CHALLENGES AND ALTERNATIVE APPROACHES FOR SIMULATING THE RESPONSE OF STEEL STRUCTURES EXPOSED TO FIRE

Hussam Mahmoud[1,*], Bruce Ellingwood[1] and Mehrdad Memari[1]
[1] Colorado State University,
Department of Civil and Environmental Engineering,
1372 Campus Delivery, Fort Collins, CO, 80523, USA
*Email: Hussam.Mahmoud@colostate.edu

ABSTRACT

Although structurally significant building fires are rare events, their occurrence can cause substantial damage and may lead to partial or complete system collapse. While fireproofing has proven to be effective in mitigating the effect of severe fires, it is rated for only a certain time and will eventually fail to provide adequate protection during a large or extended fire event. Furthermore, fireproofing typically is rated using a standard fire exposure, such as ISO 834 or ASTM E119, neither of which represent realistic fire exposures in an actual building. With the worldwide move toward performance-based fire protection engineering, understanding and quantifying system behavior through advanced numerical simulations, especially during the heating and cooling phases of realistic fire exposures, is essential for establishing proper performance-based provisions for fire engineering that ensure both safe and economical design. This paper highlights current challenges in simulating the effect of fire on steel components and frames, including proper representation of loading and boundary conditions, geometrical nonlinearities, material inelasticity, and numerical instabilities. The structural models considered include 2-D line elements, 3-D continuum elements, and multi-resolution models. In addition, the advantages and drawbacks of these models are highlighted and the implication of their features is discussed. The highlighted modeling approaches and the corresponding results shown can be used by engineers for selecting the most economical and effective techniques for simulating the response of components and structural systems to scenario fire hazards accurately.

KEY WORDS

Fire, finite element analysis, steel structural buildings, reduced beam section connections.

INTRODUCTION

The response of steel structures subjected to fire loading has gained the recent attention of structural engineers and researchers, motivated both by historical fire events, which resulted in significant structural damage and subsequent loss of life, and by the prospects of achieving better performance at reasonable cost. Current provisions do not address the effects of realistic fire conditions on the thermal and structural interactions among frame members and global system response. Therefore, there is an obvious need to quantitatively understand the behaviour of steel framed buildings under realistic fire conditions at the component and system levels. This need could be realized through experimental testing or numerical simulations. Undoubtedly both tools are essential for developing such guidelines. Generally, physical laboratory models can provide an understanding of structural behaviour under specific loading and boundary conditions and are further utilized to calibrate numerical finite element models that then can be used to evaluate the response of the structural component system under a wide range of loading scenarios, system configurations and boundary conditions.

The use of numerical modelling to evaluate the behaviour of steel elements and frames during a fire has received significant attention with recent improvement in computational capabilities. Various numerical studies have been reported in the literature to assess the effects of fire loading on the local and global response of steel structural buildings (Saab and Nethercot 1991; Gu and Kodur 2011). Numerical finite element analyses have been also performed to evaluate the behaviour of individual beams under fire loads (Vila Real *et al*. 2004). In these analyses, the slenderness ratio, various fire scenarios, the location of restraints at supports, temperature-dependent structural steel properties, level of mechanical loads have been varied. For the exposure of steel columns to elevated temperatures, various numerical analyses have been conducted, including the work of Tagaki and Deierlein (2007), who examined the effects of various parameters on stability and strength of steel columns under fire loads including different types of axial restraints, level of axial load, and eccentricity of axial load. Numerical studies were also conducted to evaluate the performance of various types of steel beam-to-column connections subjected to fire loading (Yu *et al*. 2008; Hu and Engelhardt 2014). Other numerical studies exist in the literature on beam-column connection under elevated temperature including various studies that have been conducted on single and double shear tab connections both in the US and Europe. Quiel and Garlock (2008) provided comprehensive information on modelling techniques for 3-D structural connections including suggestions on element types, integration order, meshing, material properties, contact configuration, and solving techniques. Memari and Mahmoud (2014) conducted a numerical study on the performance of 2-D steel moment-resisting frames (MRFs) with reduced beam section (RBS) connections under various single-bay compartment fire scenarios. The study showed that the global stability of steel MRFs with RBS connections is not affected by a single bay fire exposure. Agarwal and Varma (2014) arrived at a similar conclusion and showed that a perimeter column in a steel MRF is not at risk of buckling when subjected to axial demand resulting from load redistribution caused by inelastic buckling of a gravity column in a corner compartment fire. In addition, global stability and progressive collapse have been investigated in steel frames subjected to elevated temperatures (Sun *et al*. 2012).

This paper highlights challenges in conducting numerical simulations under elevated temperatures, including challenges associated with specified thermal loading, boundary conditions, material properties, and stability of numerical solutions. In addition, the paper covers alternative approaches by which the performance of structural steel connections and frames can be evaluated under various fire scenarios. This includes assessment using temperature-adaptive mechanical boundary conditions, assessment of 2-D steel frames with RBS connections, and assessment of multi-resolution steel frames with RBS connections.

SIMULATION CHALLANGES

Specified Thermal Loading

The challenges associated with thermal loadings lie in the selection of fire scenarios and the description of temperature variation with time. In the analysis of framed buildings under fire, the selection of fire cases to be analysed is generally geared towards either evaluating what might be considered a worst-case scenario or evaluating an ensemble of randomly or logically selected fire cases. The worst case scenario is typically viewed to be one compartment fire in the exterior bay in the first story of a multi-bay multi-story structure. In the case of conducting analysis under various fire cases, single and multiple floor fires as well single and multiple bay fires typically are considered. A more comprehensive, yet challenging, approach to develop such scenarios is to use a set of conditions that defines the development and spread of fire based on the distribution of combustion products throughout a building or part of a building. This set of conditions is outlined in ISO/TR-13387-2 (1999) and in the Society of Fire Protection Engineering (SFPE) Engineering Guide (2007), which accounts for both hazard and risk analysis as outlined in detail in NFPA 72 (2013).

The time-temperature curves used in fire simulations fall into two basic categories: standard fire curves, such as ASTM E119 (2012) and ISO 834 (2012), or parametric fire curves such as those generated using the Eurocode 1 (2002) or the SFPE Engineering Guide (2007) methods. The main principle behind the standardized curves is to expose a single structural member or assembly to a standard fire with designated fuel load and intensity. The component passes or fails the test depending on whether the peak temperature attained on the unexposed surface of the tested component exceeds a limiting temperature, whether the tested specimen fails in a way that allow hot gases to escape, or whether the specimen can withstand a pressure from a hose stream during a rating period of from 30 minutes to 4 hours. Many numerical studies have adopted these standardized fire curves to evaluate the response of components or sub-assemblies under elevated temperatures. The main issue with the standard fire curves is that they were developed nearly a century ago to provide prescriptive ratings and are not characteristic of a fire in a realistic modern building. They presume an inexhaustible supply of fuel during the rating period. Furthermore, they were developed assuming fuel loads that were commonly found in buildings when the test was first developed; modern fuels result in fires with significantly faster growth rates and higher radiative fractions that affect fire spread rates. In contrast, the parametric fire curves are intended to be used as design fires in compartment fire models. The parametric curves are quantitative temporal descriptions of assumed fire characteristics based on appropriate fire scenarios and include an initial post-flashover heating phase, a cooling phase, and a constant ambient temperature phase. These curves typically account for heat release rate, size of fire, yield of products of combustion, temperatures of hot gases, and time to key events such as flashover. The cooling phase in the parametric curves is both realistic and appropriate since significant structural demands may be generated in the building system due to differential cooling of elements.

Specified Boundary Conditions

Boundary conditions of varying degrees of complexity have been employed for modelling the thermal constraints provided to beams and columns during a fire (Ali *et al*. 1998; Yin and Wang 2004; Huang *et al*.

2006; Valente and Neves 1999; Yu 2006; Dwaikat and Kodur 2010; Sarraj *et al*. 2007; Hu and Engelhardt 2010; Da Silva *et al*. 2005; Kodur and Dwaikat 2009; Keller and Pessiki 2012). Figure 1 provides a depiction of some of the various boundary condition configurations that have been implemented in previous studies for evaluating the response of steel members and connection under fire. All boundary conditions require assumptions regarding the fixity of the degrees of freedom. A recent study (Mahmoud *et al*. 2015) utilized the concept of adaptive boundary conditions to account for the effect of frame in-plane restraint variation throughout the fire on the performance of moment connections, which will be discussed later in more detail. Keller and Pessiki (2012) also considered restraint variation but only in the axial direction of a member in a beam-column sub-assembly. The inclusion of proper boundary conditions in the assessment of sub-assemblies under elevated temperature is critical since the simulation results are highly dependent on the specified boundary conditions. For example, the use of fixed boundary conditions at the end of the sub-assemblies might drastically overestimate the thermally-induced axial, shear, and moment demands while the use of pinned conditions might underestimate the moment demand. One approach to alleviate such a problem is to construct the finite element model in such a way that the lengths of the beam and column representation in the model are terminated at the inflection points. This will eliminate the need for using rotational restraints at the member ends. However, the computational time might increase substantially since longer beams and columns will have to be used, which will increase the number of elements and nodes used in the simulation.

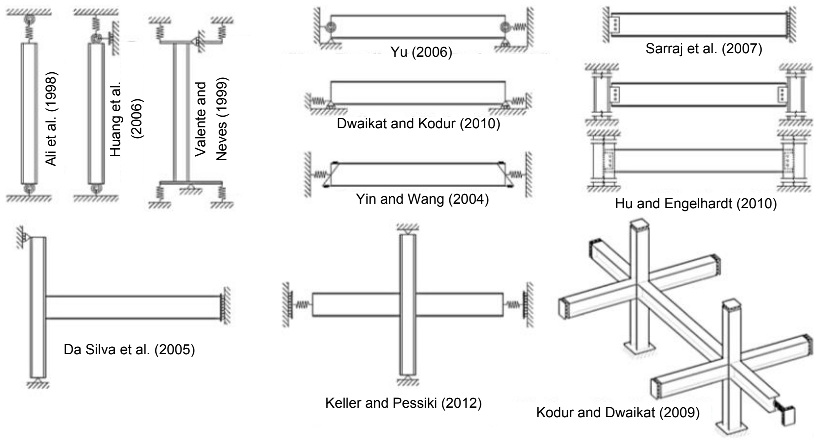


Figure 1 Implemented boundary conditions utilized in previous studies for structural members and connections under fire exposure

Specified Material Properties

Accurate thermal, mechanical, and damage models for structural materials, covering a wide range of temperature variation, must be specified in the numerical models to ensure accurate and reliable results. A quick review of existing material models under elevated temperatures reveals the need for further refinement of the existing material models to eliminate notable differences between what is adopted in existing codes. For example, the present specific heat model in ASCE (ASCE 1992) differs from its EC 3 (EC3 2005) counterpart above 200°C, with the difference especially apparent at and above 700°C. There are also differences between the two models of thermal conductivity of steel between 20°C to 1000°C, although the differences are less for specific heat. While more data exist on the mechanical properties of steel at elevated temperature in comparison to the thermal properties, large scatter still exist between the different models available in the literature. For the ASCE and EC 3 models specifically, large variation is evident for both the yield stress and elastic modulus for the full range of temperature evaluated, 20°C to 1000°C, with strong variation between 200°C to 800°C. While the stress-strain curves at elevated temperatures for both models are more rounded than at ambient, the ASCE model has a more pronounced plastic flow region than the EC 3 model. These differences in shape lead to differences in the inelastic stability limits of columns and laterally unsupported beams. Another example of differences in material properties can be found in a recent study conducted at the National Institute of Standards and Technology (NIST) where the listed properties are different than those listed in EC3 (Luecke 2014). Noteworthy, that the mechanical properties in AISC 360 (AISC 2010) are comparable to that of EC3. Thermal properties are however not provided in AISC 360.

An accurate prediction of the plastic deformation and failure of steel elements essentially requires proper capturing of material damage. Damage initiation under elevated temperature can be modelled using the Johnson-Cook damage model (Johnson and Cook 1985). The ductile damage initiation criterion defines the equivalent plastic strain as a function of stress triaxiality, strain rate, and homologous temperature. The model incorporates linear evolution of the damage variable with effective plastic displacement, which represents a linear stress-strain softening response. Even though its parameters allow its use in simulations under fire loading, the Johnson-Cook damage model has mainly been used in previous studies under ambient temperature and seldom for the evaluation of damage under elevated temperature. One of the drawbacks of the Johnson-Cook model is that it does not account for shear-dominated failures since stress triaxialities alone cannot be relied upon for predicting such failure modes (Wen and Mahmoud 2015).

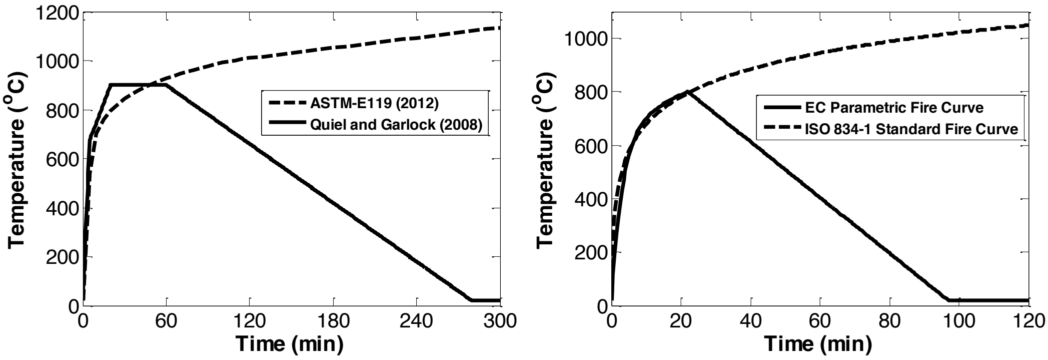
The variations in the material models highlighted above can be attributed to three main reasons. First, current knowledge on high-temperature properties of steel is based on limited material tests. Second, available high temperature material properties are only based on the heating phase of fires with no consideration to the cooling phase. Third, standard test methods for evaluating high temperature properties are non-existent; as a result, some data in the literature are based on transient tests while other data are based on steady-state tests.

Numerical Instabilities in Computational Analysis

One of the main challenges in conducting numerical simulations under fire loadings is the potential for substantial geometrical nonlinearity and material inelasticity, which can pose significant convergence challenges during the simulations. Generally, three requirements are essential for the convergence of finite element simulations: 1) completeness, 2) compatibility, and 3) stability. Completeness refers to the elements having sufficient order to capture the analytical solution in the limit of a mesh refinement process. Compatibility requires that the shape functions provide displacement continuity between elements. This is often an issue in cases where nodes are left unmerged; thereby creating unintentional discontinuities or cracks that can go unnoticed in models with substantial number of elements and nodes. Stability is typically an issue if the system of finite element equations violates certain conditions, resulting in zero-energy modes in elements in addition to excessive element distortion. This could result in the presence of an ill-conditioned stiffness matrix. While difficulties in convergence may be an indication of the onset of local or global instability, marking the presence of local buckling or system collapse, careful evaluation of the reasons for the model failing to converge may reveal other causes. Convergence can be checked using many different solution variables such as strain energy, maximum displacements, maximum stresses, and many others. However, strain energy is generally viewed as the best variable for such evaluation since it provides the smoothest convergence plots. It is generally not recommended to evaluate convergence based on maximum stress values since the maximum stress is a local rather than a global measure.

METHODOLOGY

Various numerical studies have been conducted to evaluate the response of steel frames and sub-assemblies under elevated temperatures. These studies are typically conducted using either line element models and 2-D or 3-D continuum models, or a combination of line elements and continuum elements. Boundary conditions generally are fixed, pinned, or somewhere in between (i.e. semi-rigid). The methodologies discussed below outline recent studies conducted by the authors in which various modeling techniques were used, including 3-D component level analysis using adaptive boundary conditions, 2-D system-level analysis using line elements, and 2-D system-level analysis using multi-resolution elements (3-D continuum and line elements). The simulations conducted with adaptive boundary conditions utilized the time-temperature curve shown in Figure 2 (a) (Quiel and Garlock 2008) while the simulations using the line-element models and the multi-resolution models utilized the curve shown in Figure 2 (b) (EC1 2002). All analyses were conducted using an uncoupled thermal-mechanical analysis with time-temperature curves that included both heating and cooling phases. First, a transient heat transfer analysis was conducted to obtain the transient nodal temperatures. Once the heat transfer analysis was completed, a thermal-mechanical analysis was employed by importing the corresponding nodal temperature from the results of the transient heat transfer analysis. Both material and geometric nonlinearities were included in the studies. The discussion will not focus on the details of the structures, the sub-assemblies, or the fire cases but rather on the modeling approach used. Selective results will however be highlighted in the results section.

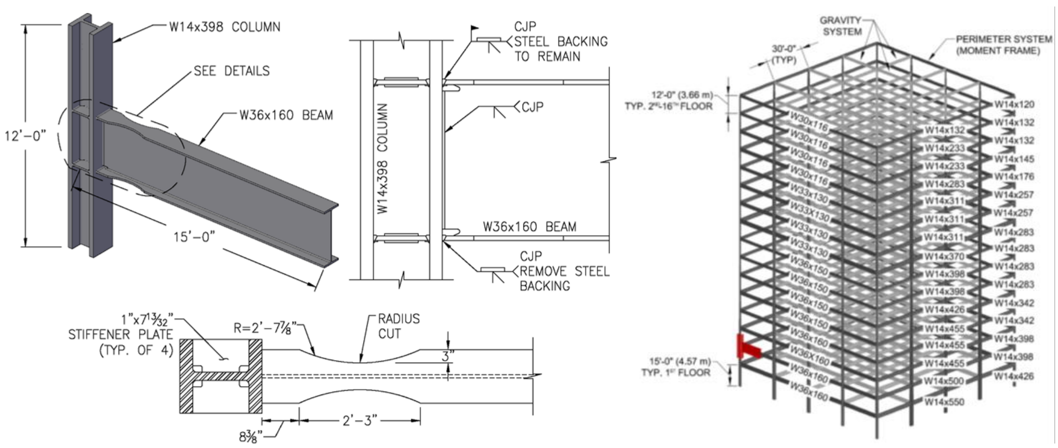


(a) (b)

Figure 2 Standard time-temperature curves compared with the other fire curves used in the simulations discussed below (a) adaptive boundary condition analysis using the fire curve in Quiel and Garlock (2008) and b) line element and multi-resolution analysis using a Eurocode parametric curve (EC1 2002).

Temperature-Adaptive Mechanical Boundary Conditions

The reduced beam section (RBS) connection shown in Figure 3 (a) is modeled using 3-D solid elements in ABAQUS and is an exterior connection at the 2nd floor of the 16-story special moment resisting frame shown in Figure 3 (b). RBS connections are fabricated by removing a portion of the beam flange near the beam-column joint to allow for energy dissipation away from the column face during seismic events, thereby reducing the seismic demand on the weld and other critical details at the beam-column interface. Two different boundary conditions are modeled. The first model, termed ‘realistic’, includes restraints in all three planar degrees of freedom (DOF), including the coupled transverse-rotational DOF, and is believed to be the most realistic representation of the planar boundary conditions. The second model is termed ‘fixed’ and has the tips of the beam and column fully fixed in all three planar DOFs to represent unrealistic worst case boundary condition.



(a) (b)

Figure 3 a) Details of the modelled RBS connection and b) location of connection in the subject frame

For the realistic restraint model, the beam-column is treated in isolation and connector elements are used at the end of the sub-assembly to model the stiffness of the surrounding frame. A translational “Cartesian” element and rotational “Rotation” element are selected for the connector elements in ABAQUS, which allows the coupled stiffness matrix for the element to be directly specified. Utilizing this element type permits the fully coupled stiffness of the surrounding frame to be modeled, which otherwise is not possible if spring elements are used. The coupled stiffness includes not only the axial, transverse, and rotational stiffness values but also the transverse-rotational stiffness of the frame excluding the modeled connection. The restraints are determined throughout the entirety of the fire simulation by developing a line element model of the whole frame and subjecting the frame to a compartment fire using a specified time-temperature curve. The simulation of the full frame model is terminated at twenty representative times throughout the fire and a series of deformation-controlled analyses is performed to evaluate the restraint stiffness provided by the framework surrounding the sub-assembly at those twenty discrete points using static analysis to populate the elements of the stiffness matrix. The stiffness values obtained are then used to update the connector elements of the 3-D connection sub-assembly to capture the evolution of restraint provided by the frame over the duration of the fire. Figure 4 provides a depiction of the transient restraint stiffness for the 16-story sub-assembly.

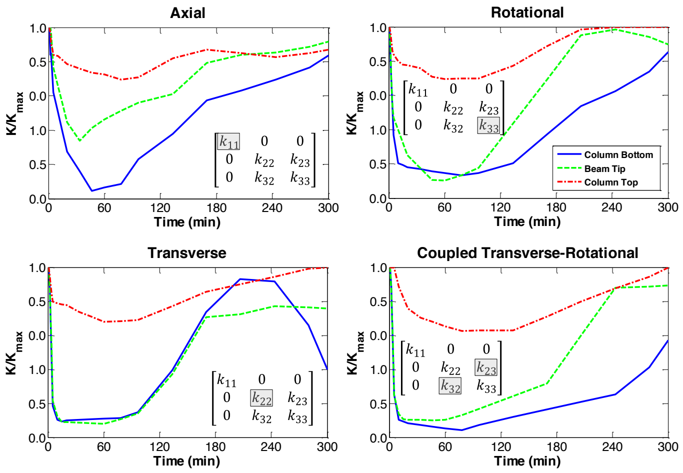
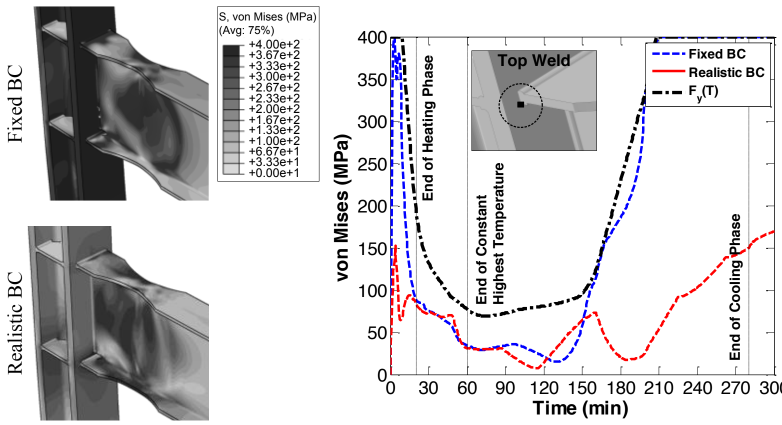


Figure 4 Normalized restraint stiffness values for connection sub-assembly in the16-story frame

The effect of boundary conditions on the damage patterns observed following the cooling phase shows large variations in the extent of local buckling in the flange and web of the beam in the reduced section for the two sets of boundary conditions as shown in Figure 5 (a). This is further demonstrated by the localized stresses at the top weld connecting the beam to the column, as shown in Figure 5 (b), where large von Mises stresses are developed in the case of fixed boundary conditions.

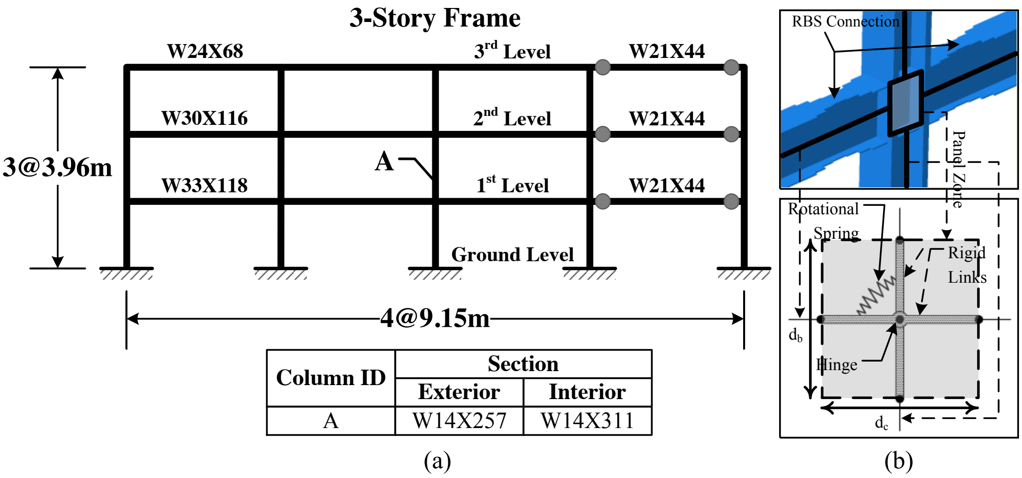


(a) (b)

Figure 5 a) von Mises stress contour in the connection under different restraint conditions and b) localized von Mises stress in the top weld at the column face.

2-D Steel Frames with RBS Connections

The special moment resisting frame shown in Figure 6 (a) is comprised of beams and columns with W-shapes and welded reduced beam section connections. In the transient heat transfer and mechanical models, 1-D line elements are used for the beams and columns. To create the connection geometry with the proper radius transition, the RBS connections are represented in the 2-D frame using piece-wise elements as shown in Figure 6 (b). The column panel zone behavior is represented using the scissors model (Krawinkler 1978) shown in Figure 6 (b), which was constructed using two rigid links that are hinged together at the mid-point and connected to the remainder of the frame using beam connectors to constrain all planar 3 degrees-of-freedom of one node to an adjacent node. In addition, a linear rotational spring with stiffness proportional to the beam and column sizes constrains the two rigid links together as shown in the figure. The P–Δ effect on the moment resisting frame, associated with the interior gravity frames, is taken into account by simulating the gravity frames using a leaning column (Yun *et al*. 2002). These models are simplistic in nature and do not pose significant challenges in their development. However, certain shortcomings might result from the use of such models including their inability to capture local buckling, strain gradients through the element thickness, and localized stresses and strains at critical details including for example cutouts and weld access holes.



Column ID	Section	
	Exterior	Interior
A	W14X257	W14X311

Figure 6 a) Elevation view and configuration of the 3-story SMRF frame and b) piece-wise approximation of the connection with a scissors model representing the panel zone

Four different fire scenarios were simulated to evaluate the response of the frame when subjected to a compartment fire, as shown in Figure 7 (a). Examples of response parameters include the developed axial force (P) in the beams (Figure 7 b) as well as Inter-story Drift Ratio (IDR) (Figure 7 c). Figure 7 (b) shows the evolution of axial forces in the beam for each fire case with axial compressive forces developed during the heating phase and axial tension during the cooling phase. Upon cooling, residual axial tensile forces remain in the beams as shown in the figure. Larger IDR values are obtained when the exterior bays are subjected to the fire loading (Figure 7 c), which can be seen in the odd-numbered fire cases. This is because when the interior spans are subjected to fire, larger lateral restraints are provided by the beams and columns at both sides of the fire-exposed bay.

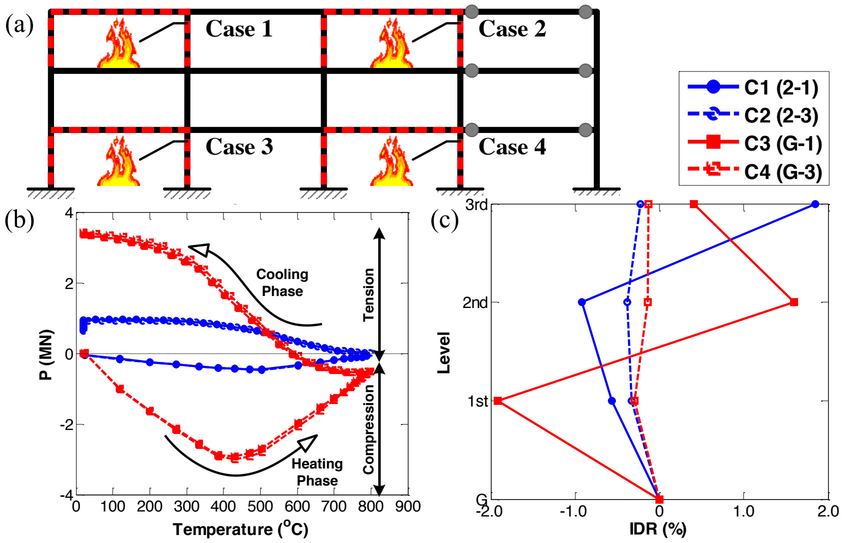


Figure 7 a) Fire cases evaluated, b) evolution of axial forces in the beam, and c) resulting inter-story drift for each fire case

Multi-Resolution Steel Frames with RBS Connections

Multi-resolution numerical models are those which employ detailed 2-D or 3-D continuum elements that are integrated with line element models. The use of continuum models captures the localized demand while the line element models provide realistic boundary conditions to the localized model, yielding the global response of the structural system. The top of Figure 8 (a) shows an example of a 3-story SMRF with RBS connection subjected to fire in the first three bays. The selected RBS connection was modelled using solid elements while the remainder of the frame was modelled using line elements. In this model the 3-D connection was extended to mid-span and mid-height of the corresponding beam and columns, respectively. The 3-D RBS connection was also laterally braced, in accordance with the AISC Seismic Design Manual (2008), to prevent out-of-plane deformation under inelastic behaviour. Multi Point Constraints (MPCs) were used to connect thermal and mechanical degrees of freedom of the line elements to those of the solid elements. All other modelling details were similar to what has been explained in the previous section. The finite element model is shown in the bottom portion of Figure 8 (a) and the deformed shape of both the frame and the 3-D connection are highlighted. The localized response at the peak and at the end of the time-temperature curve is shown in Figure 8 (b). The demand at the end of the time-temperature curve (i.e. end of cooling phase) clearly is much larger, particularly in the panel zone of the beam-to-column connection, as would be expected. Large stresses are also observed at the weld access holes and the top and bottom flange welds connecting the beam to the exterior face of the column flange. As previously mentioned, this approach allows the evaluation of the influence of the surrounding structure on the response of the connection, or the detailed modelled component, under fire loads in addition to the evaluation of the global performance of the frame. However, a shortcoming of this approach is its inability to capture the out-of-plane response of the 2-D frame as influenced by extension and contraction of girders and floor beams of the interior gravity frames, perpendicular to the plane of the 2-D moment frame.

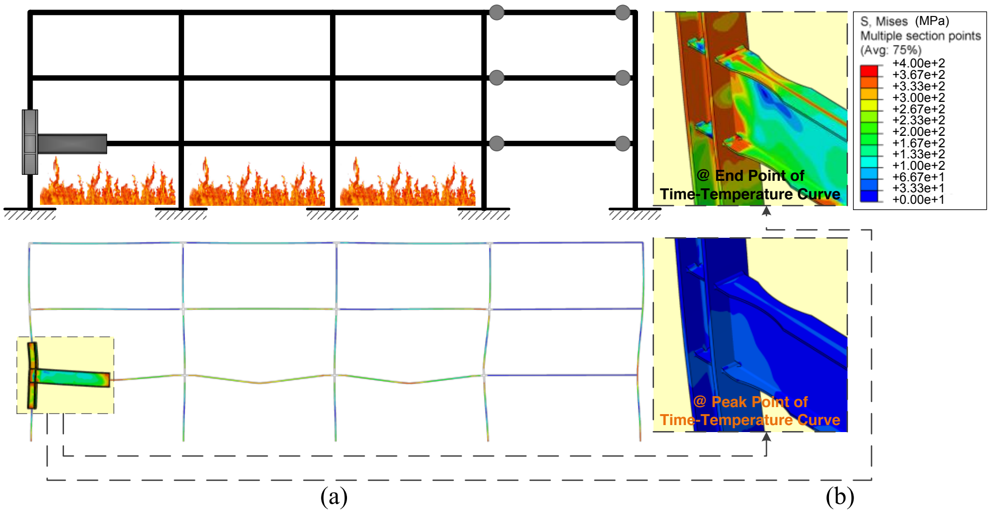


Figure 8 a) Multi-resolution frame exposed to a fire case and corresponding developed deformation, and b) von Mises stress contour in the 3-D RBS connection at the peak temperature and end of corresponding fire case

CONCLUSIONS

This paper provided an overview of the challenges associated with conducting numerical simulations under realistic building fires, including issues related to the use of simplified and unrealistic thermal loadings, idealistic boundary conditions, inaccurate and inconsistent material properties, and the lack of numerical convergence. Examples of alternative approaches for conducting numerical assessment of RBS connections and frames were presented, including the use of temperature-adaptive mechanical boundary conditions, 2-D line element models, and multi-resolution models that integrates both line and continuum elements for the evaluation of local behaviour and global response. Additional research is needed to address many of the issues mentioned to allow for more accurate prediction of response under fire loading and to support performance-based engineering of buildings for fire conditions.

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