case with a stress-strain model corresponding to static strain rate and observed the mechanical strain rate (ER) output obtained from the ABAQUS. Subsequently, for each trial, an average ER value was obtained, corresponding DIFs were calculated, and the stress-strain model was updated for both concrete and steel and used in Step-1 and Step-2 of the analysis (at room temperature conditions). During fire analysis in Step-3, the temperature ingress from the fire into concrete and the rebar is governed by their thermal properties, i.e., specific heat and thermal conductivity. Furthermore, the temperature-dependent mechanical properties, i.e., Young's modulus, yield strength, ultimate strength, and strain at ultimate strength, govern the structural response of the member. These properties were specified following the Eurocode EN 1992-1-2 [21] specifications.

2.3 Failure criteria

While simulating the PBF scenario, it is essential to consider failure limit states when evaluating response during blast loading as well as fire exposure. Step-1 of the analysis (at room temperature conditions) involves simulating gravity and live loading on the member; hence failure is unlikely as these loading conditions are well below the load-bearing capacity of the structure at ambient conditions. Subsequently, in Step-2 of the analysis (blast loading), deflection-based failure criteria in terms of support rotation are adopted as per UFC 3-340-02 [22] to check the failure of the RC column before the fire exposure. Finally, in Step-3 of the analysis, the rate of deflection and maximum deflection criteria as per ASTM E119 [23] is adopted to determine failure in the RC column during fire exposure.

3 NUMERICAL STUDY

In order to evaluate the performance of RC columns subject to the PBF scenario, finite element models were developed using the general-purpose finite element software ABAQUS [11]. The predicted response parameters from these models were used to evaluate the fire resistance period of the RC columns subject to the PBF scenario.

3.1 General

A normal strength RC column tested under combined effects of axial loading and temperature exposure as per ASTM E119 [23] standard time-temperature relation was selected from published literature [24]. The test parameters of the chosen column are summarized in Table 1.

Column Designation	Fire Exposure	Initial Load (kN)	Fire Resistance (min)	Extent of Spalling
TNC1	ASTM E119	930	278	Minor

Table 1: Summary of test parameters and results used for model validation.

Different input parameters such as the geometric details, gravity and live loading distribution, blast pressure application, fire scenario, and material properties are required to simulate the PBF scenario. The details of the column, the blast pulse and distribution, boundary conditions, and the time-temperature relation are shown in Figure 2.

The tested columns were fixed at the bottom and had only vertical translation as the degree of freedom at the top. The considered RC column had four longitudinal bars of 25mm size tied with 10 mm stirrups at a spacing of 75 mm on both ends and 145 mm in the middle. The M40 (40MPa) grade of concrete was considered with the reinforcement embedded inside with yield strength of 420 and 280 MPa for longitudinal and ties bars, respectively. Approximately 3000 mm of the entire column length was exposed to fire. The column was subject to a concentric load of 930 kN, corresponding to approximately 54% of its design capacity [25] prior to fire exposure. These loads were maintained constant throughout the fire exposure until the columns failed and could not sustain the applied loading. All four faces of the column were subject to fire exposure. The same column selected for validation was subject to varying PBF scenarios with four different blast pulses. The analysis was continued for 500 milliseconds for each case to allow for a steady state of vibration (as no damping was considered in the model) to be attained prior to fire exposure. The blast pulse details for each of the four cases are summarized in Table 2.

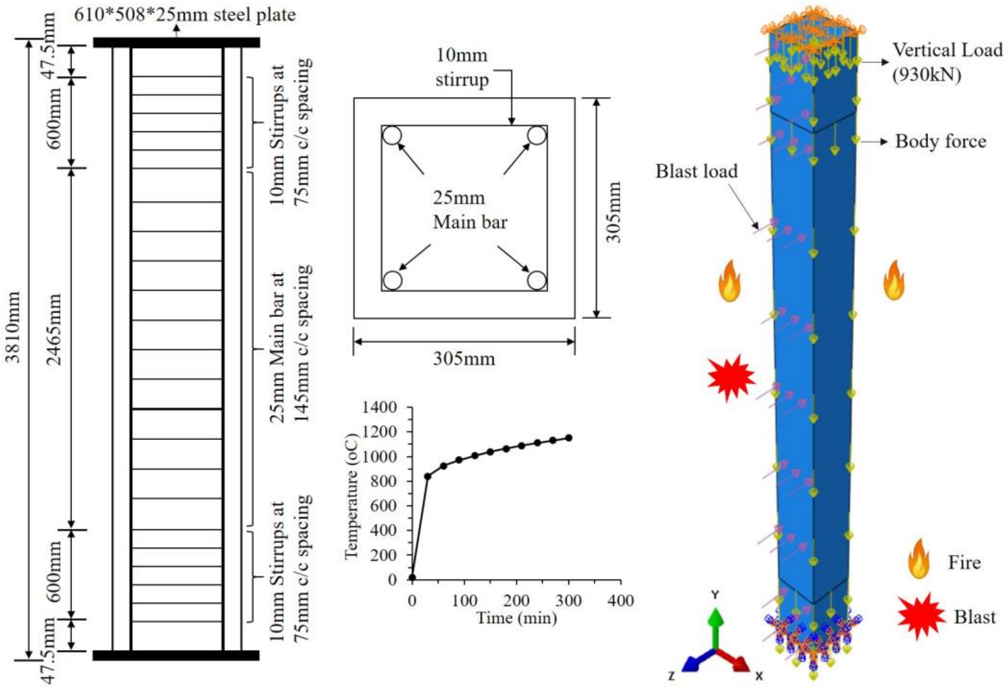

Figure 2: Dimensions, loading, and reinforcement details of the column selected for validation.

Scenario Designation	Charge Weight (Kg)	Standoff Distance (m)	Scaled Distance (m/kg$^{1/3}$)	Peak Pressure (MPa)	Peak Impulse (MPa-ms)	Scenario Classification [26]
Case-1	1000	20	2	1.056	3.638	Far-field
Case-2		16	1.6	2.08	4.798	Far-field
Case-3		12	1.2	4.895	6.944	Near-field
Case-4		11	1.1	6.268	7.789	Near-field

Table 2: Summary of blast pulses for simulating the PBF scenario.

3.2 Finite element model

The analysis was carried out using the general-purpose finite element software ABAQUS. Two sub-models were developed to simulate the effects of the PBF scenario on the RC column. A structural model was required for conducting the room temperature loading analysis, blast loading analysis, and thermo-mechanical analysis. Also, a heat transfer model is needed to calculate the evolution of temperatures within the column cross-section. Accordingly, the structural model comprised of eight-noded continuum elements with full integration (C3D8) for modeling concrete and two-node link elements (T3D2) for modeling the steel reinforcement. The heat transfer model comprised of eight-noded linear brick elements (DC3D8) for discretizing concrete and two-noded link elements (DC1D2) for steel reinforcement, with only nodal temperature as the degree of freedom. A perfect bond between rebar and concrete was simulated through the tie constraint in the heat transfer model, whereas embedded region constraint was used in the structural model. The constitutive models for concrete and steel were defined using plasticity models within the framework of the software. Concrete was modeled using a concrete-damaged plasticity model, and an elastic-plastic hardening model was used for steel reinforcement. A discretized view of the structural model of the RC column, along with the material models for both blast and fire analysis, are shown in Figure 3.

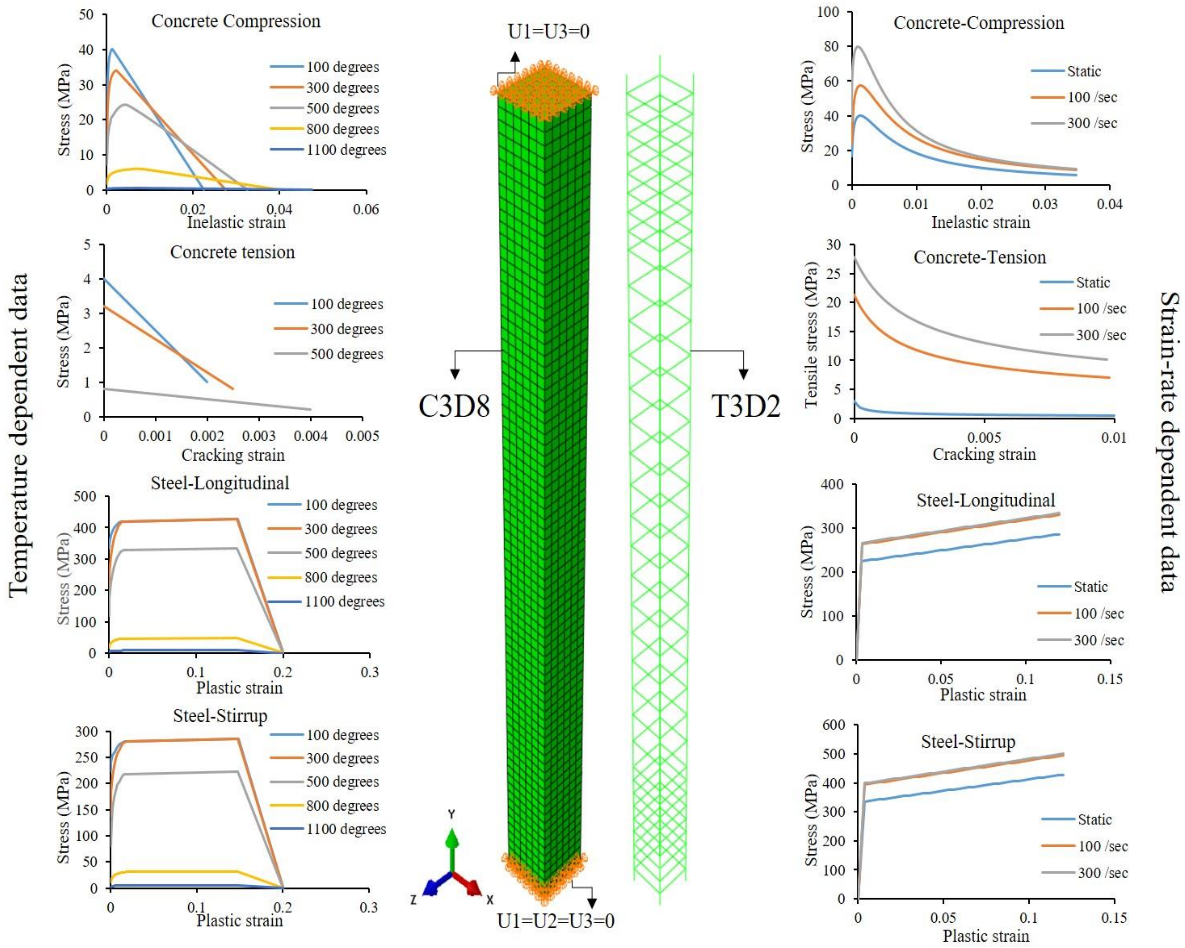

Figure 3: Discretized view of the RC column with adopted stress-strain relations for concrete and steel.

4 RESULTS AND DISCUSSION

Predictions of response parameters obtained through numerical simulation were compared against those measured during tests [24] for validation. Figure 4 shows temperature progression at various cross-sectional locations of the column as a function of time. The temperature within deeper layers of the column experiences a progressively lower rate of increase when compared with fire temperature due to low thermal conductivity and high specific heat of concrete. Overall, the measured and predicted temperatures at different cross-sectional locations are in agreement. This comparison establishes the validity of the heat transfer model.

Similarly, the axial deflections of the column measured during tests and predicted through the numerical model are compared in Figure 5. The deflection in the columns is governed by several factors, including load level, thermal expansion, and creep. The initial deflection (expansion) of the column is a consequence of thermal strains in the column due to monotonically increasing fire temperatures. Subsequently, as cross-sectional temperatures increase further, the mechanical strains begin to dominate due to temperature induced degradation in concrete and steel reinforcement as well as primary and secondary creep effects. As a result, the deflections in the column begin to decay after attaining a peak value despite fire temperatures increasing monotonically.

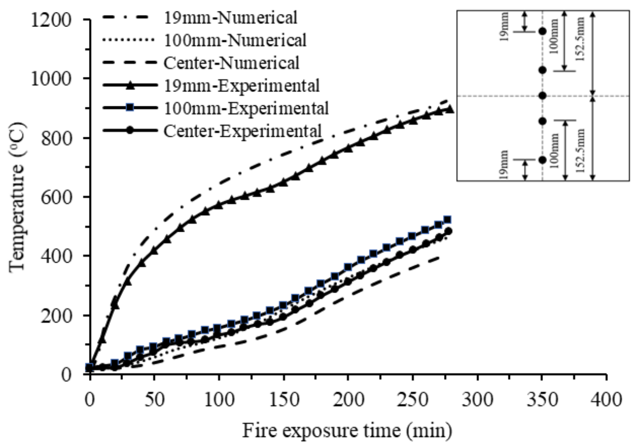

Figure 4: Comparison of measured versus predicted cross-sectional temperatures at various depths in column.

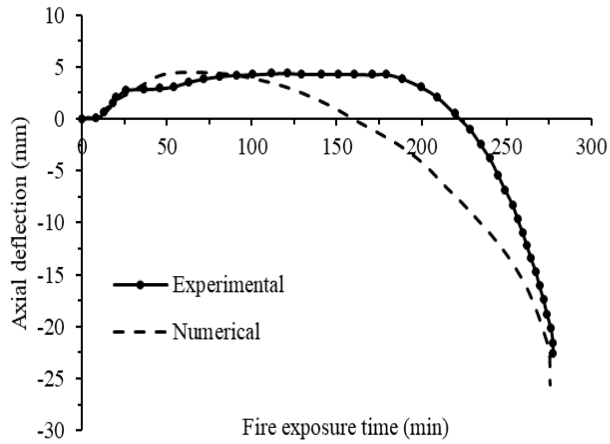

Figure 5: Comparison of measured versus predicted axial deflection during fire exposure alone.

Eventually, the increasing cross-sectional temperatures result in the yielding of reinforcement resulting in the contraction of the column followed by failure. It can be seen that the predicted and measured axial deformations agree well during the early phases of fire exposure, when the majority of the deflections occur due to thermal expansion. However, as fire exposure time progresses further, the numerical model over-predicts the contraction in the column. This can be attributed to the conservative nature of the temperature-dependent mechanical model prescribed by Eurocode 2 [21]. Nonetheless, the prediction of time to failure is in a close match with the experimental result.

The validated numerical model was modified to simulate the effects of the PBF scenario on the same RC column. In this numerical model, Step-1 comprised gravity loading as a distributed body force of 2.5 kN/m^3 and a vertical load of 930 kN on the column. Following the application of the ambient loads, blast loads were applied as pressure loading corresponding to peak pressure and pulse duration, as summarized in Table 2. Finally, after running the blast analysis for a sufficiently long time period (500 milliseconds) such that a steady state of vibration was attained, a sequentially coupled thermo-mechanical analysis was conducted to evaluate the fire resistance period for each case. During the blast analysis, it was observed that for all the considered blast cases, the central bending deflections of the RC column measured in the lateral direction were within the permissible limits recommended by UFC 3-340-02 [22]. Therefore, the considered RC column can be subject to the PBF scenario.

Figure 6 shows axial deformations on the blast loaded edge of the top face from the blast loading step for all four cases of blast loading. As expected, the top edge on the blast loaded face begins to deflect downwards initially from its original equilibrium position under static loading. The downward deflection continues to increase during the initial phases of the triangular pulse loading. Eventually, as time progresses, the amplitude of the oscillations induced by the blast pulse diminishes and attains a steady state value which does not reduce as no damping is considered in the model. Also, a noticeable residual deformation can be observed in the column depending on the scaled distance.

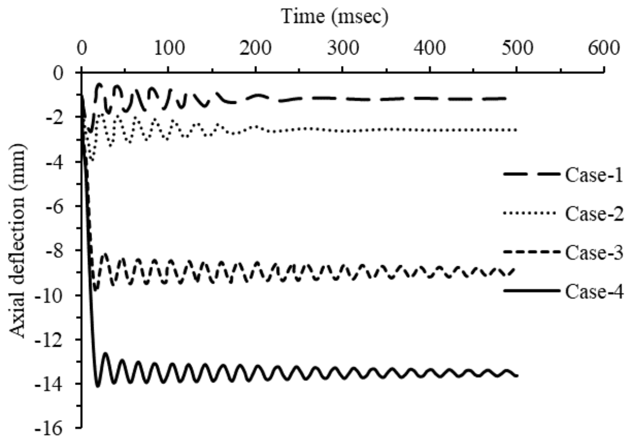

Figure 6: Predicted axial deflection at the top face of the column for different blast pulses.

Besides the blast response, a comparison of the response of the column subject to distinct PBF scenarios is contrasted with the fire response of the undamaged column in Figure 7. Clearly, the blast damaged columns subject to fire exposure exhibit a similar response to the undamaged column. Nonetheless, the initial imperfection in terms of the residual deflection and locked in stresses (as depicted for Case-2 and Case-4 in Figure 8) resulted in a significant reduction in the overall fire resistance period of the blast damaged columns. As a comparison, Table 3 shows the fire resistance period of undamaged and blast damaged RC columns (PBF). It can be observed that the fire resistance period of PBF columns reduces by almost 83% for the near-field blast (Case-4) and 22% for the far-field (Case-1).

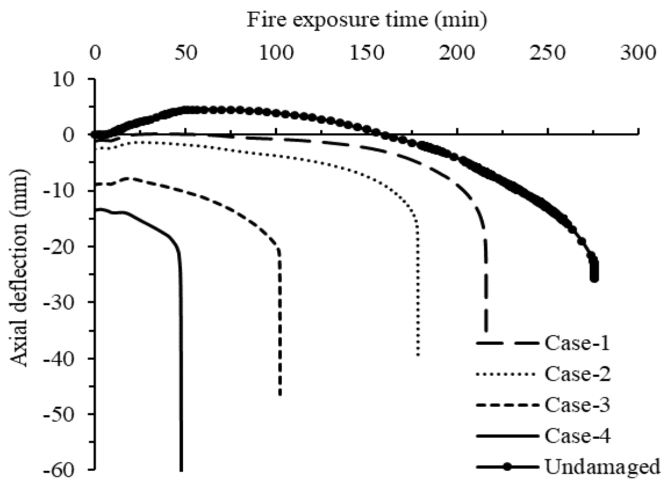

Figure 7: Comparison of axial deflection for undamaged column versus PBF scenario.

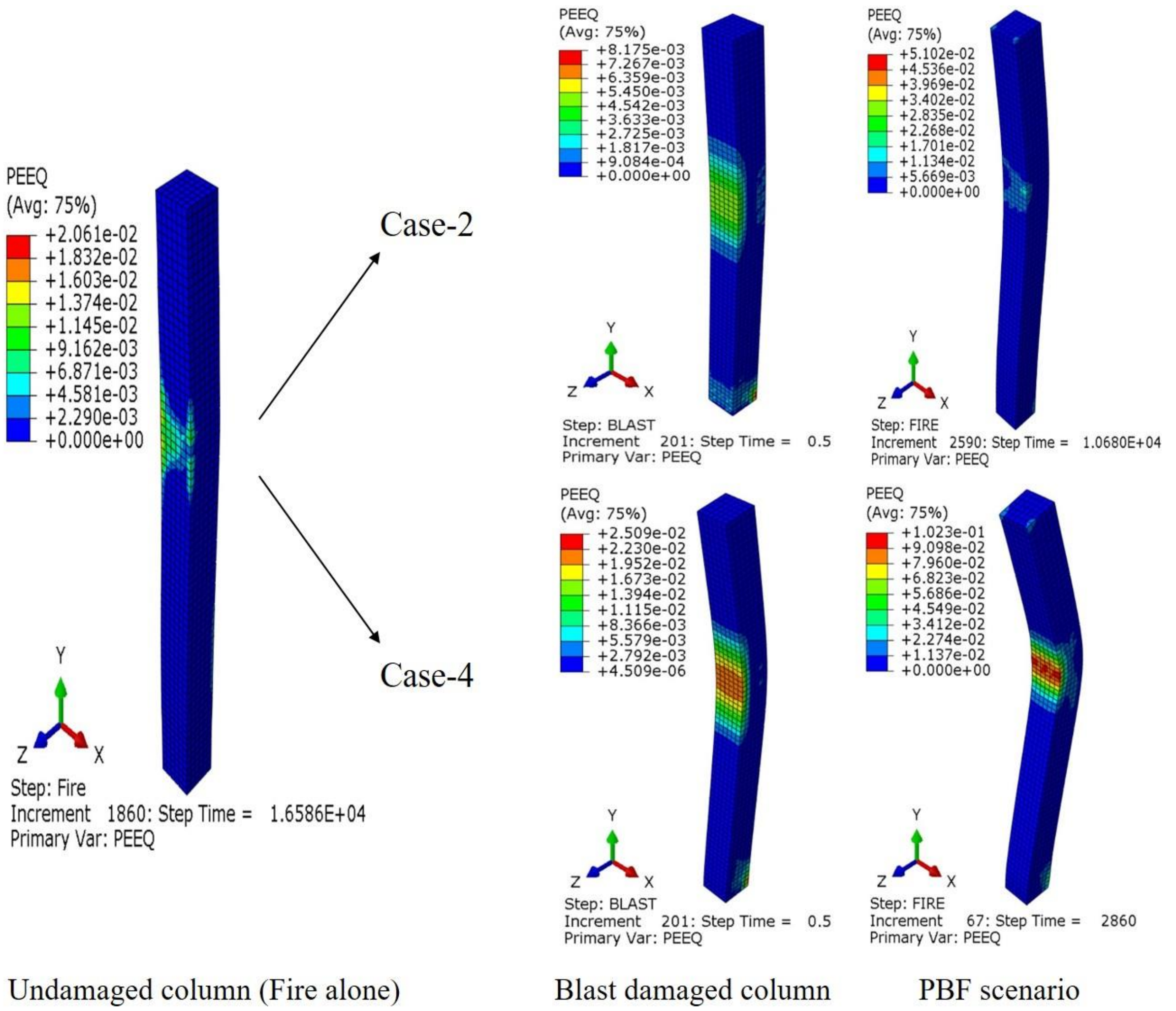

Figure 8: Equivalent plastic strain comparison of undamaged column versus PBF scenario for Case-2 and Case-4.

Loading Scenario	Residual Axial Deflection (mm)	Fire Resistance Period (min)	% Reduction
Undamaged (Fire alone)	-	276.4	-
Damaged (Case-1)	1.17	216	22.3
Damaged (Case-2)	2.56	178	35.9
Damaged (Case-3)	8.82	102.3	63.2
Damaged (Case-4)	13.6	47.7	82.8

Table 3: Comparison of fire resistance period between undamaged column versus PBF scenario.

5 CONCLUSIONS

In general, structural members or systems are designed for a required fire-resistance rating, which is defined as the duration in which a structural member or system exhibits resistance with respect to structural integrity, stability, and heat transmission. However, many of these criteria are developed for fire exposure under normal conditions without cumulative damage from the preceding blast scenario. The study presented here attempts to determine the fire resistance rating of RC axial members subject to post-blast fire. The following conclusions can be drawn based on the results of the numerical study.

- The developed continuum-based finite element model of the reinforced concrete column was validated with experimental studies on both blast and fire. The response measured during the tests agreed well with response predictions made by the numerical model, which shows the reliability of the developed numerical model in evaluating the post-blast fire scenarios of the reinforced concrete members.
- Post-blast fire scenarios can be simulated through a three-step procedure. The gravity loads acting on the reinforced concrete column were simulated as part of the first step, followed by the blast exposure in the second step at room temperature conditions. Finally, a sequentially coupled thermo-mechanical analysis was conducted in the third step.
- A comparison of the response of the column subject to distinct post-blast fire scenarios is contrasted with the fire response of the undamaged column. The initial imperfection in terms of the residual deflection and locked in stresses resulted in a significant reduction in the overall fire resistance period of the blast damaged columns.
- It can be observed that the fire resistance period of post-blast fire columns reduces by 22.3% to 82.8% at scaled distances varying from 2 to 1.1 m/kg$^{1/3}$ showing the severity of the post-blast fire scenario on the fire performance of reinforced concrete members.

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