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A Framework for Performance-Based Fire Following Earthquake Engineering

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ABSTRACT: The occurrence of fires igniting during and immediately following a seismic event represents an important design scenario that should be accounted for. Building concentration, construction type, weather conditions, and other factors can combine to create a situation in which fire following an earthquake is the principal cause of damage. Records from past earthquakes show that the damage caused by the subsequent fire can be very significant, often exceeding the damage caused by the earthquake. The current seismic design philosophy permits certain degree of damage during earthquakes, making the structures more vulnerable when exposed to the additional demand of fire loading. Fire resistance of steel frames is implemented using passive or active fire resisting systems, which have shown high variability in their sustained damage due to the seismic event. In this paper, a new framework for performance-based fire following earthquake engineering is developed and discussed. The framework is established through combining stability analysis of isolated columns with system-level finite element analysis of a steel building while accounting for randomness in parameters associated with post-flashover fire, passive fire protection, and mechanical loads. Fragility surfaces for column instability as a function of various levels of fire load density and inter-story drift ratios are produced. The results demonstrate that instability can be a major concern in steel structures, both on the member and system levels, under the sequential events and highlights the need to develop provisions for the design of steel structures subjected to fire following earthquake.

To date, limited studies have been conducted to probabilistically evaluate the performance of steel members and systems under fire loading. Assessment of structural members and systems under fire following earthquake is, however, lacking. Iqbal and Harichandran (2010) proposed a reliability-based framework to determine resistance and load factors in fire design of structural members. The study pertained to the statistics of effective random variables in fire design of structural members. Guo et al. (2013) developed a probabilistic framework to assess the fire resistance of structural members considering uncertainties associated with fire load and structural resistance parameters. The developed framework was demonstrated by analyzing a protected steel beam in order to determine probability of failure at a given level of uncertainty in natural fire event. A study has been conducted by Lange et al. (2014) to establish performance-based structural fire engineering based on extensive work conducted on performance-based earthquake engineering. Hazard, structural system, and loss domains were defined in accordance with structural fire engineering.

The above discussion sheds light on some relevant work that has been conducted on probabilistic structural fire engineering. This paper intends to expand previous studies on probabilistic structural fire to the area of fire following earthquake. Specifically, we will present a new probabilistic framework to assess performance of steel structural members and systems subjected to the cascading hazards of earthquake and fire given uncertainties in fire hazard, passive fire protection material properties, applied mechanical loads, and earthquake hazard while considering stability as a damage limit state. The usefulness of this framework is in that it can provide means by which structural design engineers can assess alternative design scenarios and select the preferred design option based on a desired probability of failure for the combined fire scenarios.

1. PERFORMANCE-BASED FIRE FOLLOWING EARTHQUAKE

The objective of performance-based engineering is to achieve a structural system, which satisfies the needs of the user, stated in terms of performance levels at different hazard levels. In the present study, a performance-based engineering framework is proposed for the cascading hazards of earthquake and fire by combining both PEER framework for earthquake hazard, Figure 1(a), and the adopted framework for fire hazard, Figure 1(b). Although this adoption for fire hazard is straightforward in concept; the definition of variables involved in the framework, e.g. intensity measure, engineering demand parameter, and damage measure, remains quite challenging due to the complex nature and extreme variability in both fire loading and performance objective levels.

Buildings need to meet minimum requirements of resistance to fire loads in terms of insulation, integrity, and stability (Lange et al., 2014). The current prescriptive fire design of steel structures is typically limited to specifying thickness of insulation materials for specific duration of fire. This prescriptive method does not take into account the probability of failure for the active or passive fire systems and the subsequent impact on integrity and stability of structural members and systems as a whole. Since the present study is concerned with the multiple hazards of earthquake and fire, stability will be the primary focus in the developed framework as discussed later. Insulation and integrity can be satisfied by non-bearing elements of building and stability can be provided by structural members and system as a whole such that objectives of safe evacuation of occupants and protection of property are met.

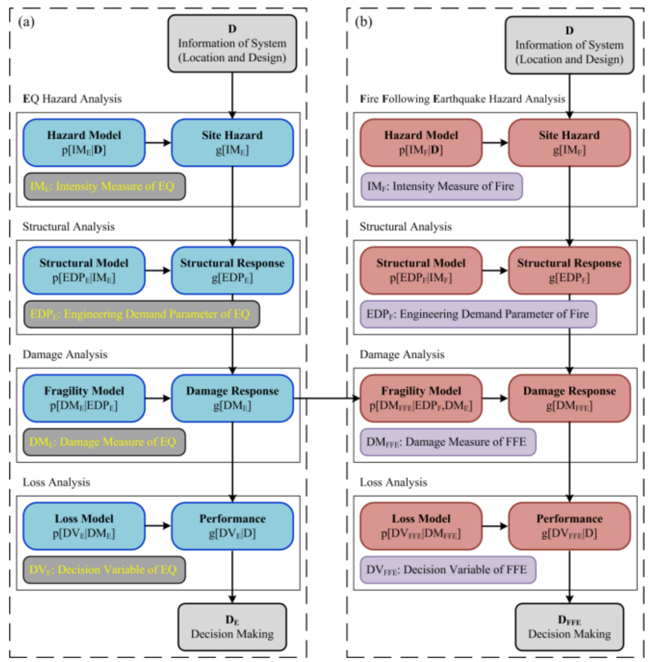

Figure 1: Performance-based fire following earthquake (PBFFE) engineering framework, a combination of (a) existing performance-based earthquake engineering framework and (b) inspired performance-based earthquake engineering for fire following earthquake multi-hazard.

An effort is placed in this paper on investigating the first 3 domains (hazard, system, and damage) of performance-based engineering for post-earthquake fire hazard, shown in Figure 1(b) in order to identify an appropriate variable for each of the domains. The intensity measure is defined by a hazard curve demarcated by frequency of exceeding an intensity measure. Fire load density could perhaps be the most suited parameter as an intensity measure for performance-based fire engineering. It is noted that the developed performance-based fire following earthquake (PBFFE) framework is devised based upon the concept of no correlation between the intensity measure of earthquake such as spectral acceleration and the intensity measure of fire such as fire load density. This is because although fire ignition after an earthquake highly depends on the intensity of the earthquake, the fire growth to flashover condition is completely independent of the earthquake intensity and is rather dependent on available fuel load and ventilation conditions of fire compartments.

The system domain analysis requires the selection of an appropriate engineering demand parameter (EDP) for performance-based engineering of post-earthquake fire hazard. In this study, stability of steel frames is chosen as the focus for the performance evaluation. Previous studies (Memari and Mahmoud, 2014; Memari et al., 2014; Mahmoud et al., 2015) have demonstrated that axial forces are the dominant demand parameter in steel structural members under elevated temperatures. The combination of axial forces in members and the subsequent effects on stability of structures could be utilized as an appropriate EDP for performance-based analysis in order to satisfy one of three parameters – insulation, integrity, and stability – required for a desired level of structural performance under fire loads.

A key step in probabilistic analysis is to define appropriate damage measures. The damage function, $g(X)$, is defined as follows in the present study

$$g(X) = \frac{S_f(X)}{R_f(X)} \tag{1}$$

where, X denotes a vector contains all random variables, $S_f(X)$ is the demand on steel column, and $R_f(X)$ is capacity of column according to the inelastic buckling stress. Failure occurs when the ratio of demand, $S_f(X)$, to capacity, $R_f(X)$, of the system is equal or greater than 1. The probability of failure is then defined as

$$P_f = P[g(X) \geq 1] \tag{2}$$

Calculating the probability of failure requires the identification of a damage measure. In this study, the onset of instability in vertical structural members (columns) is defined as damage measure under fire conditions. The capacity of columns, $R_f(X)$, is determined based on inelastic buckling stress (Memari et al., 2017) and the demand, $S_f(X)$, is evaluated according to the applied mechanical and thermal loads. Inter-story drift ratio in steel columns caused by earthquake demands can result in significant reduction in the inelastic buckling capacity of columns at elevated temperatures. Therefore, the damage measure in fire following earthquake, defined as the onset of instability in steel columns, depends on the level of inter-story drift ratio and EDP under fire loading, which is the axial force. This results in correlation between earthquake and fire hazards, which has been addressed in the damage domain of the framework for PBFFE shown in Figure 1. The equation of PBFFE can therefore be written as shown below:

$$\begin{aligned} &g(DV_{FFE}|D) = \\ \int\int\int\int\int\int \; &p(DV_{FFE}|DM_{FFE}).\, p(DM_{FFE}|EDP_F, DM_E).\, p(EDP_F|IM_F). \\ &g(IM_F|D).\, p(DM_E|EDP_E).\, p(EDP_E|IM_E).\, g(IM_E|D). \\ &d(IM_E)d(EDP_E)d(DM_E)d(IM_F)d(EDP_F)d(DM_{FFE}) \end{aligned} \tag{3}$$

in which,

$$\begin{aligned} &g(IM_F|D) = \\ &\iint p(IM_F|Flashover).\, p(Flashover|Ignition) \ldots \\ &.\, g(Ignition).\, d(Ignition)d(Flashover) \end{aligned} \tag{4}$$

All variables in Eq. (3) are defined in Figure 1. In summary, the performance-based framework introduced above forms the basis for the probabilistic analysis conducted to assess fragility of steel columns and systems at onset of instability as a damage measure. The subscript of E and F implied earthquake and fire hazards, respectively. In Eq. (4), g indicates the annual rate of an event; p denotes the complimentary cumulative distribution function of an event.

2. FRAGILITY SURFACE OF STEEL COLUMNS

The most important parameters in fire design of steel structural members are chosen as random variables for probabilistic analysis. For fire severity fire load density, compartment ventilation, compartment geometry, and thermal characteristics of surrounding surfaces are the most critical. The fire load density depends on amount, type, distribution, and characteristics of surfaces in the boundaries of the compartment according to Phan et al. (2010) and SFPE (2004). The gas temperature of fire in the compartment varies in time, t, considering energy equilibrium, opening factor, fire load density, and thermal properties of material in compartment enclosures (SFPE, 2004). In this study the random variables selected are 1) the fire load density, 2) the spray-applied fire resistive materials, and 3) the applied loads. Detailed discussion of these random variable and their treatment in the analysis can be found in Memari and Mahmoud (2018). Ultimately, the statistical treatment of these variables allows for the development of various randomly generated time-temperature curves that can be used in the simulations as shown in Figure 2.

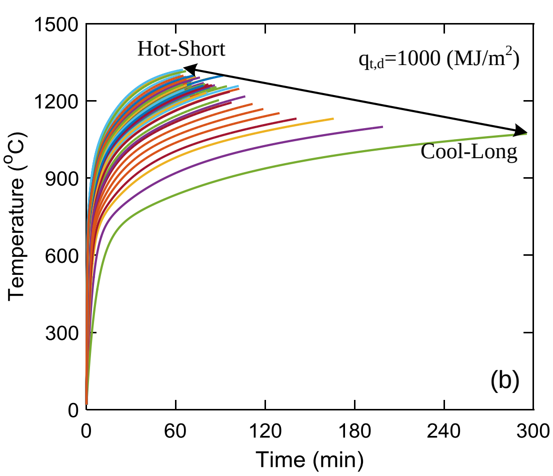

Figure 2: A set of time-temperature curves with constant fire load density of 1000 MJ/m^2.

In the present paper, the fragility analysis is conducted on steel columns by considering their instability under the cascading hazards of earthquake and fire. The developed axial force in the steel columns under specific inter-story drift ratio and elevated temperatures is selected as the predominant response parameter of steel columns under fire following earthquake. The damage measure is defined when the combined axial force demands exceeds the axial inelastic buckling capacity of the steel column. The longitudinal temperature distribution is needed to calculate the thermal demand on steel columns along with determining their capacity, which can be found by solving the conduction partial differential equation (PDE) in time considering the time-temperature curves generated for the body of steel columns.

Once the conduction transient heat transfer analysis is performed, the demand on the column and its capacity under non-uniform longitudinal temperature distribution should be determined. The capacity is determined using a finite element formulation that allows for the evaluation of buckling stress of steel columns at any inter-story drift ratio and arbitrary uniform/non-uniform longitudinal temperature profiles (Memari et al. 2017; Memari, 2016). The demand on the steel columns is assessed based on three various sources: (1) applied mechanical axial dead and arbitrary-point-in-time live loads; (2) the moment demand on steel column caused by earthquake loads if it is a member of a moment-resisting system, otherwise it will be negligible; and (3) thermal loads due to post-earthquake fire.

For the probabilistic analysis, a Monte Carlo simulation (Ang and Tang, 2006) is conducted using Latin Hypercube Sampling (LHS) method to reduce computational intensity of the simulation. The probability of failure P_f is defined as follow:

$$P_f = N_f / N \tag{8}$$

where, N_f is number of simulations in which the column fails based on a defined damage measure, and N is total number of simulations. To conduct the analysis, first a numerical model with geometrical representation of the column is developed. A displacement-controlled analysis is performed to apply a determined level of inter-story drift as an earthquake demand on column. This is performed similar to a nonlinear static pushover analysis where the column deformation at the end of conclusion of the lateral displacement analysis is considered an initial condition for post-earthquake fire loads.

In this study, a set of 100 fire curves at certain level of fire load density is generated. These 100 fire curves are converted to time-temperature curves in the body of the steel column considering samples of spray-applied fire resistance material (SFRM) described previously. A set of 100 axial demand forces is also generated according to random variation of mechanical loads. The finite difference method is employed to obtain non-uniform longitudinal temperature profiles in the steel column for each of 100 time-temperature curves. In the next step, 100 analyses are conducted to obtain the capacity of the column at specified level of inter-story drift along with the axial force demand. At this step, demand in the column caused by earthquake and fire in all 100 cases is compared to the column capacity. The repeat of this process for all levels of inter-story drift ratios and fire load densities results in fragility surface of the column under fire following earthquake according to the framework outlined in Figure 1.

To explain the process of aforementioned probabilistic analysis, an interior column is selected in the 2nd story level of 3-story moment-resisting frame, identified in the SAC steel project (FEMA 354, 2000) and designed according to the Uniform Building Code (UBC 94, 1994) for Los Angeles region. The W14×311 section and 3.96 m long column is numerically modeled. The design dead and live loads are obtained based on tributary area of the selected column according to FEMA-355C (2000).

The results of the Monte Carlo simulation for 3% inter-story drift ratio are shown in Figure 3. In this figure, each set of markers – associated with a particular fire load density – shows the values of damage function, g(X), for a set of 100 fire curves. If fire curves are labeled from 1 to 100, the values of damage function associated with each fire curve (say fire curve “i”) are connected to each other with solid color lines. The horizontal solid black line indicates the damage limit state, g(X)=1, when demand and resistance are equal. Markers located above the solid black line are an indication of failure, and those under the black line imply no failure. The same plot can be generated for the other inter-story drift ratios.

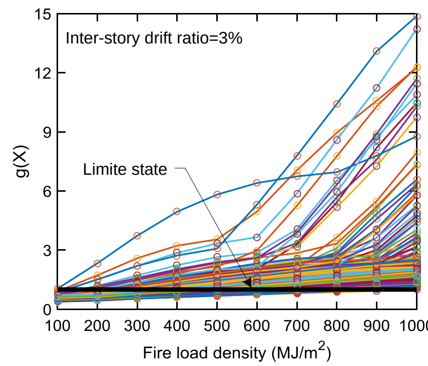

Figure 3: Value of damage function, g(X), for the selected column in various levels of fire load density at 3% inter-story drift ratio.

The probability of failure for 3% inter-story drift ratio along with the rest of inter-story drifts is plotted in Figure 4. It is seen that the probability of failure increases when the fire load density and inter-story drift ratio increase as expected. For instance, the probability of failure given 5% inter-story drift ratio is approximately 100% for all range of fire load densities. This implies that the probability of failure in a column with 5% inter-story drift is 100% with or without fire loads. The probability of failure in the steel column with 4% inter-story drift with no fire load is about 30% and is 100% at fire load density of 500 (MJ/m^2) and greater. In case of 2% inter-story drift ratio, the probability of failure reaches 84% for a maximum fire load density of 1000 (MJ/m^2). The probability of failure given 0 and 1% of inter-story drift ratio is 0.10 and 0.15, respectively, for fire load density of 1000 (MJ/m^2) as shown in Figure 4. Figure 4 expresses the probability of failure for multiple-hazard of fire following earthquake given damage measure of earthquake (inter-story drift ratio) and intensity measure of fire (fire load density).

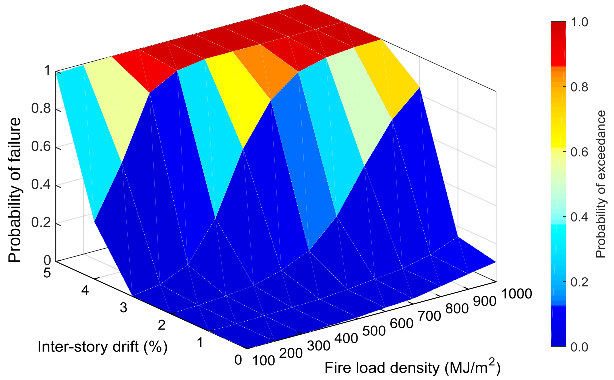

Figure 4: 3-D fragility surface.

3. CONCLUSIONS

In this paper a performance-based engineering framework was proposed to assess the performance of steel structures subjected to multiple-hazards of earthquake and fire. The framework can provide means by which structural design engineers could assess alternative design scenarios and select the preferred design option based on a desired performance level. Uncertainties associated with fire hazard, passive fire protection system, applied gravity loads, and earthquake hazard were considered. Monte Carlo simulations were employed to develop fragilities of steel columns subjected to fire following earthquake.

Fire load density was recommended as an intensity measure in the performance-based fire engineering framework. Axial force was suggested as an appropriate engineering demand parameter. The onset of instability in steel columns was chosen as damage measure. To clarify the process of the development of fragility analysis for a column, an interior column was selected in the 2nd story level of 3-story moment-resisting frame. It was assumed that post-earthquake fire occurs in the second (from left) bay of this story. It was observed that the number of failed cases increased as the fire load density increased. The value of damage function, g(X), increased with increase in the fire load density. This was more significant for fire load density of 500 (MJ/m^2) and larger. The damage function varied between 0 and 1 for fire load density of 100 (MJ/m^2), while its variation fell between 1 and 15 for fire load density of 1000 (MJ/m^2).

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