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Progressive collapse mechanisms investigation of planar steel moment frames under localized fire

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ABSTRACT

In this paper, the possible progressive collapse mechanisms of planar steel frames when one column failed under elevated temperature was studied through extensive case studies. An explicit dynamic solver was adopted, which could continue beyond local element buckling. The effects of analysis parameters such as mesh size and loading speed were investigated. And the numerical model was validated against experimental data and analysis results of other researchers. The investigated parameters included beam cross-sectional size, load ratio and location of heated column. Three progressive collapse mechanisms were found, namely, cantilever beam mechanism, pull-in force induced mechanism and high load ratio member failure mechanism, of which the last one is a new discovery. To evaluate progressive collapse of planar steel frames under fire, the cantilever beam mechanism and the pull-in force induced mechanism should be checked when the outer columns are heated, and the pull-in force induced mechanism and the high load ratio member failure mechanism need to be checked when the inner columns are heated. And the most adverse fire scenarios of a planer steel moment frame are when one of the following columns is heated: first floor column of the outmost, second and third outmost column lines or top floor column of the outmost, second and third outmost column lines.

1. Introduction

Progressive collapse refers to the phenomenon when initial local damage in a structure spreads to other structural elements as if in a chain reaction, eventually resulting in the global collapse of a structure that is grossly disproportionate to the initial damage. The catastrophic collapse of a number of very tall buildings in the WTC complex in 2001, has attracted a great deal of attention from researchers as it seems most likely that the fires initiated by aircraft impact in the towers and the burning debris spread by the collapsing towers into other buildings triggered the progressive collapses seen [1]. Considering the limitations of direct firefighter intervention in tall buildings and the enormous potential losses of life and treasure in case of collapse, it is essential to understand the performance of such buildings under fire and ensure that their structural frames possess sufficient passive fire resistance for a reasonably adequate range of realistic fire scenarios. Given that the testing of whole frames under multiple real fire scenarios is not practically feasible, the only way to study the possible failure mechanisms of such structures under fire is through computational modelling, which can then be used to develop rational design methods.

In contrast to research on progressive collapse of structures under explosion and impact, relatively limited research has been done on progressive collapse of steel structures under fire. Wang et al. [2] used a sub-model to study the performance of plane steel frames under fire. Liew [3] built a mixed-element model to study three dimensional steel frames subject to blast loading followed by a fire attack, which could capture the detailed behaviour of members and frame instability associated with the effects of high-strain rate and high temperatures. Usmani et al. [4–6] built a planar frame model to study the progressive collapse of tall building with long span truss floor systems under multi-floor fires and found two failure mechanisms (Weak Floor Collapse Mechanism and Strong Floor Collapse Mechanism). Fang et al. [7] used the temperature-dependent and temperature-independent approach, on the basis of an energy-based multi-level assessment framework proposed by Izzuddin et al. [8,9], to assess the progressive collapse of multi-storey composite frame buildings under fire. Sun et al. [10,11], studied the influence of bracing systems on the capacity of steel frames to resist progressive collapse under a localized fire. However, the possible collapse mechanisms of steel framed structures under localized fire have not been systematically studied.

In this paper, a parametric analysis was conducted to study the possible progressive collapse mechanisms of planar steel frames when one column was heated. The numerical model was developed using the commercial Finite Element Analysis software ABAQUS [12] and its

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explicit dynamic solver was employed to overcome the problems of convergence under large deformation. Analysis parameters such as loading speed and mesh size were studied and the model was validated against experimental data and analysis results of other researchers. The possible progressive collapse mechanisms of different fire scenarios were identified. And the most adverse fire scenarios for planar steel moment frames were studied.

2. Analysed frame and fire scenario

The analysed planar steel frame contains 4 bays (each 6 m wide) and 6 storeys (each 3.6 m high) as shown in Fig. 1. Uniform section dimensions are used for columns and beams. The columns are oriented to experience bending about their minor axes. A sine curve shape with a maximum deflection of 1/1000 of the column length is imposed on each column before the start of the analysis. In Fig. 1, the symbol $C_{a\text{-}b}$ (where C represents Column, a is a Roman number and b is an Arabic number) refers to the column on floor a at column line b and $B_{a\text{-}bc}$ (where B represents Beam, a is Roman number, b and c are Arabic numbers) refers to the beam between column line b and c at floor a. For example, $C_{I\text{-}3}$ stands for the first floor column at column line 3 and $B_{I\text{-}23}$ represents the beam between column lines 2 and 3 at the first floor.

This study focuses on the global behaviour and collapse mechanism of moment-resisting steel frames, which are widely used in seismic regions such as China and North America. Hence connections between columns and beams are assumed to be rigid and assumed not to fail during the analysis.

Columns are critical elements of a structure and the failure of columns may result in serious damage to the structure. The fire scenario considered in this paper involves one column of the frame to be heated. The temperature of the heated column is assumed to rise uniformly from 20 °C to 1200 °C and the rest of the structure is assumed to remain at the ambient temperature of 20 °C. Nine scenarios are considered, namely the heated column located in the first, third and fifth column lines on the first, third and sixth floors, respectively.

The column section is adopted as HW350 × 350 × 12 × 19. Four different beam section sizes are considered and for each beam section size, several load ratios are considered to study the possible progressive collapse mechanisms, as illustrated in Table 1. Uniformly distributed loads are applied to the beams and load ratio refers to the ratio of the applied load to the load capacity of the frame under ambient temperature.

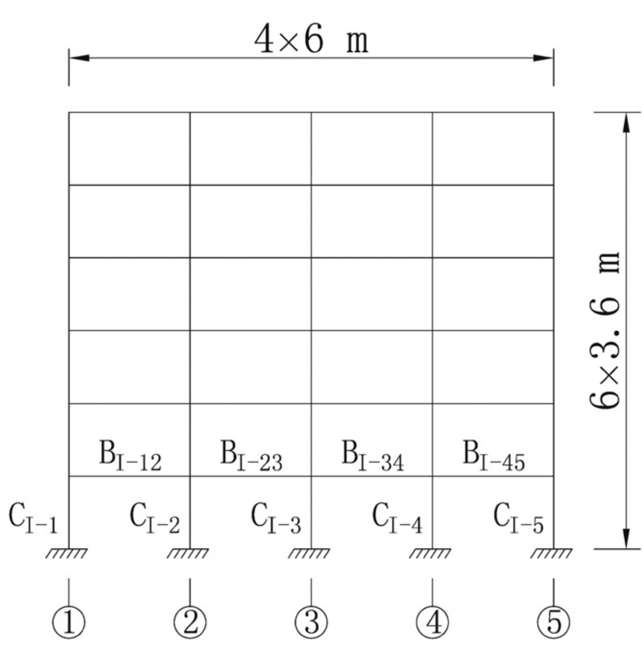

Fig. 1. Analysed steel frame.

Table 1
Model summary.

Frame	Beam	Load ratio
1	HM294 × 200 × 8 × 12	0.3, 0.5
2	HM340 × 250 × 9 × 14	0.3, 0.4, 0.5,0.6
3	HM390 × 300 × 10 × 16	0.3, 0.4, 0.5,0.6, 0.7
4	HM488 × 300 × 11 × 18	0.3, 0.4, 0.5,0.6, 0.7, 0.8

3. Finite element model and analysis

3.1. Explicit dynamic analysis procedure

The equations of motions for a structural dynamic analysis can be written as:

$$M\ddot{u} + C\dot{u} + P = F \tag{1}$$

where M is the mass matrix, C is damping matrix, P is the internal force vector and F is external force vector and u, $\dot{u}$ and $\ddot{u}$ are displacement, velocity and acceleration vectors, respectively. The acceleration can be obtained from Eq. (1) as:

$$\ddot{u} = (M)^{-1}\left(F - C\,\dot{u} - P\right). \tag{2}$$

In the explicit dynamic procedure, central difference method is used to integrate the equation of motion through the time. The motion condition of the next step is obtained based on the current step, that is:

$$\dot{u}_{(t+\Delta t/2)} = \dot{u}_{(t-\Delta t/2)} + \left(\Delta t_{(t+\Delta t)} + \Delta t_t\right)\ddot{u}_t/2 \tag{3}$$

$$u_{(t+\Delta t)} = u_t + \Delta t_{(t+\Delta t)}\dot{u}_{(t+\Delta t/2)} \tag{4}$$

where $\Delta t_{(t+\Delta t)}$ and Δt_t are time increment at step time $t + \Delta t$ and t respectively.

For explicit dynamic method, the time step must be smaller than a limiting value. The limiting value can be obtained from the highest frequency of the system as follows:

$$\Delta t_{stable} = 2\left(\sqrt{1+\xi^2} - \xi\right)/\omega_{\max} \tag{5}$$

where $\omega_{\max}$ is the highest frequency of the system and ξ is the damping ratio corresponding to the highest frequency of the system. It can be proven that the highest frequency of the element is always higher than the highest global frequency, so the stable time increment (Δt_{stable}) can be calculated approximately from the element dimension and the wave velocity of the material (c_d) as follows:

$$\Delta t_{stable} = L^e/c_d \tag{6}$$

where L^e is the characteristic dimension of the smallest element, which is usually taken as the shortest distance of any two nodes in the element.

3.2. Element and material model

The software ABAQUS has a rich element library for a large range of analysis options. The three-dimensional Timoshenko beam element (B31), which can model shear, flexure and axial deformations, is chosen to model the beams and columns. NLGEOM option is selected to enable the modelling of large displacements.