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Modeling Temperature-Induced Instabilities for Tracing the Fire-Induced Progressive Collapse in Steel-Framed Buildings

Svetha Venkatachari[1], Venkatesh Kodur[2]

Abstract
This paper presents a series of advanced simulations carried out using the finite element software ABAQUS to trace the fire-induced progressive collapse in steel-framed buildings. An approach for incorporating the full effects of high-temperature creep, fire spread, restraint conditions, local instability, and different failure limit states in the fire resistance analysis will be outlined. The evolution of temperature-induced instabilities leading to the progressive collapse in a fire-exposed steel-framed building will be discussed. Results from the numerical studies will be utilized to recommend guidelines for minimizing the onset of fire-induced instability, as well as progressive collapse, in critical buildings.

1. Introduction

Steel framing is widely adopted in multi-story buildings owing to the enhanced strength and ductility properties of steel as well as the ease of fabrication and construction of such structures. However, steel-framed buildings are more vulnerable to damage under fire exposure as steel possesses high thermal conductivity and undergoes rapid degradation of its strength and stiffness properties with the temperature rise (Kodur and Naser 2020). When exposed to severe fires steel-framed buildings can experience instability at a local or global level which can lead to the progressive collapse of a part or the entire structure (Jiang et al. 2014; Agarwal and Varma 2014). To overcome this problem steel structures always need some level of fireproofing (insulation) applied to them.

The onset of instability in steel-framed buildings under fire exposure is influenced by various factors. These instability effects alter the load paths continuously during a fire event. It is critical to identify and quantify the effects of these factors to trace fire-induced progressive collapse in steel-framed buildings. Most of these factors are dependent and influence the response of a steel framed building at different levels, namely, material level, sectional level, member level, and/or global level. For instance, the onset of instability at member and system levels is influenced by high-temperature creep effects, which in turn depend on the severity of the fire (temperature

[1] Guest Faculty, National Institute of Technology Puducherry, <svethachari@gmail.com>
[2] University Distinguished Professor, Michigan State University, <kodur@egr.msu.edu>

rise), load (stress) levels, and the extent of burning (fire spread), and so on (Venkatachari and Kodur 2021).

Steel sections are also susceptible to instabilities owing to specific design considerations (such as the use of non-compact sections in compression members) which can undergo local buckling at elevated temperatures and precipitate the onset of fire-induced collapse (Kodur and Naser 2015). Further, any detachment of fire insulation from the steel members can lead to an early onset of instability in steel members leading to sudden or abrupt failure of the structure. Such a failure can jeopardize the safety of the occupants and first responders, provide insufficient time to tackle the spread of fire and affect the integrity of the entire building itself.

The current approach to fire resistance assessment of a steel-framed building is undertaken at a sectional or member level, and this type of analysis does not fully account for the realistic load, fire spread, restraint conditions, and as well as associated instabilities that occur during a fire event (Kodur and Naser 2020). Limited studies are available in the literature that traces the system level response of steel-framed buildings under fire conditions including fire-induced collapse (Sun et al. 2012; Agarwal and Varma 2014; Jiang et al. 2015; Nguyen 2017; Rackauskaite et al. 2017; Qin and Mahmoud 2019). Even these studies do not fully account for critical factors including fire scenarios and fire spread, high-temperature creep effects, local buckling effects, and realistic failure criteria in the fire resistance analysis of steel-framed structures.

To address these knowledge gaps, this paper presents a series of numerical studies carried out on a ten-story steel-framed building using the finite element software ABAQUS. The key parameters that are to be considered to predict the onset of temperature-induced instabilities leading to the progressive collapse in a steel-framed building is outlined. Further, an approach for tracing the evolution of temperature-induced instabilities leading to the progressive collapse of a steel-framed building, while accounting for the full effects of high-temperature creep, fire spread, restraint conditions, local instability, and different failure limit states is presented. Finally, preliminary guidelines for minimizing the onset of fire-induced instability, as well as progressive collapse, in critical buildings are recommended.

2. Factors Influencing Fire-Induced Instability in Steel-Framed Buildings

Fire-induced instability in a steel member is influenced by various factors including the load (stress) level, geometry (or slenderness) of the member, restraint (support) conditions, fire severity, thermal gradients, and high-temperature properties of steel, including transient creep strains. The effect of most of these factors on the fire response of individual steel members such as beams and columns has been well studied in the literature (Kodur and Dwaikat 2010; Li and Zhang 2012; Agarwal et al. 2014; Aziz et al. 2015; Kodur and Naser 2015). On the other hand, limited studies are available in the literature that quantifies the effect of critical factors that govern the fire response of a steel structure at a system level.

The factors that influence the onset of structural instability under fire conditions leading to progressive collapse can be broadly grouped under three categories, namely, fire parameters, material parameters, and structural parameters. Fire parameters include the intensity and duration of fire exposure, number of compartments or floors burning (extent of fire spread), location of the fire (burning), etc., Fires with low to moderate severity may not be critical for tracing the fire-induced progressive collapse in steel-framed buildings. However, severe to very intense fire exposure scenarios are more likely to cause excessive damage and need to be considered for evaluating the onset of the fire-induced collapse. In addition, the extent of burning and fire location can alter the load paths and failure sequences in a fire-exposed building and are critical for evaluating fire-induced collapse.

The temperature-dependent properties of steel, concrete and fire insulation, including degradation of strength and stiffness with temperature rise, the onset of high-temperature creep, residual stresses, delamination of fire insulation, etc. are critical material parameters that influence the onset of instability in steel framed buildings. The structural parameters include the configuration of structural framing (braced or moment frame, and connection types), development of restraint forces, level (and any loss) of fire protection, continuously altering load paths, the onset of local instabilities, etc.

3. Advanced Analysis Approach for Tracing the Fire-Induced Progressive Collapse

To account for all critical factors influencing the onset of instabilities in a fire-exposed steel-framed structure, advanced analysis at a system level is to be carried out. The finite element-based program ABAQUS is used for undertaking the advanced analysis for tracing the progressive collapse of a steel-framed building under fire conditions. The fire-induced progressive collapse analysis involves a three-step procedure. The first step involves the determination of fire scenarios that are to be considered for the progressive collapse analysis. In this study, in the fire resistance calculations, the fire temperatures are traced using standard or parametric temperature-time relations specified in ASTM E119 (ASTM E119-19 2019) and Eurocode 1 (EN 1991-1-2 2002).

In the second step, thermal analysis of the fire-exposed steel members is carried out to obtain the cross-sectional temperatures in each member as a function of fire exposure time. The cross-section of the fire-exposed structural members along with any fire insulation is discretized using 4-noded DC2D4 elements. The fire temperatures are applied using convection and radiation-type boundary conditions. The parameters for these boundary conditions are defined as per Eurocode 1 (EN 1991-1-2 2002) provisions. The temperature-dependent thermal properties that are utilized in the thermal model are specified as per Eurocode 2 and 3 (EN 1992-1-2 2004; EN 1993-1-2 2005). A transient heat transfer analysis is carried out to obtain the variation of sectional temperatures with fire exposure time.

The final (third) step involves the structural analysis of the steel-framed structure at ambient and elevated temperature conditions. In this step, the load-bearing members in the steel framed building including framing and slab members are discretized using 2-noded B31 (beam) elements and 4-noded S4R (shell) elements, respectively, for undertaking the structural analysis. The connections between different members and support conditions in the building are modeled through specified joint constraints and boundary conditions in ABAQUS.

Two loading sequences are used in the structural analysis. First, the gravity loads, which are obtained as 1.2 DL + 0.5 LL (where DL and LL represent the dead load and live load respectively), and notional lateral loads, evaluated as 0.2% of gravity loading on that floor, are applied gradually to the steel framed building. Second, sectional temperatures obtained as output from the thermal analysis are applied to the fire-exposed members as predefined temperature fields. Temperature-dependent mechanical property relations for concrete and steel, as specified in Eurocode 2 and 3 (EN 1992-1-2 2004; EN 1993-1-2 2005), respectively, are given as input to the structural model.

When assigning the stress-strain data for steel in the structural analysis, it is to be specified whether transient creep is accounted for implicitly or explicitly. In scenarios, where only implicit treatment of creep is needed, the stress-strain relations specified in Eurocode 3, which incorporate the transient creep strains implicitly in their formulations, are specified as input. For incorporating creep explicitly in the fire-induced progressive collapse analysis, the creep strains that develop at elevated temperatures are computed at each time step, in addition to the mechanical and thermal strains. The built-in creep power law in ABAQUS is used to evaluate the transient creep strain at each time increment and the high-temperature creep data from tests carried out by Morovat et al. (Morovat et al. 2012) for A992 steel is used for calibration. The parameters used in the creep model are published in (Venkatachari and Kodur 2021).

Temperature-induced local buckling in fire-exposed steel members is to be included in the analysis when steel sections with high slenderness ratios are used in the fire compartments. To account for the local buckling effects in the fire-induced progressive collapse analysis, a mixed-element approach is utilized to discretize the framing members in the building. In this approach, the fire-exposed steel members are modeled using shell (S4R) elements while the other steel members are modeled using beam (B31) elements. The connection between the different elements is specified using appropriate kinematic coupling constraints in ABAQUS to achieve a pinned or fixed connection.

The progression of instability in the steel-framed structure is traced continuously throughout the fire exposure by carrying out either a nonlinear quasi-static analysis or a nonlinear dynamic explicit analysis. The appropriate analysis type is chosen based on the parameters to be included in the analysis to obtain a computationally efficient and robust solution. The response parameters, including stresses and displacements, are traced at various points in the structure including to trace the progression of instabilities in the building. At each time step, these values are used to check for failure at the member or system level based on the limiting criteria as specified in (Venkatachari and Kodur 2021). Progressive collapse is said to initiate when the system level failure limits are exceeded.

The proposed analysis approach has been validated both at the member level and system level using fire test data published in the literature. Results from the validation studies showed that the model predictions have a good correlation with the experimental temperature and displacement response. A full discussion of the validation studies is reported in (Venkatachari and Kodur 2020, 2021).

4. Numerical Studies

To quantify the effect of critical parameters on the onset of temperature-induced instabilities in steel-framed buildings, a series of numerical studies are carried out using the proposed finite element-based model. A description of the selected test building, varied parameters, and results from the numerical studies are presented in the following sections.

4.1 Test building and varied parameters

A ten-story steel-framed building, designed for NIST (Liang et al. 2006) for examining the robustness of the structure to resist disproportionate collapse, is used for this study. Fig. 1 shows the elevation and plan of the selected test building. A992 (Grade 50) steel is used for the framing members. The floor system in the building comprises an 83 mm lightweight reinforced concrete (compressive strength = 21 MPa) slab topping a 76 mm deep ribbed metal deck. The framing members are 2-hour fire rated as per IBC 2021 (IBC 2021) provisions while the thickness of the concrete slab is found to provide a fire rating of 1 hour and no external fire insulation is applied to the floor slab. Full details of the building are presented in (Venkatachari and Kodur 2020).

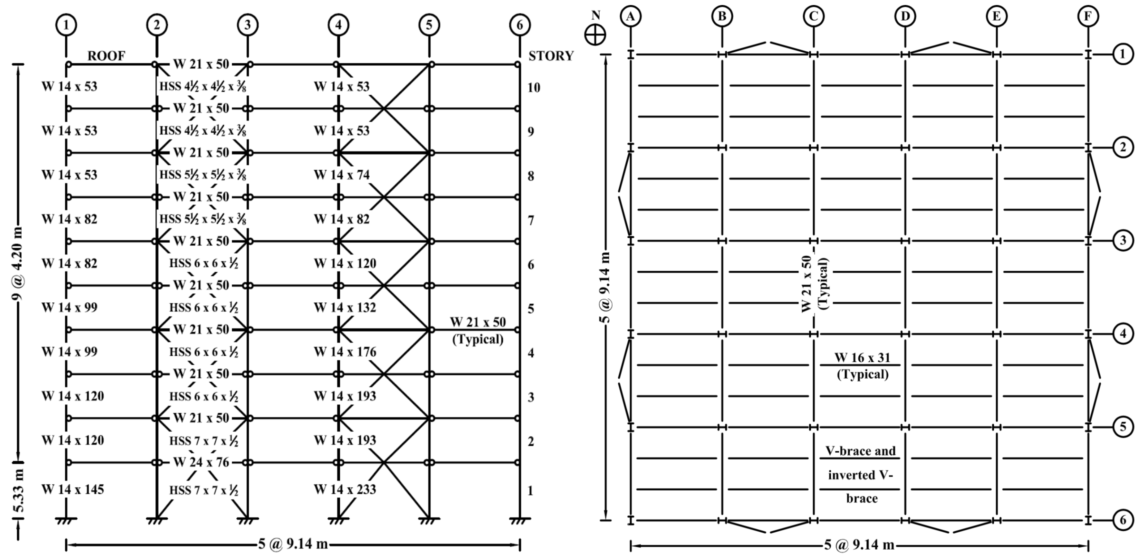


Figure 1: Elevation and plan of the selected test building

A total of 16 cases are presented under three broad categories: fire parameters, material parameters, and structural parameters. Under fire parameters, the influence of the type of fire exposure, the extent of burning, and the spread of fire from one region to another are quantified. Fig. 2 and Fig. 3 show the types of fire exposure and fire spread scenarios considered in the study. Under material parameters, the effect of high-temperature creep on the onset of temperature-induced instabilities is evaluated for different fire scenarios. The influence of including creep effects explicitly in the fire resistance analysis is compared with that of an implicit treatment to creep effects. Under structural parameters, two cases are considered. In the first case, the effect of including local instabilities in the fire-induced progressive collapse analysis is quantified. In the second case, the influence of including the transverse framing and composite floor slab in tracing the progression of instabilities is evaluated. Table 1 summarizes the parameters considered for the numerical studies.

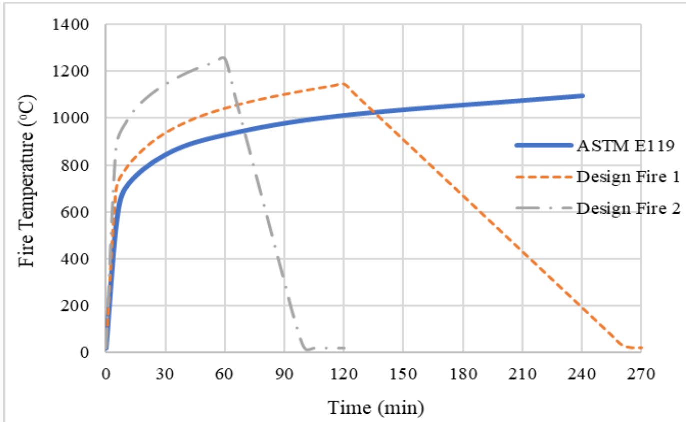


Figure 2: Fire exposure scenarios considered for the study

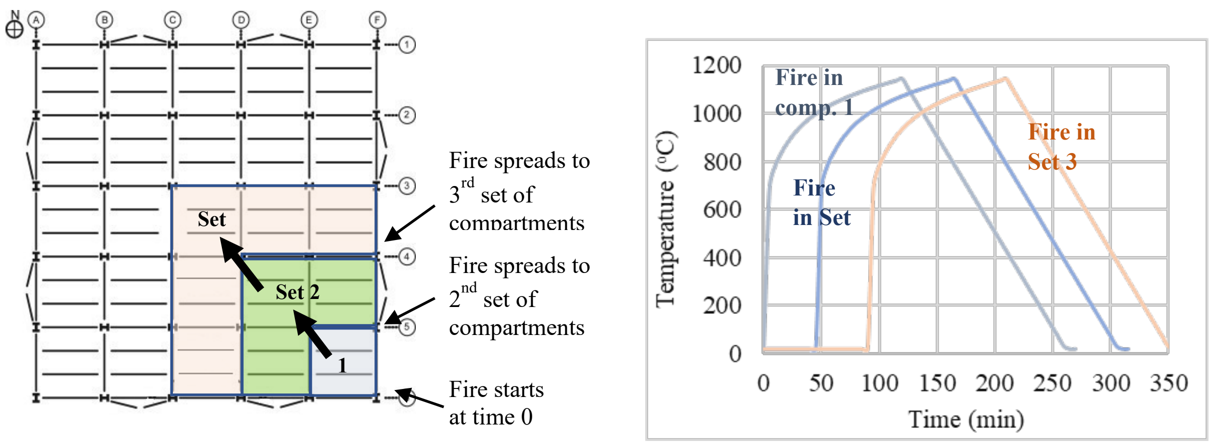


Figure 3: (a) Horizontal fire spread scenario in one story (b) Temperature-time variation in the fire-exposed compartments

4.2 Results from numerical studies

The results from the numerical studies carried out are presented under three sections, namely, the effect of fire parameters, the effect of high-temperature creep, and the effect of structural parameters.

Effect of fire parameters:

To quantify the effect of the type of fire exposure, fires with different intensities are applied to the steel-framed building in the compartments shown in Table 1. Under ASTM E119 fire exposure, the steel members experience a rapid increase in sectional temperatures due to the steep rise in fire temperatures in this scenario. Whereas, under the design fire scenarios, the temperature progression in steel members is more gradual. In the case of design fire 2, the steel temperatures do not exceed 550°C due to the shorter burning duration of fire exposure than the other two fire scenarios. The lateral displacement at the top story level of the building is traced under the different fire exposure scenarios (Fig. 4). It is seen that the lateral displacement begins to increase rapidly under the ASTM E119 scenario and design fire at around 120 min and 100 min, respectively, leading to the onset of progressive collapse under these fire conditions. Under design fire 2, no collapse occurs due to the shorter burning durations and lower sectional temperatures in the steel members.

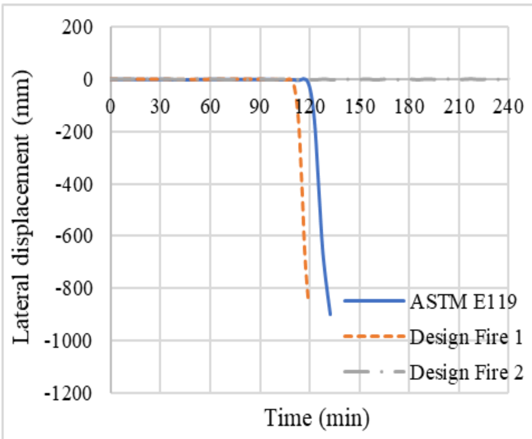
Table 1: Varied parameters for the numerical simulations

Varied parameter	Fire compartments	Fire scenario	Analysis regime	Failure time (min)	Occurrence of progressive collapse (Yes/No)
Fire parameters					
Fire intensity	Floor 1 –D-F & 4-6	ASTM E119	Dynamic	132.5	Yes
	Floor 1 – D-F & 4-6	Design Fire 1	Dynamic	119	Yes
	Floor 1 – D-F & 4-6	Design Fire 2	Dynamic	NF[1]	No
Extent of burning	Floor 1 – E-F & 2-4 (one story)	ASTM E119	Dynamic	NF	No
	Floors 2-3 – E-F & 2-4 (two stories)	ASTM E119	Dynamic	NF	No
	Floors 2-4 – E-F & 2-4 (three stories)	ASTM E119	Dynamic	165	Yes
Fire spread	Horizontal fire spread (HF): Floor 1 – C-F & 3-6	Design Fire 1	Dynamic	154	Yes
	Vertical fire spread (VF): Floors 1-3 – D-F & 4-6	Design Fire 1	Dynamic	100	Yes
Material parameters					
High-temperature creep	Floor 2 – A & 3-5 (implicit creep)	ASTM E119	Quasi-static	219	Yes
	Floor 2 – A & 3-5 (explicit creep)	ASTM E119	Quasi-static	191	Yes
	Floor 2 – A & 3-5 (implicit creep)	Design Fire 1	Quasi-static	143	Yes
	Floor 2 – A & 3-5 (explicit creep)	Design Fire 1	Quasi-static	131	Yes
Structural parameters					
Local instability	Floor 9 – A & 3-5	ASTM E119	Quasi-static	199	No
	Floor 9 – A & 3-5	ASTM E119	Quasi-static	179	No
Load Paths	Floor 2 – F & 2-4	ASTM E119	Dynamic	210	Yes
	Floor 2 – E-F & 2-4	ASTM E119	Dynamic	NF	No

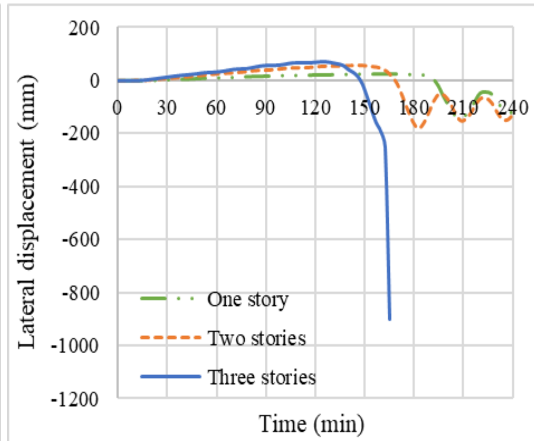
1. NF – No failure

Multiple stories can be impacted by fire due to specific design considerations adopted in steel-framed buildings, such as ventilated cladding systems that allow the propagation of fire through the air cavity behind the cladding. When the extent of burning is limited to two compartments in one or two stories, no global collapse is initiated. This can be seen from the lateral displacement response (Fig. 4) which shows that the lateral displacement at the top story level stabilizes with time in these scenarios. However, when the fire exposure is considered in three stories, with two compartments burning on each floor, progressive collapse is initiated at 165 min as seen from the runaway type of response in Fig. 4(b).

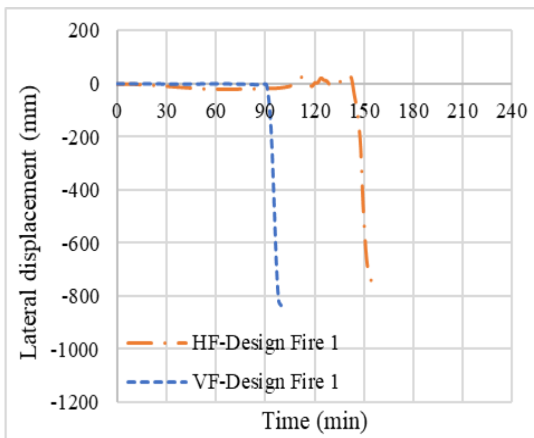
To study the effect of fire spread from one region of the building to another, two scenarios are considered. In the first scenario, a horizontal spread of fire within one story is simulated with the progression of fire and fire temperatures considered as per Fig. 3. In the second scenario, a vertical fire spread is considered, with fire starting in four corner compartments (between gridlines D-F & 4-6) on floor 1 and then spreading to floors 2 and 3 at a 45 min interval. From Fig. 4(c) it can be seen that both scenarios result in the onset of global failure. However, the vertical fire spread scenario is more severe due to the larger extent of burning and multiple failures in structural members on different floor levels compared to the scenario with horizontal fire spread. The deformed shape of the building before collapse under the two fire spread scenarios is shown in Fig. 5.



(a) Varying fire intensity

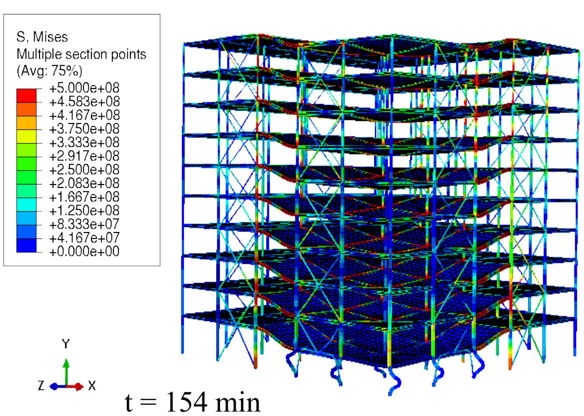


(b) Varying extents of burning

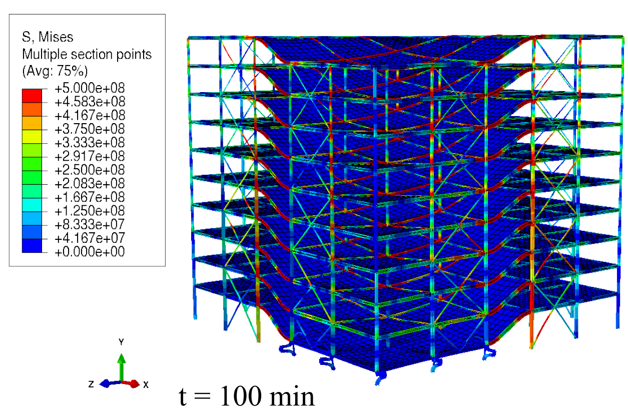


(c) Varying fire spread scenarios

Figure 4: Effect of fire parameters on the onset of fire-induced progressive collapse



(a) Horizontal fire spread with Design Fire 1



(b) Vertical fire spread with Design Fire 1

Figure 5: Deformed shape of the building under varying fire spread scenarios

Effect of high-temperature creep:

To quantify the effect of high-temperature creep on the onset of fire-induced collapse, the fire resistance analysis is carried out under two fire scenarios, namely, ASTM E119 fire exposure and design fire 1. In each scenario, the effect of incorporating high-temperature creep through the implicit approach and explicit approach is evaluated. Fig. 6 shows the lateral displacement of the steel frame at the top story level with fire exposure time under different fire scenarios predicted using models with implicit creep and explicit creep. Under severe fire exposure, as in the case of ASTM E119 fire scenario and design fire 1, it is seen that the onset of instability is highly sensitive to the method in which creep is incorporated in the analysis. The analysis using explicit creep provides conservative estimates of the failure time in steel-framed buildings.

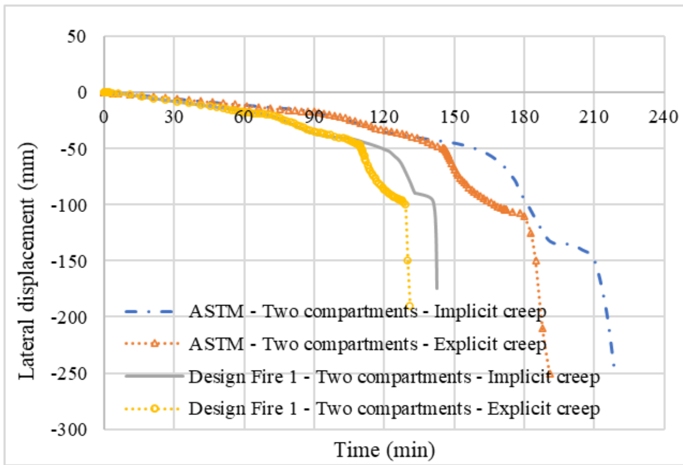
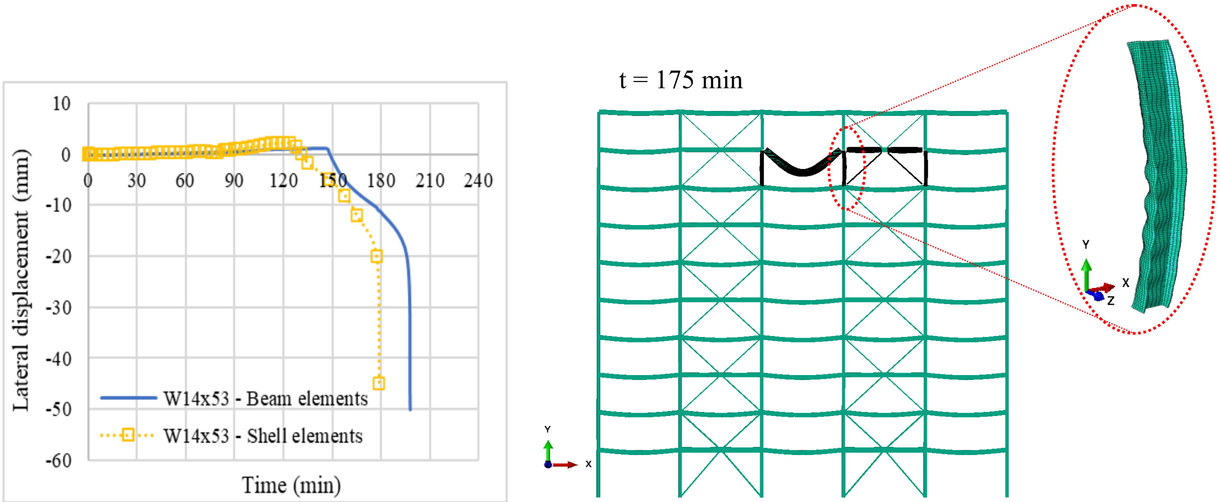


Figure 6: Effect of high-temperature creep on the onset of fire-induced progressive collapse

Effect of structural parameters:

To quantify the effect of including local instabilities in the fire-induced progressive collapse analysis, two scenarios are considered. In the first scenario, all steel structural members are discretized using beam elements, which are not capable of capturing any distortions in the cross-section such as local buckling, warping, etc. In the second scenario, a mixed-element approach is applied where the steel members in the fire compartments are modeled using shell elements and the remaining members are modeled using beam elements. Fig. 7 shows the lateral displacement at the top story of the frame for the two scenarios and the deformed shape of the frame when the mixed-element approach is applied in the fire resistance analysis. The middle column W14x53 experiences local buckling under fire conditions and hence, the model that includes the effect of local instabilities predicts a failure time of 179 min, 20 min lesser than that predicted using the model neglecting the effect of local instabilities.

In the final study, the effect of including the transverse framing and floor slab in the fire-induced progressive collapse analysis is evaluated. Two structural configurations are considered; one with only the 2D steel frame along gridline F (Fig. 1) and the other including the entire 3D structure with the framing in two directions and the floor slab. Results from the analyses show that the while the 2D building frame subjected to fire exposure in two compartments experiences collapse at 210 min, the 3D building frame with similar fire exposure does not undergo collapse (Fig. 8). In the case of the 3D building, although several members in the fire-exposed compartments fail, the structure is able to redistribute the loads to the neighboring members in all directions, and hence, does not collapse. Moreover, the presence of the composite floor slab limits the deflection in the beams, thereby adding to the stability of the structure.



(a) (b)

Figure 7: Effect of local instabilities on the onset of fire-induced progressive collapse

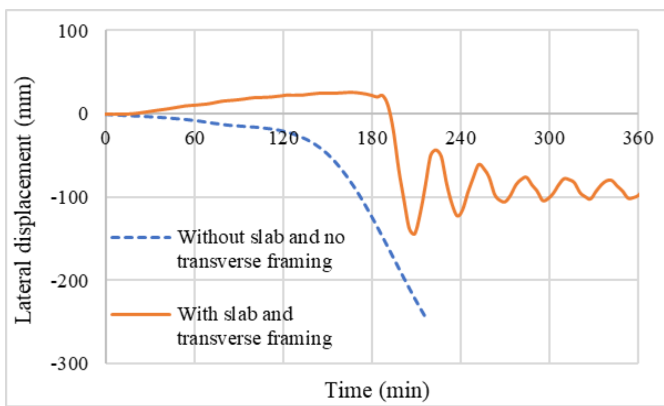


Figure 8: Effect of varying load paths on the onset of fire-induced progressive collapse

5. Preliminary Guidelines for Minimizing the Onset of Fire-Induced Collapse

Fire-induced progressive collapse analysis is highly complex and computationally expensive. Moreover, the design for fire-induced collapse may not be critical for all buildings. Hence, it is recommended that only buildings that are at a high-risk level from the perspective of fire hazards may be designed for fire-induced collapse. Such high-risk buildings can be identified using guidelines specified for ambient conditions such as the GSA guidelines (GSA 2013). Additional factors which are specific to fire conditions such as the severity of fire exposure and the extent of fire spread that is likely to occur in a building can be evaluated in identifying scenarios that are critical for fire-induced collapse.

Buildings that are in the high-risk category can be provided with suitable design features, such as the recommendations in the FEMA 426 (BIPS 06/FEMA 426 2011) document, to reduce the potential for fire-induced collapse. In addition, features that are critical for minimizing the vulnerability of the building to fire hazards such as the design for fire protection to steel members, compartmentation, etc. are needed. In the case of high-risk buildings, implementation of performance-based fire design approaches using advanced analysis can yield optimum fire resistance strategies that are unique for a specific building. The advanced analysis approach presented in this paper can be applied to a series of probable fire and loading scenarios to determine the robustness of the building against fire-induced collapse. Further, the results from each case can be utilized to develop strategies such as improving the steel sections, connections, fire protection system, etc.

6. Conclusions

The advanced simulations presented in this paper, which evaluated the influence of several critical parameters on the onset of instabilities leading to the collapse of steel-framed buildings, led to the following findings:

i. The severity of fire exposure, the extent of burning, and fire spread have a significant influence on the onset of fire-induced progressive collapse in a steel-framed building. Under severe fire exposure (peak fire temperatures over 1000ºC and burning for more than 2 hours), fire-induced progressive collapse is triggered when the fire spreads to three stories with 15% of the floor area burning on a single floor.
ii. Temperature-induced creep strains influence the onset of instability in steel-framed structures. Neglecting transient creep effects in the fire-induced progressive collapse analysis can result in an underestimation of global failure times.
iii. Neglecting the effect of local buckling in the finite element model can result in failure times that are 10% higher than models that consider local buckling effects using shell elements for fire-exposed framing members.
iv. The composite floor slab and transverse framing add significant robustness against fire-induced collapse. Neglecting their contribution results in unrealistic load distributions and failure times.

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