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Simulating the Response of a 10-Storey Steel-Framed Building under Spreading Multi-Compartment Fires

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Abstract

This paper presents a numerical investigation on the structural response of a multi-story building subjected to spreading multi-compartment fires. A recently proposed simple fire model has been used to simulate two spreading multi-compartment fire scenarios in a 10-story steel-framed office building. By assuming simple temperature rising and distribution profiles in the fire exposed structural components (steel beams, steel column and concrete slabs), finite element simulations using a three-dimensional structural model has been carried out to study the failure behavior of the whole structure in two multi-compartment fire conditions and also in a standard fire condition. The structure survived the standard fire but failed in the multi-compartment fire. Whilst more accurate fire models and heat transfer models are needed to better predict the behaviors of structures in realistic fires, the current study based on very simple models has demonstrated the importance and necessity of considering spreadingmulti-compartment fires in fire resistance design of multi-story buildings.

Keywords: Fire resistance, High-rise building, Spreading multi-compartment fires, Numerical investigation, Simple fire model

1. Introduction

Prescriptive fire codes are based on the concept of fire compartmentation, which contains a fire within the enclosed space of origin for a specified period of time (Janssens, 2008). The compartmentation process is accomplished by providing fire-resistive floor, wall, and ceiling assemblies and by protecting openings and penetrations through enclosure boundaries. However, fire spreading across compartments are commonly observed in accidental fires, no matter whether the buildings are compartmentalized or not. Tall buildings are more vulnerable to experiencing multi-compartment fires than low-rise buildings, mainly because of the limitations in fire service intervention for tall buildings. Although fire spreading across compartments or floors may cause progressive collapse of tall buildings (Flint et al., 2007; Rackauskaite et al., 2017), multi-compartment fire scenarios are usually not considered in the evaluation of structural safety for tall buildings (Jiang et al., 2014a; Jiang and Li, 2017a). An important reason for that is the lack of suitable theoretical models for spreading multi-compartment fires.

The behavior of a realistic fire is complex, which depends on many parameters such as fire load (amount and distribution), ventilation, combustion (or burning rate), compartment size and geometry, and thermal properties of compartment boundaries (Quintiere, 2006). Depending on whether flashover will happen or not, realistic fires are usually divided into pre- and post-flashover fires. Flashover is the rapid transition between the primary fire which is essentially localized around the item first ignited, and the general conflagration within the compartment when all fuel surfaces are burning (Drysdale, 1999). For small and middle scaled compartments with sufficient fuel and ventilation, the potential fires will develop to flashover and be characterized as post-flashover fires. For large scale enclosures, flashover is unlikely to occur and the fires are characterized as pre-flashover fires. Post-flashover fires are generally believed to pose the largest risk to structural safety and are usually considered in fire resistance design (Jiang et al., 2014, 2015), while pre-flashover fires are found to cause structure failure by both experimental and theoretical studies (Kamikawa, 2006; Agarwal et al., 2014; Choe et al., 2018; Zhang et al., 2013a, 2013b, 2015, 2018) and should also be considered.

So far, with increase in complexity, empirical correlations (e.g. nominal fire curves and parameter fire curve), zone models and field models have been developed to model realistic fires. Empirical correlations are usually based on test data on single compartment fires which are inapplicable to multi-compartment fires, while zone and field models are theoretical models complied in numerical codes which are capable of modeling both single and multi-compartment fires. This paper numerically investigates the response of a 3D steel-framed structure subjected to simulated spreading multi-compartment fires by zone model. The similar structure has been studied by Jiang and Li (2017b), in which the response of the structure

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subjected to various (single) compartment fires was investigated. The purpose of this study is to demonstrate the importance of considering spreading multi-compartment fires in fire resistance design.

2. Methodology

2.1. Fire Model

In this study, zone model was used to predict the thermal environment in a fire compartment. In zone model, the gas within compartment is generally divided into one, or a few, control volumes (zones), and for each zone, the physical parameters such as gas temperature and species concentrations are assumed to be spatially uniform. Then, from the mass and energy conservation principle as well as the ideal gas law, a set of ordinary differential equations (ODEs) are derived. In this type of model, the physical details of the gas within a zone are not considered, while mass and energy transport between zones is calculated by modelling the relevant fire sub-processes: combustion, fluid flow and heat transfer (Fu and Hadjisophocleous, 2000). Particularly, the two-zone model CFAST (Peacock et al., 2017) was used to do the fire simulations in the following studies.

In addition to the two-zone model assumption, the following assumptions are also adopted in calculating spreading multi-compartment fires (Zhang, 2017):

- Fire spread from one compartment to an adjacent compartment occurs when the temperature rise of the unexposed surface of a connection boundary (wall, ceiling, or floor) reaches a critical value of 139°C. This assumption has been made in accordance with the criterion for heat transmission in ASTM E119-16a (2016); and
- Fire in a compartment ignites immediately when the temperature rise of the inner surface of any of the compartment boundaries (walls, ceiling and floor) reaches 139°C; and
- Except the vents which are connected to the outside environment, there are no additional holes in the solid boundaries which connect compartments.

It is recognized that neither the two-zone model used in this study can fully simulate the physics of a realistic fire, nor the assumptions of fire spread given above can perfectly reflect the real conditions in practice, e.g., fire spread through electrical holes in compartment boundaries is not unlikely (Beji et al., 2014). Nevertheless, those assumptions are used in our study because, according to our literature survey, there is no available multi-compartment fire model for structural fire analysis, and we believe that the studies based on those assumptions provide valuable information for fire resistance design.

2.2. Heat Transfer Model

Fire protections on the steel columns and beams were considered in this study. A linear temperature history was assumed for the protected members (Quiel and Marjanishvili, 2011; Neal et al., 2012; Sun et al., 2015), varying from an initial 20°C to a predefined critical temperature (600°C for beams and 550°C for columns) according to the fire rating of protected steel members. It was assumed that the temperature varied linearly beyond the critical temperature until reaching the gas temperature. Four-side and three-side fire exposures were assumed for the heated columns and beams, respectively. The temperature of the upper flange of heated beams was assumed 75% of that of the web and lower flange. The temperature distribution through the depth of the heated slab was taken from the Eurocode 3 (European Committee for Standardization, 2005c).

2.3. Structural Model

In this study, an explicit dynamic analysis was carried out in LS-DYNA. The three-dimensional Hughes Liu beam element was used to model the steel columns and beams. This element had an integrated cross-section, and the command *INTEGRATION_BEAM was used to define an I-shape section. The arrangement of the integration points was achieved by an integration refinement parameter k. A value of $k = 2$ was taken in this study where 7 and 6 integration points were arranged for the flange and web, respectively. The slab was modelled by a layered composite shell formulation (*PART_COMPOSITE), in which a distinct structural material, thermal material, and thickness can be specified for each layer. This allows distinct layers to be specified for the reinforcement, except for concrete through the thickness of the slab. This also allows a coupled thermal-mechanical analysis using the same slab elements.

The material type of MAT_202 (MAT_Steel_EC3) was used for steel beams and columns at ambient and elevated temperatures. The material type of MAT_172 (MAT_CONCRETE_EC2) was used to model the reinforced concrete slab at ambient and elevated temperatures. The temperature-dependent stress-strain properties of these two material types refer to Eurocode 3 (European Committee for Standardization, 2005b) and Eurocode 2 (European Committee for Standardization, 2004), respectively.

3. Fire-Structural Analysis

3.1. Prototype Building and Finite Element Model

A multi-story moment resisting steel-framed composite frame was studied. The structural layout and member dimensions were based on the prototype building in Cardington fire tests (Kirby, 1997). The frame had five bays of 6 m, five spans of 9 m and eight stories of 4 m, as shown in Fig. 1. All connections were assumed rigid in this study, which is a common practice for steel-framed buildings in seismic zones.

All the primary beams were taken as 356×171×51 UB. The edge and internal columns were taken as 305×305×

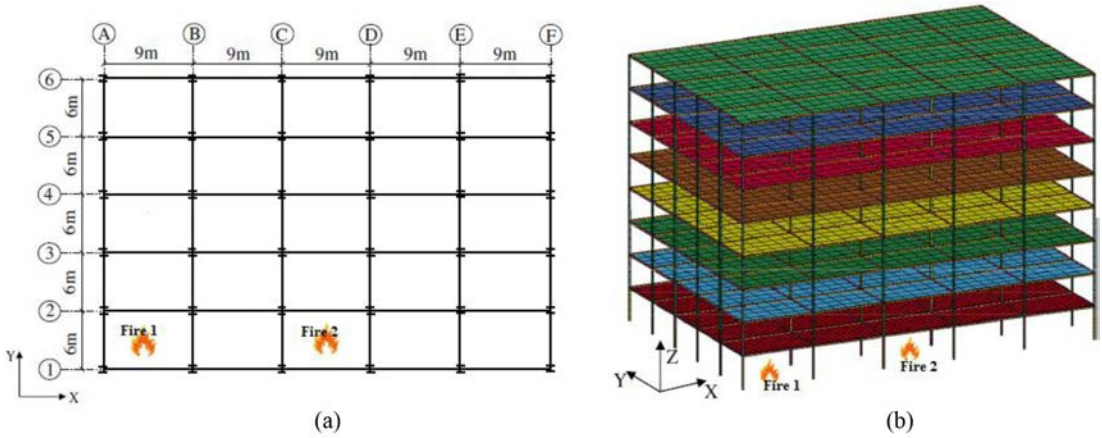


Figure 1. Model of a 3D multi-storey moment resisting frame: (a) plan view; (b) finite element model. (Jiang and Li, 2017a).

137 UC and 305×305×198 UC, respectively. No secondary beam was simulated in the model. This is partly to simplify the modeling process, and consider the fact that it is generally accepted that secondary beams are left unprotected which will experience high temperatures and lose their stiffness and strength quickly. The load-bearing capacity for these two columns was about 7000 kN and 10,000 kN. The reinforced concrete flat slab had a thickness of 120 mm and reinforcement bars in a diameter of 12 mm and spacing of 200 mm (a mesh of 565 mm^2/m). The concrete cover of reinforcement bars was 30 mm from the bottom of the slab. The Young's modulus and yield strength of steel beams and columns were 200 GPa and 355 MPa, respectively. The compressive strength of concrete was 35 MPa and the yield strength of reinforcement was 500 MPa.

A uniformly distributed load $q = 6$ kN/m^2 was imposed on the slab (a load ratio of 0.3 for composite beams and 0.2 for slabs). The load ratio of a member is defined as the ratio of the applied load to its load-bearing capacity. This was calculated according to the fire design load using Dead + 0.5Live (European Committee for Standardization, 2004) where the dead load was 4.86 kN/m^2 including the self-weight, ceiling, serves etc. and the live load was 2.5 kN/m^2. This applied an axial load of 2500 kN on the internal columns (load ratio of 0.25), 1250 kN on the edge columns (load ratio of 0.18), and 630 kN on the corner columns (load ratio of 0.09). The steel columns and primary beams were fire protected to achieve fire resistance rating of 3 h and 2 h, respectively (IBC, 2015).

3.2. Finite Element Model

The finite element program LS-DYNA with explicit integration scheme was used for structural analysis. An initial imperfection of length/1000 was imposed on columns. An element size of 0.75 m × 0.75 m was used for the ground floor slab (i.e., a mesh of 8×12) and 1.5 m × 1.5 m for all the upper slabs (a mesh of 4×6). Ten elements were meshed for all the columns on the ground floor and 4 elements for the columns on the upper floors. The mesh of the frame is depicted in Fig. 1b. A sensitivity analysis was first carried out to determine the appropriate time scale. It was shown that a duration of 90-min heating of ISO 834 standard fire (for gas temperature of 1000°C) can be scaled to a 12-second explicit dynamic analysis in LSDYNA. This produces quasi-static responses before the buckling of the heated column and dynamic responses after. This time scale was used in all the analyses below, not only significantly saving the computing time but considering the dynamic effects to the remaining structures due to sudden local failure. The validation of the numerical models including the effect of mesh size, initial imperfection, and time scale is presented in the reference (Jiang and Li, 2017b).

3.3. Fire Scenarios

Two fire scenarios were considered as given by Zhang (2017). Fire scenario 1 initiates in a corner compartment (Fire 1 in Fig. 1) and fire scenario 2 starts in a middle compartment (Fire 2 in Fig. 1). Fig. 2 shows the heat release rate curve for each compartment fire and Fig. 3 show the CFAST models for fire scenarios 1 and 2. The steel beams and columns are not included in the CFAST models, while the heat sink effect of steel members might be important (Zhang and Li, 2013).

4. Results

4.1. Fire Behavior

Figs. 4a-b show the predicted fire curves for different compartments in fire scenario 1 and 2, and Fig. 4c gives the compartment labels. In fire scenario 1, the initial fire in the bottom corner compartment ("Comp 1-1") first spread to the upper compartment ("Comp 2-1"), then

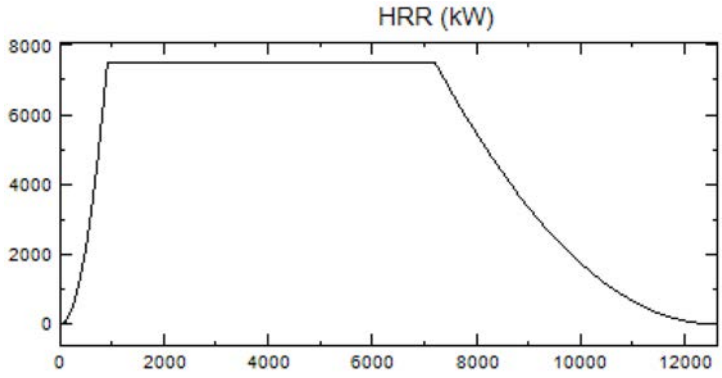


Figure 2. Heat release rate for each compartment.

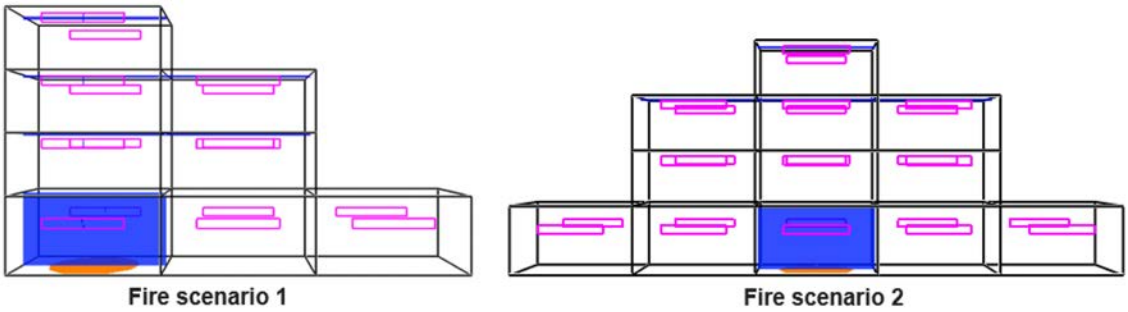


Figure 3. CFAST Numerical models.

spread to the right compartment ("Comp 2-1"), because the boundary wall is thicker than the ceiling slab. While the shapes of the fire curves in different compartments are similar the peak fire temperatures are slightly different, because the heat fluxes transferred through compartment boundaries are different.

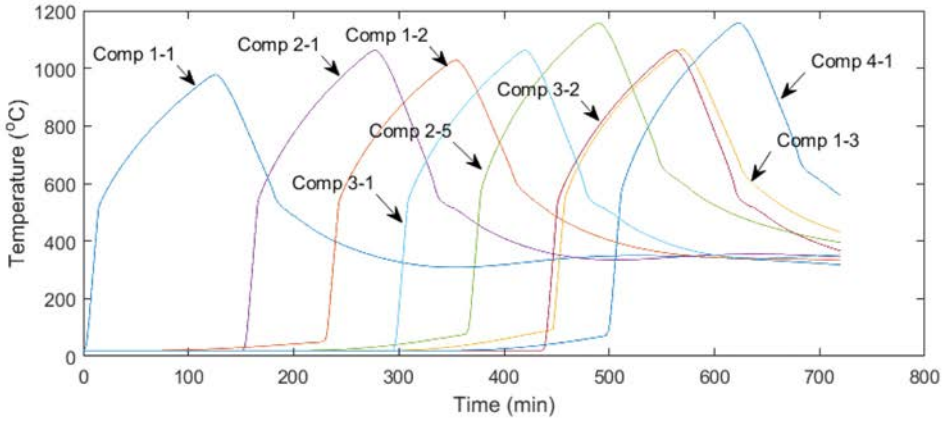
4.2. Temperature Response

A typical temperature history curve of protected columns and beams is shown in Fig. 5 where the gas temperatures of standard fires and spreading fires are also presented. As mentioned in Section 2.2, a linear temperature history was assumed for the protected members, varying from an initial 20°C to a predefined critical temperature (600°C at 120 min for beams and 550°C at 180 min for columns). For standard fires, it was assumed that the temperature varied linearly beyond the critical temperature until reaching the gas temperature. While for spreading fires with a cooling phase, it was assumed that the steel temperature varied linearly until reaching the gas temperature, and followed the cooling path of the gas temperature curve.

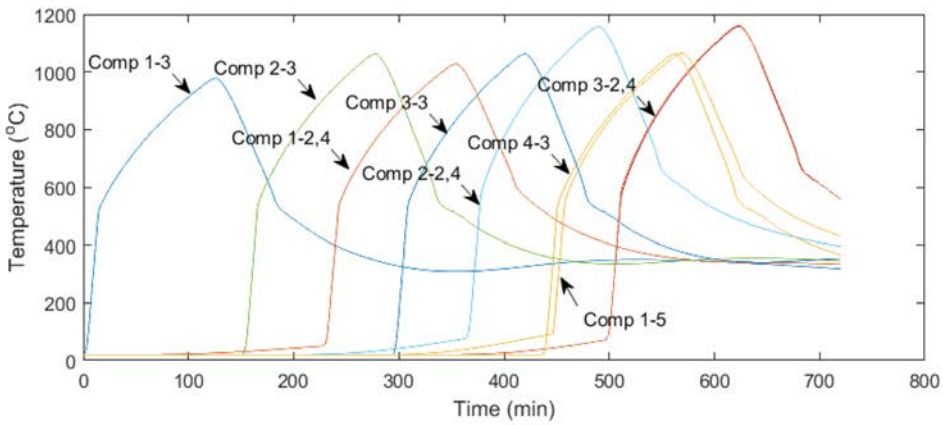
4.3. Structural Response

The structural responses of the protected frame under spreading fires are presented in this section. A comparison of collapse behavior of protected frames under standard and spreading fires is made in the next section. The collapse mode of the frame and axial displacement of the columns are shown in Figs. 6 and 7, respectively. The results showed that the frame collapsed for both fire scenarios 1 and 2. The collapse was triggered by the buckling of the heated columns in the source fire compartment, followed by the failure of the adjacent columns. The buckling of columns spread from the source fire compartment to the farther locations of the frame, accompanied by global lateral drift of the frame.

For the fire scenario 1 at corner as shown in Fig. 7a, the four heated columns in the compartment 1-1 buckled at about 165 min when the temperature of columns reached about 500°C. The failure time of the heated column was determined as the time when its top returned back to its initial position before heating (i.e., zero axial displacement). After this point, there was a sudden increment in the axial displacement and reduction in the axial force of the buckled column. The temperature of the heated columns at failure (500°C) was 50°C less than the predefined critical temperature of columns (550°C). This is due to the increased compression from the restrained thermal expansion as well as the lateral displacement at the top of the column from the deflection of the heated slab. After the buckling of the columns in the source compartment, the loads sustained by them were redistributed to the adjacent columns. However, it is interesting to note that the adjacent columns did not buckle immediately but withstood for another 50



(a) Fire scenario 1



(b) Fire scenario 2

4-1	4-2	4-3	4-4	4-5
3-1	3-2	3-3	3-4	3-5
2-1	2-2	2-3	2-4	2-5
1-1	1-2	1-3	1-4	1-5

(c) Compartment labels

Figure 4. Predicted gas temperatures for different fire scenarios.

mins (from 165 min to 210 min) until the buckling of B3 at about 210 min. This long withstanding period of the adjacent columns is attributed to the development of deflection of the heated slab which increased the lateral displacement of the adjacent columns.

Similar to the fire scenario 1, the buckling of the heated columns in the source fire compartment 1-3 occurred at about 165 min when the temperature of columns reached about 500°C. After the buckling of the heated columns, the adjacent columns along the short span (C3 and D3) failed first at 210 min, followed by failure of those along the long span (B2 and E2) at 270 min.

5. Discussion

The effect of fire spreading on the collapse behavior of

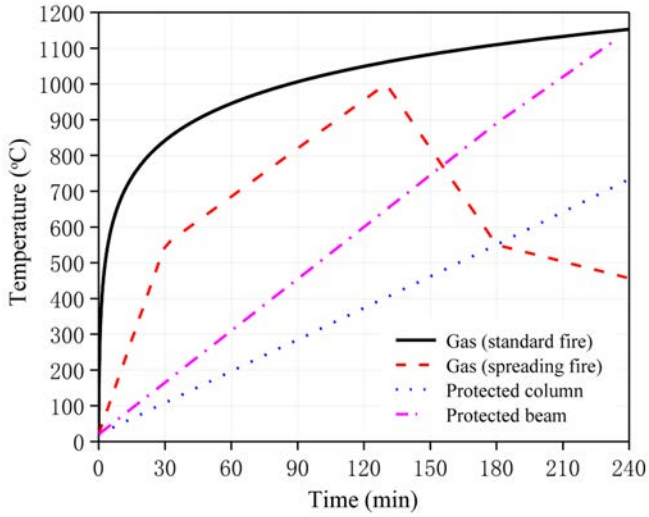


Figure 5. Temperature-time history of steel members heated in the fire compartment.

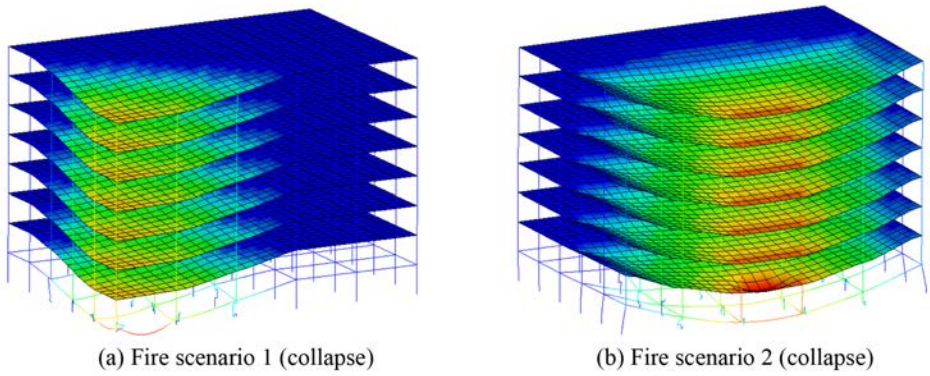


Figure 6. Collapse mode of the protected frame under spreading fires.

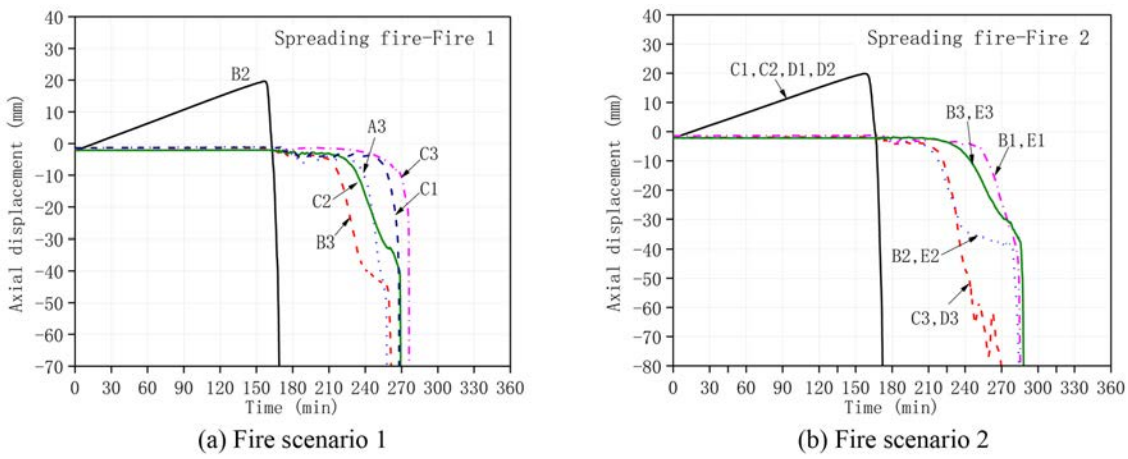
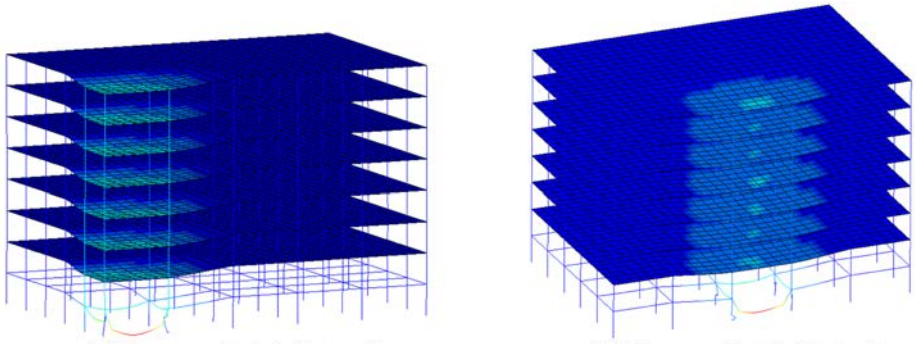


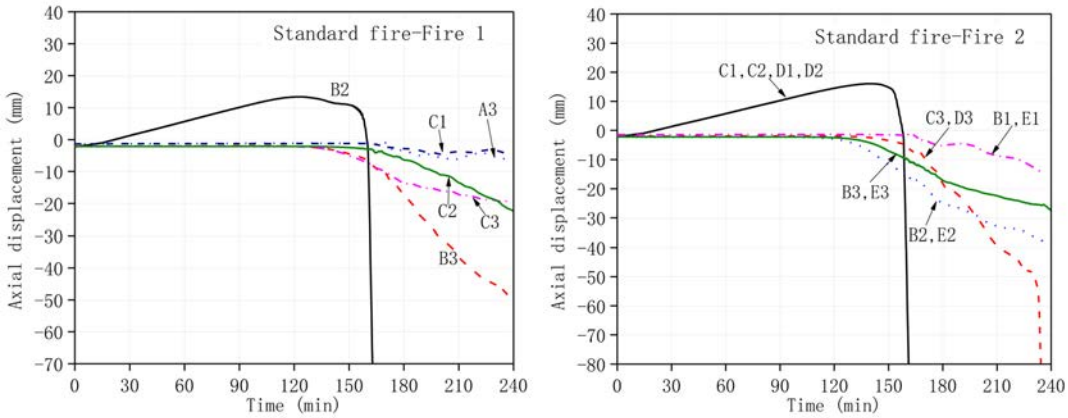
Figure 7. Axial displacements of the columns in the source fire compartment under spreading fires.

the frame is discussed in this section, by comparing to that under ISO standard fires. It was assumed that only one compartment was subjected to the standard fire (compartment 1-1 for fire scenario 1 and compartment 1-3 for fire scenario 2). Compared to the collapse of the frame under spreading fire, no global collapse of the frames occurred



(a) Fire scenario 1 (withstand) (b) Fire scenario 2 (withstand)

Figure 8. Collapse mode of the protected frames under standard fires.



(a) Fire scenario 1 (b) Fire scenario 2

Figure 9. Axial displacements of the columns in the fire compartment under standard fires.

under the standard fire although suffering from collapse of the fire compartment, as shown in Fig. 8. This indicates the higher severity of spreading fires since they are more prone to cause progressive collapse of structures. The fire spread to adjacent compartments led to the failure of adjacent columns (Fig. 9), triggering the global collapse. This is partly due to the degraded stiffness and strength of these columns at elevated temperatures. This may be also due to the reduced load-bearing capacity of slabs through tensile membrane action at elevated temperatures.

6. Conclusions

This paper numerically investigates the response of a 10-story steel-framed building subjected to spreading multi-compartment fires. The calculations are based on a simple fire model and assume simplified temperature rising and distribution profiles in the affected structural members. Although it has been fire protected according to the prescriptive codes, the investigated building, which indeed survived the standard fire exposure, collapsed in the spreading multi-compartment fires. This demonstrate that spreading multi-compartment fires may pose much worse scenarios than a standard fire, and therefore, should be considered in the fire resistance design of high-rise buildings.

Acknowledgements

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