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Experimental and numerical study on thermal-structural behavior of steel portal frames in real fires

Guobiao Lou [a,c], Chenghao Wang [c], Jian Jiang [b,*], Yaqiang Jiang [b], Liangwei Wang [b], Guo-Qiang Li [a,c]

[a] State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai 200092, China
[b] Sichuan Fire Research Institute, Ministry of Public Security, Chengdu 610036, China
[c] College of Civil Engineering, Tongji University, Shanghai 200092, China

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ABSTRACT

This paper presents experimental and numerical investigations on the collapse behavior of a 12 m × 6 m steel portal frame exposed to fire. A real fire test is conducted with a 4 m × 6 m fire compartment at the corner of the frame. Extensive thermal and structural responses of the frame are measured and presented. The experimental results show that the fire compartment collapses after 20-min heating with a critical column temperature of 850 °C. It is found that the measured gas temperatures are higher than the ISO fire, but lower than the parametric fire specified in EN 1991-1-2, indicating the underestimation of the thermal exposure for standard fires and unrealistic estimation for parametric fires. For steel temperatures, the prediction from parametric fires is in a reasonable agreement with the measurements by predicting a similar maximum temperature at a delayed time. The prediction from standard fires significantly underestimated the steel temperatures by up to 300 °C. The significant non-uniform gas temperature distribution along the frame height shows a two-zone model with a temperature gap of 200 °C. The maximum temperature of the members beyond the fire compartment exceeds 500 °C due to the spread of hot smoke. A numerical model is established and validated against experimental results. A potential outward collapse mode of the frame is predicted. It is suggested that a more realistic description of fire scenarios is still needed for a performance-based structural fire design, based on a better consideration of ventilation conditions and thermal properties of boundary enclosure.

1. Introduction

Steel portal frames are the dominant structural form for single-storey industrial and commercial buildings, provided that the design details are cost effective and economical. One of the major disadvantages of steel structures is its sensitivity to fire as steel materials will lose most of its stiffness and strength at about 600 °C. Compared to multi-storey buildings having fire resistance requirements to prevent structural collapse, single-storey buildings are mainly required to prevent fire spread to adjacent buildings [1]. Important factors relevant to limiting fire spread are the presence of fire-resistant external walls and/or the specified minimum spacing between the adjacent buildings. Although external walls within a certain distance of the site boundary are required to have a fire-resistance rating, steel columns and rafters are always permitted to be unprotected, and are not required to achieve the level of fire resistance required for walls. In addition to limiting fire spread, the life safety of fire fighters in case of fire-induced collapse of portal frames has received growing attention. This is probably attributed to the fact that most fires appear to develop after hours when there are no or few occupants still present in the building.

An inward collapse of the frame is always preferred (Fig. 1a) since it can not only limit the fire spread to adjacent buildings but may help to extinguish fire inside the frame. In contrast, the outward collapse (Fig. 1b) should be prevented since it will further endanger the safety of fire fighters who assist with extinguishing the fire outside the frame. Therefore, the safety of firemen and properties can be guaranteed by ensuring no collapse or collapse towards the inside. On the other hand, the external walls (made of concrete or steel panels) may detach from the supported columns and fall outwards, thus potentially endangering the life of fire fighters. The outward collapse of a wall is possible due to its thermal gradient caused by heating the inside surface of the wall which may be in a different direction to that of the frames. Current fire

* Corresponding author.
E-mail address: jiangjian_0131@163.com (J. Jiang).

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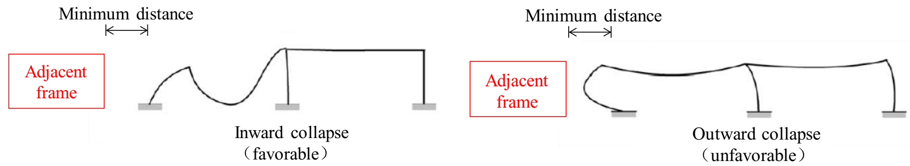

Fig. 1. Different collapse modes of steel portal frames.

resistance design of steel portal frames recommends to enhance the connection between walls and columns, and simultaneously strengthen the base stiffness of columns to resist the overturning moment that will occur when the roof collapses [1]. In this case, the external walls will deform together with the steel frame whose collapse mechanism plays a key role in the overall stability of steel portal frames. It is worthy of noting that a frame may collapse during either heating or cooling phase of fire [2,3]. It is evident that large tensile forces can develop in the fire-exposed beam due to contraction when it cools down [4–6], which may cause the failure of beam-to-column connections. The concrete slab may reach maximum temperature during the cooling phase [7]. In addition, steel portal frames may suffer from localized fires where flashover is unlikely to occur where the structural behavior is quite different from that to a standard fire [8]. There is also a high possibility of travelling fires in portal frames, leading to a different collapse behavior compared to a uniformly heated frame [9–11].

A number of recent fires in industrial structures have drawn attention to a current lack of understanding about the progressive collapse of steel portal frames under fire conditions. Compared to multi-story and tall buildings [12–15], steel portal frames, as one type of long-span structures, are always more prone to collapse due to its low level of structural redundancy and high level of fire loads. However, there are few studies on the progressive collapse behavior of long-span structures exposed to fire. Ali et al. [16] studied the sensitivity of collapse mode of steel portal frames to fire scenarios, load ratios, frame dimensions. The results showed that a frame may collapse outwards if the fire localized to the column where the heated portion of the beam was limited and not sufficient to provide catenary forces to pull the columns inwards. Song et al. [17] investigated the influence of column base conditions on the failure mechanisms of portal frames in fire. The results indicated that the current design method may provide unconservative results. Bong et al. [18] addressed the necessary level of base fixity to ensure an inward deformation of frames. They also concluded that it was not necessary to protect portal frames unless the designer wished to ensure that the columns and walls remain standing. Moss et al. [19] studied the influence of support conditions, axial restraints, fire severities on the collapse mode of steel portal frames in fire. It was found that some level of base fixity should be provided to ensure a favorable inward collapse mode. EI-Heweity [20] investigated the failure temperature of steel portal frames exposed to fire. A failure criterion depending on the formation of plastic hinges was proposed. Johnston et al. [21] also addressed the importance of column base fixity in preventing outward collapse of portal frames. They demonstrated that the joint rigidity and various fire scenarios should be considered to allow for a conservative design. Garcia et al. [22] studied the behavior of steel portal frames with fire-resistant steel and also intumescent coatings. The results showed that a combination of these two methods was the best choice from both economic and structural views.

However, all these above-mentioned studies focus on numerical modeling, while experimental studies are lacking. Wong [23] conducted fire tests on a scaled steel portal frame (scale ratio of 1:5). The frame did not collapse since the fire lasted only for about 10 min, and some parts of the frame were heated to just over 900 °C. Pyl et al. [24] conducted a full-scale fire test on a cold-formed steel portal frame. The tested frame collapsed after 62-min natural-fire exposure with a critical temperature of 750 °C. The catenary action in rafters was observed in the test. However, limited information on the thermal and structural responses of these two tests was provided. Therefore, it is necessary to conduct full-scale fire tests on hot-rolled steel portal frames (the most common type) to investigate its collapse behavior, and also to provide validation references for the numerical modeling, which is one objective of this study.

On the other hand, most previous numerical simulations adopt standard fires as thermal loads which are quite different from realistic fire scenarios of portal frames. The selection of standard fires is partly due to the lack of accurate parametric natural fire models for long-span structures [25]. The parametric temperature-time curves specified in EN 1991-1-2 [26] applies to compartment fires with a maximum compartment height of 4 m, which are not available for portal frames of which the height always exceeds 5 m. Another objective of this study is therefore to provide extensive data of gas and steel temperatures for developing calculation methods of parametric fire models used for portal frames.

This paper experimentally and numerically investigated the collapse behavior of 3D steel portal frames under natural fire conditions. The portal frame had a span of 12 m, column spacing of 6 m and eaves height of 5.4 m. The overall deformation and detailed failure of the frame were shown. The gas temperatures at different horizontal and vertical locations of the fire compartment were measured. The in-plane and out-of-plane displacements of columns and rafters were also recorded. Numerical simulations of the tested frame were conducted and validated against experimental results. The numerical model was used to predict the global collapse model of the frame.

2. Test setup

2.1. Dimension of the frame

A 12 m × 6 m single-storey steel portal frame was constructed and tested, as shown in Fig. 2. The portal frame was designed based on a Chinese code [27]. The span of the portal frame is 12 m, and the spacing of columns is 6 m. The eaves height (the height of the top of the column) is 5.4 m, while the apex height (maximum height at mid-span) is 5.8 m with a roof pitch of 1:15. The dimension of the frame has been commonly used in China. A fire compartment of 4 m × 6 m surround by a fire wall was established at one side of the frame (Fig. 3b). The dimension and material of the structural members are listed in Table 1. The cross-sections of the frame columns and rafters are H200 × 150 × 6 × 8. Circular steel tubes of Φ16 × 3 (diameter × thickness) are used for the column bracing and roof bracing. The purlins have a lipped channel section of C140 × 60 × 20 × 3 and spacing of 1.5 m. The details of the frame bay are shown in Fig. 3a. The two bays of the steel frame are connected by three connecting beams LG1, LG2, LG3 (Φ125 × 5), as shown in Fig. 3b. The portal frame is covered by wall claddings and roof claddings with a thickness of 0.5 mm, as shown in Fig. 2b. The cladding is made of stone wool sandwich panel. The nominal yield strength of the columns, rafters, column bracings is 345 MPa, while that of other members such as roof bracings and purlins is 235 MPa.

2.2. Scheme of loading

2.2.1. Lateral load

Lateral loads were applied to the frame according to a design wind