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Progressive Collapse of Steel High-Rise Buildings Exposed to Fire: Current State of Research

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Abstract

This paper presents a review on progressive collapse mechanism of steel framed buildings exposed to fire. The influence of load ratios, strength of structural members (beam, column, slab, connection), fire scenarios, bracing systems, fire protections on the collapse mode and collapse time of structures is comprehensively reviewed. It is found that the key influencing factors include load ratio, fire scenario, bracing layout and fire protection. The application of strong beams, high load ratios, multi-compartment fires will lead to global downward collapse which is undesirable. The catenary action in beams and tensile membrane action in slabs contribute to the enhancement of structural collapse resistance, leading to a ductile collapse mechanism. It is recommended to increase the reinforcement ratio in the sagging and hogging region of slabs to not only enhance the tensile membrane action in the slab, but to prevent the failure of beam-to-column connections. It is also found that a frame may collapse in the cooling phase of compartment fires or under travelling fires. This is because that the steel members may experience maximum temperatures and maximum displacements under these two fire scenarios. An edge bay fire is more prone to induce the collapse of structures than a central bay fire. The progressive collapse of buildings can be effectively prevented by using bracing systems and fire protections. A combination of horizontal and vertical bracing systems as well as increasing the strength and stiffness of bracing members is recommended to enhance the collapse resistance. A protected frame dose not collapse immediately after the local failure but experiences a relatively long withstanding period of at least 60 mins. It is suggested to use three-dimensional models for accurate predictions of whether, when and how a structure collapses under various fire scenarios.

Keywords: Progressive collapse, Influencing factors, Collapse mechanism, Fire scenario, Bracing system

1. Introduction

The traditional way of determining the fire resistance of a structure is to test its critical members in a standard fire (e.g., ISO 834 fire). Such tests are conducted on simply supported members with failure criteria in terms of failure of members, limit of deformation, rate of deformation or limiting temperature. Since the Broadgate Phase 8 fire and the subsequent Cardington fire tests (Kirby, 1997) in the 1990s, the global behavior of steel framed structures in fire has received increasing concern. It is confirmed that steel members in real multi-story buildings have significantly greater fire resistance than isolated members in standard fire tests, due to the realistic member dimension, boundary condition, and fire scenario. Especially since the collapse of Word Trade Tower (WTC) under the terrorist attack on September 11, 2001, there have been growing interests in understanding progressive collapse resistance of structures under accidental loads such as blast, impact and fire (Hayes et al., 2005; Khandelwal et al., 2008; Menchel et al., 2009; Sasani et al., 2011). The term "progressive 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" (ASCE, 2005). It implies that large displacements, even failure, of individual structural members are acceptable given the prevention of a global structural collapse. An important lesson resulting from the collapse of WTC is that prescriptive fire resistance ratings of individual structural members do not guarantee the adequate performance of a whole building system (Cowlard et al., 2013).

Current research has focused on exploring the potential collapse mechanism and proposing corresponding measures to mitigate or prevent the structural collapse. The collapse mechanism of steel structures exposed to fire depends on many influencing factors such as load ratios, strength of beams and columns, connections, slabs, fire scenarios, bracing systems, fire protections and their interaction. The objective of this paper is to review current design and research approaches on these influencing factors to figure out the key factors to enhance the resistance of structures against fire-induced progressive collapse.

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2. Design Approaches

The progressive collapse is a relatively rare event as it requires both an abnormal loading to initiate the local damage, and a structure that lacks adequate continuity, ductility and redundancy to resist the spread of failure. The assessment of collapse performance of structures and measures for mitigating disproportionate collapse can be found in various design codes (GSA, 2003; ASCE 7, 2005; DoD, 2010). ASCE 7 (2005) proposes two general approaches that attempt to reduce the potential of progressive collapse: direct design and indirect design. Direct design approaches include the alternate path method and specific local resistance method. The former is applied by instantaneously removing the potentially damaged column (simulating the local damage) and assessing the progressive collapse resistance of the remains to ensure there are alternative load transferring paths to bridge over the missing member. The location of column removal is given in GSA, as shown in Fig. 1 where the perimeter columns at mid-span and at corner, as well as interior columns adjacent to the corner are selected. If the building collapses due to the removed member, the specific local resistance method can be used to design this key member to withstand abnormal loads without exceeding a specified level of damage. In the indirect design approach, the structural resistance against progressive collapse is considered implicitly through the provision of minimum levels of strength, continuity and ductility, such as catenary action of the floor slab, redundant structural systems, etc. A tie force approach is provided by DoD (2010), which prescribes a tensile force capacity of the floor or roof system to allow the transfer of load from the damaged portion of the structure to the undamaged part, as shown in Fig. 2.

The alternate path methodology is more applicable to blast or impact loads rather than fire loads, although it is typically considered to be "threat independent". Firstly, the duration of fire (in hours) is much longer than that of blast (in milliseconds), and thus the behavior of structures exposed to fire is a quasi-static process until the failure of heated members (Richard Liew and Chen, 2004). Secondly, the time when a structure collapses (i.e., fire resistance) is a key factor apart from whether it collapses. This is to say the duration the structure can resist collapse is of great importance. This fire resistance against structural collapse depends on the failure process of heated members which should be explicitly simulated in numerical models. Thirdly, all structural members (beams, columns, slabs) in a fire compartment are heated and interrelate with each other (e.g., the thermal expansion of beams and floors may push columns away which contributes to its premature buckling). This interaction cannot be simulated by simply removing the heated columns. In addition, only one column is removed each time in the alternate path method compared to several columns simultaneously heated in the case of fire. Therefore, the local failure of a structure should be included in the collapse analysis of structures exposed to fire to ensure an accurate prediction of both collapse time and collapse mode. The design methods for the prevention of fire-induced progressive collapse of structures is lacking.

3. Influencing Factors on Collapse Mechanism

In case of fire, the local failure mentioned in the definition of progressive collapse is the failure of steel members and slabs in the fire compartment. The failure of these heated components will cause failure of adjacent connections, beams and columns at ambient temperatures, and thus lead to the collapse of the whole structure. Under fire conditions, whether a structure collapses, when it col-

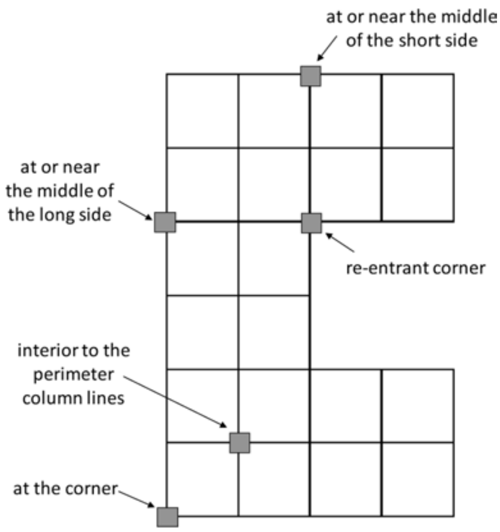

Figure 1. Potential location of column removal in a framed structure (GSA, 2003).

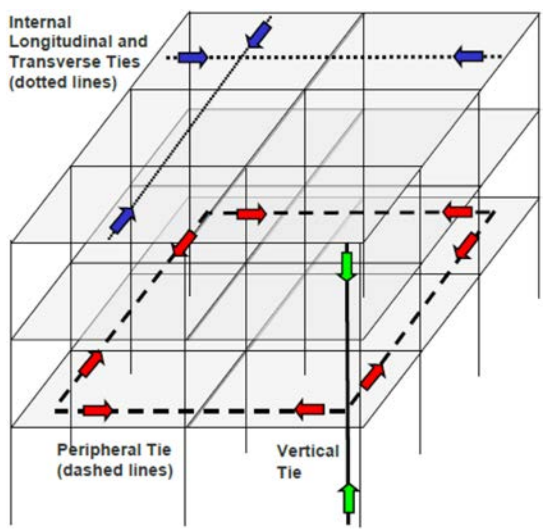

Figure 2. Tie forces in a framed structure (DoD, 2010).

lapses, and how it collapses are the main concern of engineers and researchers. A state-of-the-art review of influence factors, such as load ratios, beams, columns, slabs, connections, fire scenarios, bracing systems, fire protections on these issues is presented in the following sections.

3.1. Influence of Load Ratios

The load ratio, defined as the ratio of the imposed load to the load-bearing capacity of a member, significantly affects the collapse response of structures. In the structural fire design, a relatively low load ratio is always achieved since a relatively small design load (e.g., Dead +0.5Live) is considered compared to that (e.g., 1.4Dead +1.2Live) taken in the ambient design. This is to avoid considering two accidental events at the same time. Taking columns for example, the load ratio in fire design varies in a range of 0.2~0.3, compared to a level of 0.5~0.6 in ambient design. It was evident that a high load ratio will potentially lead to a global downward collapse of structures due to the buckling of all the columns on the fire affected floor (Sun et al., 2012a; Jiang et al., 2014a; Jiang et al., 2015a). It was also found that a frame with a high load ratio of 0.6 may collapse due to the buckling of all columns on the ground floor even if the fire occurred on the upper floor (Jiang et al., 2015a). The influence of load ratio is related to the critical temperature of steel members, that the higher the load ratio, the lower the critical temperature, and thus the earlier the collapse occurs. This indicates that the load ratio affects both the collapse mode and collapse time.

3.2. Influence of Beams

The influence of heated beams lies in its pulling-out effect on the connected columns at the early stage of heating, and pull-in effect on the columns at high temperatures. The former is due to the thermal expansion of the beam at elevated temperatures, while the latter is caused by tensile forces developed in the beam (catenary action) due to its large deflection. The survival of beams in fire will increase the lateral displacement in the column which generates a large P-D effect. The failure of beams will also lead to the loss of lateral resistance of the column, resulting in its premature buckling.

Richard Liew et al. (1998) investigated the collapse behavior of a three-bay three-storey frame under standard fire. It was found that the control parameter leading to frame collapse is the buckling of internal columns rather than the beam mechanism. If the columns were fire-protected, the collapse limit state was governed by the limiting deflection of the beam (span/20). Ali et al. (2004) found inward and outward collapse modes of 2D single-storey steel frames exposed to standard fires. The former could be triggered by the catenary action of the heated beam at high temperatures in a compartment fire. The latter occurred when the fire was localized to the column which buckled outward due to the thermal expansion of the adjacent beam at a relatively lower temperature. Sun et al. (2012a) pointed out that large beam sections could result in a global downward collapse, due to the more uniform load redistribution provided by the strong beam. Jiang et al. (2014a) mentioned that the collapse modes of steel frames with strong and weak beams were column failure mechanism and beam failure mechanism, respectively. The former mechanism was due to the buckling of the columns below the heated floor represented by a global collapse of the frame, while the latter was initiated by the premature development of plastic hinges at the ends of beams denoted by an obvious lateral drift of the heated floor.

It is concluded that the thermal expansion of the heated beams at low temperature and catenary action at high temperature have great effects on the collapse mechanism of steel frames exposed to fire. It is recommended to avoid using a large beam section since strong beams, acting as a rigid floor, will lead to an undesirable sudden collapse of structures.

3.3. Influence of Columns

The fire-induced progressive collapse of steel buildings is always triggered by the sequential buckling of columns, no matter whether the connection (or beam) fails or not. Fang et al. (2012) identified two collapse modes: single-span failure and double-span failure. The former occurred when the fire-affected column maintained its strength in fire, or the upper ambient floor can offer sufficient resistance for the heated floor. The latter was associated with the buckling of the heated column. Fang et al. (2013) proposed that sufficient structural robustness cannot be guaranteed through applying fire protection only to the steel columns due to the risk of shear failure of the unprotected beam-to-column connections. Applying fire protection to both columns and connections can be highly effective. It was recommended to employ column web stiffeners to prevent premature failure of the column web in compression. Agarwal and Varma (2014) studied the fire-induced progressive collapse of a 10-storey steel building under natural fire with cooling phase. It was found that the gravity columns played a key role in the overall stability of the building. This is because gravity columns have the highest load ratio (45~50%) since they are designed to resist gravity load alone and thus had smaller cross section, compared to moment resisting columns (5~11%) which are larger and stronger to be designed to resist lateral load. The results showed that, for a corner fire on the fifth storey, the interior gravity column failed when its temperature reached 560°C, while the perimeter moment resisting columns withstood the fire. Therefore, it is more critical to protect columns rather than beams to prevent a collapse.

3.4. Influence of Slabs

Traditionally, floor slabs are used to support loads through

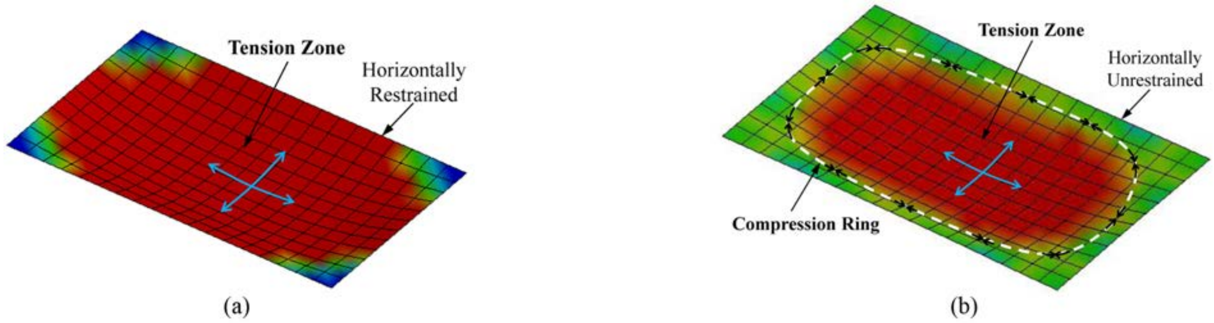

Figure 3. Tensile membrane action of reinforced concrete slabs: (a) with horizontal restraints; (b) without horizontal restraints (Jiang et al., 2018).

a bending mechanism or acts as the compression flange of composite beams. At large deflections, the slab undergoes tensile membrane action, provided the slab's perimeter is vertically supported and horizontally restrained, as shown in Fig. 3a. It is also possible for the tensile membrane action to occur in two-way spanning floors that are vertically supported but horizontally unrestrained. In this case, the slab supports load by a tension zone in the center provided by the reinforcement and a "compression ring" forming around the edges to balance the tensile forces (Fig. 3b). The merits of incorporating tensile membrane action into the structural fire design of steel-concrete composite slabs have prompted the elimination of the fire protection of interior supporting steel beams, and thus to optimize the construction cost of steel-framed structures.

Usmani et al. (2003) investigated the stability of WTC tower exposed to fire alone. The results showed that the collapse of the tower was mainly due to the thermal expansion effect rather than the material effect of loss of strength and stiffness since the temperature of columns was found within 400°C when the collapse occurred. The collapse was triggered by the buckling of external columns due to the loss of its lateral support provided by the composite truss floor systems. The loss of stiffness in floors was due to the material softening and buckling induced by restrained thermal expansion. The details of this collapse mechanism were further studied by Usmani (2005) and Flint et al. (2007), and it was found that the main reason for the collapse was the low membrane capacity in compression of the truss floor. Based on the stiffness of floors, Lange et al. (2012) proposed two collapse mechanisms: a weak floor failure mechanism and a strong floor failure mechanism (Fig. 4). The former was initiated by the buckling of the adjacent floor below the fire-exposed floor which experienced large membrane compressions. If the floor was strong enough, the external column would collapse due to the formation of plastic hinges in it on the fire-exposed floors. Quiel and Garlock (2008) compared the numerical results of 2D and 3D models of steel frames in fire. It was concluded that including the slab in the 2D structural analysis has no effect on the response since the slab undergoes tension. However, for the heat transfer analysis, the effect of slab should be considered to accurately predict the overall temperature distribution of steel beams due to the heat sink effect from the slab. Fang et al. (2013) proposed that increasing reinforcement over the hogging moment regions of joints can effectively improve the overall robustness of the structures where the rupture of rebars may govern the collapse mode. Strengthening the slab resistance through employing larger reinforcement ratio or increasing the deck thickness was also effective. Pham and Tan (2013) concluded that tensile membrane action in slabs is feasible and an effective solution for preventing progressive collapse of buildings under column loss scenarios. Greater tensile membrane forces can be mobilized in the central region due to the participation of beam reinforcement and slab top reinforcement. While in the outer region, the compressive ring of concrete can be strengthened by slab hogging moment. The studies by Agarwal and Varma (2014) showed that the loads carried by the failed column could be transferred to the neighboring columns through catenary action of slabs. Increasing the reinforcement in the slab (greater than the minimum shrinkage reinforcement) can facilitate uniform load redistribution, and thus reduce the risk of failure spread and collapse of the structure. Li et al. (2017) conducted standard fire tests on full-scale composite slabs. The tensile membrane action of slabs was found, and the effect of secondary steel beams was investigated.

Therefore, it is necessary to ensure the vertical support of the slabs to form tensile membrane action. It is recommended to increase both sagging and hogging reinforcement to enhance the tensile membrane action of the slab.

3.5. Influence of Connections

Beam-to-column connections play an important role in the structural stability of steel structures in fire. The capability of connections to sustain large tensile forces and rotations in fire directly affects the load distribution from the beam to columns, and further influence the survival of the building in fire. Evidence from the collapse of WTC tower (FEMA, 2002, NIST, 2005) demonstrated that the

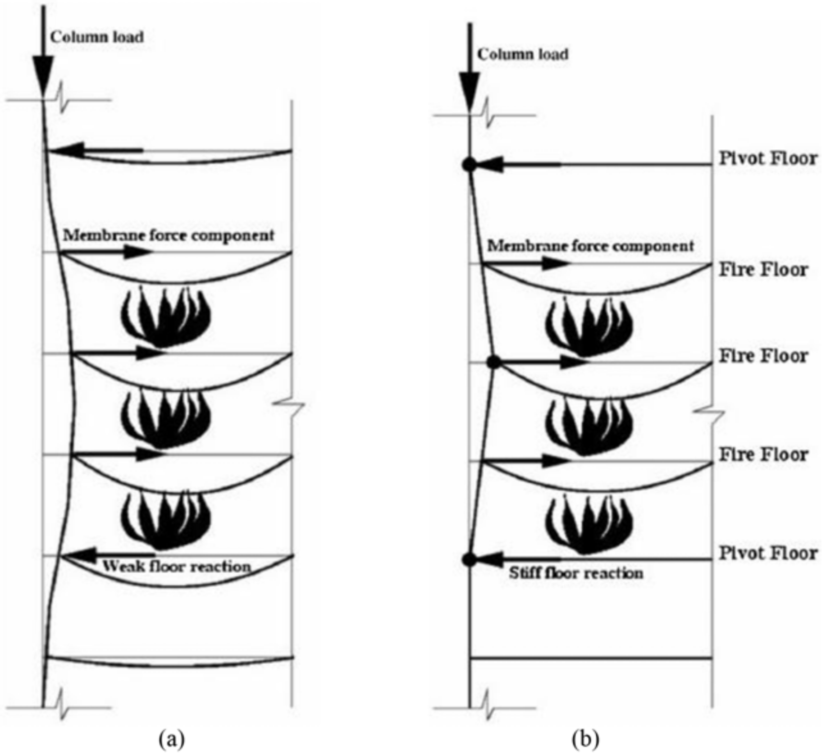

Figure 4. Collapse mechanism of tall buildings subject to multi-floor fire (Lange et al. 2012): (a) weak floor failure mechanism; (b) strong floor failure mechanism.

failure of connections led to the loss of lateral support of external columns which led to the collapse of the tower. The results of Cardington fire tests also showed how the connections help the building survive the fire without progressive collapse. Therefore, preventing the failure of connections is essential for the collapse resistance of structures in fire. This is always achieved by insulating connections with fire protection materials. Without fire protection, the temperature of joint region is much lower than the middle of the beam due to the large mass concentration and thermal shielding effect from slabs. It is specified in EN 1993-1-2 (2005) that the temperature of the joint is 62~88% of that of the lower flange of the heated beam at midspan. Considering the maximum temperature of 1000°C for the lower flange, the difference of average temperature of the joint can be more than 200°C.

Previous studies focus on the experimental and numerical investigations on isolated connections or sub-assemblages (AI-Jabri et al., 1998; Wang et al., 2007; Qian et al., 2008). Recently, Wang et al. (2011) conducted ten fire tests on medium-scale restrained steel sub-frames, including various sizes of columns and types of connections (such as fin plate, web cleat, flush endplate, flexible endplate and extended endplate connections). The experimental results showed that failure of connections only occurred when the beam was in catenary action. The beams were able to undergo very large deflections (span/8~span/6) without failure. If catenary action in the beam was considered in the structural robustness against collapse, the effects of different columns and different joints should be considered. The results also showed that the flexible end plate connection performed the poorest, followed by flush end plate and fin plate connections. The web cleat connections performed the best. The use of light columns with a relatively small cross section may prevent the failure of connections due the catenary action of the beam. Based on the experimental results, three levels of modelling of endplate connections were proposed by Chen and Wang (2012). These included a detailed model with solid elements for connection and steel members, hybrid model with spring element for connection and solid elements for steel members, reduced model with spring element for connection and beam elements for steel members. Seven fire tests were conducted by Haremza et al. (2013) on composite steel-concrete sub-frame where the concrete slab was included in the beam-to-column connection. The results showed that the compressive axial forces in the restrained beam can increase the rotation capacity and ductility of connections.

In contrast to the numerous investigations on the failure behavior of connections itself, studies on its effect on the collapse behavior of whole structures is lacking. Wald et al. (2009) carried out fire tests on the eight-storey building at Cardington. It showed that the connections were

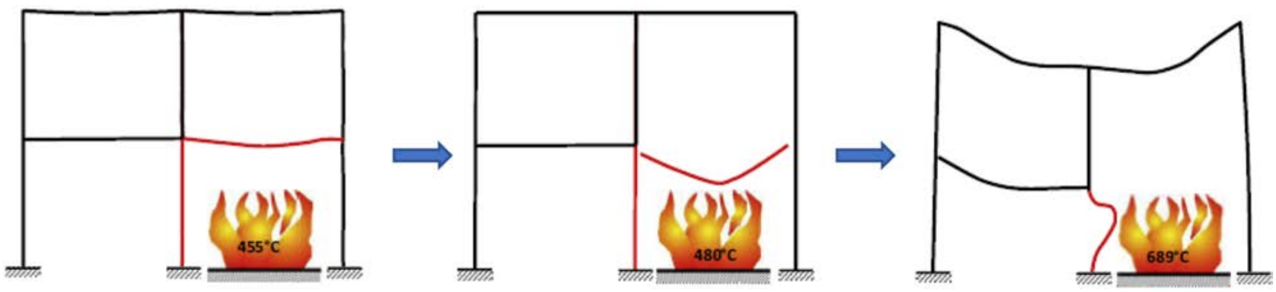

Figure 5. Progressive collapse of a frame with failure of connections (Sun et al., 2015).

subject to large axial force in a magnitude level of 300 kN during the heating and cooling phases. The tie forces calculated by EN 1991-1-7 (2006) were lower (unsafe) compared to the measured forces in the test. Yu et al. (2010) found that the effective typing of joints to prevent collapse can be improved by using a more rigid connection, increasing tensile capacity of concrete in composite slabs, using a decking profile with higher moment resistance, adding tensile reinforcement near the joint. Fang et al. (2012) identified that the shear failure of joints can happen either before or after the buckling of the heated column. Fang et al. (2013) proposed failure criteria of joints and overall system. It was assumed that the structure collapsed if the ductility limits of surrounding ambient joints were exceeded. The results showed that the fire protection to joints can be effective in avoiding punching shear under fire. Agarwal and Varma (2014) found that all the connections attached to the buckled interior column exposed to fire failed immediately after the column failure. The failure of connections can be prevented by increasing the reinforcement in the slab which significantly reduces the deflection of slabs. Sun et al. (2015) studied the effect of ductility of connections on the collapse resistance of steel frames exposed to fire. It was found that both tensile and compressive ductility of the connections contributed to the fire resistance of the beams, and also prevented the detachment of beams (Fig. 5). A beam with a longer span required higher ductility in its connection to achieve the specified level of fire resistance.

Therefore, there is limited research on the progressive failure of the connection components and their effect on the collapse resistance of whole frames in fire.

3.6. Influence of Fire Scenarios

The influence of fire scenarios includes time history of gas temperature in the fire compartment (standard or natural fire), location of fire (internal or external; lower floor or upper floor), number of fire compartments (single or multiple compartments in fire) and spread of fire (i.e., travelling fire).

The standard fire curves (ISO 834 or ASTM E119) represent only the fully developed phase of fire which is considered as the worst-case fire in enclosure. It is evident that standard fire curves cannot exhibit the behavior of real fires which include three phases of growth, fully developed, and decay of the fire. To better represent a realistic fire, natural fire curves (or parametric fire) are developed by taking into account the geometry of the compartment, ventilation condition, fire load density, thermal characteristics of materials. The primary difference between standard and natural fire curves is that the latter accounts for the cooling phase and always has a lower maximum temperature, as shown in Fig. 6. It is found that a frame may collapse in the cooling phase in a high-ventilation fire due to the less rapid temperature rise in the column than the beam because of the large cross-section of the column (Richard Liew et al., 1998; Lien et al., 2009; Agarwal and Varma, 2014). The temperature of the column will continue to increase in the cooling phase, leading to the collapse when it reaches the limiting temperature. In contrast, it will take a longer time for the frame to collapse (at 30 min) in the low-ventilation fire, compared to the collapse at 13 min in the standard ISO fire. In addition, a structure will experience plastic deformation at the early stage of a fire due to the restrained thermal expansion and induce a considerable permanent deformation after the fire is put out which may cause great damage, even collapse, of the structure (Lien et al., 2009). Neal et al. (2012) pointed out that the fire type (standard or natural

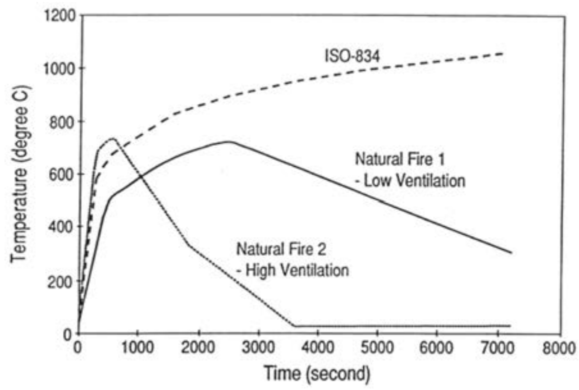

Figure 6. Comparison of temperature-time curves of ISO standard fire and natural fires. (Richard Liew et al., 1998).

fire) had negligible effect on the collapse behavior of unprotected frame since unprotected members failed early in the fire where different fire types had similar temperature time history. For protected frames, the natural fire with a decay phase could lead to a longer fire resistance. The natural fire for open plan compartments is the critical knowledge gap for performance-based design of structures in fire, and there is a new research direction toward large-compartment fire and travelling fire (Cowlard et al., 2013). Recently, Lou et al. (2018) conducted real fire tests on full-scale steel portal frames. It was found that the temperature distribution in the frame was significantly non-uniform, and the frames collapsed asymmetrically.

A fire may occur in the interior or exterior of a frame, and also occur on its lower floor or upper floor. Generally, a fire on the ground floor is more severe than that on the upper floor since the ground-floor columns have the largest load ratio. However, it is also necessary to consider the upper-floor fire that columns on the upper floors had a smaller size cross section and thus a faster temperature increase, compared to columns on the lower floor. It was found that the edge bay fire was more prone to induce the collapse of structures than the central bay fire (Jiang et al., 2014a; 2015a). It was also found that the most dangerous situation is the frame subjected to high load ratios exposed to a central bay fire where its progressive collapse may occur as early as 250°C (Jiang et al., 2014a). Four collapse modes were proposed by Jiang et al. (2014b) including the global and local downward and lateral collapse. The collapse mechanism of frames was in the form of lateral drift of the frame above heated floors for an edge bay fire and downward collapse of frames along the heated bay for a central bay fire. For multi-compartment fires, it was found that the spread of fire in the vertical direction had little effect on the collapse mode of structures, while a horizontally distributed fire scenario was prone to cause a global downward collapse of structures (Jiang et al., 2014b). Neal et al. (2012) pointed out that the upper floor fire resulted in a longer survival time compared to the lower floor fire if the beams were not protected. This is because less gravity load was transferred on the upper floor and thus the beam which spans double bays withstood the fire longer. For the protected beams, the fire in the upper floor led to a shorter survival time due to the fact that the collapse mechanism was governed by the failure of columns where upper floor columns had a faster temperature increase and thus shorter withstanding time. This indicates that the effect of fire scenarios depends on the size and fire protection of steel members. Kilic and Selamet (2013) concluded that the location of fire did not significantly change the collapse mechanism as long as the fire was contained on a single floor. This conclusion was questionable because it assumed that all the columns on one floor were heated which was an extreme situation. Nigro et al. (2014) used a probabilistic approach integrating Monte Carlo simulation to assess the probability of failure of structures in fire. This approach was to identify the most critical fire scenario.

Both the standard and natural fire curves assume a uniform temperature distribution in the compartment considering the occurrence of flash-over. A flash-over is the near-simultaneous ignition of most of the directly exposed combustible material in an enclosed area. The assumption of flash-over is valid for a relatively small compartment (i.e., small compartment fire), up to 500 m^2 of floor area without openings in the roof and for a maximum compartment height of 4 m (EN 1991-1-2, 2005). Flash-over is unlikely to occur in large or open compartments, and thus a localized fire should be taken into account where a non-uniform temperature is assumed. Ali et al. (2004) found that a frame under a small compartment fire will collapse inward due to the catenary action of the heated beams which drive the columns inward. When the fire localized to the column, the column will buckle outward pushed by the expanding beam at a relatively low temperature.

Observations from realistic fires such as those in WTC tower and Windsor Tower have revealed that the fire in large open areas travels across the floors rather than burning simultaneously for the duration. Indeed, combustible materials in large compartments are consumed at a rate governed by the ventilation condition, leading to a non-uniform temperature in the compartment. A review of research on travelling fire can be found in the reference (Behnam and Rezvani, 2015, Rackauskaite et al., 2015). Most previous studies focus on the formation of travelling fire or its effect on the behavior of structural members. The behavior of structures against progressive collapse under travelling fires is not well understood. Generally, a travelling fire has two fields: the near-field (flame) and the far-field (smoke), as shown in Fig. 7. The spread of fire can produce larger beam deflection than does simultaneous heating of multiple compartments, and there was possibility that a frame collapsed during the cooling phase (Bailey et al., 1997). Richard Liew et al. (1998) found that the frame can survive the fire scenario where all bays in one storey were heated simultaneously, but not in the case of fire spread to adjacent two compartments. This is because extra compression was induced in the cooling beam in the source compartment provided by the heating of adjacent beams. It was found that the maximum deflection and residual deflection of the beam in the source compartment were higher if fire spread was considered. This indicates the importance of considering the possibility of fire spread in the determination of the required collapse resistance of the building exposed to fire. If not, it is suggested to ensure the fire partition to effectively prevent the spread of fire from one compartment to another. Behnam and Rezvani (2015) pointed out that the frame was more vulnerable to travelling fire compared to standard fire. However, the collapse mechanism of structures under travelling fire is still not clear, and thus further

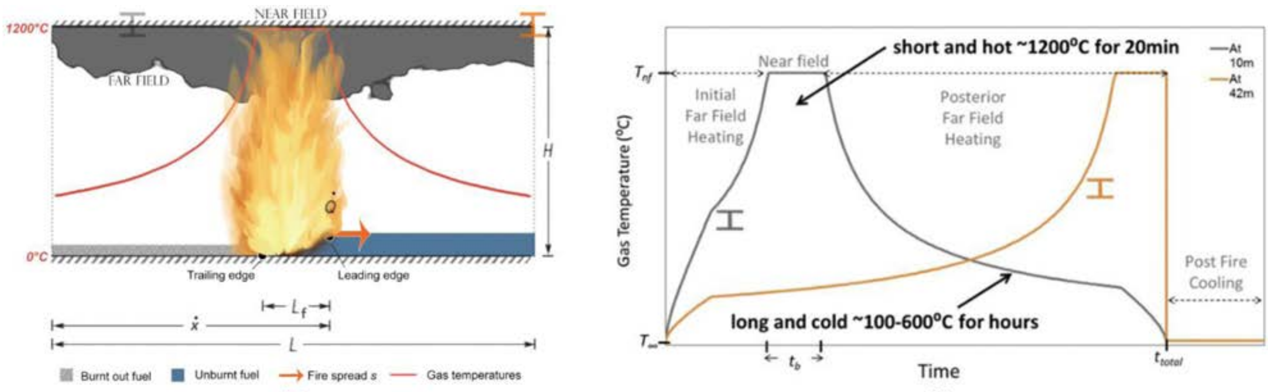

Figure 7. Illustration of a travelling fire: (a) Definition of the near field and far field; (b) distribution of gas temperature. (Rackauskaite et al., 2015).

work should be done.

3.7. Influence of Bracing Systems

Preventing the spread of local failure is the key to ensure the resistance to disproportionate collapse. Increasing structural redundancy is an effective way for this purpose to enhance the robustness of structures against collapse. Some attempts have been made by using bracing systems to enhance redundancy of structures at ambient temperatures and provide alternative load redistribution path after a local failure. Bracing systems are most commonly used in a building to resist lateral loads induced by seismic or wind actions. Two types of bracing systems are always used: vertical bracing system placed along the entire height of the building and horizontal bracing system placed on individual floors (e.g., hat bracing on the top floor), as shown in Fig. 8.

The hat bracing is effective to uniformly redistribute loads to adjacent columns, and thus delay or prevent the collapse of structures (Flint et al., 2007). However, it failed to resist the lateral drift of columns which may lead to a global downward collapse (Sun et al., 2012b). A vertical bracing system can act as a barrier to prevent the spread of local failure to the rest of structures (Jiang et al., 2015b). It is thus recommended to use a combined bracing system in practical design. Sun et al. (2012a) studied the effect of lateral bracing systems on the collapse resistance. The bracing system was modelled by axial elastic spring with different stiffness. It was found that the global failure of the frame was not sensitive to the lateral stiffness, but was governed by the sequential buckling of columns. The collapse behavior of braced steel frames was further studied by Sun et al. (2012b) where the braces were explicitly modelled. It was found that the hat truss acted as a rigid beam across the top storey of the frame to distribute the vertical reaction forces between adjacent columns. The collapse of the frame was triggered by the buckling of bracing members in compression. The application of stronger bracing members increased the collapse temperature but generated larger axial forces in the adjacent column, leading to its premature buckling. This indicates that the hat truss has a limited capacity to avoid the pull-in of columns in the heated floor. Increasing the strength and stiffness of bracing members have the potential to prevent the collapse. The vertical bracing system can effectively prevent the spread of local failure from bay to bay, and also increase the lateral restraint of the frame, reduce the pull-in effect of the columns. Jiang et al. (2015b) recommended an interior arrangement of vertical bracings which effectively prevented the spread of local damage to the rest of structures. Talebi et al. (2014) investigated the application of buckling restrained braces (BRB) on the prevention of progressive collapse of steel frames in fire. It showed that BRBs provided an enhanced collapse resistance for the frame due to its prevention of buckling of bracing members.

3.8. Influence of Fire Protections

Fire resistance of steel-framed structures has traditionally

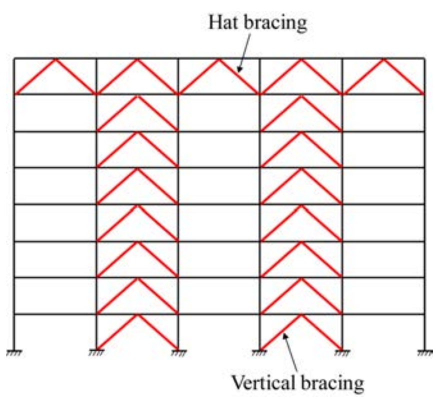

Figure 8. Schematic of practical layout of bracing system in a framed structure.

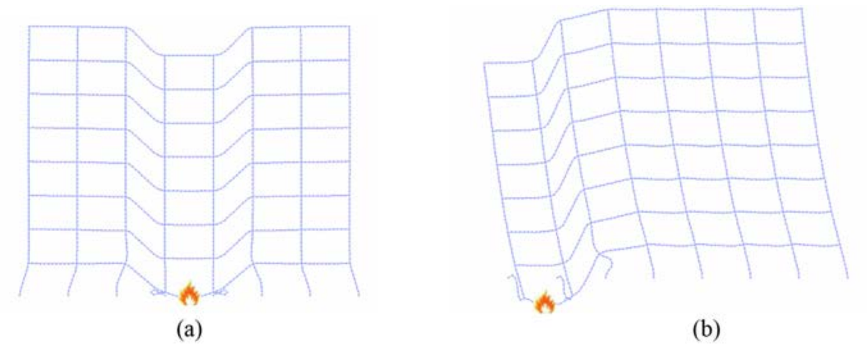

Figure 9. Collapse mode I - general collapse with combined lateral drift of the frame and buckling of columns: (a) fire at midspan; (b) fire at edge.

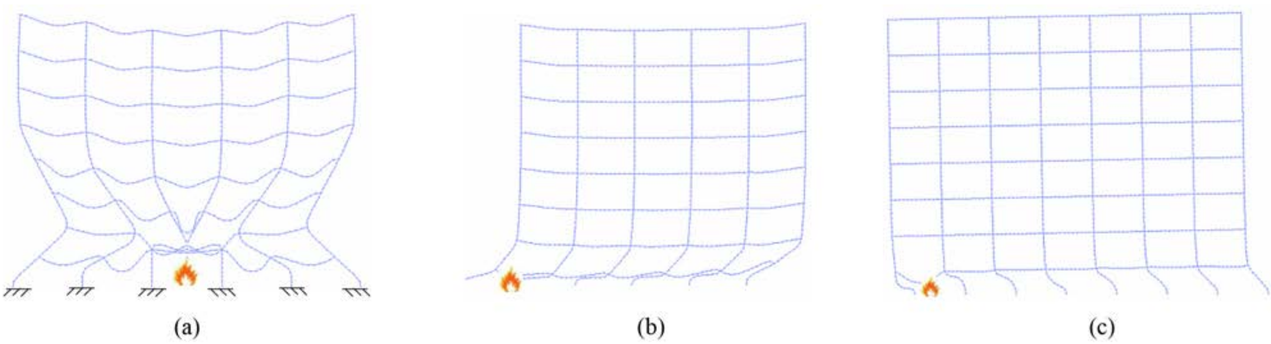

Figure 10. Collapse mode II - lateral drift collapse: (a) frame with weak beams; (b) frame with high load ratio; (c) frame with hat bracing.

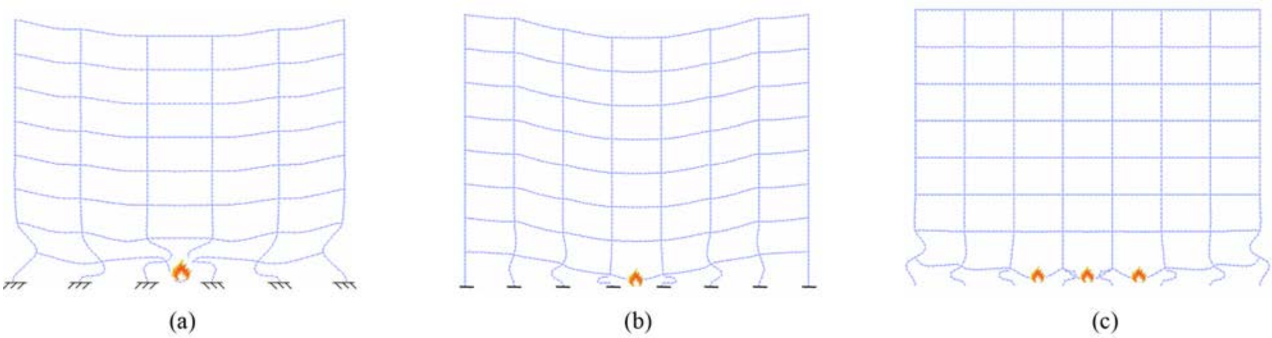

Figure 11. Collapse mode III - global downward collapse: (a) frame with strong beams or high load ratio; (b) frame with vertical bracing; (c) frame with combined hat and vertical bracing.

been ensured by applying insulating materials around the steelwork, such as sprays, boards, blankets, and intumescent coatings (Xu et al., 2018). The application of fire protections will delay the temperature rise in the steel members and enhance their fire resistance. Quiel and Marjanishvili (2012) studied the fire resistance of a damaged steel building against progressive collapse. The middle column on the ground floor was removed in a four-bay and five-storey frame, and a fire was assumed in the middle two bays. It was found that the unprotected frame collapsed due to sagging failure of beams at 10 min. The protection of beams alone led to the buckling of the heated unprotected columns and the collapse time was extended to 30 min. The 1-h fire protection for both beams and columns resulted in a 1.5 h fire resistance with a collapse model of sagging failure of beams which is more ductile and preferred than the column failure. This indicates the importance to protect columns. Furthermore, Neal et al. (2012) considered a combination of fire protection of beams and columns. They concluded that fire protection had an important effect on the collapse resistance. The unprotected beam always failed before the column because it experienced a faster temperature increase due to its three-sided fire exposure. If the beam was protected, the collapse mode and time were affected significantly by the fire location and the fire type. Fang et al. (2013) proposed that the application of fire protection is not always an effective way to increase the collapse resistance for a localized fire with limited fire affected area. Fire protection may even lead to an undesirable reduction in overall resistance due to the elimination of thermal expansion which can enhance the rotation capacity and ductility of joints. The collapse mechanism of an 8-storey braced steel frame with concrete slabs was studied by Jiang and Li (2017b). It was found that the fire protection of steel members had a significant influence on the resistance of structures against fire-induced collapse. A protected frame did not collapse immediately after the local failure but experienced a relatively long withstanding period of at least 60 min (Jiang and Li, 2017b). This indicated that the overall fire resistance of the frame against global collapse was somewhat 1-hour longer than that of individual members.

4. Discussion

4.1. Potential Collapse Mode

The collapse mode of steel framed structures exposed to fire has been comprehensively investigated through 2D and 3D models. It is confirmed that the progressive collapse of a structure will be triggered by the buckling of adjacent columns at ambient temperatures with or without lateral drift in them. The lateral drift of columns is driven by the catenary action of the heated beams or tensile membrane action of the heated floors. Three collapse modes of steel buildings are summarized based on the review of various influencing factors in previous sections, as shown Figs. 9~11, respectively. These are: general collapse mode, lateral drift collapse mode and global downward collapse mode. The general collapse is the most common collapse mechanism where the collapse is due to the buckling of adjacent columns experiencing obvious lateral drift (Fig. 9). If the catenary action in the beam is significant (weak beam) or the load ratio is high, the lateral drift of the frame will govern its collapse (Fig. 10). A frame with hat braces under an edge fire may also collapse laterally. If the lateral drift of the frame is restrained, the frame will globally collapse downward (Fig. 11). The application of vertical bracing or multi-compartment fires may lead to this downward collapse mode.

These collapse modes are concluded mainly based on small compartment fires, their feasibility for large open-plan compartment fires (localized fire and travelling fire) should be further checked. Research on the measures to mitigate or prevent the progressive collapse of steel buildings as well as practical design approaches is still lacking.

4.2. 2D Model vs 3D Model

The advantage of using 3D models over 2D models is to account for more realistic fire scenarios, load redistributions and beneficial effect of slabs. A comparison between 2D and 3D models (Jiang and Li, 2017b) showed that the 2D model produced conservative results by underestimating the collapse resistance, and it cannot capture the load redistribution in a 3D model where more loads were distributed along the short span than those along the long span. Although, the 2D model and 3D model may lead to similar results on whether and how a frame collapses in some cases, but they provide quite different predictions on when the frame collapses in most cases. This is because the 2D model cannot fully consider the redundancy in a real structure which will significantly delay the collapse. Therefore, 3D models should be used to make an accurate prediction of the collapse mode and collapse time of frames exposed to various fire scenarios.

5. Conclusions

This paper reviewed the influencing factors on the progressive collapse mechanism of steel framed buildings exposed to fire. Three collapse modes were found for various load ratios, strength of beams and columns, fire scenarios, layouts of bracing systems. The following conclusions can be drawn:

(1) The key influencing factors on the collapse mode are load ratio, fire scenario, bracing layout, and fire protection.
(2) A high load ratio is prone to cause global downward collapse of a frame which is undesirable.
(3) It is found that the collapse of the frame is governed by the stability of columns rather than deflection of beams. It is desirable to strengthen the columns by fire protections or column web stiffeners, or to use relatively weak beams to facilitate the formation of catenary action in beams which may lead to a ductile collapse mechanism.
(4) The tensile membrane action in slabs is effective to prevent the collapse of the frame. It is necessary to ensure the vertical support of the slabs for the formation of tensile membrane action. It is recommended to increase both sagging and hogging reinforcement to enhance the tensile membrane action of the slab.
(5) Preventing the failure of connections is essential for the collapse resistance of structures in fire. The failure of connections can be prevented by increasing the reinforcement in the slab, using rigid connections, applying fire protection and reducing the size of columns.
(6) It is found that a frame may collapse in the cooling phase and under travelling fire rather than the heating phase and standard fire, respectively. This is because the steel members may experience maximum temperature and maximum displacement under

these two fire scenarios. An edge bay fire is more prone to induce the collapse of structures than the central bay fire. Multi-compartment fires in the horizontal plane, as the severest fire scenario, will lead to global downward collapse. A fire in the upper floor may lead to a shorter survival time due to the buckling of adjacent columns which have a faster temperature increase.

(7) A combination of hat and vertical bracing systems is recommended to enhance the collapse resistance of structures in fire. Increasing the strength and stiffness of bracing members have the potential to prevent the collapse.

(8) The application of fire protections on steel members has a significant influence on the resistance of structures against fire-induced collapse. A protected frame may not collapse immediately after the local failure but experienced a relatively long withstanding period of at least 60 min. The collapse mode and time of protected frames may be affected significantly by the fire location and the fire type.

(9) It is necessary to use three-dimensional models for accurate predictions of the collapse mode and collapse time of frames exposed to various fire scenarios.

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