Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Atlanta, Georgia, April 21-24, 2020

Role of transient creep in fire induced progressive collapse of steel framed buildings

Venkatachari S[1], Kodur V K R[2]

Abstract
This paper examines the influence of temperature induced transient creep on the onset of instability in steel framed buildings under severe fire exposure. A numerical model is developed in ABAQUS to trace the temperature induced transient creep effects in steel framed structures subjected to fire exposure. The transient creep strain that develops under fire conditions is explicitly accounted for in the fire resistance analysis, in addition to temperature induced property degradation in steel. The developed model is applied to predict the overall response of the structure, as well as failure pattern of a 10-story braced steel framed building under fire exposure. Results from the fire resistance analysis indicate that transient creep strains become dominant when the steel temperature in the member increases beyond 600^{o}C. Further, the results also show that neglecting an explicit treatment of transient creep in the model formulation can lead to unrealistic prediction of failure paths and thus, higher failure times in a steel framed building.

1. Introduction

Steel framed buildings under severe fire exposure can experience loss of stability at local, member or system level, and in rare cases, can experience partial or full collapse of the structure. Fire induced collapse in steel framed buildings gets initiated with the onset of instability in a single or multiple set of critical members, such as columns or main beams, and can propagate through the system leading to failure of the respective compartment, story or the entire structure. The onset of instability in steel members is primarily due to temperature induced degradation in strength and stiffness properties of steel, combined with significant levels of transient creep strain that develops at temperatures beyond 600^{o}C (Wang and Kodur 2019). Loss of structural members from the structural system can lead to high stress levels in the remaining members. These high stress levels can produce high creep strains at temperatures beyond 600^{o}C, which can lead to an early onset of instability in the structure.

In current simplified fire resistance analysis approaches, the effect of temperature induced transient creep strain is often neglected. In advanced fire resistance analysis, undertaken at

[1] Ph.D. Candidate, Michigan State University, <venkat28@msu.edu>
[2] University Distinguished Professor, Michigan State University, <kodur@egr.msu.edu>

member and system level, only partial creep strains are implicitly accounted through temperature dependent stress-strain relations as in Eurocode 3 (EN 1993-1-2 2005). Recent studies (Kodur et al. 2010; Morovat et al. 2014) have shown that such implicit treatment of creep does not capture the realistic deformation response of members in the latter stages of fire exposure and thus overpredict the failure time (or fire resistance) of a member. The extent of temperature induced transient creep deformations in steel members primarily depend on the magnitude, duration and rate of development of stress and temperature level attained in steel under fire exposure. Limited studies (Kodur and Dwaikat 2010; Li and Zhang 2012; Morovat et al. 2014; Kodur and Aziz 2015) have evaluated the temperature induced creep effects on the buckling load of steel columns and fire response of restrained beams at elevated temperature. However, the influence of creep on the overall fire response of steel framed buildings, specifically on the onset of instability at member and system levels, has not been studied.

To address the aforementioned knowledge gaps, a numerical model is built in ABAQUS to trace the overall response of the steel framed building under fire exposure, and specifically incorporate the evolution of temperature induced instability at member and system levels. The transient creep strain that develops under fire conditions is explicitly accounted for in the fire induced progressive collapse analysis, in addition to temperature dependent degradation in thermal and mechanical properties of steel and fire insulation, as well as effects of geometric nonlinearity. The developed numerical model is applied to predict the behavior of a ten-story steel framed structure exposed to fire. The influence of temperature induced transient creep on the overall fire response of the structure, including development of alternate load paths and failure patterns, is evaluated.

2. Numerical Model

In order to evaluate the effect of transient creep on the evolution of temperature induced instability at member and system levels, a numerical model is developed in finite element analysis package, ABAQUS. This model, in addition to various other factors, can explicitly account for the transient creep strain that develops under fire conditions. The details of the numerical model, discretization of the structure, material property relations and failure limit states adopted in fire resistance analysis are discussed in the following sections.

2.1 General Procedure

The analysis procedure consists of a sequentially coupled thermal analysis followed by structural analysis. For the thermal analysis, the steel members are exposed to temperatures as encountered in a fire. A nonlinear transient heat transfer analysis is carried out to obtain the temperature variation with fire exposure time at specific nodes in the member cross-section and these temperatures are subsequently applied on steel members during the structural analysis as thermal load.

The approach for structural analysis is based on the alternate load path criterion recommended in Unified Facilities Criteria (UFC 4-023-03 2016) and GSA guidelines (GSA 2013). The method is extended to analyze the stability of a steel framed structure under fire exposure conditions. The structural frame is subjected to a gravity load of 1.2 D + 0.5 L (where, D and L represents the dead load and live load respectively), as per ASCE 7-16 (ASCE/SEI 7-16 2016) recommendations for extraordinary (low-probability) events such as fires. To account for the effects of initial system imperfections, an additional lateral notional load is applied at each story level corresponding to 0.2% of gravity loading in that floor (ANSI/AISC 360-16 2016). The progression of instability in the steel framed structure at room and elevated temperature is taced by carrying out a nonlinear static analysis in multiple steps. In the first step, the gravity and lateral loads are gradually applied on the structural frame at room temperature until the loads are stabilized. In the second step, the structural frame is subjected to thermal loading obtained from the output of the thermal analysis as predefined fields in the model. The analysis continues in incremental time steps until the failure of the first structural member (or members). In the third step, the failed member(s) is (are) removed at a fixed temperature, and the system is allowed to stabilize under the redistributed loads. Removal of a column is accompanied by an increase in gravity loads in floor areas above the failed column by a dynamic load factor of 2 to account for the inertial effects (UFC 4-023-03 2016). In the following step, the analysis further continues in incremental time steps with the application of thermal loading until the failure of the subsequent member. The above procedure is repeated until the end of fire exposure duration or occurrence of global instability (failure), whichever occurs first.

2.2 Discretization

The thermal model uses a 2-D four-noded DC2D4 element for discretization of cross-section of the steel member and fire insulation. This element has only one active degree of freedom, i.e., the temperature at each node. For the structural analysis, 2-D model of the steel frame is adopted for computational efficiency. The steel framing members (i.e. beams, braces and columns) are discretized using the 2-D three-noded B22 beam element. This beam element has a quadratic order and has three degrees of freedom at each node, namely, two translations (in x and y directions) and one rotation (about z direction). The quadratic Timoshenko beam element can capture large strains, specifically which occurs due to transient creep under fire conditions, and shear deformations.

2.3 Material Properties

To simulate the response of the steel braced frame under fire conditions, temperature-dependent relations specified in Eurocode 3 (EN 1993-1-2 2005) are used to model thermal and mechanical behavior of steel at elevated temperature. In this study, spray-applied fire resistive material (SFRM) formed the fire protection material on structural framing members. The properties of the SFRM type insulation, namely, density, thermal conductivity and specific heat are assumed to be 220 kg/m3, 0.15 W/m-K and 1200 J/kg-K respectively (Kodur and Fike 2010). Fire exposure on steel members is specified through time-temperature relations based on specific fire scenario. Convective heat transfer coefficient and emissivity of 25 W/m^2-^oC and 0.8, respectively, is used for the fire exposed faces and 10 W/m^2-^oC and 0.2, respectively, for the unexposed faces of the structural member (i.e. slabs). Stefan-Boltzmann radiation constant (σ_s) of 5.67×10^{-8} W/m^2-K^4 is assumed for evaluating radiative heat transfer coefficient.

2.4 Explicit model for accounting transient creep in analysis

Transient creep strain that develop in steel is explicitly accounted through the built-in CREEP power law model in ABAQUS. The strain hardening form of power law (shown in Eq. 1) is adopted, as it is relevant for fire resistance analysis where the stress state of the material varies with time.

$$\dot{\varepsilon}^{cr} = (A\tilde{q}^n[(m+1)\bar{\varepsilon}^{cr}]^m)^{\frac{1}{m+1}} \quad (1)$$

in which, $\dot{\bar{\varepsilon}}^{cr}$ is the creep strain rate, $\tilde{q}^{n}$ is the stress, $\bar{\varepsilon}^{cr}$ is the creep strain and A, n and m are temperature dependent creep material parameters. The high temperature creep data from tests carried out by Morovat et al. (Morovat et al. 2012) for A992 steel is used to calibrate the creep material model in ABAQUS using nonlinear regression fitting technique. The model is capable of accounting for primary and secondary creep strains that occur at elevated temperature. The values of parameters A, n and m at different temperatures are tabulated in Table 1.

Table 1: Parameters for creep model

Temperature	Parameters		
T (^{o}C)	A $(MPa)^{-n}(min)^{-m-1}$	n (1)	m (1)
400	1.3E-8	1.77	-0.810
500	6.62E-10	2.294	-0.711
600	1.21E-10	2.921	-0.744
700	5.6E-11	3.788	-0.288
800	5.97E-9	3.525	-0.137

2.5 Failure limit states

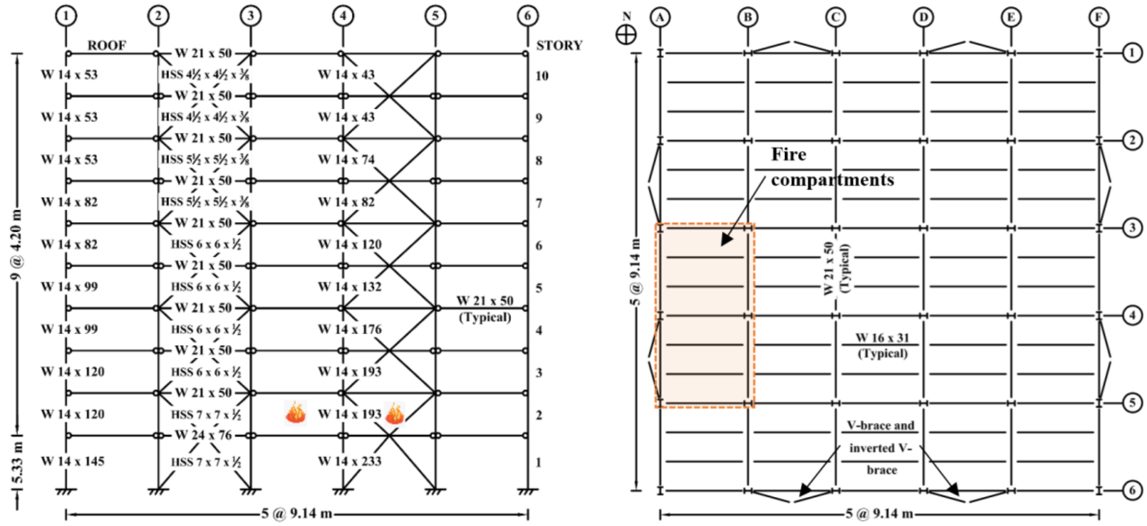
The failure of a member (local failure) is said to have occurred when one of the following limit states is reached (BS 476-20 1987).

i. The demand to capacity ratio (DCR) exceeds 1 (unity) in a structural member.
ii. At any fire exposure time, the deflection of the beam (or slab) exceeds $L/20$ (mm) or the rate of deflection exceeds $L^2/(9000d)$ (mm/min), where L is the length of the beam (or span of slab) (mm), and d is the effective depth of the beam (or slab) (mm).
iii. At any fire exposure time, the vertical displacement of the column exceeds $0.01h$ mm or the rate of deflection exceeds $0.003h$ (mm/min), where h is the height of the column (mm).

The global failure, on the other hand, is taken as the time when the structure is no longer able to maintain static equilibrium (or collapse initiation).

3. Case Study

The above numerical model is applied to evaluate the evolution of temperature induced instability, leading to possible failure of a ten-story braced steel framed building (designed by NIST (Ghosh 2006)) under fire exposure. Fig. 1 shows the elevation and plan of the building along with the steel sections used for the framing members. All beams, columns and bracing members are assumed to be protected with SFRM type insulation with 2 h fire rating. The exterior frame along “A” is selected for this case study. Fire is assumed to start in two interior compartments (as shown in Fig. 1(b)) in the second story and is contained within these compartments for the entire duration. ASTM E119 standard fire exposure (ASTM E119-19 2019) is adopted to calculate the fire temperature in the compartment. The structure is exposed to fire for 240 min.



(a) (b)

Figure 1: Details of braced steel framed building (a) Elevation (b) Plan

Figs. 2, 3 and 4 show the sectional temperatures in different steel members in the fire-affected compartments. As the frame considered in the analysis is an exterior frame, the beams are subjected to a 2-sided fire exposure from the bottom. The column along gridline A4 and braces are exposed to fire from three sides, while the columns along gridlines A3 and A5 are subjected to 2-sided fire exposure. Owing to the fire insulation provided, the temperatures within the cross-section of the members increase slowly in the initial period of fire exposure. Then, the rate of temperature rise further slows down as the increase in fire temperature reduces with time.

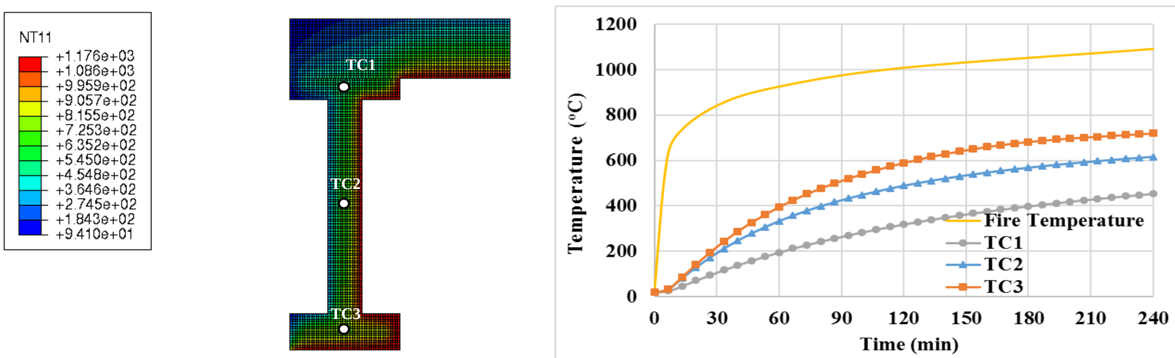


Figure 2: Sectional temperatures in beam W21x50 (A-3-4 and A-4-5) with 2-sided fire exposure

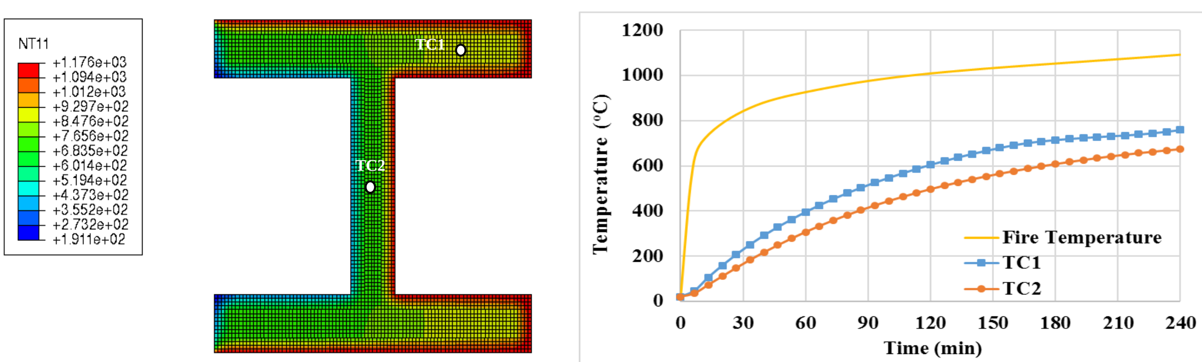


Figure 3: Sectional temperatures in column W14x193 (A4) with 3-sided fire exposure

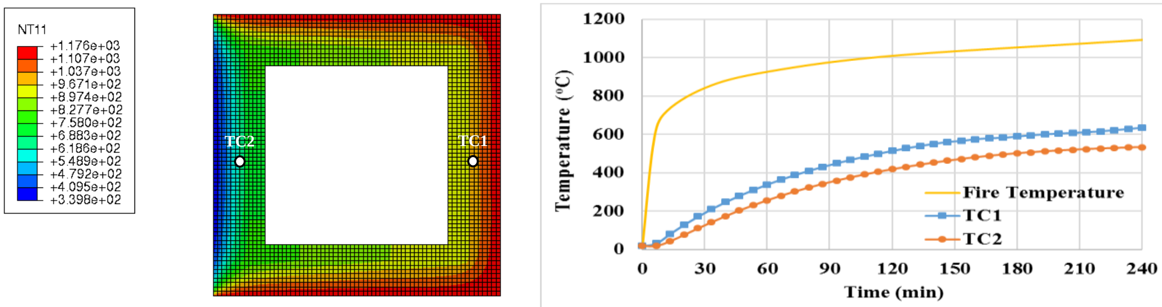
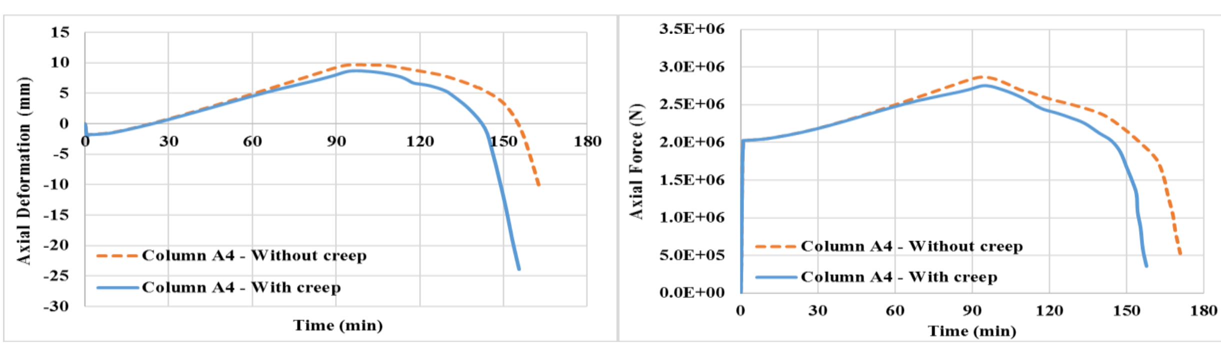


Figure 4: Sectional temperatures in brace HSS7x7x1/2 with 3-sided fire exposure

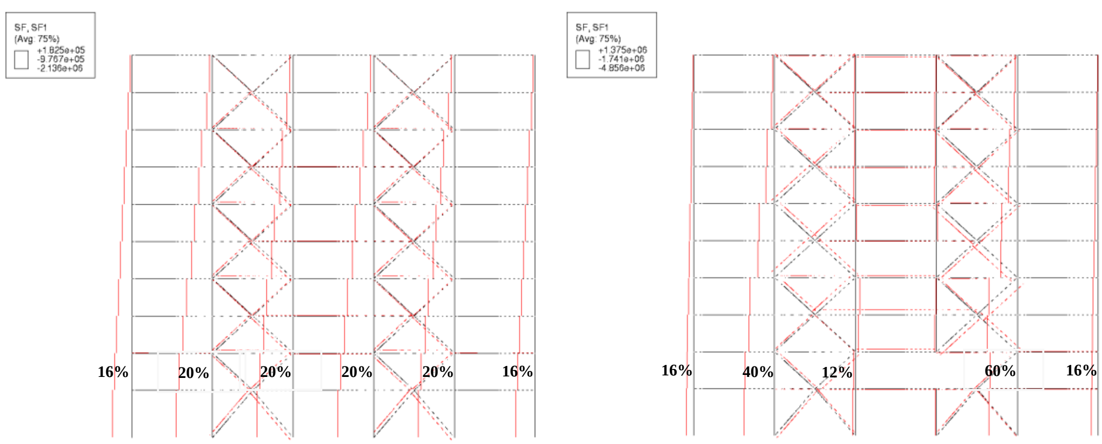
The results from the structural analysis are presented for two cases; one incorporating creep explicitly and the other without incorporating creep explicitly. Fig. 5 shows the axial deformation and axial force response of the steel column along gridline A4. The column initially experiences an axial force of 2100 kN (equal to 20% of its yield capacity) and then, is subjected to fire from three sides. In the initial stage, the axial deformation response is primarily governed by the onset of temperature induced thermal strains without much contribution from mechanical and transient creep strains. During this stage, there is minimal degradation in mechanical properties of steel due to cross-sectional temperatures lesser than 400°C and stress levels in the column remain below 25%. In this stage, the column undergoes expansion, and the responses predicted using the analyses “with creep” and “without creep” are similar. With fire exposure time, as the temperature in the cross-section rise beyond 400°C, there is larger degradation in the strength and stiffness properties of steel and stress levels rise to 40%. This results in an increase in mechanical strain and also onset of transient creep strain which offsets some of the thermal strain. In this stage, the column continues to expand as this column is lightly loaded; however, the rate of expansion reduces. The difference between the responses predicted using the models with and without creep start to emerge in this stage. In the final stage, when steel temperatures increase beyond 600°C and stress levels on the column reach to about 50%, mechanical and transient creep strains dominate the response as can be seen for the case of analysis “with creep”. The axial deformation of the column shifts from expansion to contraction. Instability begins to set in when the axial force in the column drops to zero and ultimately column fails at about 154 min in the analysis “with creep”. During this stage, the difference in the deformation and force response predicted using the models with and without creep becomes evident. The model “without creep” underestimates the extent of axial deformation and axial force developed in the steel column and hence, overestimates the time at which instability sets in and predict the failure time to be at 163 min.



(a) (b)

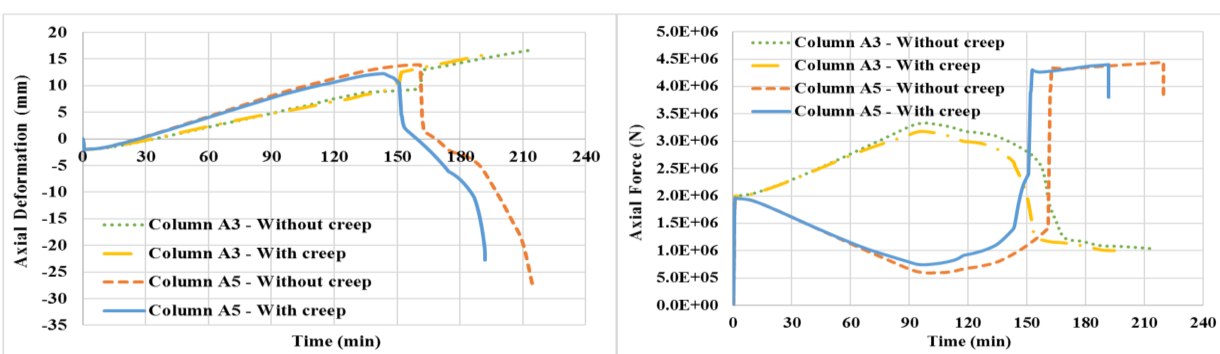
Figure 5: (a) Axial deformation and (b) axial force in column along gridline A4

In the subsequent analysis step, the failed Column A4 is removed. Due to redistribution of forces, the brace adjoining the failed column immediately fails in the same step and hence, is also removed from the frame. Fig. 6 shows the axial force distribution in the braced steel frame following the application of gravity and lateral loading at room temperature and the redistributed forces in the frame at 155 min after the removal of Column A4 and left brace. The initial stress levels in the columns in the fire affected story are 20% and lesser. Following the removal of one column and brace, Column A5 and Column A3 (which are at a temperature of about 500^{o}C) experiences a stress level of 60% and 12% respectively. The Column A2 (which is still at room temperature) is stressed to 40%. Fig. 7 shows the axial deformation and axial force response of Column A3 and A5. As Column A3 and Column A5 are exposed to fire from only two sides, the axial deformations in these columns are lesser when compared to Column A4. Following the removal of Column A4 and brace, the deformation of Column A5 rapidly increases (in the downward direction) due to high stress levels. The models "without creep" and "with creep" predict a failure time of 219 min and 191 min respectively. The difference in failure times using models with and without creep is about 28 min, as compared to 9 min difference obtained in the case of failure of Column A4. The redistribution of forces following the removal of Column A4 and left brace leads to higher stress levels. This, when combined with higher steel temperature, results in significantly higher amount creep strain and in turn, a higher difference in the failure times using models with and without creep.



(a) (b)

Figure 6: Axial force profile of steel frame at (a) room temperature following the application of initial gravity and lateral loads (b) 155 min of fire exposure using analysis "with creep"



(a) (b)

Figure 7: (a) Axial deformation and (b) axial force in columns along gridline A3 and A5

Fig. 8 shows the progression of mid-span deflection in the beams in the fire affected compartments. Beam A-3-4, located in the unbraced bay, shows much higher deflections as compared to Beam A-4-5 which is in the braced bay. Although both the beams are subjected to the same level of temperature, the Beam A-4-5 experiences a larger axial force than Beam A-3-4. Hence, the creep strains developed in Beam A-4-5 is much higher as compared to that developed in Beam A-3-4. In both beams, the model "without creep" underestimates the deflections that occur in the beams. This results in unrealistic predictions of deformations in the adjoining members in the frame.

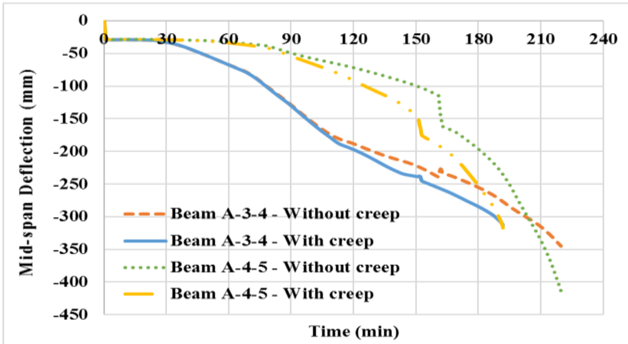


Figure 8: Mid-span deflection of beams in the fire affected compartments

In the subsequent analysis step (using models with or without creep), removal of Column A5 results in failure of multiple braces in higher floors in the bay above the failed columns due to high levels of redistributed loads. Removal of these braces results in the failure of the two right bays throughout the height of the frame. Consequently, the building is likely to experience progressive collapse at this stage and the analysis is terminated.

4. Limitations and Future Direction

The focus of this study is limited to quantifying the effect of temperature induced creep on the progression of instability in a steel framed structure exposed to fire. Specific attention is given to study the evolution of instability at member and system levels, as well as development of alternate load paths, which will influence the likelihood of progressive collapse, when creep effects are explicitly accounted for in the analysis. In the current FE model, a 2-D braced steel frame is considered for computational efficiency. This approach does not fully account the beneficial effect arising from the restraint offered in a 3-D framing and floor system, present in a building. To address this, the study is currently being extended to include the 3-D framing and slabs in the numerical model to realistically predict the fire response and failure times at a system level. The analysis, with further complexity, is also being extended to include other structural systems including buildings with moment frame, rigid core, or mixed framing configurations.

5. Conclusions

Based on the results of the case study presented, the following conclusions can be drawn on the effect of transient creep on the evolution of instability in steel framed structures exposed to fire.

i. Neglecting transient creep effects in the fire resistance analysis of a steel framed structure results in underestimation of deformations at a member level, specifically at stages close to failure and thus, overestimation of failure time (or fire resistance).

ii. When the cross-sectional temperatures in the member are below 400^{o}C and stress levels are below 30%, the transient creep effects tends to be negligible. In such scenarios, implicit treatment of creep is found to be sufficient for predicting the fire response of the steel framed structure. Explicit treatment of creep is needed when temperatures in the member reaches above 600^{o}C and stress levels exceed 50%.
iii. Following the failure of one or more members in the system, redistribution of loads results in higher stress levels in the adjoining members in the fire affected compartments. In this case, creep effects dominate the response and leads to early onset of instability in the fire exposed steel structure.

Acknowledgments
The authors would like to express their gratitude to S. K. Ghosh and Associates, Inc. for sharing the design document of the ten-story braced steel framed building used in this study.

References

ANSI/AISC 360-16 (2016) Specifications for structural steel buildings. American Institute of Steel Construction, Chicago, IL

ASCE/SEI 7-16 (2016) Minimum Design Loads and Associated Criteria for Buildings and Other Structures. American Society of Civil Engineers, New York

ASTM E119-19 (2019) Standard Test Methods for Fire Tests of Building Construction and Materials. ASTM International, West Conshohocken, PA

BS 476-20 (1987) Fire tests on building materials and structures – Part 20: Method for determination of the fire resistance of elements of construction (general principles). BSI, UK

EN 1993-1-2 (2005) Eurocode 3-Design of Steel Structures, Part 1-2: General Rules for Structural Fire Design. Commission of European Communities, Brussels, Belgium

Ghosh SK (2006) Assessing ability of seismic structural systems to withstand progressive collapse: Design of steel braced frame buildings. Report - S. K. Ghosh Associates, Inc., Palatine, IL

GSA (2013) Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance. U.S. General Services Administration, Washington, D.C.

Kodur V, Dwaikat M (2010) Effect of high temperature creep on the fire response of restrained steel beams. Materials and structures 43:1327–1341

Kodur V, Dwaikat M, Fike R (2010) High-temperature properties of steel for fire resistance modeling of structures. Journal of Materials in Civil Engineering 22:423–434

Kodur V, Fike R (2010) Guidelines for improving the standard fire resistance test specifications. In: Advances in the State of the Art of Fire Testing. ASTM International

Kodur VK, Aziz EM (2015) Effect of temperature on creep in ASTM A572 high-strength low-alloy steels. Materials and Structures 48:1669–1677

Li G-Q, Zhang C (2012) Creep effect on buckling of axially restrained steel columns in real fires. Journal of Constructional Steel Research 71:182–188

Morovat MA, Engelhardt MD, Helwig TA, Taleff EM (2014) High-Temperature Creep Buckling Phenomenon of Steel Columns Subjected to Fire. Journal of Structural Fire Engineering 5:189–202. https://doi.org/10.1260/2040-2317.5.3.189

Morovat MA, Lee JW, Engelhardt MD, et al (2012) Creep properties of ASTM A992 steel at elevated temperatures. Trans Tech Publ, pp 786–792

UFC 4-023-03 (2016) Design of Buildings to Resist Progressive Collapse, With Change 3. Department of Defense, Washington, D.C.

Wang W, Kodur V (2019) Material Properties of Steel in Fire Conditions. Academic Press, Cambridge, MA