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Structural Performance of a Five-Story RC Frame under Post-Earthquake Fire

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ABSTRACT

Post-earthquake fires (PEF) constitute a significant risk to civil structures, as they can cause major structural damage. To decrease the risks associated with PEF, it is important to understand how buildings react during such catastrophes. In this paper, a simplified method was used to analyze a five-story RC frame. The method first evaluates the earthquake damage in terms of concrete cover spalling and residual deformations. Assuming different durations of fire exposure, the moment-curvature diagrams for different sections of the frame and the axial stiffnesses of the frame elements are then calculated. Finally, the frame performance is evaluated. The examined frame was found to likely fail rapidly during PEF incidents, thus not allowing adequate evacuation time for the residents.

Keywords: Post-Earthquake Fire, Seismic Spalling Length, Residual Deformation, Distributed Plasticity, Collapse Time.

INTRODUCTION

Performance-based design (PBD) has become widely accepted in structural engineering. Previous studies examined the different performance levels under seismic and gravity loads [1]. However, few studies examined utilizing PBD for structures experiencing multi-loading scenarios such as post-earthquake fire (PEF) incidents [2]–[5].

The literature provides experimental data about the influence of seismic damage on the fire capacity of RC members and frames. Seismic residual deformations were found to reduce the fire endurance of reinforced concrete (RC) columns significantly [6][7] and of RC beams slightly [8]. Kamath et al. [3] and Shah et al. [4] explored the effect of PEF on RC frames. It was concluded that seismic damage increases the temperature within the section core and decreases the lateral capacity of RC frames.

Numerical investigations addressing this topic are based on several assumptions. Vitorino et al. [5], [9], [10] ignored the effects of the seismic residual deformations during the fire stage. Behnam et al. [11], Behnam & Ronagh [12], and Ronagh & Behnam [2] estimated the seismic residual deformations for different performance levels using SAP2000 pushover analysis. All of the mentioned authors estimated the seismic damage based on the description given in FEMA 356 [13] for the different performance levels. Then, they utilized the finite element software SAFIR [14] to define the structural performance during the PEF.

This paper utilizes a relatively accurate and simplified method to examine the structural performance of a 3-bay 5-story RC frame exposed to PEF. The seismic damage and seismic residual deformations are first evaluated. Then, the moment-curvature (M-φ) relationships and axial stiffnesses of the frame's damaged and intact sections are estimated at different fire durations [15]. Finally, the frame was modelled using the distributed plasticity approach to examine its structural performance during the PEF [16].

PROTOTYPE CONCRETE FRAME

The 3-bay 5-story RC frame analyzed by Behnam & Ronagh [12] is explored in this paper. The geometry and dimensions of the frame are presented in Figure 1. The frame was designed based on ACI 318 [17]. The concrete compressive strength and steel yield strength were 25 MPa and 400 MPa, respectively. The applied loads were a dead load of 8.0 kPa and a live load of 2.5 kPa. Behnam & Ronagh [12] assumed the fire to be affecting the first two floors.

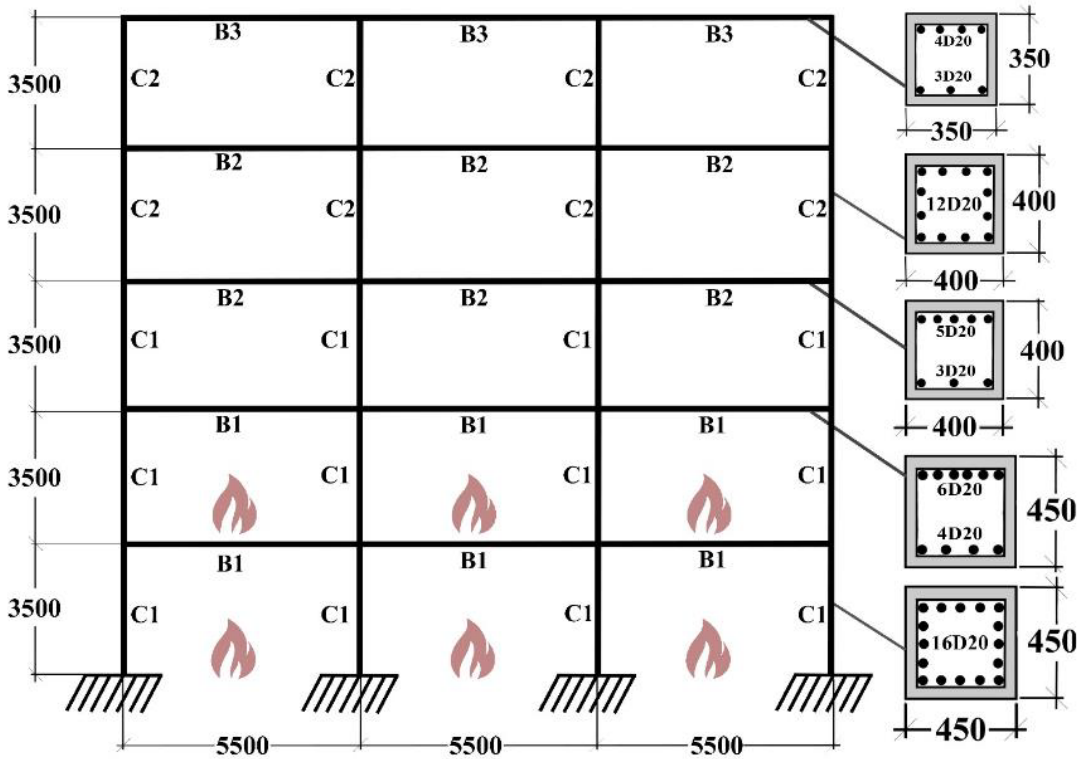

Figure 1. The geometry of the considered frame (All dimensions in mm)

ANALYSIS OF THE FRAME UNDER PEF

This section provides details about the conducted steps to estimate the frame's structural performance during exposure to PEF.

1. Maximum seismic inter-story drift

The maximum roof displacement is assumed 2%, which matches the value chosen by Behnam & Ronagh [12] for the life safety seismic performance level (LS). Instead of conducting pushover analysis to define the inter-story drifts, they were estimated by assuming that the deformation mode follows the first mode of vibration. Their values are given in Table 1.

2. Seismic residual deformations

Behnam & Ronagh [12] applied the lateral loads as a shock followed directly by the PEF, which resulted in high values for the residual deformations. In this paper, the yield drifts, Δ_y, were estimated using the procedure proposed by Priestly et al. [18]. The residual deformations were then estimated using Eq (1) [19], where Δ_{peak} is the maximum inter-story drift. Table 1 presents the yield and residual deformations for each floor.

$$\Delta_r = \begin{cases} 0 & for\ \Delta_{peak} \leq \Delta_y \\ 0.3(\Delta_{peak} - \Delta_y) & for\ \Delta_y \leq \Delta_{peak} \leq 4\Delta_y \\ (\Delta_{peak} - 3\Delta_y) & for\ \Delta_{peak} \geq 4\Delta_y \end{cases} \tag{1}$$

Table 1. Yield and residual deformations for each floor

Floor #	Inter-Story Drifts (mm)	Inter-story Yield Deformation (mm)	Inter-story Residual Deformation (mm)
1	61	38	7
2	70	38	10
3	81	38	13
4	82	43	12
5	56	43	4

3. Concrete damage

The effect of the seismic cracks was not considered in the analysis as they were found not to affect the temperatures within the concrete core [20][21][22]. To estimate the spalling length, the columns and beams of the frame were modelled as cantilevers using the finite element software OpenSees [16]. Each member was discretized into twenty displacement-based beam-column elements to accurately capture the local response. Concrete 01 and steel 02 from the OpenSees library [16] were utilized to model the confined concrete, unconfined concrete, and steel reinforcement. The free ends of the members were pushed to the inter-storey drifts listed in Table 1. Concrete spalling was assumed to initiate at a compressive strain of 0.004 [23][18]. For the examined case, the concrete cover spalling affected the side concrete covers of the columns, the covers perpendicular to the load direction. The spalling lengths for the columns on the first and second floors were evaluated and reported in Table 2. The beams did not experience any concrete cover spalling.

Table 2. Spalling length values for the columns

Column Location		l_{sp} *(mm)*
First Floor	Exterior Column	102
	Interior Column	210
Second Floor	Exterior Column	100
	Interior Column	178

4. Section Characteristics

The modified sectional analysis method [15] was utilized to construct the M-φ relationships and calculate the axial stiffness and the thermal strains of the different structural elements at various standard fire durations. Figures 2, 3 and 4 show example M-φ plots for intact beam B1, intact interior 1st-floor column C1, and damaged interior 1st-floor column C1. Table 3 shows example values for the axial stiffness of the 1st-floor columns and B1. Considering different standard fire durations, Tables 4 and 5 show example values for the thermal axial strains (ε_{th}) of the 1st-floor interior column and the thermal axial strains (ε_{th}) and thermal curvatures (φ_{th}) of beam B1.

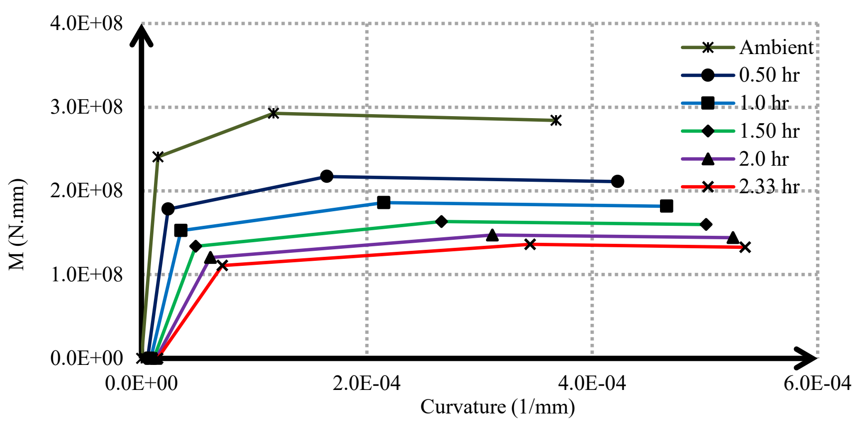

Figure 2. Moment curvature relationships of intact beam B1

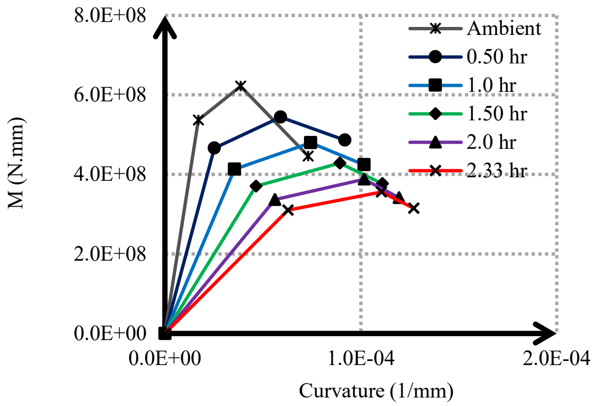

Figure 3. Moment curvature relationships of intact interior 1st-floor column C1

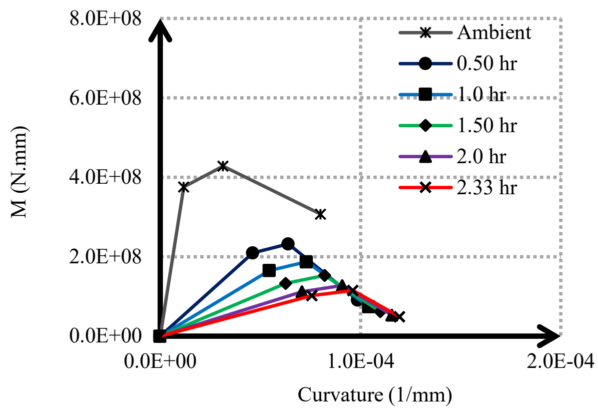

Figure 4. Moment curvature relationships of damaged Interior 1st floor column C1

Table 3. The axial stiffnesses of example frame elements (units in N)

Fire Duration (hour)	Ambient	0.5	1.0	1.5	2.0	2.33
1st-floor Intact Interior C1	4.17E+09	1.86E+09	890E+06	550E+06	448E+06	336E+06
1st-floor Damaged Interior C1	3.55E+09	1.22E+9	636E+6	445E+6	326E+6	269E+6
B1	3.73E+09	1.59E+9	994E+6	742E+6	570E+6	481E+6

Table 4. Axial thermal strains for the first-floor interior column C1

Fire Duration (hr)	**Intact**		**Damaged**	
	ε_{th} (mm/mm)	**Δ_{axial} (mm)**	**ε_{th} (mm/mm)**	**Δ_{axial} (mm)**
0.50	1.96E-03	6.0	5.94E-03	2.5
1.0	3.85E-03	11.9	6.87E-03	2.9
1.5	5.38E-03	16.6	7.54E-03	3.2
2.0	6.54E-03	20.1	7.99E-03	3.4
2.33	7.36E-03	22.7	8.34E-03	3.5

Table 5. Axial thermal strains and thermal curvatures of beam B1

Fire Duration (hr)	ε_{th} (mm/mm)	F_{axial} (N)	φ_{th} (1/mm)	$M_{Fixed\text{-}end}$ (N.mm)
0.50	1.73E-03	2.75E+06	5.46E-06	41.1E+6
1.0	3.04E-03	3.02E+06	8.3E-06	36.0E+6
1.5	4.08E-03	3.03E+06	11E-06	30.2E+6
2.0	4.83E-03	2.75E+06	13E-06	25.5E+6
2.33	5.32E-03	2.56E+06	14.4E-06	20.7E+6

5. Modelling of the seismically damaged RC frame

The frame is modelled, as shown in Figure 5, using the distributed plasticity elements of OpenSees [16]. Each of the beams and columns of the first two floors is divided into four segments. For the damaged elements, the length of the end segments was taken equal to the spalling length. The M-φ relationships that were constructed in step 3 defined the characteristics of the distributed plasticity elements. The residual seismic deformations are imposed on the floors by applying them as inter-story floor displacements. The axial thermal strains in the columns are modelled as induced axial deformations, Δ_{axial} in Table 4. These deformations are calculated as the sum of the axial deformations of the damaged and intact parts of the column. The axial thermal strains and thermal curvatures in the beams are modelled using equivalent axial forces (F_{axial}) and fixed-end moments ($M_{Fixed\text{-}end}$), respectively. Values of these axial forces and moments are given in Table 5.

Results

The frame was loaded by the specified gravity loads and analyzed at different fire durations. Figure 6 shows the comparison between the predicted lateral drift of the first floor and that by Behnam & Ronagh [12]. The analysis was conducted twice, once using the calculated residual drifts and the other using the values used by Behnam & Ronagh [12] The lateral drift-fire duration curve, Figure 6, matched that of Behnam & Ronagh [12] when their residual drifts were utilized. When the final stable residual drifts were utilized, the curve had the same trend, but the frame was able to withstand the fire for a longer duration. If the frame was not seismically damaged, lateral drifts would not have been an issue and failure would initiate either due to beams reaching their flexural capacity or the columns reaching their axial capacity.

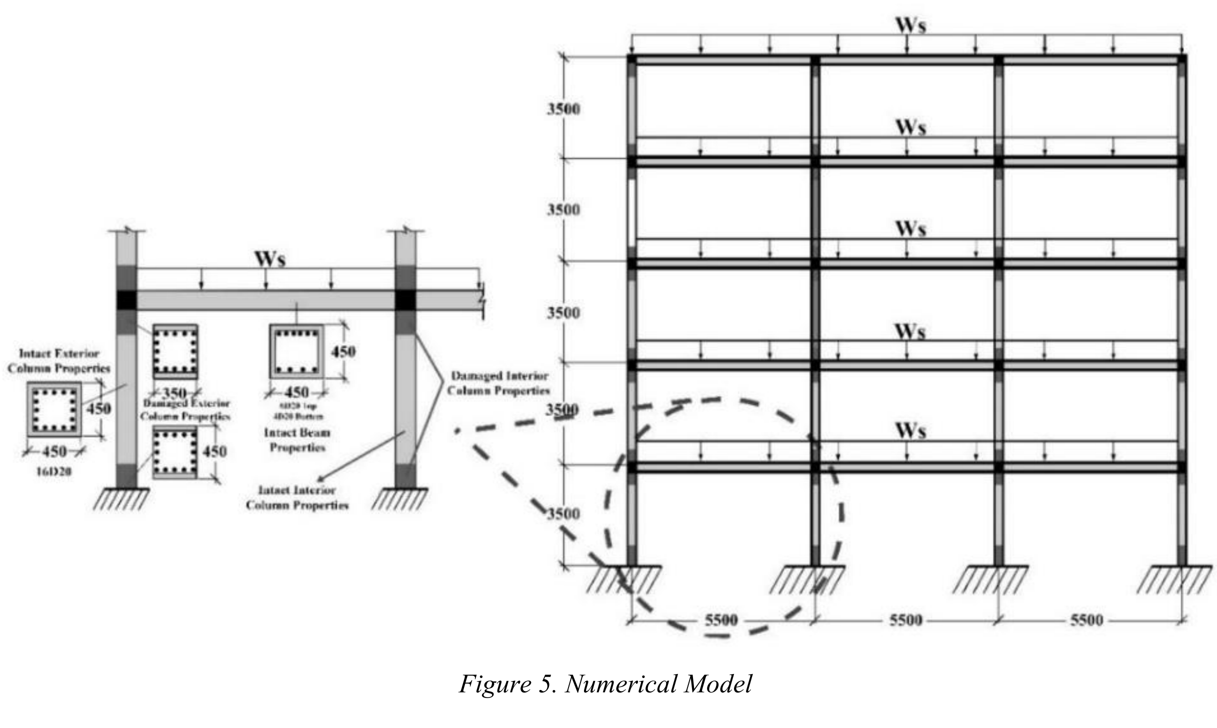

Figure 5. Numerical Model

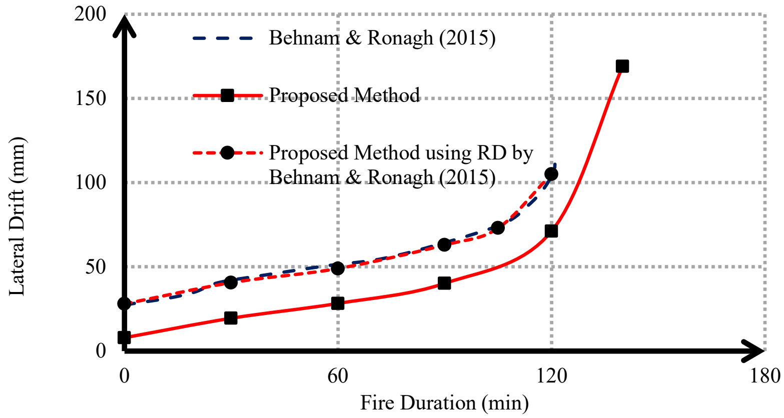

Figure 6. The lateral drift of the first floor

CONCLUSIONS

In summary, a simplified method capable of estimating the structural performance of RC frames under PEF is examined in this paper. It can be easily applied to further examine the performance of RC structures exposed to PEF. The method account for seismic damage and seismic residual deformations. Fire analysis relied on the modified sectional analysis method [15], which makes the user involved in the calculations, instead of having a black box solution. The main conclusions of the study are as follows.

- The use of the distributed plasticity modelling approach is promising in the field of structural fire engineering.
- Representing the residual deformations, the thermal axial strains and the thermal curvatures using equivalent forces or deformations can solve the complications associated with multi-scenario loading.
- Seismic damage alters the $(M - \varphi)$ behaviour of RC sections during fire exposure, where the moment capacity reduces and the curvature increases.
- Seismic residual deformations accelerate the rate of the collapse of RC frames under fire.

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