AN APPROACH FOR POST-EARTHQUAKE FIRE ASSESSMENT OF STEEL MRF BUILDINGS

R. Chicchi¹ and A.H. Varma²

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Structures have traditionally been designed by conducting two dimensional (2D) analyses of structural response to individual hazards. This work considers the cascading hazard, post-earthquake fire or fire following earthquake, by using three dimensional (3D) finite element building models of a 10-story case study building with a perimeter moment frame system. This modeling approach can simulate building behavior that includes inelastic deformations, instability failures, connection damage at elevated temperatures, and the effect of temperature on material strength and stiffness. Seismic loads are applied through the application of nonlinear time history ground acceleration records. Fire loads are modeled using parametric time-temperature curves from Eurocode and two-dimensional heat transfer analysis to determine internal temperatures of structural members. Incremental dynamic analysis as well as incremental fire analysis are used to explore a range of hazard intensities. Results from post-earthquake fire studies at the first story are explained in this paper. Findings show that gravity columns are the most vulnerable component in post-earthquake fire (PEF) hazards, regardless of the level of seismic damage. Seismic damage may cause additional damage but gravity column failure continues to be the initiating failure mode in all fire following earthquake scenarios studied.

¹Assistant Professor, Department of Civil and Architectural Engineering and Construction Management, University of Cincinnati, Cincinnati, OH 45221 (email: Rachel.Chicchi@uc.edu)
²Professor, Lyles School of Civil Engineering, Purdue University, West Lafayette, IN 47907

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Structures have traditionally been designed by conducting two dimensional (2D) analyses of structural response to individual hazards. This work considers the cascading hazard, post-earthquake fire or fire following earthquake, by using three dimensional (3D) finite element building models of a 10-story case study building with a perimeter moment frame system. This modeling approach can simulate building behavior that includes inelastic deformations, instability failures, connection damage at elevated temperatures, and the effect of temperature on material strength and stiffness. Seismic loads are applied through the application of nonlinear time history ground acceleration records. Fire loads are modeled using parametric time-temperature curves from Eurocode and two-dimensional heat transfer analysis to determine internal temperatures of structural members. Incremental dynamic analysis as well as incremental fire analysis are used to explore a range of hazard intensities. Results from post-earthquake fire studies at the first story are explained in this paper. Findings show that gravity columns are the most vulnerable component in post-earthquake fire (PEF) hazards, regardless of the level of seismic damage. Seismic damage may cause additional damage but gravity column failure continues to be the initiating failure mode in all fire following earthquake scenarios studied.

**Introduction**

Post-earthquake fires have recently become recognized in the structural engineering community as a potentially very dangerous hazard scenario in areas of high seismicity. This hazard, known also as fire following earthquakes, is a cascading hazard event wherein a seismic event occurs and causes a fire that follows. This works aims to understand the interrelationships between the two hazard types and the damage they cause through analytical FEM modeling of a case study building. It follows a methodology that has been explained in more detail in Chicchi and Varma [1].

**Case Study Building Design**

The case study building structure was designed using traditional methods in order to later analyze

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\(^1\)Assistant Professor, Department of Civil and Architectural Engineering and Construction Management, University of Cincinnati, Cincinnati, OH 45221 (email: Rachel.Chicchi@uc.edu)

\(^2\)Professor, Lyles School of Civil Engineering, Purdue University, West Lafayette, IN 47907

using more sophisticated, performance-based methods. The design conforms to U.S. building codes and standards: IBC [2], ASCE 7 [3], and AISC 360 [4]. The structure is a 10-story office building, designed for hazard levels associated with Los Angeles, CA.

**Building geometry and structural framing system**

The building consists of a three bay by five bay layout (shown in Figure 1) with each bay measuring 25 ft (7.63 m) x 25 ft (7.63 m). Each story height is 12 ft (3.66 m). Openings for stairs, elevators and utilities have been omitted for model simplicity. The structure employs a moment frame system along the perimeter as the lateral force resisting system. The interior frames of the structure are gravity frames with simple shear tab connections.

![Building framing plan](image)

Each floor system was designed for a 65 pounds per square foot (psf) dead load and 50 psf live load. Typical gravity beams at each level are W14x22 and the interior girders are W18x35. Each floor slab is a 3”-20 gauge (75 mm) composite deck with 2 ½” (65 mm) light-weight concrete topping. The studs, used to develop composite action between the steel and the deck, are ¾” diameter and 4 ½” long. Each beam was designed to achieve a minimum of 25% composite action. Welded wire reinforcement (152 mm x 152 mm MW10) is embedded in the concrete slab. The gravity framing for the roof is the same as the typical floors. Table 1 provides a summary of the member sizes for the primary structural components.

<table>
<thead>
<tr>
<th>Story</th>
<th>MF Beams</th>
<th>MF Columns (Exterior)</th>
<th>MF Columns (Interior)</th>
<th>Gravity Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>9-10</td>
<td>W21x83</td>
<td>W14x176</td>
<td>W14x283</td>
<td>W8x24</td>
</tr>
<tr>
<td>7-8</td>
<td>W24x94</td>
<td>W14x257</td>
<td>W14x311</td>
<td>W8x40</td>
</tr>
<tr>
<td>5-6</td>
<td>W27x94</td>
<td>W14x283</td>
<td>W14x342</td>
<td>W12x58</td>
</tr>
<tr>
<td>3-4</td>
<td>W30x108</td>
<td>W14x342</td>
<td>W14x370</td>
<td>W14x74</td>
</tr>
<tr>
<td>1-2</td>
<td>W30x124</td>
<td>W14x370</td>
<td>W14x398</td>
<td>W14x90</td>
</tr>
</tbody>
</table>

The perimeter moment frames were designed as special moment resisting frames because the structure is classified as seismic design category D. This means that the frames must comply with ductility and detailing requirements per the AISC Seismic Provisions [5]. The equivalent lateral
force procedure, which is a linear static procedure outlined in ASCE 7, was used for the seismic design. Wind loads were calculated using the Main Wind Force Resisting System Directional Procedure (Chapter 27) in ASCE 7-10 [3]. The basic wind speed is 110 mph and the exposure category is B. The building base shear due to wind is 388 kips and 805 kips due to seismic.

Fire protection design
The fire resistance rating for the structural members is determined from the International Building Code [2]. The structure is classified as building occupancy B and Type IIA. This corresponds to a 1-hour fire resistance rating (FRR) for all structural members. The thickness of fireproofing required to achieve a 1-hour FRR for each structural member was determined using the prescriptive W/D approach outlined in Ruddy et al. [6] based on fire tests conducted by Underwriters Laboratory [7]. The composite deck is not fire protected, as it can already achieve the performance rating without fireproofing.

FEM Modeling Approach

ABAQUS, a commercially available finite element method software, was used to develop three-dimensional (3D) models of the case study building structure. These nonlinear inelastic models can simulate inelastic deformations, instability failures, connection damage at elevated temperatures, and the effect of temperature on material strength and stiffness. This modeling approach provides a way to assign time history records and temperature changes directly to the model in order to analyze the effects of seismic and fire hazards in a more sophisticated manner than traditional, simplified design and analysis methods. The modeling approach was adapted from Agarwal [8], who created the building model to analyze a structure subjected to compartment fires.

Beams and columns were modeled using 2-node beam elements that are an approximation of a 3D solid element using Timoshenko beam theory. The composite slab was modeled using 4-node, reduced integration shell elements. The composite slab was conservatively modeled as 2 1/2” thick light-weight concrete, to match the concrete above the deck flutes. The steel deck is idealized as rebar embedded in the slab, using the built-in embedded rebar option in ABAQUS. The rebar area matches the area of the steel deck and is applied only in the strong direction of the deck. The shear studs are modeled using rigid connectors. A schematic of the modeling approach for the composite floor system is shown in Fig. 2.

The perimeter of the building consists of moment frames with fixed beam to column connections. These connections were modeled as rigid connectors. This simplification was justified based on work by Yang et al. [9] which found through experimental testing that the tested moment connections maintained design strength up to 650°C and with only a 25% reduction in stiffness. When subjected to seismic loading, it was presumed that the connections were designed with adequate strength and ductility for the plastic hinges to form outside of the connection region. With this assumption, the moment-rotations at the joints are tracked to assess connection failure. More detailed modeling of the stud connections and moment connections, including potential failure modes, should be incorporated in future work.

The simple (shear) connections in the building were modeled as shear-tab connections using wire connector elements that include the axial force-axial displacement-moment-rotation-temperature behavior, based on work by Sarraj [10] and Agarwal [8]. During the seismic loading step, idealized pinned connections were used to represent gravity connections instead of the equivalent gravity connectors. Work by Chi et al. [11] showed that the gravity frame contribution to the lateral stiffness of the structure was primarily attributed to the gravity columns themselves
and not the gravity connections and, thus, this was viewed as an adequate simplification to the model which allowed for ease of importing between the seismic and fire building models. The material models for steel and concrete were developed using Eurocode, which considered stress-strain relationships at elevated temperatures. For steel, elastic behavior and inelastic behavior with isotropic hardening were captured at ambient and elevated temperatures. The material model for concrete is based on two failure mechanisms: tensile cracking and compressive crushing. ABAQUS provides a built-in concrete damaged plasticity model which predicts this behavior.

![Figure 2. Schematic of modeling approach for composite floor system [8].](image)

**Hazard Selection and Application**

**Earthquake Selection and Scaling**

Ground motions were selected based on actual earthquake records obtained from the online PEER database [12]. Seven notable earthquakes in the Los Angeles, CA vicinity were selected, as shown in Table 2. Each record was scaled in order for the mean value of the records to be greater than the design response spectrum in the range of 0.2T to 1.5T.

<table>
<thead>
<tr>
<th>EQ #</th>
<th>Event Name</th>
<th>Station Name</th>
<th>Year</th>
<th>Magnitude</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley-02</td>
<td>El Centro Array #9</td>
<td>1940</td>
<td>6.95</td>
<td>2.1</td>
</tr>
<tr>
<td>2</td>
<td>Kern County</td>
<td>Taft Lincoln School</td>
<td>1979</td>
<td>7.36</td>
<td>3.0</td>
</tr>
<tr>
<td>3</td>
<td>San Fernando</td>
<td>Castaic – Old Ridge Route</td>
<td>1971</td>
<td>6.61</td>
<td>3.3</td>
</tr>
<tr>
<td>4</td>
<td>Imperial Valley-06</td>
<td>Aeropuerto Mexicali</td>
<td>1989</td>
<td>6.53</td>
<td>2.5</td>
</tr>
<tr>
<td>5</td>
<td>Mammoth Lakes-02</td>
<td>Mammoth Lakes H.S.</td>
<td>1994</td>
<td>5.69</td>
<td>3.8</td>
</tr>
<tr>
<td>6</td>
<td>Loma Prieta</td>
<td>Palo Alto –SLAC Lab</td>
<td>1952</td>
<td>6.93</td>
<td>2.0</td>
</tr>
<tr>
<td>7</td>
<td>Northridge</td>
<td>Santa Monica City Hall</td>
<td>1980</td>
<td>6.69</td>
<td>2.4</td>
</tr>
</tbody>
</table>

**Incremental Dynamic Analyses**

The nonlinear dynamic procedure can be used to design and analyze structures for specific ground motion records; however, this does not give an adequate picture of the progression of behavior. Therefore, incremental dynamic analysis (IDA) is commonly used in conjunction with the nonlinear dynamic procedure to create a parametric analysis of building behavior to seismic loads. In the IDA approach, each ground motion record can be scaled using an intensity measure (IM) to generate response curves. Once the demand is determined through scaling of the IM, the capacity must be evaluated through a damage measure (DM). This procedure of incrementally scaling multiple ground motions aims to capture the variability in amplitude and frequency of the building response, providing smaller dispersion and, thus, increased confidence in the range of results. PGA
Fire Selection and Scaling

Fire Location
There is a lot of uncertainty in defining and locating the fire hazard. Engineering judgement must be used to determine the critical locations for analysis while remaining realistic about the likelihood of each fire event. For the purposes of this study, only compartment fires are studied. In this context, each compartment represents one bay of the structure. Compartment fires were explored at a lower (1st), mid (5th) and upper (9th) level of the building, studying both interior and exterior compartments. Only first floor results will be presented in this paper.

Three compartments were analyzed. These compartments are classified as Case A (corner compartment), B (exterior compartment in third bay of long size of building), and C (interior, center compartment), as shown in Fig. 3. When the ground motion was prominent in the transverse direction, Case B2 was studied instead of Case B.

Figure 3. Compartment fire locations.

Fire time-temperature curve
Parametric time-temperature curves were developed using Annex A of Eurocode 1 [13] for compartment fires. These parametric curves are preferable to other standard curves (ASTM E119 and ISO 834) because they include a cooling phase and vary depending on three factors: thermal inertia of the enclosure (b), opening factor (O), and fire load density (q_{t,d}). Three fire time-temperature curves were selected, as shown in Fig. 4.

Figure 4. Fire time-temperature curves used in analyses.

\[ [O(m^{1/2}), b(J/m^3s^{1/2}K), q_{t,d}= (mJ/m^2)] \]
These represent short, mid and long duration fires. The heating phase of Fire 1 matches ISO 834 and ASTM E119 standard time-temperature curves. Results from only Fire 1 will be presented in this paper.

**Heat Transfer Model**

In order to determine the internal temperatures of the structural members, ABAQUS was used to conduct 2D heat transfer analyses. The cross-section of each member in the compartment fire was modeled as a 2D part and exposed to each parametric time-temperature curve. Fire-proofing was modeled at the thickness required to achieve a 1-hour fire resistance rating per the prescriptive method, using thermal properties provided in Ruddy et al. [6]. The temperatures within a wide-flange section are recorded at five locations across the section (known as integration points), as shown in Fig. 5. These nodal temperatures are then input into the building model at five minute intervals.

**Incremental fire analyses**

The incremental dynamic analysis approach used in earthquake engineering can be applied to fire scenarios using different intensity measures and damage parameters. Unlike IDA which commonly uses PGA or $S_a$ as the intensity measure, there does not seem to be a consensus among researchers on the most indicative intensity measure to use for fire. Ultimately, peak fire temperature within the compartment was the chosen intensity measure. For each fire time-temperature curve, the temperatures were scaled by 0.75, 1, and 1.5, and the vertical story deflection was recorded as the damage parameter. While this approach is beneficial for capturing a range of fire intensities, peak fire temperature does not necessarily correlate with failure, as very short, hot fires may not cause high temperatures within the structural members. Additional care should be taken to select an alternative fire intensity measure, such as fuel load density, if fragility functions are going to be created from the results.

**IDA Results**

Ground motion intensities of 0.75, 1, and 1.25 times the PGA for each earthquake were studied. The designation EQ2-1 indicates earthquake ground motion 2 scaled by 1 times the PGA. Fig. 6 shows the maximum story drift ratios that were recorded at each level for the seven different earthquakes scaled by different PGAs. The earthquake ground motions with PGA*0.75 (Fig. 6(a)) resulted in story drift ratios below 0.03, with many of the earthquake responses less than 0.015. The only earthquakes that resulted in drift ratios greater than 0.015 were ground motions 1, 4 and 7. Fig. 6(b) shows the drift ratio response of the structure when subjected to 1 times the PGA of the design basis earthquake. Again, most drift ratios are below 0.015 with the exception of earthquakes 1, 4, and 7. According to the Seismic Design Manual [11], special moment frame connections must be capable of withstanding 0.04 radians of interstory drift. Earthquake 7 surpasses this limit slightly.
Results have shown that earthquakes 4 and 7 are strong ground motions that can cause deflections at or beyond the acceptable criteria for special moment frame systems. Alternatively, four of the ground motions are relatively weak, not resulting in very much deformation or damage. The current scaled earthquakes were deemed acceptable because they cover a range of earthquake intensities. However, more careful scaling of ground motions could have been conducted in order to scale all seven ground motions so that they more closely fit the response spectrum.

As explained previously, the moment frame connections were modeled as fixed connectors which cannot fail. However, ductile damage was incorporated into the material models in order to simulate fracture at interstory drift ratios beyond 0.04. Whenever these interstory drifts exceeded this limit, fracture occurred. Fracture occurs primarily in the moment frame beams within the protected zone, particularly at the lower three levels of the structure.

**IFA Results**

The results of Fire 1 at the first story will be discussed. The three primary failure modes that were observed are: column failure, bay failure, and system collapse. Renderings of these failure modes are shown in Table 3. Column failure occurs when a column is no longer able to support axial loads. Bay failure is when two or more columns fail, resulting in deformations across the bay. In column and bay failures, sagging of the adjacent bays occurs, but system collapse is avoided through redistribution of the failed column loads. System collapse is defined as member failures that result in collapse of more than one bay of the structure. While avoiding system collapse is paramount, efforts should be made to minimize the level of damage within the structure due to fire. This can be achieved by preventing column collapse through implementation of more robust gravity columns or increased fireproofing.

Table 4 shows failure modes and times of failure for Fire 1 scaled at 0.75, 1 and 1.5 times the peak fire temperature for corner (A), edge (B) and interior (C) compartments. These results are for all post-earthquake fire scenarios where the earthquake did not cause moment frame beam-to-column connection fracture. These are also the results of the fire-only analysis scenario. The designation Fire 1-1.5 indicates Fire 1 (per Fig. 4) scaled by 1.5 times the peak fire temperature.
Table 3. Fire and post-earthquake fire failure modes.

<table>
<thead>
<tr>
<th>Failure Type</th>
<th>Failure Mode Renderings</th>
<th>Plan View (showing extent of deflection)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Failure</td>
<td><img src="image" alt="Column Failure" /></td>
<td><img src="image" alt="Plan View" /></td>
</tr>
<tr>
<td>Bay Failure</td>
<td><img src="image" alt="Bay Failure" /></td>
<td><img src="image" alt="Plan View" /></td>
</tr>
<tr>
<td>System Failure</td>
<td><img src="image" alt="System Failure" /></td>
<td>Varies</td>
</tr>
</tbody>
</table>

No failure is observed for the Fire 1-0.75 case, where the peak fire temperature reaches 690°C. For Fire 1-1, interior gravity column failure (as shown in Table 3) occurs at 98 minutes at the corner compartment. As the W14x90 gravity column is subjected to elevated temperatures, its strength is reduced and it eventually buckles as it reaches its load carrying capacity. The framing system is able to redistribute loads to adjacent columns in order to prevent system collapse. The column maintains essentially constant axial load despite exposure to high temperatures. This is because the gravity column is relatively unrestrained. A moment frame column exposed to the same temperatures would experience increased axial load due to restrained thermal expansion caused by the fixed connections framing into the column. When the gravity column reaches an internal temperature of 524°C, it collapses.

Table 4. Post-earthquake fire failure modes and times of failure for Fire 1

<table>
<thead>
<tr>
<th>Location</th>
<th>Fire 1-0.75</th>
<th>Fire 1-1</th>
<th>Fire 1-1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Failure Type</td>
<td>Time of Failure (min)</td>
<td>Failure Type</td>
</tr>
<tr>
<td>1A</td>
<td>none</td>
<td>-</td>
<td>Column</td>
</tr>
<tr>
<td>1B</td>
<td>none</td>
<td>-</td>
<td>Bay</td>
</tr>
<tr>
<td>1C</td>
<td>none</td>
<td>-</td>
<td>System</td>
</tr>
</tbody>
</table>
At compartment 1B, two gravity columns are exposed to the compartment fire, causing gravity column buckling at its critical temperature. This response is the same as compartment 1A except that two columns buckle at once, causing more widespread damage (see Table 3). The interior compartment (1C) exposes four gravity columns to the compartment fire, resulting in failure of all four columns at once. The slab and framing system must try to span three bays which leads to large deformations and tensile forces that cannot be resolved. Thus, system collapse occurs. Because the system does not fail until well after the 1-hour resistance rating for which it was designed (allowing for adequate time for occupants to escape), its design criteria is achieved. However, when evaluating the structural resilience of the system, which relates to the speed of recovery after a hazardous event, the above mentioned failures are not resilient because they would require significant repair and replacement of structural members. Efforts should be made to increase structural resilience by decreasing the likelihood of these three failures from occurring.

The same failure modes are observed for Fire 1-1.5. The time of failure occurs more quickly (at 50 minutes) because this is the time that it takes for the high intensity fire to heat up the column to 524°C.

These studies show that, for structural systems with members that are all designed for a 1-hour FRR, gravity columns are the most vulnerable component within the structure regardless of the level of seismic damage. This is because gravity columns have a high demand to column capacity ratio at ambient conditions, known as the utilization ratio. This ratio increases as elevated temperatures lower the column capacity, eventually resulting in column collapse. The utilization ratio for the W14x90 gravity column at the first story is 0.60 as compared with only 0.07 for the W14x398 moment frame columns. Because of the vulnerability of the gravity columns, seismic and fire damage is decoupled from one another, and the influence of seismic damage on fire resilience in this case study example is negligible.

Based on these studies, gravity column failures consistently initiate the first failure mode; however, in high seismic areas, fractures within the moment frame system may lead to an additional, subsequent failure mode (such as system collapse) after column failure. Specifics of these findings will be provided in future work.

Conclusions

The vast majority of post-earthquake fire studies to date have been modeled in two dimensions, neglecting potential failure modes within the gravity framing system. This paper shows that this role should not be overlooked, as gravity columns are extremely vulnerable to fire and post-earthquake fire hazards, particularly in cases where all members are protected to the same level of fire resistance rating. Gravity column sizes or fireproofing thickness can be increased to make the columns more resistant to thermal loads. By preventing column failure modes from occurring, the overall multi-hazard resilience of the system can be improved. While this work is limited to a specific case study building, the modeling approach and methodology can be applied to different structural systems with varying hazards and fire resistance ratings.

Acknowledgments

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References


