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# Effect of bracing systems on the fire-induced progressive collapse of steel structures

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# Effect of bracing systems on the fire-induced progressive collapse of steel structures

**ABSTRACT**

This paper investigates progressive collapse mechanisms of braced steel frames subjected to various fire scenarios using OpenSees, an open-source object-oriented software developed at UC Berkeley. The OpenSees framework has been recently extended to deal with structural behavior under fire conditions by authors. This paper summaries the key work done for this extension and focuses on the application of the developed OpenSees to study the effect of different bracing systems on the fire-induced progressive collapse resistance of steel-framed structures. The study considers two types of bracing systems (vertical and hat bracing) and different fire scenarios such as single and multi-compartment fire on the ground floor and second floor. Four collapse mechanisms of steel frames in fire are found through parametric studies. These are general collapse characterized by the collapse of the heated bay followed by lateral drift of adjacent cool bays, global collapse of the whole frame due to the buckling of ground floor columns, local and global lateral drift modes of collapse caused by catenary action developed in the heated beams under large deflections. All the collapse mechanisms are triggered by the buckling of the heated columns. The thermal expansion of heated beams at early heating stage and their catenary action at high temperature have great influences on the collapse mechanisms. The vertical bracing systems has positive effects on increasing the lateral restraint of the frame against local or global drift, while when arranged at edge bays of frames they negatively contributes to the spreading of a local damage to a global collapse in the form of sequential buckling of adjacent columns through load-transfer mechanisms. For a more realistic arrangement of vertical bracings inside the frame, the bracing acts as a barrier to restrain the spread of local damage to the rest of the frame. Instead, using hat bracing can effectively optimize the load-transfer path through a more uniform redistribution of loads in columns and enhance the resistance of structures against progressive collapse. The application of vertical bracing systems alone on the steel frames to resist progressive collapse is proved to be unsafe and a combined vertical and hat bracing system is recommended in the practical design.

**KEYWORDS:** OpenSees; progressive collapse; steel frame; bracing system; fire scenario; collapse mechanism

## 1 INTRODUCTION

Steel structures have many advantages such as lightweight, high strength, appealing architecture, ease of erection, and recyclable use of materials. These advantages make them particularly suitable for application in high-rise and very tall buildings in China and elsewhere in the world. However, steel structures are not inherently fire resistant because much of the strength of steel is lost when the steel temperature reaches 600$^{o}$C or above during a fire. Due to the high-rise and often landmark nature of such buildings, the probability of them being subjected to long sustaining fire is high. When such an incident occurs, despite fire protection, the risk of some members losing their local load-bearing capacity is very high due to a multiple of feasible reasons such as more severe fire exposure than designed, loss of fire protection due to impact (the case of World Trade Centre) or lack of durability.

If the structure were to have low resistance against progressive collapse after local failure of some components, consequent catastrophic progressive collapse could take place, causing tremendous tragedy as a result of loss of lives and properties and immeasurable societal impact.

The progressive collapse of structures 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 7 2005). The assessment of collapse performance of structures and measures for the mitigation of disproportionate collapse can be found in various design codes (GSA 2003; ASCE 7 2005; DoD 2010). 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. Recent large building frame tests (Cardington fire tests) in real fire conditions as well as investigations on the collapse of WTC under terrorist attack have shown that detailing of connections and structural redundancy are key to enhance robustness of structures against fire-induced progressive collapse. A redundant structure is one with extra internal load paths or external supports in excess of the minimum required for stability. The redundancy allows a structure to transfer loads retained previously by damaged components to its surrounding parts through a variety of load paths to prevent a local or global failure. Bracing systems, as one of the effective measures to enhance the redundancy of structures, are most commonly used to resist seismic and wind loading. Several researchers have studied the potential of braced frames to mitigate progressive collapse of structures. Khandelwal et al. (2009). investigated the progressive collapse resistance of steel frames with concentric and eccentric bracing. It was concluded that an eccentrically braced frame is less vulnerable to progressive collapse than a special concentrically braced frame. Lotfollahi and Alinia (2009) studied the effect of tension bracing on the collapse mechanism of steel moment resisting frames. It was suggested that the use of weakened beams/columns or large size bracings should be avoided and beams should be designed stronger beams than the current design recommendations. Kim et al. (2011) studied the collapse mechanisms of braced frames with various bracing configurations and deduced that the inverted-V type possessed superior ductile behavior during progressive collapse. Asgarian and Rezvani (2012) proposed a new algorithm to investigate capacity of concentrically braced frames against progressive collapse. The minimum residual capacity and the most critical locations of element loss as well as element removal impact factor for the frames were determined. Results showed that the frame with two braced bays had more robustness for mitigating progressive collapse. Fu (2012) studied the response of a concentrically braced multi-storey steel composite building under consecutive column removal scenarios using a 3-D finite element modelling approach. The results showed that the formation of plasticity were strongly related to the column removal sequences where the corner column removal scenario was of the most danger.

In a fire situation, the concept of bracing system can also be applied to the design against progressive collapse of steel framed structures. However, relevant researches are lacking. Ali et al. (2004) studied the collapse modes and lateral displacements of single-storey steel-framed buildings exposed to fire. Two collapse modes were found including inward collapse due to catenary action of the heated beam and outward collapse resulting from the thermal expansion of the heated beam. The results showed that the lateral displacement of frames increased with the increase of spatial extent of fire and roof weight which may affect the minimum clearance between frames and firewalls. It also indicated that the creep should be considered for high roof loads and tall columns. Usmani (2005) proposed a possible progressive collapse mechanism for tall frames such as the WTC twin towers in fire. The mechanism involved a complete deformation sequence of frames, from initial thermal expansion, followed by the buckling and subsequent tensile membrane behavior of the heated floors, to the column buckling due to the weakened lateral restraint from the floors. Huang and Tan (2006) proposed a new sub-frame model and isolated member model to ascertain the fire resistance of beams and columns subjected to compartment fires. Takagi and Deierlein (2007) indicated that the variability in the high-temperature yield strength of steel is the most significant factor in the collapse probability assessment of steel-framed buildings in fire. Fang et al. (2011) proposed multi-level system models for progressive collapse analysis of structures exposed to fire. Two robustness assessment approaches namely temperature-dependent and temperature-independent approaches were carried out using the proposed models. The latter ignored the temperature effect but considered the model reduction due to the heating by removing several heated members of the structures. Quiel and Marjanishvili (2012) used a multi-hazard approach to evaluate the performance of a damaged structure subjected to a subsequent fire. Fang *et al.* (2012) conducted a realistic modeling of a multi-storey car park under a vehicle fire scenario. Three failure modes such as single-span failure, double-span failure and shear failure were proposed. Simplified robustness assessment methods of car parks under localized fire were proposed (Fang et al. 2013). Lange *et al.* (2012) proposed two collapse mechanisms of tall buildings subjected to fire on multiple floors, namely, a weak floor failure mechanism and a strong floor failure mechanism. A simple design assessment methodology was proposed. Sun *et al.* (2012a) carried out static-dynamic analyses of progressive collapse of steel structures under fire conditions using Vulcan. The influences of load ratios, beam size and horizontal restraint on the collapse mechanisms were discussed. The same procedure was then used to study the collapse mechanisms of bracing steel frames exposed to fire (Sun *et al.* 2012b). The results indicated that a combined hat and vertical bracing system can enhance the robustness of structures to resist the progressive collapse. However, the analysis was conducted on a 2D frame model with five bays and four storey which was supposed to be inadequate to consider the redundancy effects. Neal et al. (2012) studied the fire resistance of a prototype steel high-rise building taking into account of various fire protection and fire scenarios. The results showed that a subsequent fire may lead to building collapse if the building sustained localized damage to the extent where nearby fire protection was removed or damaged.

The only reliable method, at present, of predicting the complex structural behavior observed in any whole frame structure in fire is to model the frame using a finite element code incorporating geometric and material nonlinearity with increasing temperature and suitable thermal expansion coefficients. Many finite element programs have been written to simulate the structural behavior at elevated temperature. These include specialist programs such as ADAPTIC (Song 1995; Izzuddin 1996), SAFIR (Franseen 2000; Vila Real *et al.* 2004), VULCAN (Bailey 1995; Huang 2000) and commercial packages such as ABAQUS (Gillie *et al.* 2001, 2002) , ANSYS (Kodur and Dwaikat 2009; Cai *et al.* 2012), MIDAS, etc. Although specialist programs are cost-effective to purchase and easy to use they lack generality and versatility because they are always developed to focus on some special feature of structural behavior in fire and limited in a relatively small number of users and developers. The commercial packages have a large library of finite elements and excellent GUIs to enable efficient and detailed modeling of structural responses to fire and also allow user subroutines for modeling special features of structural behaviors. Despite obvious advantages commercial packages require substantial recurring investment for purchase and maintenance that often make them unaffordable for researchers and deter new entrants to the field. An alternative to commercial packages and specialist programs is open source software, where the source codes of the software is made available for anyone to download, modify, and use (mostly for free).

OpenSees is an open-source object-oriented software framework developed at UC Berkeley (McKenna 1997). OpenSees has so far been focused on providing an advanced computational tool for analyzing the non-linear response of structural frames subjected to seismic excitations. Given that OpenSees is open source and has been available for best part of this decade it has spawned a rapidly growing community of users as well as developers who have added considerably to its capabilities over this period, to the extent that for the analysis of structural frames it has greater capabilities than that of many commercial codes.

The main objective of this paper has been to study the influence of bracing systems on the progressive collapse resistance of steel moment resisting frames (MRF) in fire using the OpenSees framework developed by authors (Jiang 2012; Jiang *et al.* 2013). The details of the extension in OpenSees and corresponding verification and validation can be found in these references. After a brief introduction of developments in OpenSees, uing an implicit dynamic analysis (Newmark method) in OpenSees, parametric studies were carried out first on the progressive collapse analysis of braced steel frames under single-compartment fire on the ground floor. The effect of vertical and hat bracing on the axial forces and end moments in the heated members are then investigated. In addition to the single-compartment, multi-compartment fires on the ground floor and second floor, respectively were considered to investigate the effectiveness of the bracing systems against progressive collapse of frames.

## 2. BACKGROUND OF DEVELOPED OPENSEES

This study has been carried out using OpenSees which has currently been developed by authors at the University of Edinburgh and Tongji University for analyzing the behavior of structures in fire. The extended two-dimensional modeling capability of structures in fire has been embedded in the released OpenSees 2.4.0. A big picture of the development of OpenSees is to provide a complete and fully automated software framework for the fire model, heat transfer model and structural model. The current development of OpenSees focuses on the mechanical behavior of structures under pre-defined temperature distribution. In this stage no fire and heat transfer models are developed in OpenSees. The extensions involve creating a new thermal load pattern class and modifying existing material, section and element classes to include temperature dependent messages. Figure 1 briefly illustrates the hierarchy of the existing and developed classes in OpenSees. More details can be found in reference (Jiang *et al.* 2013a). A thermal load class Beam2dThermalAction was created to store the temperature distribution in members which was classified as an elemental load.  The storage of temperatures was defined through the depth of the beam section by coordinate (*LocY*) and the corresponding temperature (*T*). At this stage a total of 2, 5 and 9 temperature points are available, respectively. New temperature dependent material classes for steel and concrete (Steel01Thermal and Concrete02Thermal) were derived by modifying the existing corresponding material classes (Mazzoni *et al.* 2007) according to Eurocodes (ENV 1992,1993). The Opensees currently supports both distributed plasticity and concentrated plasticity based Euler-Bernoulli beam-column elements. Moreover, the distributed plasticity beam-column elements can be classified into the typical displacement-based (DispBeamColumn) and force-based beam-column elements (ForceBeamColumn) (Spacone and Filippou 1992). Both these two beam/column elements have been modified to include temperature related interfaces (DispBeamColumn2dThermal and ForceBeamColumn2dThermal). A variety of solution algorithms are available in OpenSees for static and dynamic analyses (Mazzoni *et al.* 2007). The load control, displacement control and arc-length control methods can be used for static analyses with various iteration methods for nonlinear problems such as the Newton-Simpson method. For dynamic analyses, explicit integration methods such as central difference methods and implicit integration methods such as the Newmark method and HHT method are available in the existing framework of OpenSees. The existing analysis algorithms in OpenSees are inherently compatible with the developed classes by authors and can be used directly for the progressive collapse of structures.

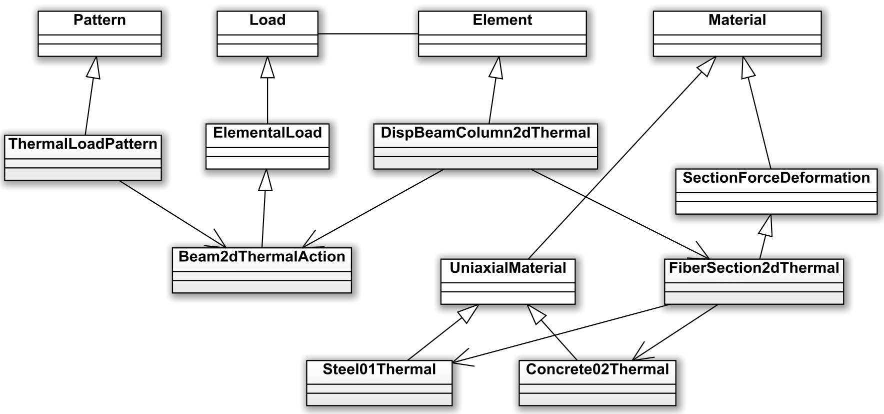



Figure 1 Class diagram for thermomechanical analysis in OpenSees.

The static analyses of structures in fire using developed OpenSees have been extensively verified and validated by authors (Jiang 2012; Jiang *et al.* 2013). The OpenSees framework provides various static solution algorithms to  facilitate the convergence such as Newton method, Modified Newton method, Arch-length method, etc. (Mazzoni *et al.* 2007). However, when using a conventional static procedure for progressive collapse analyses, it will often subject to a fatal singularity in the stiffness matrix when one or more structural members fail or buckle where a dynamic procedure has to be used. In this study, an existing implicit dynamic procedure in OpenSees, i.e. Newmark method ($\beta$ =0.8 and $\gamma$ =0.45), is used to conduct the progressive collapse analysis of steel frames under fire conditions. The validation of the combined performance of the developed structural fire model and existing dynamic analysis framework can be found in the reference (Jiang et al. 2013c, submitted to Advances in Structural Engineering).

The reason for selecting implicit over explicit analysis solution scheme is because an implicit analysis solves the system of equations for each increment and performs Newton-Raphson iterations until it reaches convergence while explicit analysis does not attempt to reach a converged solution for each time step. For that reason an explicit analysis typically uses many more time steps than an implicit one. Franssen and Gens (2004) have suggested that the numerical damping is accurate enough for most "structures in fire" applications since there are no highly dynamic effects present despite fire's transient nature. They proposed increasing the Newmark parameters "$\beta$" and "$\gamma$" when using the Newmark integrator. A similar procedure is followed in this paper by adding numerical damping when conducting dynamic analyses of structures in fire. This has been achieved in OpenSees by using the Newmark integrator with the values suggested (0.8 and 0.45) by Franssen and Gens (2004).

## 3. DETAILS OF STEEL FRAMES STUDIED

The main objective of this paper is to investigate the effectiveness of bracing systems in mitigating progressive collapse of steel moment resisting frames exposed to fire. Taking into account of the high structural redundancy as well as the computational efficiency, a 2D steel frame of seven bays with 6m span and eight storey with 4m storey height was modeled in this study, as shown in Figure 2. Two types of bracing systems were taken in this study. These are a "hat truss" and a vertical bracing system. The configuration of bracing systems is supposed to have great influence on its effectiveness against progressive collapse of frames. To filter the effect of configuration of bracings as well as make simplicity but without losing generality, in this paper, the hat bracing was reduced to a series of rigid beams cross the top storey of the frame model, whilst the vertical bracing was represented by a series of lateral restraints on each storey to restrain the horizontal movement of the frame. Both the beams and columns in the compartment exposed to fire were heated and the adjacent compartments were left at ambient temperature. In this way, the catenary action of the heated beam due to large deflections was considered. Uniform temperature distributions based on the temperature-time curve defined in the standard fire ISO834 were assumed in the heated members, not only along their length but across the depth of the cross-section. Two ground-floor fire scenarios was used in this study. Fire 1 is a fire occurring in the central bay and Fire 2 represents a fire in the edge bay. The Newmark dynamic analysis was carried out in OpenSees to study the behavior of the steel frame under fire conditions. The Newmark parameters were taken as 0.8 and 0.45, respectively. The corotational geometrical transformation in OpenSees was used to consider the geometric nonlinearity (Taucer and Filippou 1991).

In this case, all the beams and columns are taken as UB 305x165x40 and UC 254x254x89, respectively. A mesh of 8 and 12 elements were employed for each beam and column, respectively. The temperature dependent bilinear plastic material (Steel01Thernal) was used for steel members. The strain hardening was adopted with a slope of 1% of the initial modulus of elasticity to facilitate the convergence of the analysis. The modulus of elasticity and yield strength of steel at ambient temperature were taken as 200GPa and 280MPa, respectively. The properties of the steel material at elevated temperature referred to Eurocode 3 (ENV 1993-1-2 2005).

In this study rigid connections between beams and columns are assumed, in which their failure and fracture were not considered in the analyses. The hat truss bracing systems and vertical bracing systems are reduced to rigid beams on the roof and lateral restraints against horizontal displacements, respectively. Hence, the buckling and failure of the bracing members were ignored in this paper.

Although several idealizing assumptions have been made the study gives an insight into the performance of bracing systems in redistributing forces within the frame and preventing progressive collapse after the buckling of the heated columns.

The frame without bracing is firstly analyzed under various fire scenarios, followed by applying the two bracing systems separately and in combination. Single-compartment fires at central and edge bays were used for a detailed investigation of progressive collapse mechanisms of braced steel frames. The effect of bracing systems against collapse of frames under multi-compartment fire conditions were then studied.

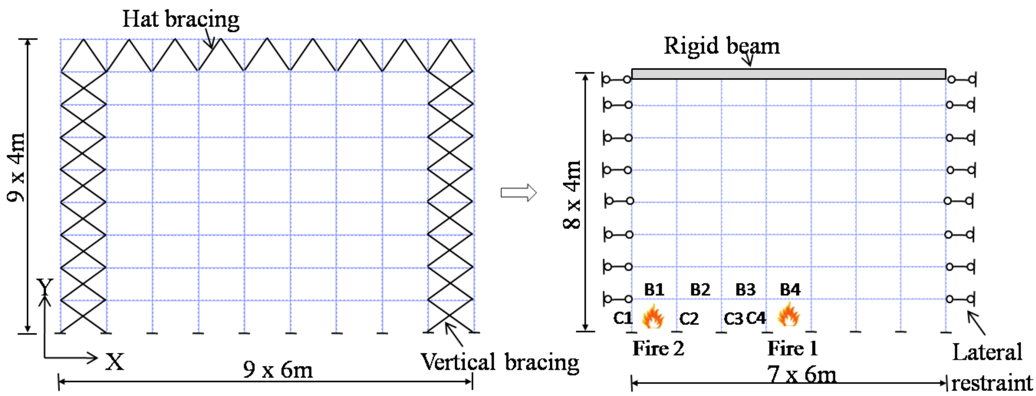



Figure 2 Schematic of the steel frame in fire modelled in OpenSees

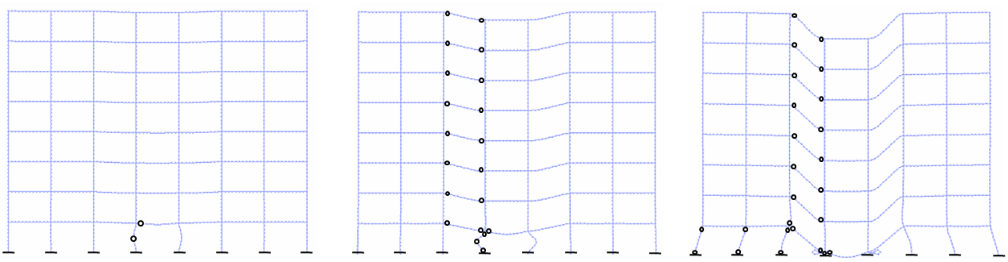
## 4. PROGRESSIVE COLLAPSE ANALYSIS OF STEEL FRAMES

### 4.1 Case 1: Behavior of frames without bracing

#### 4.1.1 Central bay fire (Fire 1)

Figure 3 shows the collapse procedure of the steel frame under central bay fire (Fire 1), plotted together with the formation of plastic hinges in beams and columns against temperature. Due to the symmetry, the plastic hinges are plotted on half the model. It can be seen that the collapse is triggered by the buckling of the heated column and aggravated by the pull-in of the upper remainder of the frame above the heated floor. At the early heating stage the heated compartment expands outwards through the thermal expansion of the heated beam and columns. Additional compression forces are generated in the heated members due to the restrained thermal expansion by the surrounding cool structure. The compression forces in the heated column C4 increases first as the yield strength of steel material keeps constant before 400 $^{o}$C. Once the compression forces in C4 exceeds its buckling load, the column buckles at around 540$^{o}$C as shown in Figure 3a. Beyond this point the floor above the heated column C4, losing vertical supporting and having to sustain the vertical loads previously carried by the heated column, experiences large deflection which leading to the formation of plastic hinges at the ends of beams at the adjacent bay at 650 $^{o}$C, as show in Figure 3b. On the other hand, due to the large deflection, tension force can be generated in the heated beams, i.e. catenary action, after 600 $^{o}$C, as shown in Figure 6a. The tension force in the beam starts to gradually pull in the upper frame, leading to the formation of plastic hinges at the two ends of ground floor columns when the temperature reaches 750 °C as shown in Figure 3c. In this way a local mechanism forms among the ground columns where the P-Δ effect will aggravate the their drift inwards and complete collapse of the whole frame occurs.

Figure 6a shows the variation of the normalized axial force and end moment in the heated beam and column, enveloped by the yield force (Mp or Fy). Mp and Fy are the plastic moment and axial yield force of the beam/column section, respectively. The declined yield force envelop takes into account of the material degradation at elevated temperature. The heated beam B4 experiences compression forces before 600 °C and tension afterwards. The axial forces and end moments in the heated column C4 increases first due to the restrained thermal expansion and reduces later after 400 °C, where the yield strength of steel starts to reduce according to Eurocode 3. It is noted that the end moment reduces to zero at about 540 °C while the axial compression force in the column reaches its yielding limit. After this point the axial force in the column follows the yield force envelop and moments at the ends of the column increases again, even in excess of the yield limit due to the large rotation overwhelming the effect of material degradation. Figure 7a shows the development of axial forces in columns C1-C4 against temperature. It is clear that the loads previously sustained by the buckled heated column C4 is redistributed on C3 alone. The other columns sway before the load redistribution.

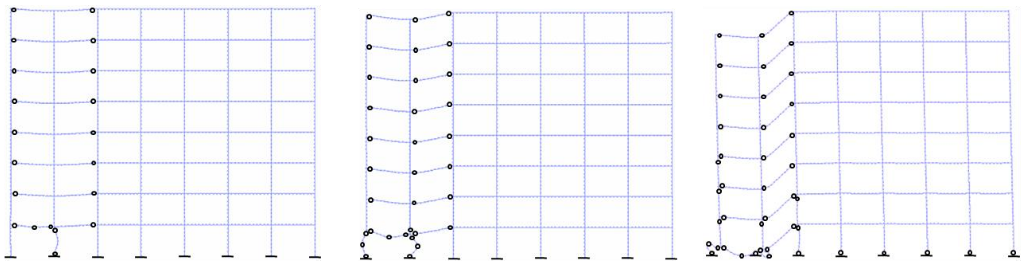


(a) 540°C (b) 650 °C (c) 750 °C

Figure 3 Failure process of the steel frame without bracing under Fire 1

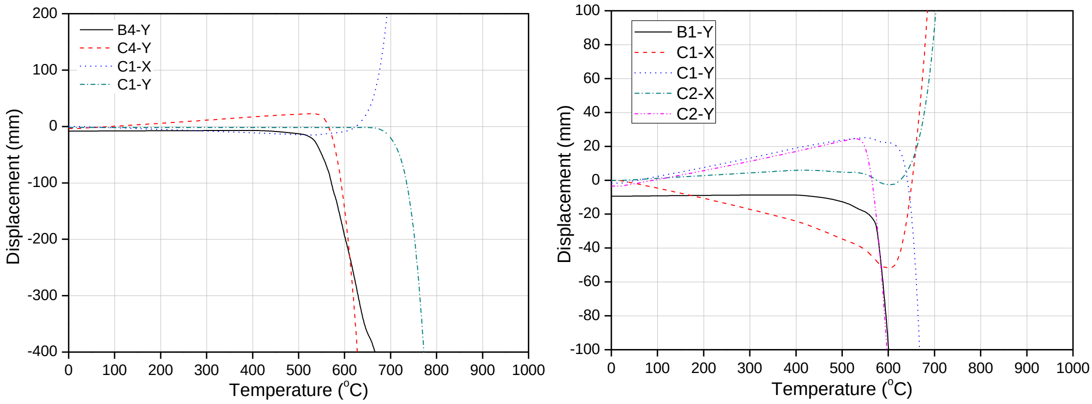
4.1.2 Edge bay fire (Fire 2)

The collapse procedure of the steel frame subjected to Fire 2 is depicted in Figure 4. Similar to Fire 1, the collapse of the frame is triggered by the buckling of the heated columns. The inside heated column C2, sustaining twice as much as load of the edge column C1, buckles first at about 550°C. The buckling of the two heated columns leads to large deflection of the heated bay, causing plastic hinges form in beams at adjacent bay, as shown in Figure 4b. Without the support of the column, the deflection of the beams above the column on the first floor accelerates under large compression forces caused by their restrained thermal expansion. The material degradation at elevated temperature aggravates the deformation of the beams. As the deflection increases, the load-bearing capability of the beams changes from bending to catenary action where tension forces are generated in the beams, pulling the edge column inward after 800 °C as shown in Figure 4c. The lateral drift of the heated column generates great P-Δ effects in it which leads to its large vertical displacements and finally results in the collapse of the frame. Similar to Fire 1, the forces sustained by the heated columns are transferred to the adjacent columns C3 alone with the other columns drift away in a rash.



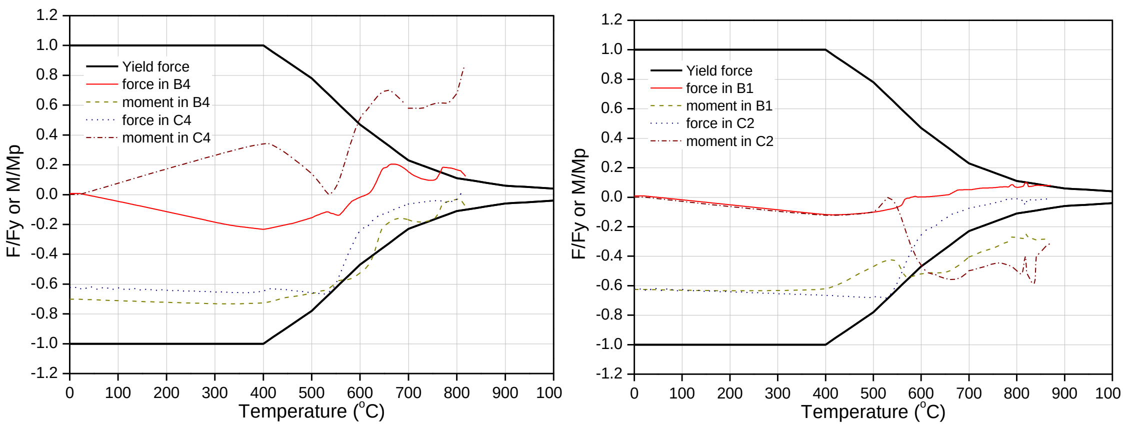
（a）550°C （b）670 °C （c）800 °C

Figure 4 Failure process of the steel frame without bracing under Fire 2



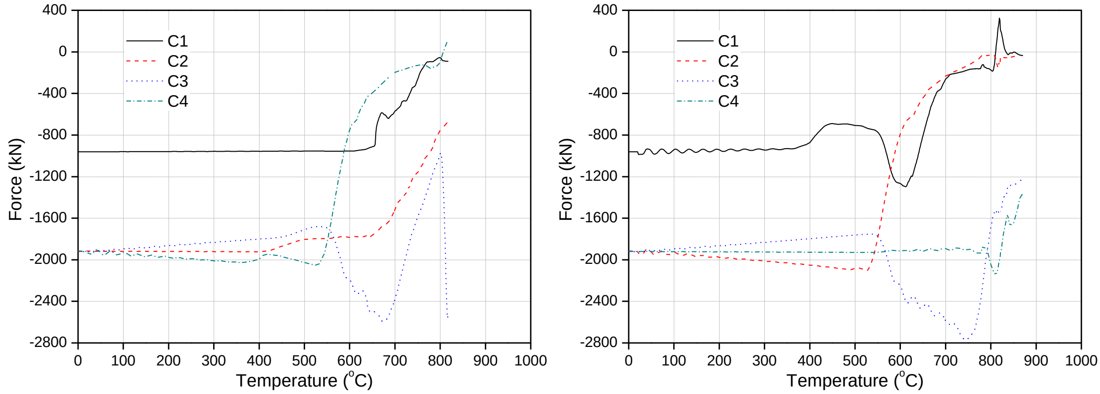

(a) central bay fire (Fire 1) (b)edge bay fire(Fire 2)

Figure 5 Displacements of the heated members in the frame without bracing




(a) central bay fire (Fire 1) (b)edge bay fire(Fire 2)

Figure 6 Forces in the heated members in the frame without bracing



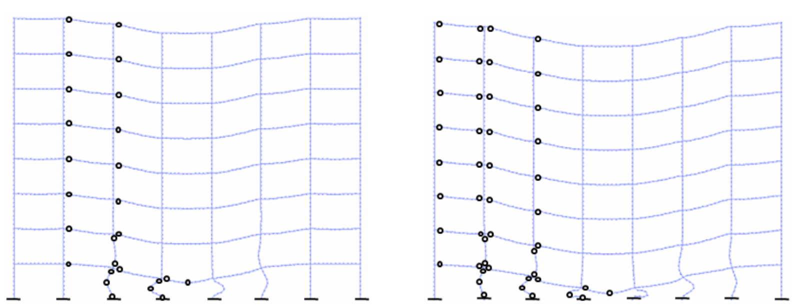

(a) central bay fire (Fire 1) (b) edge bay fire(Fire 2)

Figure 7 Axial forces in the ground floor columns in the frame without bracing

Three stages can be identified for the collapse of steel frames exposed to fire: (1) the buckling of the heated column as the trigger of the collapse; (2) Catenary action, generating in the heated beam under large deflection, pull in the surrounding parts of the frame; (3) the pull-in of columns, leading to the formation of plastic hinges at their ends, cause the global collapse of structures. It can be concluded that the load-redistribution capacity and lateral restraints are two significant factors affecting the robustness of structures against progressive collapse in fire. In the following sections, vertical bracing and hat bracing are to be applied to enhance the structural resistance against progressive collapse.

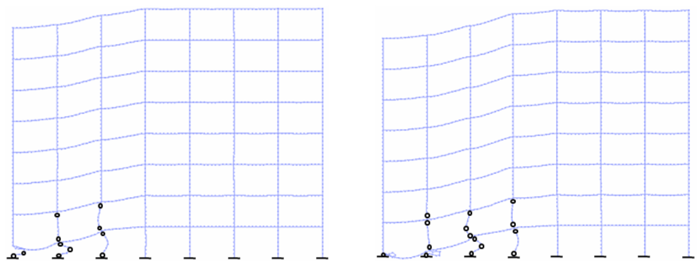
### 4.2 Case 2: Frames with vertical bracing alone

The failure modes of frames subjected to ground floor fires show obvious drift phenomenon caused by the catenary action in beams above the buckled columns. The horizontal drift of frames can be restrained using vertical bracing. The collapse pattern of laterally braced frames after buckling of the heated columns under Fire 1 and Fire 2 are illustrated in Figure 8 and 9, respectively. Similar to the unbraced case, the buckling of heated columns occurs first during the initial heating stage. Instead of horizontal drift, the frame experiences sequential buckling of columns, spreading from the heated bay to the neighboring bays. The loads previously sustained by the heated column are redistributed sequentially in the other columns, as shown in Figure 10.



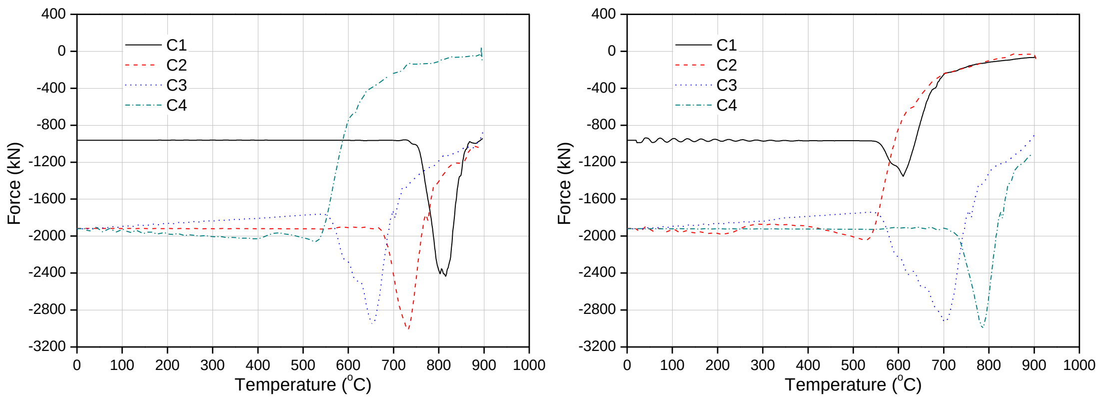
（a）660 °C （b）750 °C

Figure 8 Failure process of the steel frame with vertical bracing under Fire 1



（a）700 °C （b）800 °C

Figure 9 Failure process of the steel frame with vertical bracing under Fire 2




(a) central bay fire (Fire 1) (b) edge bay fire(Fire 2)

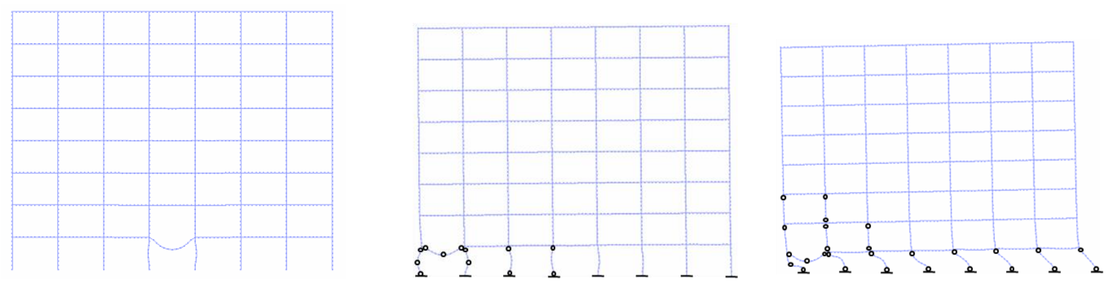
Figure 10 Axial forces in the ground floor columns in the frame with vertical bracing

### 4.3 Case 3: Frames with hat bracing alone

The application of the vertical bracing can resist the sway of the frame but at the expense of sequential buckling of columns in which way the local damage is transferred to the surrounding components, leading to a global collapse. If the columns are strong enough to resist the redistribution load, the frame is safe. Otherwise, the local damage will spread to the adjacent bays and trigger the domino effect, leading to a global failure which is considered to be a more dangerous situation. It is believed that a locally heated frame may progressively fail, eventually generating an overall instability, unless it has sufficient force redistribution capability to continue carrying its vertical loads. If the force redistribution capacity of the frame is sufficient, it may only collapse locally rather than lose overall stability. A hat bracing is an effective mean of distributing vertical loads to columns and its effect is studied for the two fire scenarios in this section.

By applying the hat bracing to the frame in the form of a rigid roof, the steel frame under central bat fire (Fire 1) survives in collapse, accompanied by large deformation in the heated members, with final deformation mode as shown in Figure 11a. The loads previously sustained by the buckled column are shared by all the other ground floor columns simultaneously, as shown in Figure 12a, avoiding the reloading on the individual adjacent column alone as observed in Case 2.

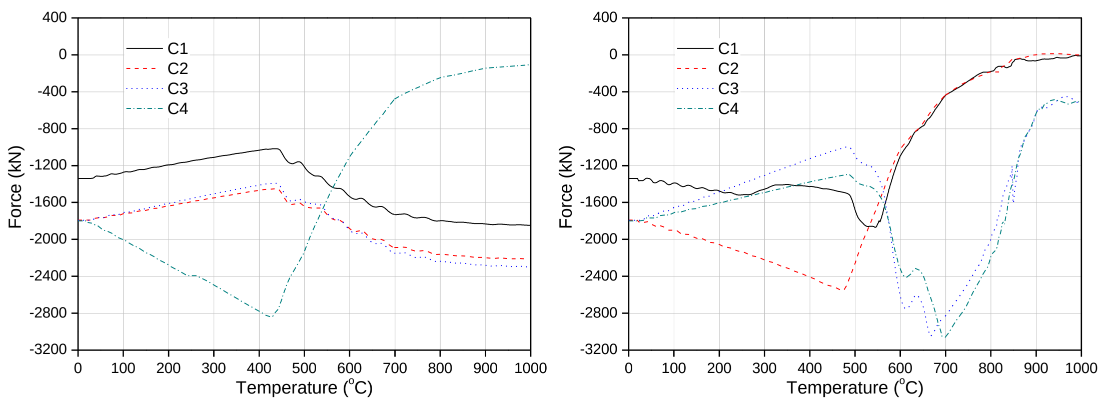
It is not the case for frames under edge bay fire (Fire 2). Figure 11b shows its failure process after the buckling of the heated columns. The reduction in the compression forces in the buckled heated columns are compared to sharp redistribution of loads in the adjacent columns sequentially, leading to their buckling with a short interval, as shown in Figure 12b. Beyond this point plastic hinges start to form at the ends of all the columns on the ground floor as shown in Figure 11b. This may be attributed to the fact that the thermal expansion of the heated beam push the two heated columns outward asymmetrically and the P-d effects resulting from the large UDL generate great additional moment at the bottom of the frame which leads to the premature formation of plastic hinges in them. The development of plastic hinges in the ground floor columns makes the frame a mechanism and drift laterally, eventually leading to the downward collapse of the whole frame.



660 °C    750 °C

(a) Fire 1（stand）    (b) Fire 2

Figure 11 Failure process of the steel frame with hat bracing




(a) central bay fire (Fire 1)    (b) edge bay fire(Fire 2)

Figure 12 Axial forces in the ground floor columns in the frame with hat bracing

**4.4 Case 4: Frames with combined vertical and hat bracing**

From the results presented, it can be seen that a bracing system can enhance capability of a steel frame to resist progressive collapse under fire conditions. A vertical bracing can effectively prevent a localized or global drift of frames but has high potential of sequential buckling of columns, spreading the local damage to the whole frame. In contrast, the application of a hat bracing can increase the load-transfer capacity after local failure, but obvious drift of frames can still induce progressive collapse of steel frames under edge bay fire as shown in Figure 11b. Therefore, it is proposed that a combination of these two bracing systems might provide a greater variety of load-transfer paths and sufficient lateral restraint. Figure 13 shows the final mode of the frame under ground floor edge bay fire where it stand the progressive collapse, although several the ground columns buckle.

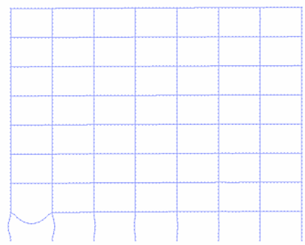
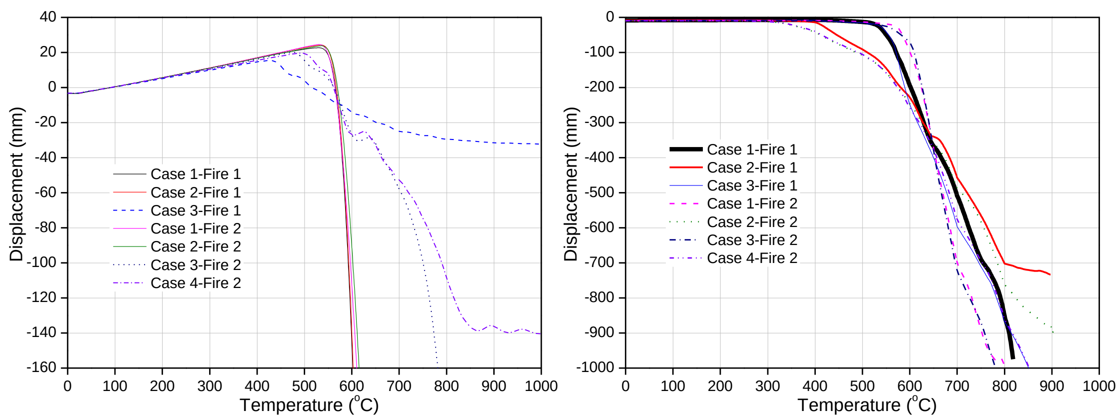


Figure 13 Deformation mode of the steel frame with combined bracing under Fire 2

Based on the four cases studied, it is concluded that the pull-in of columns is one of the main factors contributing to progressive collapse of the frame. The pull-in of columns is initiated by catenary action in the heated beam above the buckled heated columns and aggravated by P-Δ effect. The application of bracing systems increase the redundancy of the structure, and provides alternative load-redistribution paths. The effect of bracing systems on the development of axial forces and moments in the heated members is of great concern and is illustrated in Figures 14-17.

The vertical displacement of the top of the heated column (C4 for Fire 1 and C2 for Fire 2) and midspan deflection of the heated beam (B4 for Fire 1 and B1 for Fire 2) for various bracing systems are illustrated in Figure 14. The deflection of the beam was taken the net value where the components due to columns vertical displacements were subtracted. From Figure 14a, it can be seen that the columns experience upward thermal expansion first followed by downward movement after their buckling. The earliest buckling of the column occurs in the case of the frame with hat bracing under central bay fire (Case 3 in Fire 1) at 400 $^{o}$C, almost 100 $^{o}$C lower than the hat braced case under edge bay fire (Case 3 in Fire 2). This corresponds to the largest compression forces developed in the heated column as shown in Figure 15. A sudden drop of vertical displacement of the top of the column is observed in the frame without bracing and with vertical bracing. The arrangement of a hat bracing alone slow down the reduction with a less stiffer slope which occurs in both fires. Especially, two plateaus occur in the vertical movement of columns in the frame with combined bracing systems under Fire 2. The first occurs at about 600 $^{o}$C, lasting for a period of 50 $^{o}$C while another plateau happens after 850 $^{o}$C when the compression in the column becomes almost zero. This means the frame can stand for the removal of the heated columns without collapse.




(a) heated columns (b)heated beams

Figure 14 Displacements in heated members with various bracing systems

For the midspan deflection of the heated beam as shown in Figure 14b, it can be seen that the lateral bracing advance the large deflection of the heated beam but in a flatter slope, compared with unbraced case. The advancement in the deflection is dominated by the larger compression forces generates due to the stiffer lateral restraints from the bracing, compared with  adjacent frames of its own. At high temperatures after 600 °C, large tension forces generate in the heated beams due to the lateral restraints from vertical bracing which resists the loading through catenary action, leading to a smaller deflection in a relatively mild slope.

It is noted that, from Figure 15, the application of hat bracing increases axial compression forces in the heated column, considering a global restraining against thermal expansion from all the other columns in the hat braced frame. The combination of vertical bracing to the frame with hat bracing under edge bay fire (Fire 2) causes a reduction in the axial forces in the heated column.

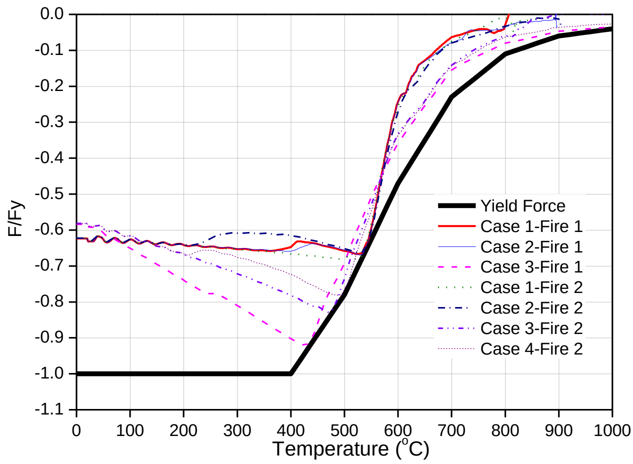



Figure 15 Axial forces in heated columns for various bracing systems

The axial force in the heated beam, as shown in Figure 16, changes from compression to tension. The compressive forces in the heated beam is due to the restrained thermal expansion by the remainder of the frame and its reduction is governed by the material degradation at elevated temperatures. The hat bracing has little effect on the axial forces in beams. The heated beam in the frame under edge bay fire for all bracing cases has a smaller axial force (both in compression and tension), compared with that in the case under central bay fire. This means the frame provides greater lateral restraint on the central bay fire than the edge bay fire. It is interesting to note that the forces in all the heated beams reduce to zero at about 600 °C. It may be explained that at that point plastic hinges form at both ends of the heated column which becomes a mechanism whilst the heated beam above the column loses its vertical support suddenly, experiencing large deflection which accelerate the transformation from compression to tension.

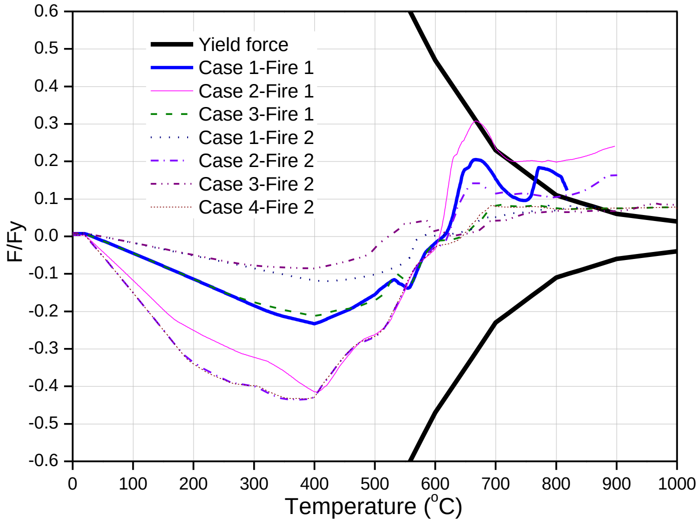



Figure 16 Displacements in heated beams for various bracing systems

As shown in Figure 17, the moment at the end of heated columns increases in the early heating stage and then reduces to zero followed by another increment (a following reduction is seen for frames with hat braces (Case 3)). The first increment in end moments of columns is to resist the increasing rotation of the heated beam and the following reduction is caused by the reducing yield strength of steel material after 400 $^{o}$C. The second increment in the moment is due to the large end rotation of columns which overwhelming the reduction in the material properties at elevated temperature. For the unbraced or laterally braced frame(Case 1 and 2), the heated columns experiences large end rotations in which the moment increases beyond the yield force envelop. This is unrealistic, due to the strain hardening predefined in the steel material. This effect leads to a earlier formation of plastic hinges at the ends of heated columns at about 600 $^{o}$C, which is 200 $^{o}$C lower than the comparable 800 $^{o}$C for frames with hat bracing. In contrast, the end moment of heated columns in the frame with hat bracing is dominated by the material degradation as temperature increases beyond 550 $^{o}$C with a declined variation following the yield force path.

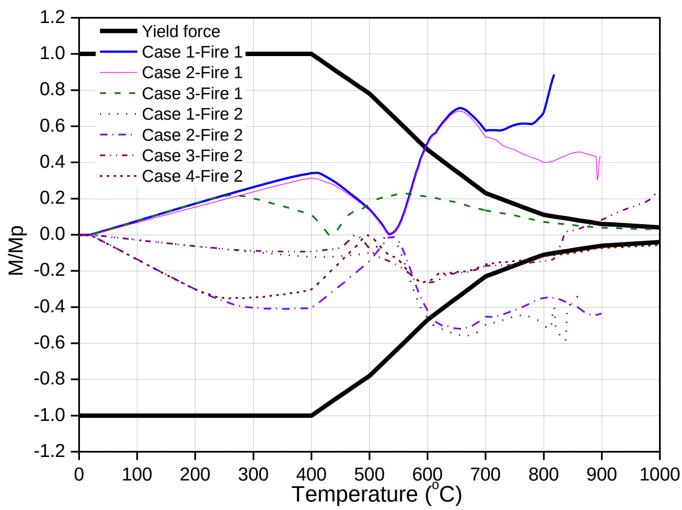



Figure 17 End moments in heated columns for various bracing systems

The variation of the moment at the end of heated beams is depicted in Figure 18 against temperature. In general, the end moment in the heated beam of frames under central bay fire (Fire 1) follows the yield force envelop after a slight increment before 400 $^{o}$C. In contrast to central bay fire, the vertical bracing in the frame under edge bay fire (Fire 2) lead to an obvious increment in the moment due to large compression force generating in the beam, causing large deflections and end rotations. There observed an apparent increment in the moment of beams at about 550 $^{o}$C after a period of reduction. This may be attributed to the sudden drop of the midspan deflection of the heated beam, leading a accelerating formation of plastic hinges forms at both its ends. After 600 $^{o}$C, the end moment in the heated beam is dominated by material degradation at elevated temperatures.

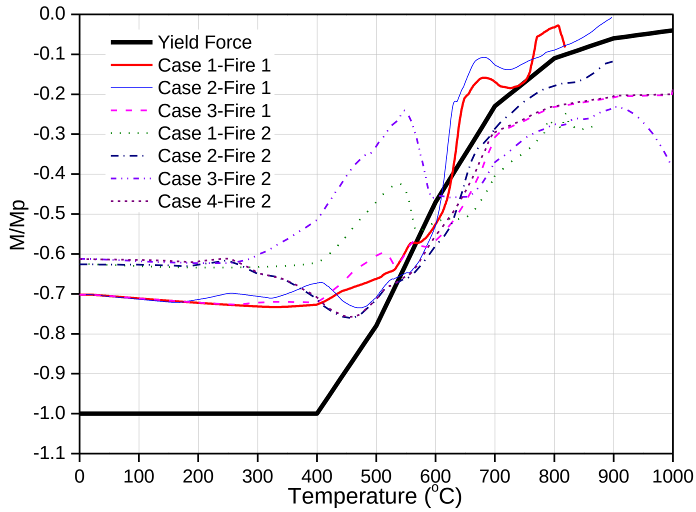



Figure 18 End moments in heated beams for various bracing systems

**4.5 Discussions**

From the parametric studies above, it is found that the hat bracing can effectively prevent the progressive collapse of steel frames under single-compartment fire at the central bay. The frame under edge bay fire can survive the collapse with a combined bracing system consisting of hat and vertical bracings. It had been observed from a number of accidents in a real fire that many elements of the structure may be heated simultaneously, i.e. fire may occur in more than one compartment (multi-compartment fire) which is supposed to be a more severe fire scenario than a single-compartment fire. To this end, the effectiveness of bracing systems against progressive collapse of steel frames under multi-compartment fire is investigated in this section. In this paper, a three-compartment fire horizontally distributed in the frame were used, located on the ground floor and second floor, respectively.

*4.5.1Multi-compartment fires*

Figures 19 and 20 show the failure model of steel frames subjected to central bay multi-compartment fire with hat bracing and combined bracing, respectively. In both cases global downward collapse can be observed. For the frame with hat bracing alone, the beams in the three heated compartment experience larger deflection to the extent that great tension force generate which drives the heated storey to move inward. Different from the failure mode shown in Figure 3 in the form of pull-in of the upper storey of the frame above the heated compartment, the existing of rigid beams across the top storey of the frame, representing the hat bracing, causes the heated floor to move inward followed by the global downward movement of the upper frame. The addition of lateral bracing to the hat braced frame restrain the locally lateral movement of the heated floor, but initiates sequential buckling of all the ground floor columns, finally leading to a global downward collapse.

To consider a less severe fire scenario of a three-compartment fire on the second floor, the frame horizontally restrained by vertical bracing experiences the buckling of columns at edge bays adjacent to the heated bay in both sides of the frame. In comparison to the global collapse as show in Figure 19, the application of hat bracing can effectively prevent the progressive collapse of the frame under second floor multi-compartment fire, although large deflection of heated beams occurs. The survival of the frame in progressive collapse is due to the smaller compression force in the second floor columns when heated, considering less gravity loads sustained than that in columns on the ground floor. The residual load-bearing capacity of heated columns can still provide sufficient vertical supporting on the heated beam which limits the development of the deflection of the heated beams. Hence, the tension forces in the heated beam is not high enough to pull the columns in.

Similar to Figure 20, the combined bracing system is also unable to resist the progressive collapse of frame under multi-compartment fire at the edge bay (as shown in Figure 23). If the same fire occurs on the second floor, the redistribution of loads lead to the buckling of columns at the adjacent bays on the ground floor, causing a sequential buckling of upper columns storey by storey which is shown in Figure 24.

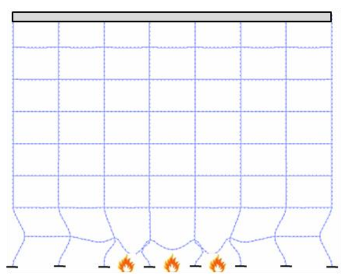


Figure 19 Failure mode of the frame with hat bracing under central multi-compartment fire on the ground floor

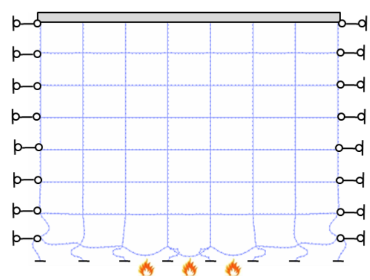


Figure 20 Failure mode of the frame with combined bracing under central multi-compartment fire on the ground floor

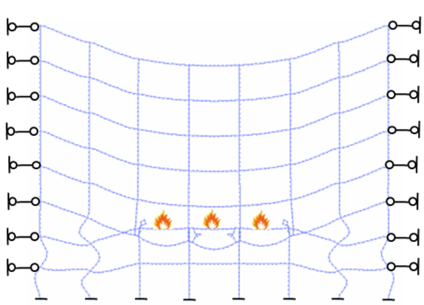


Figure 21 Failure mode of the frame with vertical bracing under central multi-compartment fire on the second floor

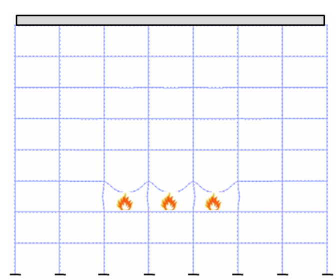


Figure 22 Failure mode of the frame with hat bracing under central multi-compartment fire on the second floor

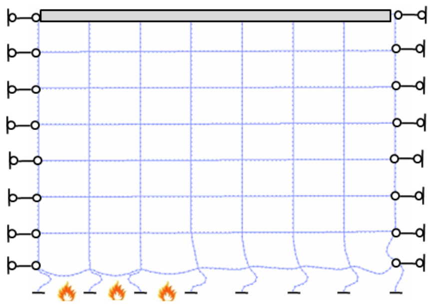


Figure 23 Failure mode of the frame with combined bracing under edge multi-compartment fire on the ground floor

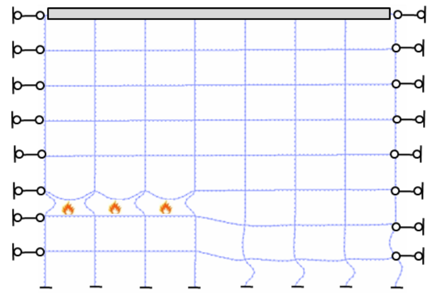


Figure 24 Failure mode of the frame with combined bracing under edge multi-compartment fire on the second floor

*4.5.2 Interior vertical bracing systems*

It is recognized that the application of vertical bracing systems along the edge bays of a steel frame is not common. The effect of a more realistic vertical bracing systems arranged in the interior bays of the frame on its progressive collapse should be discussed. From the collapse patterns of steel frames with combined vertical and hat bracing systems as shown in Figure 13, 20 and 23, the hat bracing dominate the collapse procedure and the failure mode in the form of the buckling of all the bottom columns. Hence, the application of interior vertical bracing systems alone is considered in this study. The corresponding arrangement is shown in Figure 25 where the vertical bracing systems are installed at the second bay and central bay, respectively.

Figure 26 and 27 shows the failure mode of the frame with second bay and central bay vertical bracing under Fire 1, respectively. In comparison with Figure 8 where the vertical bracing system is arranged at the edge bay, it can be seen that, as the movement of the vertical bracing from the edge to the center of the frame, the buckling o the columns occurs within the range between the left and right bracing systems. The rest of the frame outside the bracing systems drifts inside driven by the tension forces developed in beams as well as the large deflection of heated bays due to the column buckling. This indicates that the resistance of steel frames against progressive collapse can benefit from that the arrangement of vertical bracing systems in the inner of the frame, considering that the lateral drift mode may slows down the progressive collapse which is more acceptable or less dangerous than the downward collapse mode.

For a edge bay fire (Fire 2), the collapse mode of the frame with the second bay vertical bracing is shown in Figure 28 (the central bay case is similar). Similar to the case Fire 1, the buckling of columns is confined locally in the heated bay, indicating that the vertical bracing system inside the frame can effectively restrain the spread of the buckling of columns.

Figure 29 and 30 show similar failure mode of frames under edge bay multi-compartment fire with inner vertical bracing systems, respectively. The inner vertical bracing acts as a barrier to cut the spread of local damage to the rest of the structure.

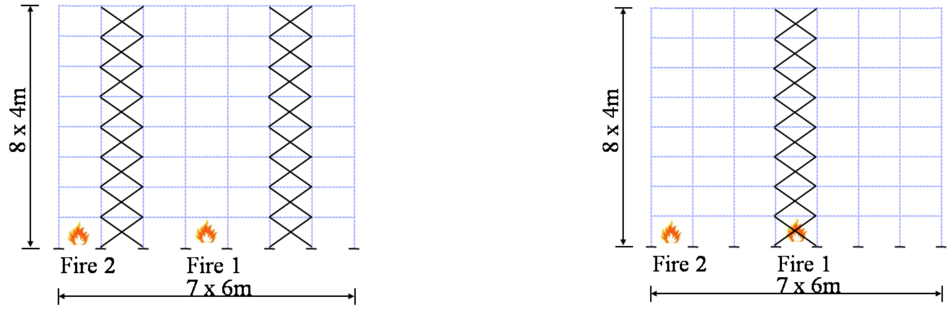



Figure 25 Arrangement of vertical bracing systems inside the frame model

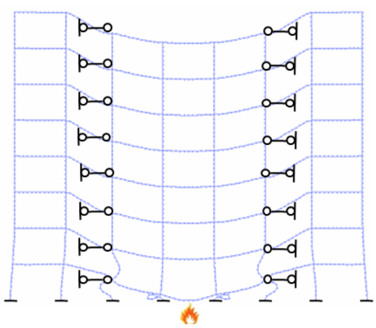


Figure 26 Failure mode of the frame with the second bay vertical bracing under Fire 1

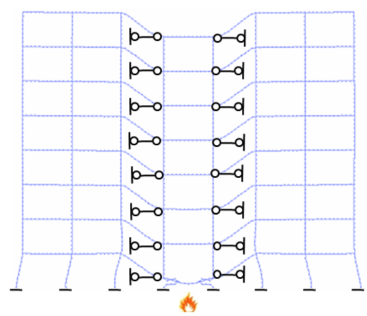


Figure 27 Failure mode of the frame with the central bay vertical bracing under Fire 1

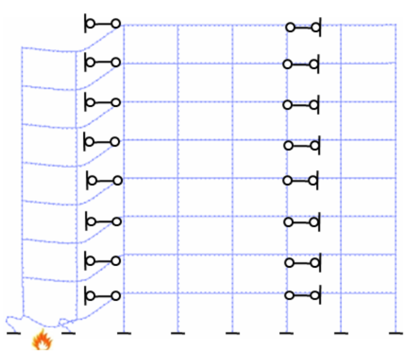


Figure 28 Failure mode of the frame with the second bay vertical bracing under Fire 2

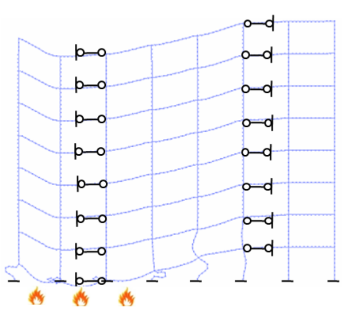


Figure 29 Failure mode of the frame with the second bay vertical bracing under edge bay multi-compartment fire

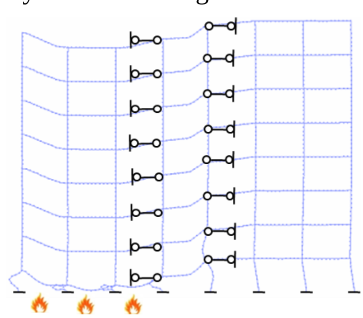


Figure 30 Failure mode of the frame with the central bay vertical bracing under edge bay multi-compartment fire

In summary, a total of four collapse mechanisms of braced steel frames are found in this study listed as follows:

F1. General: Heated bay collapses and upper storey above the heated floor drifts laterally. (as shown in Figure 3,4,26-30)

F2. Global downward collapse: the whole frame collapses downward due to the buckling of columns. (as shown in Figure 20, 21, 23, 24)

F3. Local lateral drift : The collapse is induced by local lateral drift of the heated floor. (as shown in Figure 19)

F4. Global lateral collapse: the whole frame collapses laterally due to the formation of plastic hinges at ends of ground floor columns. (as shown in Figure 11)

The effect of bracing systems on the resistance against progressive collapse of frames subjected to different fire scenarios are concluded in Table 1. It is found that the multi-compartment fire, ground floor fire and edge bay fire are severer than the single-compartment fire, upper floor fire and central bay fire, respectively. The vertical bracing systems can change the collapse mechanisms of frames but unable to prevent the progressive collapse. The hat bracing system can prevent the collapse of frames under single-compartment fire but has nothing to do with the multi-compartment fire, even the combined bracing.

Table 1 Collapse mechanisms of steel frames with various bracing systems under various fire scenarios

| Bracing system | Failure modes | Fire scenario | | |
|---|---|---|---|---|
| | | Horizontal location | Vertical location | No. of bays in fire |
| No | F1 | C/E | G/U | 1/3 |
| Vertical | F2 | C/E | G/U | 1/3 |
| Hat | No collapse | C | G | 1 |
| | | | U | 1/3 |
| | F3 | C | G | 3 |
| | F4 | E | G | 1/3 |
| Combined | No collapse | C/E | G | 1 |
| | | C | U | 1/3 |
| | F2 | C/E | G | 3 |
| | | E | U | 3 |

Note: C-Central; E-Edge; G-Ground; U-Upper.

## 5. CONCLUSIONS

The effectiveness of bracing systems in preventing progressive collapse of steel frames under various fire scenarios have been investigated in this study using the developed OpenSees framework. The conclusions may be drawn as follows:

1. In general, the collapse of steel frames in fire is triggered by the buckling of the heated columns followed by the buckling of adjacent columns at the same storey of the heated column or below. The collapse mode is characterized through collapse of heated bay followed by lateral drift of upper storey of the frame above the heated floor. The thermal expansion of the heated beams at low temperature and catenary action at high temperature have great effects on the collapse mechanisms of steel frames exposed to fire.

2. Using vertical bracing can increase the lateral restraint against local or global drift in the frame through the sequential force-redistribution on adjacent columns. When the bracings are arranged at edge bays of frames, the load-transfer mechanism may spread the local damage to the neighboring bays which will lead to a global downward collapse of steel frames through sequentially buckling the columns on the ground floor. The vertical bracing system can slow down the collapse by sequentially buckling the columns through load-redistribution in them one by one. However, its application alone in the steel frame under fire conditions is unsafe.

3. For a more realistic arrangement of the vertical bracing inside the frame, the bracing system can effectively restrain the horizontal development of the buckling of columns and acts as a barrier to cut down the spread of local damage to the rest of structures. It is recommended that the vertical bracing system be arranged inside the frame to mitigate the fire-induced progressive collapse of structures

4. Alternatively, the hat bracing can effectively enhance the resistance of steel frames against progressive collapse. This is done through uniform force-redistribution in columns. However, local lateral drift of the heated floor occurs in the hat braced frame under multi-compartment fire on the ground floor, which leads to a global collapse of the frame.

5. The fire-induced progressive collapse of steel frames can be prevented using a combined vertical and hat bracing system which is recommended in the practical design of structures in fire.

6. The load-redistribution capacity and lateral restraints provided by the frame itself or external supports are two significant factors which affect the robustness of structures against progressive collapse exposed to fire. In addition, the pull-in of columns is one of the main concerns contributing to progressive collapse of the frame.

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