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Disproportionate collapse of 3D steel-framed structures exposed to various compartment fires

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ABSTRACT

This paper numerically investigates disproportionate collapse resistance of three-dimensional steel-framed structures exposed to compartment fires. The effect of fire protections (low, medium, high) as well as fire locations (corner, edge and interior) on collapse modes and load redistribution schemes is studied. The results show that the frames do not collapse immediately after this local failure but experience a relatively long withstanding period of at least 60 min. This is attributed to the increasing deflection of heated slabs, resulting in increased lateral displacements of adjacent cool columns which governs their buckling. This indicates that the "fire rating" of a structure against global collapse is somewhat 1-hour longer than that of individual members. It is found that the fire protection of steel members has significant effect on the resistance of structures against fire-induced disproportionate collapse. The frames with a medium level of fire protection (2-hour fire rating for columns) withstand the fire. A comparison between 2D and 3D models shows that the 2D model produces conservative results by underestimating the collapse resistance of structures. It cannot capture the load redistribution in a 3D model where more loads are distributed along the short span than those along the long span. The presence of slabs for delaying the global collapse cannot also be simulated by a 2D model. It is recommended that the fire protection of perimeter columns should be enhanced to 2-hour fire rating and slabs should be protected to delay and prevent the collapse of structures.

1. Introduction

The traditional structural fire design is based on standard fire tests on individual structural members under idealized loading and boundary conditions [1]. Fire resistance of steel-framed structures has traditionally been ensured by applying insulating material around the steelwork, such as sprays, boards, blankets, and intumescent coatings. These significantly increase the construction cost, while evidence from real fires suggests that reduced levels of fire protection may be sufficient. Simple calculation methods such as limiting temperature method (accounting for load ratio) and moment capacity method (accounting for non-uniform temperature in the cross-section) [2] have been proposed to better determine the performance of structures in fire. However, these approaches typically apply to isolated members and result in limited saving in fire protection compared to prescriptive methods based on standard fire tests. Cardington fire tests [3] showed that steel members in real multi-storey buildings had significantly greater fire resistance than isolated members in the standard fire test. For example, the unprotected steel beams had a predicted limiting temperature of about 650 °C but experienced temperatures over 1000 °C in the test without any indication of collapse of the frame. This improvement is attributed to the tensile membrane action in the floor slab at large displacements and elevated temperatures.

Especially since the collapse of the Word Trade Tower (WTC) under terrorist attack on September 11, 2001, there has been considerable interest in understanding the disproportionate collapse of tall buildings in fire. The disproportionate collapse is defined as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it" [4]. This indicates that large displacements (even failure) of individual structural members are acceptable provided that structural collapse is prevented. Fire protection may be reduced or eliminated in some steel members if, in the absence of these members, an alternative load path can be provided by other undamaged structural members.

A comprehensive review of fire-induced collapse mechanisms of steel structures can be found in reference [5]. Usmani et al. [6] investigated the stability of Word Trade Center and the results showed that the WTC tower might still collapse under the fire condition alone due to the degradation of lateral support of columns provided by the composite truss floor system. Two collapse mechanisms, namely a weak floor failure mechanism and a strong floor failure mechanism were

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proposed [7]. Ali et al. [8] studied the collapse modes and lateral displacements of single-storey steel-framed buildings exposed to natural fire. The outward and inward collapses of heated columns were found, resulting from the thermal expansion and catenary action of the heated beam, respectively. Quiel and Garlock [9] compared the predicted behavior of high-rise steel frames under fire using 2D and 3D modes. The results showed that the 2D model was sufficient to produce a reasonable prediction if the slab was considered in the thermal analysis of beams (but not in the structural analysis). Fang et al. [10] proposed multi-level system models for disproportionate collapse analysis of structures exposed to fire. Two robustness assessment approaches namely temperature-dependent and temperature-independent approaches were carried out. The latter ignored the temperature effect but considered the model reduction due to the heating by removing several heated members of the structures. Sun et al. [11] studied the influence of load ratios, beam sizes and horizontal restraints on the collapse mechanisms. The results showed that a lower loading ratio and a larger beam section might result in a global collapse while a higher loading ratio and a smaller beam section led to a local collapse. Jiang et al. [12–14] studied the effect of bracing and fire scenarios on the collapse mechanisms of steel frames under fire using OpenSees. Four collapse modes were proposed including the global and local downward and lateral collapses. Menari and Mahmoud [15] studied the response of steel moment resisting frames with reduced beam section connections under fire. The results showed that the global stability of the frame was not affected by single bay fire exposure.

Most of previous studies focus on two-dimensional plane frames. Although capable of capturing some key issues of the fire-induced collapse mechanisms of structures, they fail to fully consider the load redistribution path in a realistic structure, and the effect of floors, i.e. the tensile membrane action of floors at large displacements and elevated temperatures. More recently, there has been some research on three-dimensional steel framed structures. Pyl et al. [16] investigated the fire safety of a 3D single-storey industry portal frame where limited load transfer paths were available. Kilic and Selamet [17] investigated the fire-induced collapse modes of a 49-storey steel frame. It indicated that the location of the fire on a single floor did not significantly change the collapse mechanism. This conclusion was questionable because it assumed that all the columns on one floor were heated which was an extreme situation. Agarwal and Varma [18] studied the fire-induced disproportionate collapse of a 3D 10-storey steel buildings. It was found that the columns played a key role in the overall stability of the building. However, a corner fire on the fifth storey was considered which was not the severest fire scenario for the disproportionate collapse of frames. Therefore, there remains a lack of understanding of disproportionate collapse mechanism of three-dimensional structures under fire, in which load redistribution path and tensile membrane action of floors can be reasonably considered.

This paper presents numerical analyses of disproportionate collapse of three-dimensional multi-storey steel frames exposed to fire, using commercial software LS-DYNA with explicit integration scheme. A 3D steel frame with 5 bays (6 m), 5 spans (9 m) and 8 storey (4 m) was designed and analyzed by varying the fire locations and fire protection of steel columns and beams. Reinforced concrete slabs were also modelled. Four compartment fires at the corner, middle of a long edge, middle of a short edge and the interior of the frame were modelled. Three levels of fire protections (low, medium, high) were considered for the columns and beams in the fire compartment. The collapse mode and load redistribution scheme of frames subjected to these fire scenarios and fire protections were investigated. A plane frame with the same dimension, load, fire scenario as the 3D model was created to investigate the difference between them for the prediction of local and global failure of frames.

2. Modelling of prototype frame

A multi-storey moment resisting steel-framed composite frame was modelled in LS-DYNA. The structural layout and member dimensions were based on the prototype building in Cardington fire tests [3]. The frame had five bays of 6 m, five spans of 9 m and eight storey 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. It is recognized that the type of connections (rigid or pinned) affects the load redistribution and collapse mode of frames, and generally rigid connections are beneficial for the collapse resistance of frames for a higher level of load redistribution when a column fails than pinned connections. This means the assumption of rigid connections may lead to unconservative results. As a preliminary study and avoiding the sensitivity of collapse mode on the distribution of pinned and rigid connections, rigid connections were assumed in this study and further work is underway to discuss the influence of connections on the collapse resistance of steel-framed structures exposed to fire.

All the primary beams were taken as $356 \times 171 \times 51$UB. The edge and internal columns were taken as $305 \times 305 \times 137$UC and $305 \times 305 \times 198$UC, respectively. The steel columns and beams were modelled by three-dimensional Hughes Liu beam elements. The local buckling cannot be simulated by this element (only global buckling was considered). The load-bearing capacity for these two columns was about 7000 kN and 10,000 kN. No secondary beams were modelled since they were generally unprotected in the practical engineering, thus subjecting to high temperature and significantly low strength. An initial imperfection of *length*/1000 was imposed on columns. All the columns were arranged with strong axes along the short span (Fig. 1a), which is a typical layout to supplement the weak resistance to lateral forces in the short span direction.

Flat reinforced concrete slabs were modelled instead of composite slabs with profiled decking. This was to consider the effect of concrete floors but prevent the difficulties in using shell element to simulate the ribbed composite slab. 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 and concrete through the thickness of the slab. The steel beam shared the same node with the slab to form a composite beam with a rigid connection between the slab and steel beam [19]. The reinforced concrete 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.

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). This was calculated according to the fire design load using Dead + 0.5Live [20] 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 load ratio of a member is defined as the ratio of the applied load to its load-bearing capacity.

The Young's modulus and yield strength of steel beams and columns were 200GPa and 355 MPa, respectively. The compressive strength of concrete was 35 MPa and the yield strength of reinforcement was 500 MPa. The MAT_202 (MAT_Steel_EC3) was used for steel beams and columns at ambient and elevated temperatures. The temperature-dependent material properties refer to EC3 [2]. The material MAT_172 (MAT_CONCRETE_EC2) was used to model the reinforced concrete slab at ambient and elevated temperatures. The stress-strain curves in this material for the concrete and reinforcement at ambient and elevated temperature are as specified in EC2 [21].

The validation of the numerical models including the effect of mesh size, initial imperfection, and time scale is presented in the reference [22]. To save the computing cost as much as possible, an element size of 0.75 m $\times$ 0.75 m was used for the ground floor slab (i.e. a mesh of 8×12) and 1.5 m $\times$ 1.5 m for all the upper slab (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