ADVANCED METHODS FOR PERFORMANCE-BASED ASSESSMENT OF STEEL BUILDINGS UNDER THE EFFECTS OF EARTHQUAKE AND FIRE

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Abstract

Fire ignitions in buildings during or immediately following an earthquake have been noted in previous seismic events. Records from past earthquakes show that the damage caused by the subsequent fire can be substantial, often exceeding the damage caused by the earthquake. The current earthquake design limit states allow structures large degrees of damage accumulation under major earthquakes as long as lives of the occupants are protected. While this design approach has been proven successful in ensuing life safety, permitting large damage accumulation in the structure might result in fire ignition, subjecting the structure as well as the occupants to high risk of fire exposure. As such, it is imperative to consider the combination of these loading scenarios in analysis and design of structures in high seismic regions. The development of guidelines and provisions require advances in analysis and testing tools that support understanding the performance of individual components along with vulnerability of structural systems under fire following the earthquake. In this paper, a set of analysis methods that have been developed and utilized to evaluate the performance of structural elements and systems subjected to fire following seismic events will be introduced. These methods comprise of analytical formulations on structural components, detailed finite element models of structural systems, and hybrid simulations. Code provisions that facilitate design procedure for such loading scenario, developed through the integration of the results obtained from these various methods, will also be introduced. Moreover, the paper will highlight the role of simplified finite element models as well as advanced experimental methods in understanding the response of steel structures as a system under the sequential hazards. A new framework for performance-based fire following earthquake engineering will also be presented. This framework allows for the integration of all mathematical and numerical tools while taking into account uncertainty in critical variables that impact member and system vulnerability under such cascading event. These variables include post-flashover fire, passive fire protection, structural resistance, and mechanical loads. The results demonstrate the benefit of conducting the component-level analysis and the value in integrating various methods for vulnerability assessment at the system level.

Keywords: fire following earthquake; finite element; numerical methods; experimental tests; performance-based analysis.

1. Introduction

The response of steel structure buildings subjected to the combined hazard of fire following earthquake has gained recent attention. This has been motivated by recoded historical post-earthquake fire events, which demonstrated the significant loses, both in life and property, predominantly in large metropolitan areas [1,2]. Despite the potential extensive damage, post-earthquake fire scenario is not considered a loading case in current seismic design approach. Existing earthquake design philosophies allow specific degrees of damage in the structural members and connections under strong ground motions, which could result in spalling of fireproofing, leaving the structural members vulnerable to post-earthquake fire. With the substantial damage recorded in recent events and the associated life losses, there is a need for understanding and quantifying the response of steel buildings subjected to the multi-hazard event of earthquake and fire in high seismic regions. Until recently, there has been limited studies have been conducted on the response of steel structures, both on the global and local scales, under fire following earthquake scenarios [3-5]. In the last decade, however, there has been a surge in studies exploring the effect of fires following earthquakes on structural steel elements and systems [6-13].

In this paper, different analysis methods are presented to evaluate the performance of structural elements and systems subjected to fire following seismic events. These methods are targeted at assessment of structures the component level using analytical formulations and at the system level using detailed finite element models and hybrid simulations. Code provisions that facilitate design procedure for such loading scenario, developed through the integration of the results obtained from these various methods, will also be introduced. Moreover, the paper will highlight the role of finite element models as well as advanced experimental methods in understanding the response of steel structures as a system under the sequential hazards. A new framework for performance-based fire following earthquake engineering will also be presented. This framework allows for the integration of all mathematical and numerical tools while taking into account uncertainty in critical variables that impact member and system vulnerability under such cascading event. These variables include post-flashover fire, passive fire protection, structural resistance, and mechanical loads. The results demonstrate the benefit of conducting the component-level analysis and the value in integrating various methods for vulnerability assessment at the system level.

2. Member-Level Analysis

A flexibility-based finite element approach is introduced that has the capability to predict the geometrically nonlinear response of a beam-column element subjected to variable temperature distribution along its length and constant temperature throughout the cross section based on Euler-Bernoulli beam theory. The flexural element, shown in Fig. 1(a), is assumed to have a non-uniform longitudinal temperature distribution with T_i and T_j being the nodal temperatures at either end. Since the elastic modulus of steel is a function of temperature and degrades at elevated temperatures, the nodal temperature at each end of the element will result in temperature-dependent modulus of elasticity $E(T_i)$ and $E(T_j)$. The modulus of elasticity along the length of the element, x, can therefore be written as:

$$E(x) = E(T_i)\left(1 + \frac{\zeta x}{L}\right) \quad (1)$$

where, L is the length of finite element and ζ is calculated as follows:

$$\zeta = \frac{E(T_j)}{E(T_i)} - 1 \quad (2)$$

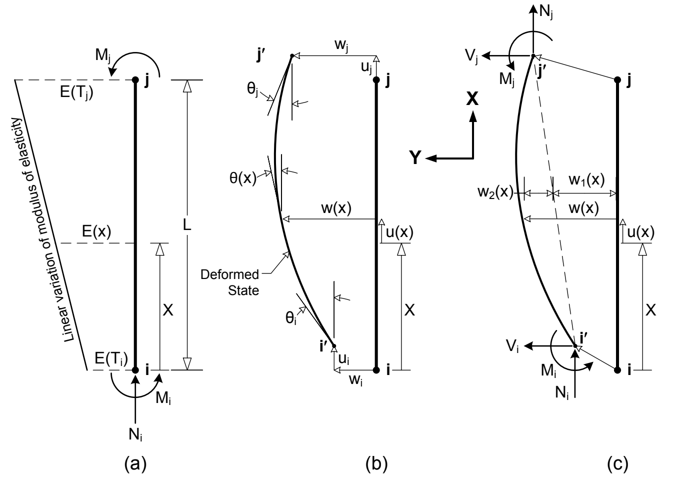


Fig. 1 – (a) Flexural element subjected to non-uniform longitudinal temperature and three applied external nodal forces (b) the deformed state of element with all nodal deformation variables (c) the deformed state of element with all nodal force variables

The first-order stiffness and geometric stiffness matrices necessary for the stability analysis can be extracted from the three sets of kinematic (Eq. (3)), equilibrium (Eq. (4)), and material law (Eq. (5)) equations. Both stiffness matrices were verified against linear elastic and inelastic analyses, where details available in [6, 7, 11].

$$\boldsymbol{u} = \int_0^L \boldsymbol{\Omega}^T . \boldsymbol{\gamma}\, dx \tag{3}$$

In Eq. (3); $\boldsymbol{u}$ is a vector of relative displacements and rotations, $\boldsymbol{\gamma}$ is a vector of strains, and $\boldsymbol{\Omega}$ is a matrix that transforms strains into displacements and rotations. Equilibrium equation is

$$\boldsymbol{R}(\boldsymbol{x}) = \boldsymbol{\Omega} . \boldsymbol{f} + \boldsymbol{R_2}(\boldsymbol{x}) \tag{4}$$

where; $\boldsymbol{R(x)}$ is a vector of the cross sectional forces, $\boldsymbol{R_2(x)}$ is a vector of cross-sectional forces developed because of the inclusion of the (P-δ) effect, $\boldsymbol{f}$ is a vector that represents the applied external nodal actions and, $\boldsymbol{\Omega}$ is a matrix that correlates the applied external nodal actions to those developed internally in the cross-section. Finally, material law equation is

$$\boldsymbol{R}(\boldsymbol{x}) = \mathbf{k_s}(\mathbf{x}) . \boldsymbol{\gamma} \tag{5}$$

where; $\mathbf{k_s(x)}$ is the section stiffness matrix.

Once first-order and geometric stiffness matrices were derived, a wide variety of analyses was conducted on steel columns including "modal analysis" to determine eigenvalues (elastic buckling force) and eigenvectors (elastic buckling mode shapes) along with "nonlinear inelastic analysis" to obtain the critical stress at the onset of instability. Additionally, this methodology allows investigating the impact of multiple variables on the elastic and inelastic response of steel columns, i.e. various temperature-dependent material characteristics, longitudinal temperature profiles, boundary conditions, inter-story drift ratios, among others.

Fig. 2 shows the buckling stress of a pinned-pinned steel column subjected to various levels of inter-story drift ratios and non-uniform longitudinal temperature profiles. This indicates that increase in inter-story drift causes significant reduction in the inelastic buckling stress of steel columns. This reduction varies from one temperature profile to another. However, it is seen that the buckling stress for inter-story drift ratio of 5% reaches less than 40% of its value when no inter-story-drift is present. In other words, permanent residual rotation in steel columns caused by the earthquake can result in significant reduction in buckling capacity of the column.

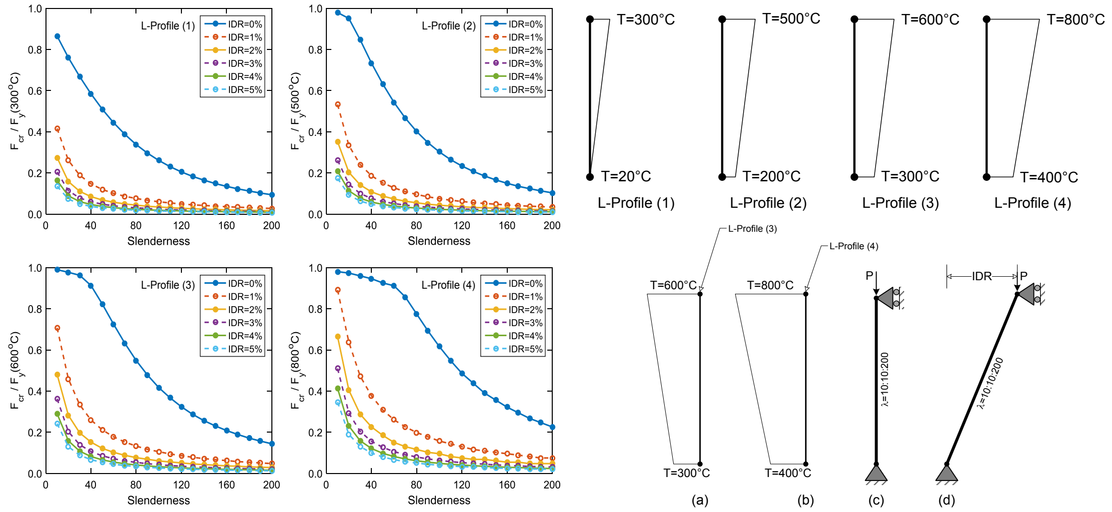


Fig. 2 – The buckling stress in the pinned-pinned column at various inter-story drift ratios and longitudinal temperature profiles

3. System-Level Analysis

3.1 Numerical Finite Element Simulations

This section demonstrates utilization of finite element simulations, using the commercial software ABAQUS [14] to provide insight into the effects of earthquake-initiated fires on medium-rise steel moment resisting frame with reduced beam section connections, which have become common frame type in modern earthquake-resistant design following the 1994 Northridge and 1995 Kobe earthquakes. The earthquake simulations are conducted using nonlinear time-history analysis where the frame is subjected to a suite of near-field and far-field ground motions. With the state of the structure following the earthquake used as the initial condition for the fire analysis, the uncoupled thermal-mechanical analysis is performed with a specified time-temperature curve applied at the reduced beam section connections. A 3-step analysis procedure is performed to simulate the post-earthquake fire scenarios in these MRFs. First, the frames are analyzed under gravity loads. Second, ground motion records are applied at the ground and lower levels, if any, to simulate earthquakes using dynamic time-history analysis. Third, thermal-mechanical analysis is conducted to simulate post-earthquake fires and the corresponding stresses. To perform the third step, a transient heat transfer analysis first is conducted to obtain the transient nodal temperatures followed by a mechanical analysis utilizing these nodal temperatures to determine the fire-induced actions and deformations.

Piece-wise reduced beam section connections are employed in all rigid joints of the 2-D MRF, as shown in Fig. 3, to create the RBS connection geometry with the proper transition in accordance with FEMA-350 [15]. The scissor model (Fig. 3) is employed to represent the panel zone at the beam-to-column joints [16]. The seismic lumped masses are distributed among the beam to column joints of the MRF. The gravity loads are divided into two parts. First, the gravity loads associated with the MRF are applied as distributed vertical forces along the beams at each story level. Second, the gravity loads associated with the interior gravity frames per tributary area are applied as concentrated loads to the leaning columns at the corresponding story levels. The representation of the gravity frames with the leaning columns is needed to account for the P-Δ effects [17]. The leaning columns are modeled using truss elements and are connected to the main MRF at the floor levels using multi-point constraint (MPC) links, as shown in Fig. 3. These columns are axially stiff and pinned at the basement and floor levels (Fig. 3); hence, they have no effect on the lateral stiffness of the main MRF. Further details are provided in Memari et al. [8].

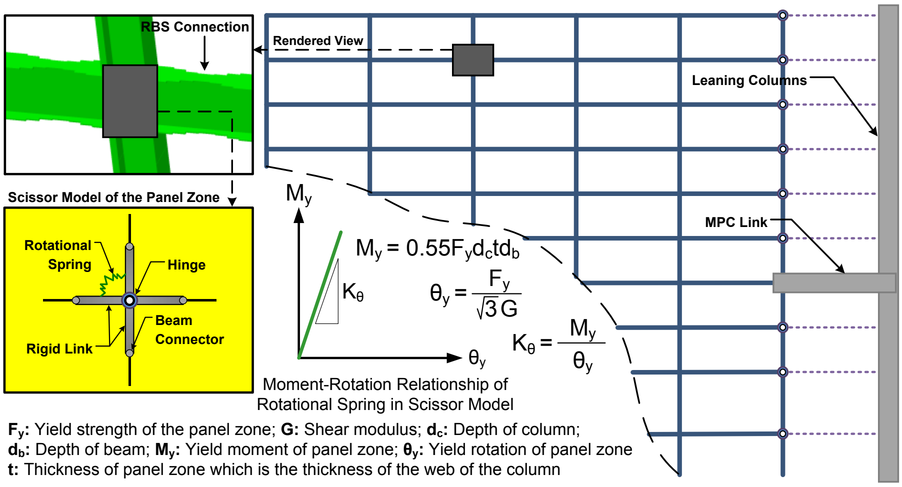
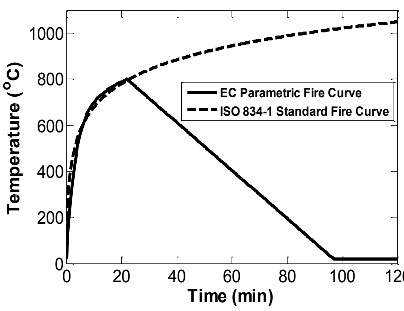


Fig. 3 – Details of the leaning columns, RBS connections, and scissor model

The period of vibration for the first three mode shapes in the present study have an excellent agreement with those obtained by Gupta and Krawinkler [18]. To validate the thermal-mechanical analysis, a small-scale steel frame is chosen, which has been previously tested at elevated temperatures by Rubert and Schaumann [19] and a good agreement was observed, confirming the approach used in this study. Time-temperature curve is employed to simulate a realistic fire event in the numerical analyses. The Eurocode parametric fire curve [20], shown in Fig. 4, is used in the present study because it has the capability of representing all three different phases in a fire event including an initial heating ramp, a cooling phase, and a constant ambient temperature. Two post-earthquake fire scenarios are considered. These two fire scenarios are applied at one-third and two-thirds the height of the MRF starting from the ground level and are denoted as FFE-1/3H and FFE-2/3H, where FFE is the acronym for Fire Following Earthquake. The FFE-1/3H fire scenario includes 1st - 3rd levels in the 9-story MRF, while the FFE-2/3H fire scenario includes 1st - 6th levels in the 9-story MRF. The fire is applied to all spans of a given story level only at the location of the RBS connections. The structural performance level of the buildings under the individual earthquakes and both post-earthquake fire scenarios (measured in terms of IDR) is summarized in Fig. 4. With few exceptions, the structural performance level of the MRFs under the earthquakes is not affected by the post-earthquake fires and the potential for systems collapse does not appear to be imminent as a result of these applied post-earthquake fires. However, scenarios which result in asymmetric heating of the frame may give rise to excessive P-Δ effects, leading to the possibility of collapse.



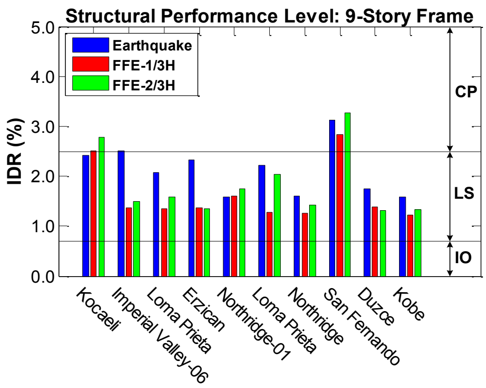


Fig. 4 – Fire curves (left) and structural performance level of the studied MRF under the earthquakes and both post-earthquake fire scenarios (right)

The performance of RBS type of connection has not been addressed under fire loading in the past. Furthermore, it is essential to assess their response under fire loads after residual damages caused by the earthquake loads. Here, an exterior connection at the ground level of the 9-strory frame is analyzed in detail under a post-earthquake fire scenario to evaluate its local behavior. To evaluate the local behavior of the selected RBS connections under earthquake loads and also capture stiffness and strength degradation of the surrounding frame due to post-earthquake fire loads, a multi-resolution numerical model of the MRF systems is created as illustrated in Fig. 5. The selected RBS connection is modeled using solid elements while the remainder of the frames is modeled by line elements. Further details are provided in [11].

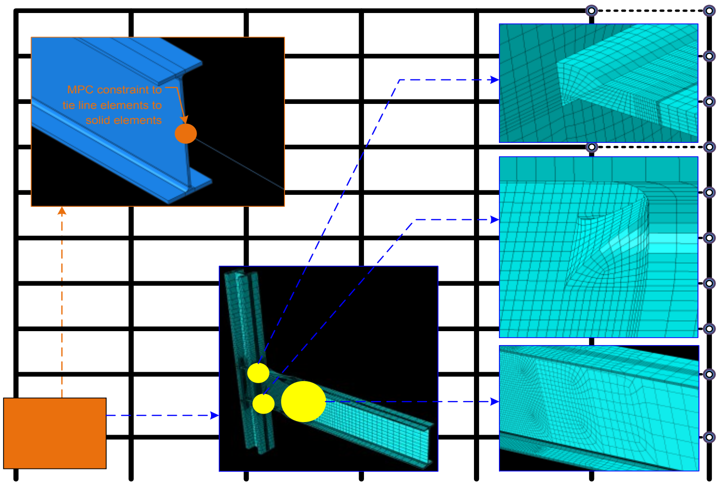


Fig. 5 – Details of multi-resolution modeling technique

As shown in Fig. 6, the residual deformations and stress history resulting from the earthquake has significant effects on the response of RBS connection to post-earthquake fire loads. Furthermore, Fig. 6 shows significant local buckling in both top and bottom reduced flanges along with web at their location under post-earthquake fire load. In addition, very high von Mises stress is concentrated in the flanges and web at the location of reduced section.

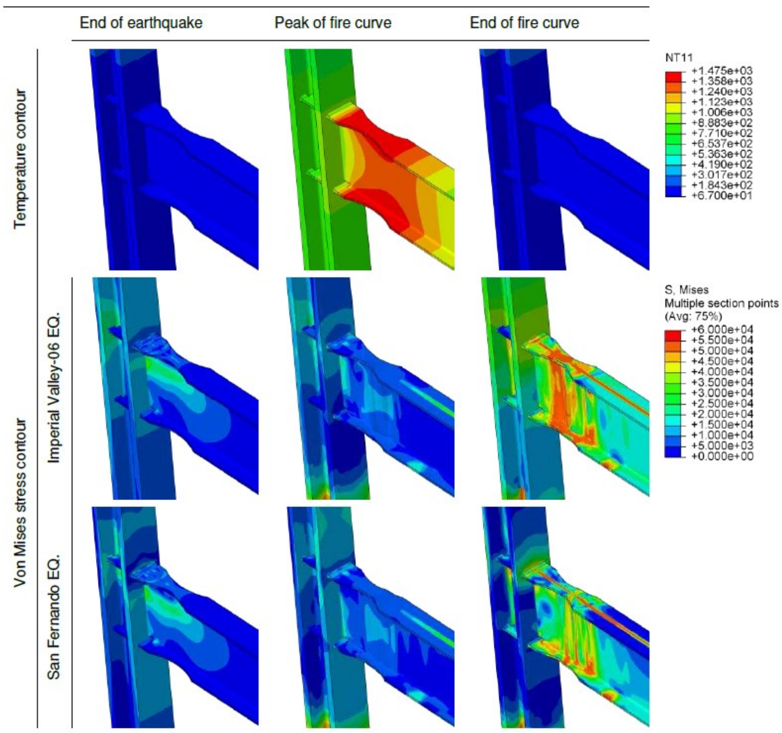


Fig. 6 – The deformation and von Mises stress distributions in the 3-D RBS connection

3.2 Hybrid Simulation

This section discusses the application of hybrid simulation to assess the performance of steel structures subjected to elevated temperatures with emphasis on post-earthquake fire response of the structure. A specific attention is given to the stability of steel columns under elevated temperatures in the absence and presence of permanent inter-story drift caused by a prior earthquake. A small-scale steel building, that is 4-bay (in both x- and y-axes) and 4-story high, is utilized in this study as shown in Fig. 7. The location of the building is assumed to be in downtown Los Angeles on soil type C per ASCE 7-16 [21]. The story height is 1,220 mm (48 in.) and the span width is 1,524 mm (60 in.). The perimeter lateral seismic-resisting systems are steel special concentrically-braced frames (SCBF) that are designed according to the AISC LRFD Specification 360-16 [22]. The interior frames are only gravity frames, designed to carry the vertical loads.

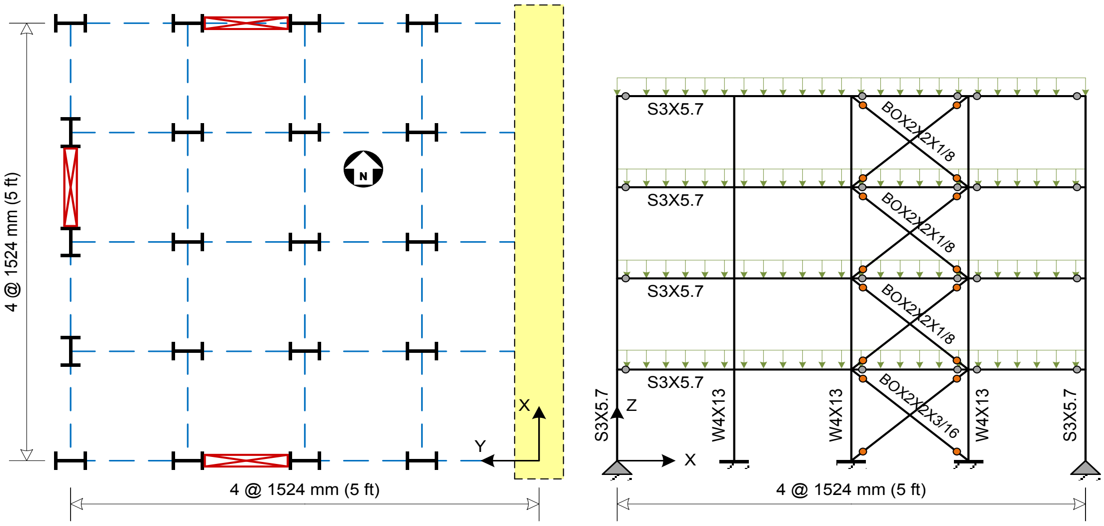


Fig. 7 – Configuration of the small-scale braced frame

In this framework, the structure is divided into two parts, namely the integration numerical module and the substructure module as shown in Fig. 8. The integration numerical module includes a numerical finite element model of the entire structure except for the substructure that is represented with a physical specimen. The numerical module was developed and housed on a desktop at Colorado State University (CSU). The Structural Laboratory at CSU was also utilized for the testing portion of the simulation. A Network Interface for Controllers (NICON) presented in Zhan and Kwon [23] is utilized to establish communication between the two modules discussed above. NICON can receive a vector of displacements and rotations from the integration numerical module and send the displacement command to an actuator controller at each time step of the simulation by using the communication protocol discussed in Huang and Kwon [24]. Once the displacement commands are imposed onto the specimen, the vector of measured displacements and rotations along with a vector of restoring forces and moments are sent back to NICON. When the compatibility between the integration module and the substructure is achieved, NICON passes the restoring forces and moments to the integration numerical module for calculating the displacement increment for the next step.

Prior to conducting the hybrid fire simulation, a numerical heat transfer analysis for the structure is performed. The result is used to define the thermal load in the integration numerical module at each time step. In the simulation, NICON controls the analysis interval based on the predefined length of each time step to ensure the synchronization of the thermal load. In this framework, the temperature of the physical specimen is synchronized with the temperature profile of the numerical model by running real-time simulation. The nodal temperature in the numerical integration module at each time step during the simulation is predetermined by the numerical heat transfer analysis. In verifying the earthquake simulation, the result of the standalone numerical model is compared to the numerical hybrid simulation. The numerical hybrid simulation comprises of the numerical integration module along with the numerical substructure module – far left column in the first story – both created in ABAQUS. An excellent agreement is observed for roof drift and column inter-story drift between the standalone and hybrid numerical analysis. To verify the post-earthquake fire simulation, fire load is applied to the far-left span of the first story. In the thermal analysis, ASTM E119 [25] is used as fire load and is directly applied to the top beam and adjacent columns of the span as a thermal boundary condition. The horizontal and vertical displacements at the interface node (Node 2 in Fig. 8) are compared to verify the hybrid simulation method. An excellent agreement was observed between the numerical hybrid simulation and the standalone numerical model, verifying the accuracy of the hybrid simulation [9]. Four hybrid fire following earthquake simulations are carried out in this study, each representing different level of inter-story drift in the steel columns (Table 1).

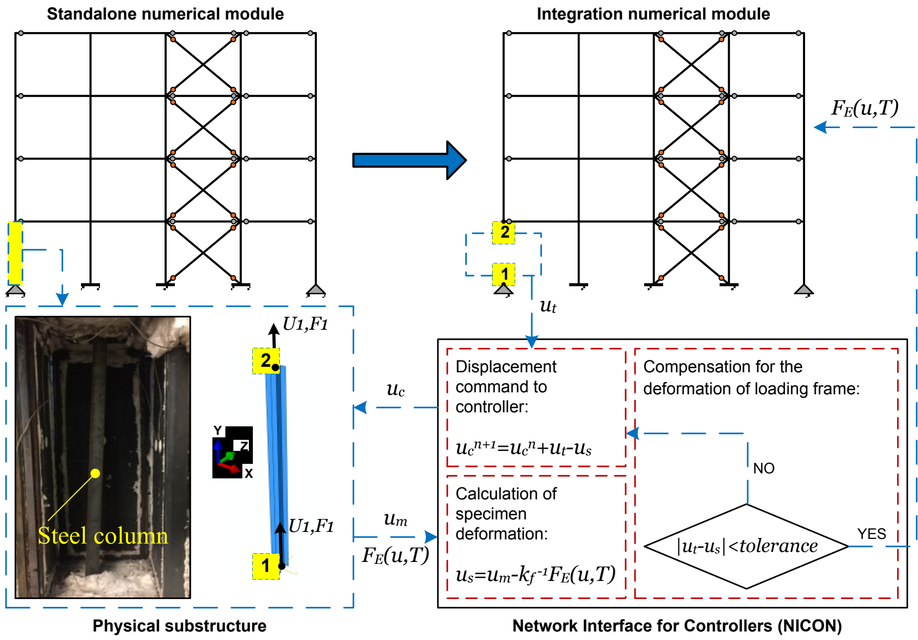


Fig. 8 – Substructuring configuration of the structure for hybrid fire simulation

Table 1 – Testing matrix

Test ID	Column Inter-story Drift (%)	Roof Drift (%)	No. of Buckled Braces	Type of Test/Analysis
Test A	0	0.00	0	Fire
Test B	2	1.35	6	FFE
Test C	3	2.04	6	FFE
Test D	4	2.76	6	FFE

Fig. 9 shows comparison of displacements and forces among all tests. Both plots clearly highlight the effects of residual drift, caused by an earthquake, on the response of the columns subject to fire. Specifically, an increase in the IDR resulted in increase in deformation of the column due to temperature load, which means smaller stiffness and lower level of axial force in the column. Moreover, increase in the IDR accelerates column buckling since the column inter-story drift imposes additional bending moment demand in addition to the axial force.

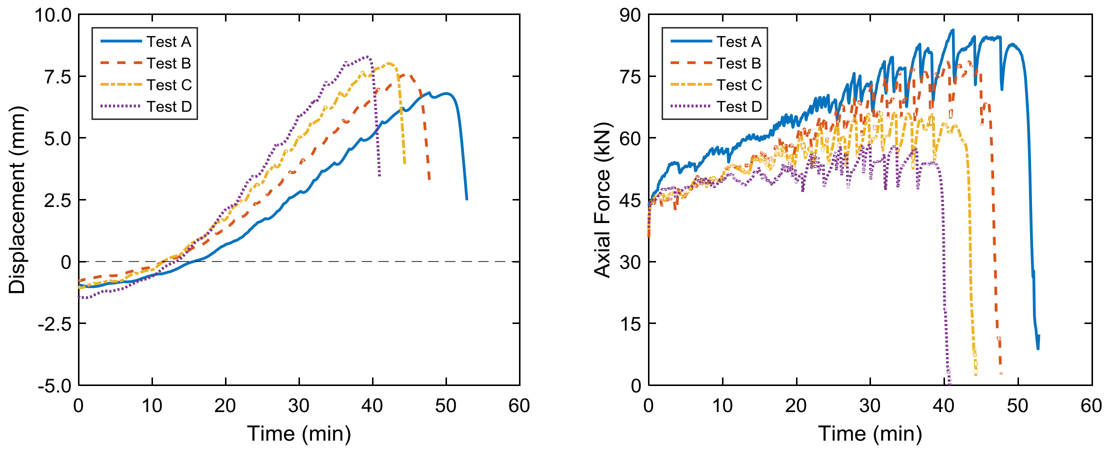


Fig. 9 – Displacement and force comparison among all hybrid tests

4. Performance-Based Analysis

In this section, a new methodology is presented in which the member level and the system level analyses are for performance-based design of steel frames under the cascading events is presented. Performance-based engineering, which has been widely used in earthquake engineering, provides an opportunity to structural engineers to devise optimal and robust solutions for the design of structures under earthquake loads given all existing constraints. Performance-based earthquake engineering (PBEE) addresses performance of structures on a system-level based on probabilistic assessment of collapse. In summary, this framework includes 4 domains: (a) hazard domain, (b) system domain, (c) damage domain, and (d) loss domain. In this study, efforts are placed on investigating the first 3 domains (hazard, system, and damage domains) in PBFE framework in order to identify an appropriate variable for each of them. The hazard analysis results in identifying intensity measure of fire hazard. Several parameters have been considered as intensity measure of fire in the past studies, e.g. maximum gas temperature, duration of fire, peak temperature in a compartment, heat flux, among others. While these parameters are a viable option to serve as an intensity measure, fire load density, which is used in this study, could perhaps be the most suited parameter as an intensity measure for performance-based fire engineering. The system domain enforces the selection of an appropriate engineering demand parameter for performance-based fire engineering. In this study, vertical stability of steel frames is chosen as the focus for the performance evaluation. This is because in this study emphasis is placed on fire following earthquakes in which vertical stability, as oppose to lateral, under fire is of concern. The use of vertical stability allows for meeting one out of three parameters – insulation, integrity, and stability – required to satisfy a desired level of structural performance under fire loads.

A performance-based engineering framework is needed to be defined for the cascading hazards of earthquake and fire by combining both PEER framework for earthquake and the adopted framework for fire. The developed framework for the combined hazards is outlined in Fig. 10 below. This 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. Therefore, these two hazards can be assessed independently up to the step where the damage caused by the earthquake has significant effects on the response of the structural member or system to fire loads.

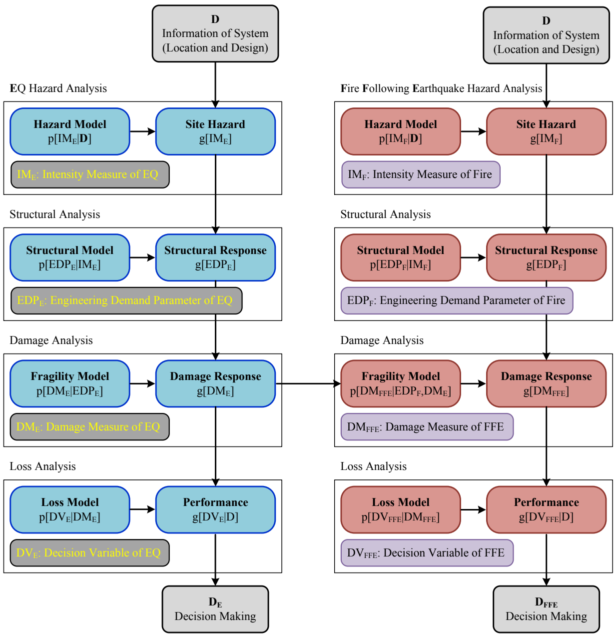


Fig. 10 – The proposed framework for performance-based fire following earthquake engineering [10]

The geometry of steel column is considered as Fig. 11 in the numerical analysis. To run member-level probabilistic analysis, a Monte Carlo simulation is conducted using Latin Hypercube Sampling method. To conduct the analysis, first a numerical model with the 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.

A set of “N” fire curves at certain level of fire load density is generated using Latin Hypercube sampling of the related stochastic variables. It results in N fire curves that are converted to time-temperature curves in the body of the steel column considering samples of passive fire protection material described previously. A set of N axial demand forces is also generated according to stochastic variation of mechanical loads. The finite difference method is employed to obtain non-uniform longitudinal temperature profiles in the steel column for each of N time-temperature curves. In this study, for each random variable, 100 samples are produced. In summary, prior to calculating the load demand on a column, the available information would include the deformed state of a column at a certain level of inter-story drift along with longitudinal distribution of temperature and applied mechanical loads. In the next step, N (100) analyses are conducted to obtain the ultimate interaction demand of axial force and moment on a column. The demand caused by inter-story drift and applied axial forces are calculated using the flexibility-based formulation with minimal modifications. The demand caused by the thermal loads is calculated using uncoupled thermal-mechanical analysis. The capacity is also determined using the flexibility-based framework for all N (100) combinations of fire load and axial force demands. At this step, the developed axial and moment demands in the column caused by earthquake and fire in all cases (N=100) is compared to column capacity to determine the damage measure of the column under each case. Therefore, the probability of failure can be calculated by dividing the number of failed samples (N_f) by the total number of samples (N). The repeat of this process for all levels of inter-story drift ratios and fire load density will result in fragility surface of column of interest as shown in Fig. 11.

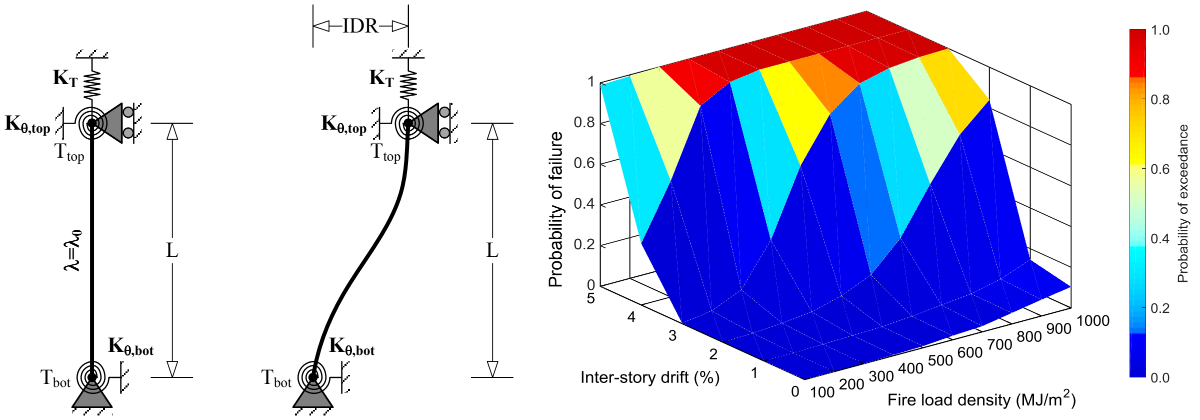


Fig. 11 – The mechanical model of steel column under fire following earthquake (left) and the 3-D fragility surface of the selected steel column given inter-story drift ratio and fire load density (right)

5. Conclusion

In this paper, various analysis methods are presented and highlighted to evaluate the performance of steel frames under the cascading effects of earthquakes and fires. The methods varied from analytical formation at the component level (column), to numerical finite element models and hybrid simulations at the system level, to performance-based engineering method. The presented methods and their integration can offer a new direction towards performance assessment of steel frames under the combined hazards.

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