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Progressive Collapse Mechanisms of Steel Frames Exposed to Fire

Jian Jiang[1], Guo-Qiang Li[2,]* and Asif Usmani[3]

[1]College of Civil Engineering, Tongji University, Shanghai 200092, China
[2]State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai 200092, China
[3] School of Engineering, the University of Edinburgh, Edinburgh EH9 3JF, United Kingdom

Abstract: OpenSees is an open-source object-oriented software framework developed at UC Berekeley. The OpenSees framework has been recently extended to deal with structural behavior under fire conditions. This paper summaries the key work done for this extension and focuses on the application of the developed OpenSees to study the fire-induced progressive collapse mechanisms of steel structures. The implicit dynamic analysis method (Newmark method) is applied and the influences of the load ratios, beam sizes and fire scenarios on the collapse behavior of frames are investigated. Single-compartment fire scenarios in the central bay and edge bay are considered, respectively. A total of four collapse mechanisms of steel frames are proposed by varying the three influencing factors. Most of the collapse of steel frames is 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 most common collapse mode of steel frames are in the form of lateral drift of frames above the heated floor together with downward collapse of frames along the heated bay. As the load increases, the collapse behavior of structures is dominated by a downward collapse of the whole frame with little sign of the upper frame drift. The collapse modes of steel frames with strong and weak beams are column failure mechanism and beam failure mechanism, respectively. The former mechanism is due to the buckling of the columns below the heated floor represented by a global collapse of the frame and the latter is initiated by the premature development of plastic hinges at the ends of beams denoted by an obvious lateral drift of the heated floor. Generally, the edge bay fire is more prone to induce the collapse of structures than the central bay fire. It is 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.

Key words: progressive collapse, fire-induced, collapse mechanism, steel frame, load ratio, beam size, fire scenario.

1. INTRODUCTION

The traditional approach for evaluating the fire resistance of structures (based on prescriptive building codes) is by testing individual structural members under a standard fire, where the member capacity is associated with a limiting temperature. This approach does not consider natural fire scenarios and the enormous associated uncertainties. Furthermore, the behavior of structural members in isolation entirely ignores the structural interactions a member would experience as part of the whole structure. The unscientific nature of prescriptive approaches has led to gradual and accelerating adoption of performance-based design approaches, characterized by much greater reliance on scientific understanding and numerical modeling technologies.

Since the Broadgate Phase 8 fire in London and the subsequent Cardington fire tests, researchers have began

* Corresponding author. Email address: gqli@tongji.edu.cn; Fax: +86-21-6598-3431; Tel: +86-21-6598-2975.

to investigate and understand the behavior of whole composite steel-framed structures under fire conditions. Especially since the collapse of the Word Trade Tower (WTC) under terrorist attack on September 11, 2001, there has been considerable interest in understanding the fire-induced progressive collapse of tall buildings. The 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 7 2005). 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 the mitigation of disproportionate collapse can be found in various design codes (GSA 2003; ASCE 7 2005; DoD 2010). ASCE 7 (2005) proposes two general approaches for reducing the possibility of progressive collapse: Direct Design and Indirect Design. Direct Design approaches include the Alternate Path (PA) method which requires that the structure be capable of bridging over a missing structural element in the event of a localized damage and the Specific Local Resistance (SLR) method which requires the building provide sufficient strength to resist a specific load. With Indirect Design, the structural resistance of the 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 Forces (TF) 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.

Largely driven by the need to improve design approaches, intensive researches on structural robustness to resist progressive collapse have been undertaken for the past decade. Quintiere *et al.* (2002) proposed that the compression buckling of the truss rod was the main trigger to the further collapse of the WTC towers. Huang (2002) studied the progressive collapse of steel frames in fire using FEMFAN, a finite element program developed at the Nanyang Technological University. An isolated beam/column model was proposed and the influences of boundary conditions, load levels, member slenderness ratios and cross-section thermal gradients on the progressive collapse of heated members were studied. It was found that the oversimplified boundary conditions adopted by the current design codes led to an unsafe critical temperature of steel members. The creep of steel may dominate the behavior of heated steel members beyond 400ºC. Usmani *et al.* (2003) carried out a 2D numerical modeling of the WTC tower subjected to fire alone regardless of the damage caused by the terrorist attack. The analysis showed that the collapse was initiated by a stability mechanism resulting from geometry changes in the structure caused by thermal expansion effects and indicated that the collapse was due to a major fire event. 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. Takagi and Deierlein (2007) investigated the collapse performance of steel-framed buildings under fire conditions. The results indicated that the variability in the high-temperature yield strength of steel is the most significant factor in the collapse probability assessment. 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. 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.

To enable the investigation of structures against fire-induced progressive collapse accurately and efficiently, however, considerable further developments of modeling technologies are required. 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).

Taking 3D thermomechanical analysis of structures subjected to random fires in ABAQUS for example, a heat transfer analysis must be carried out on a mesh of continuum solid elements to establish the temperature evolution on sufficient points in the structure. The same mesh can of course be used for simulating the mechanical response. This however is a very computational expensive approach and also not very accurate compared to the much more accurate structural elements (beam-column or frame). However if an analyst chooses structural elements, currently ABAQUS only allows five temperature points on the cross-section of a 3D beam-column element. This makes an accurate analysis of the heat transfer meaningless as the temperature resolution obtained is not usable in a structural frame model. The authors have found this to be a severe limitation in their use of ABAQUS. This is another important reason for the search for a more suitable software platform for modeling structures in fire. OpenSees fitted the bill perfectly and offered excellent capabilities of simulating structural response to earthquakes offering the possibility of a multi-hazard simulation capability, e.g. fire following an earthquake.

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.

This paper presents the utilization of OpenSees to investigate the progressive collapse mechanisms of steel frames under fire conditions. The detailed introduction of the extension of OpenSees for thermomechanical analysis of structures by authors can be found in references (Jiang 2012; Jiang *et al*. 2013). Parametric studies were carried out by performing implicit dynamic analysis (Newmark method) in OpenSees to investigate the influence of the load levels, beam strength and fire scenarios on the collapse modes of steel frames exposed to single-compartment fire. Two fire scenarios were used: the central bay fire and edge bay fire. Various collapse mechanisms of structures were found by varying the three influencing factors.

2. EXTENSION OF OPENSEES FOR THERMOMECHANICAL ANALYSIS

The OpenSees framework has currently been developed by the research team at the University of Edinburgh for thermomechanical analysis of structures. 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. 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. 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). The class hierarchy of new classes added in OpenSees can refer to references (Jiang *et al.* 2013a). 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 which will be validated in the following sections.

3. VALIDATION OF THE DEVELOPED OPENSEES

The static analyses of structures in fire using developed OpenSees have been extensively verified and validated by the 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 (β = 0.8 and γ = 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 will be demonstrated in the following sections.

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 "β" and "γ" 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).

3.1. Shallow Toggle Frame Tested at Ambient Temperature

A shallow two-bar toggle frame was proposed by Williams (1964). The elastic modulus of the members is 71 kN/mm^2 and they are of rectangular cross-section with a width of 19.13mm and a depth of 6.17 mm, as shown in Figure 1. The two ends of the frame are fully fixed. Figure 2 shows the comparison of the load-displacement relationships of the test results and numerical results from both OpenSees and ABAQUS. The Arch-length method was used for the static analysis in OpenSees. It is evident that the developed thermomechanical model in OpenSees works well with the existing dynamic procedure and can handle the stability problems.

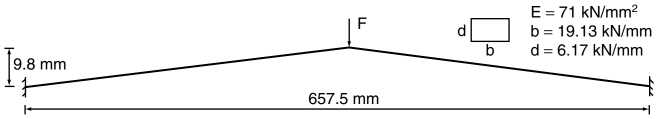


Figure 1. William toggle frame (William 1964)

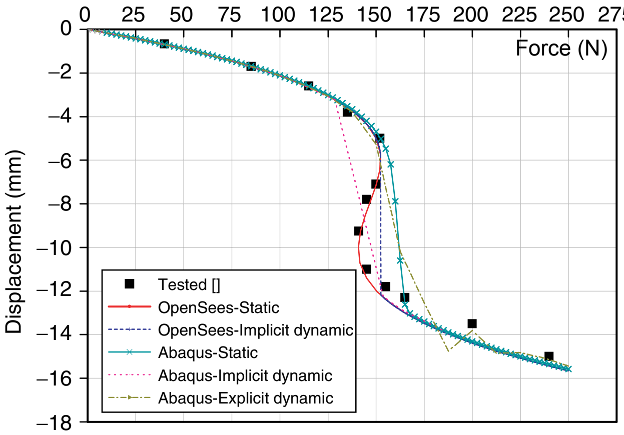


Figure 2. Comparison of predicted numerical results and test results

3.2. Steel Frames Tested at Elevated Temperature

A series of tests on plane steel frames at elevated temperatures were performed in Germany (Rubert and Schaumann 1986). A schematic diagram of two steel frames EHR3 and ZSR1 are shown in Figure 3. The braced two-bar frame (HER3) was subjected to a uniform temperature rise and only one bay of the two-portal frames (ZSR1) was uniformly heated. All

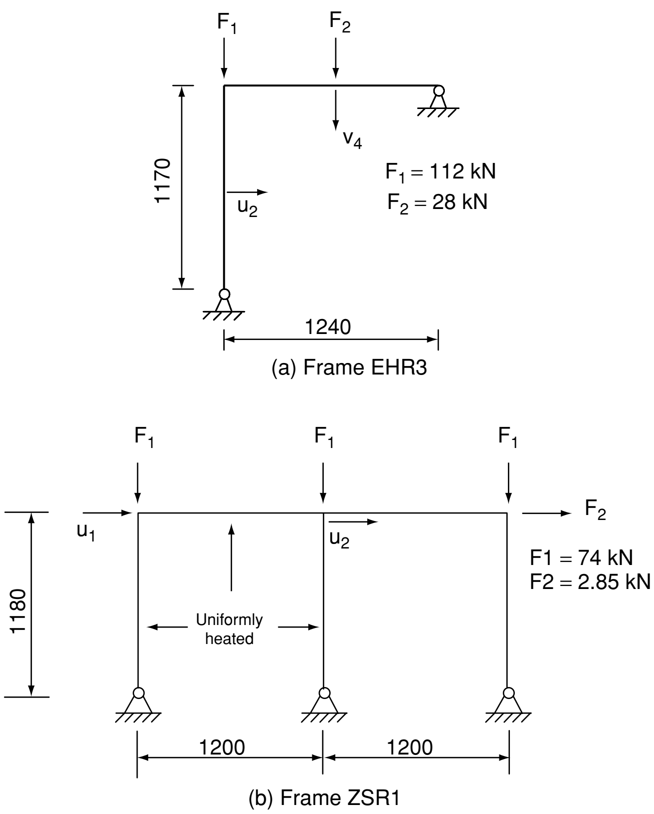


Figure 3. Schematic of the tested steel frames (mm)

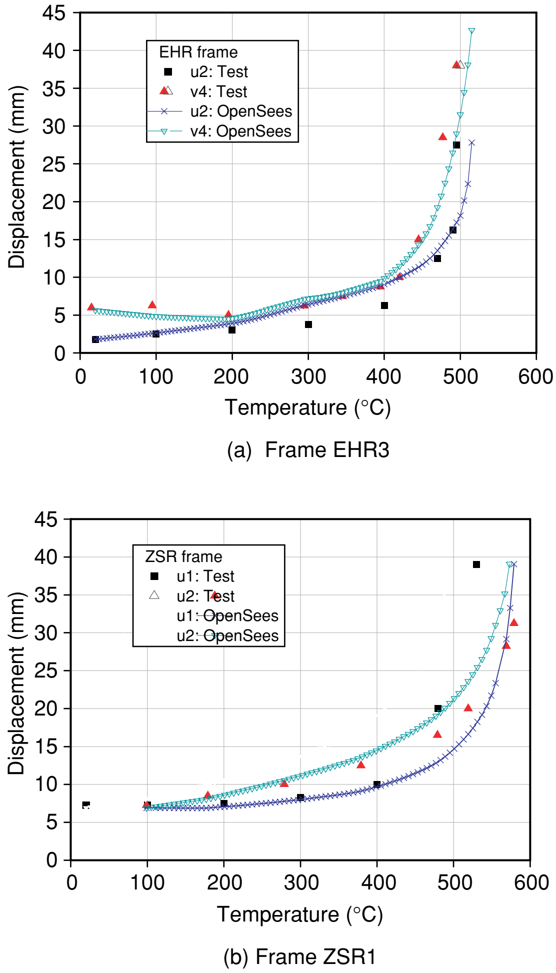


Figure 4. Comparison between predicted and test deflection results

structural elements were made of IPE80 I-shaped steel. The yield stresses and modulus of elasticity are 382 N/mm^2 and 210 N/mm^2 at ambient temperature for EHR3 and 355 N/mm^2 and 210 N/mm^2 for ZSR1, respectively. Comparisons between the predicted deflections by OpenSees and the test results illustrated in Figure 4 show satisfactory agreement.

4. PROGRESSIVE COLLAPSE ANALYSIS OF STEEL FRAMES IN FIRE

4.1. Details of Steel Frames Studied

The main objective of this paper is to investigate the progressive collapse mechanisms of steel-framed structures under different fire scenarios. Hence, considering both computational efficiency and structural representation, a 2D steel frame of five bays with 6 m span and eight storey with 4 m storey height is modeled in this study, as shown in Figure 5. Sun *et al.* (2012a) studied the collapse mechanisms of a steel frame where only the columns were heated. In this paper, both the beam and columns in the compartment exposed to fire were heated and the adjacent compartments were left at

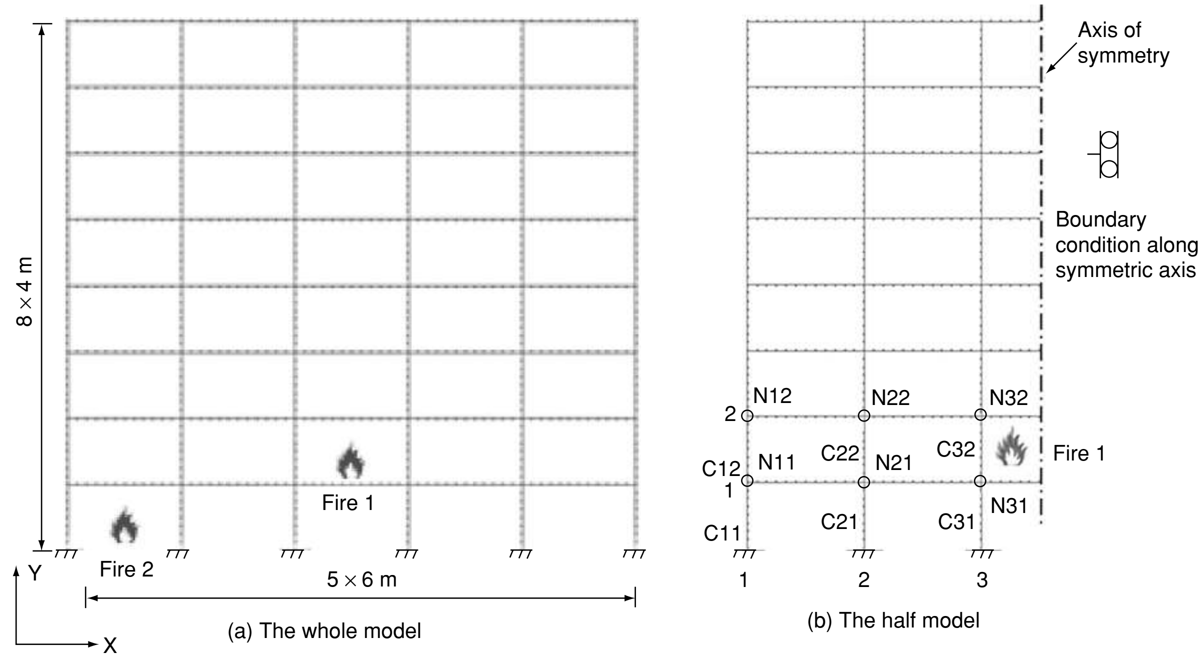


Figure 5. Schematic of the steel frame exposed to fire modelled in OpenSees

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. Figure 5(a) shows the two fire scenarios used in this study. Fire case 1 is a fire occurring in the central bay on the second floor. Fire case 2 represents a fire in an edge bay on the ground floor. For Fire case 1, only half of the frame was analyzed due to the symmetry as shown in Figure 5(b). 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). For each fire scenario, a series of cases, varying in load levels and beam strength, have been conducted to deeply understand the collapse mechanisms of frames. The deformation and resisting forces in the beams and columns on the ground two floors were output in a scaling ratio of 1:1 to explain the collapse mechanisms of the steel frame. The locations of these nodes and columns are labeled in Figure 5(b) where N and C denote node and column, respective. The first and second subscript numbers represent the corresponding locations in the bay and storey, respectively.

Respectively, 8 and 12 elements were employed for beams and columns. All the columns are taken as UC 254 × 254 × 89 in all the analyses in this paper. Temperature dependent bilinear plastic material 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).

4.2. Central Bay Fire (Fire Case 1)

4.2.1. Influence of vertical loadings

Two uniformly distributed loads (UDL) (50 kN/m and 65 kN/m) are applied vertically on all the beams of the steel frame. In this case, the beams is taken as UB 305 × 165 × 40. Figure 6 shows the collapse modes of steel frames under the two UDLs. It is found that all the collapses of frames are initiated by the buckling of the heated column followed by the sequent bucking of the adjacent columns. For the case of the frame with UDL of 50 kN/m, there are obvious horizontal movements of the frame above the heated storey before the buckling of other columns as shown in Figure 6(a). As the UDL increases, the column buckling advances and the whole frame collapse downward without horizontal drift of upper frame when UDL is 65 kN/m as shown in Figure 6(b). The detailed process of each collapse mechanism is presented in the following sections in comparison with plastic hinges formed in the frame.

(1) Case 1 Steel frames in Fire case 1 under UDL of 50 kN/m

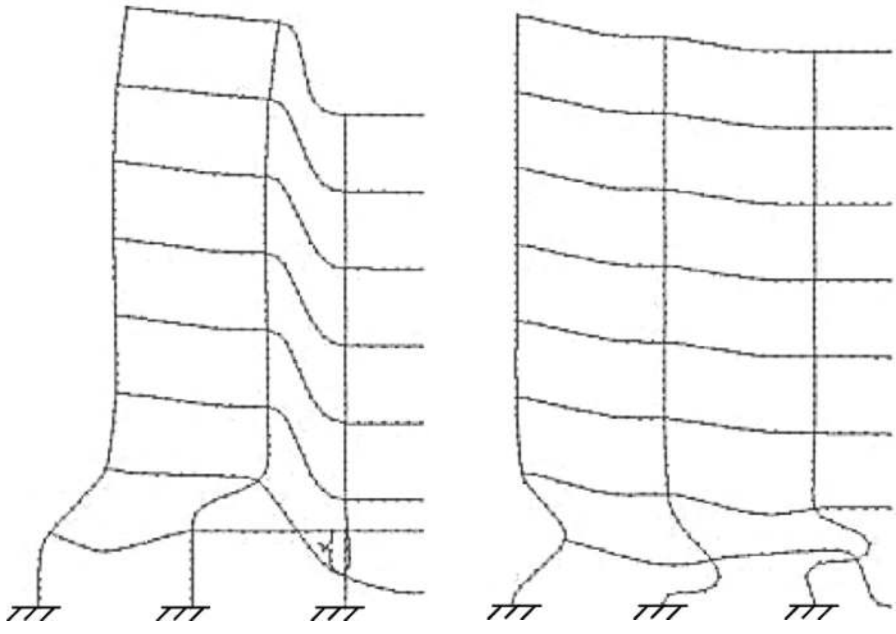


Figure 6. Collapse mechanisms of steel frames in Fire 1 under different loads

Figure 7 shows the collapse procedure of the steel frame under the UDL of 50 kN/m. At the early heating stage the heated compartment is pushed up and left by the thermal expansion of the heated columns and beam as shown in Figure 7(a). Additional compression force is generated in the heated columns and beam due to the

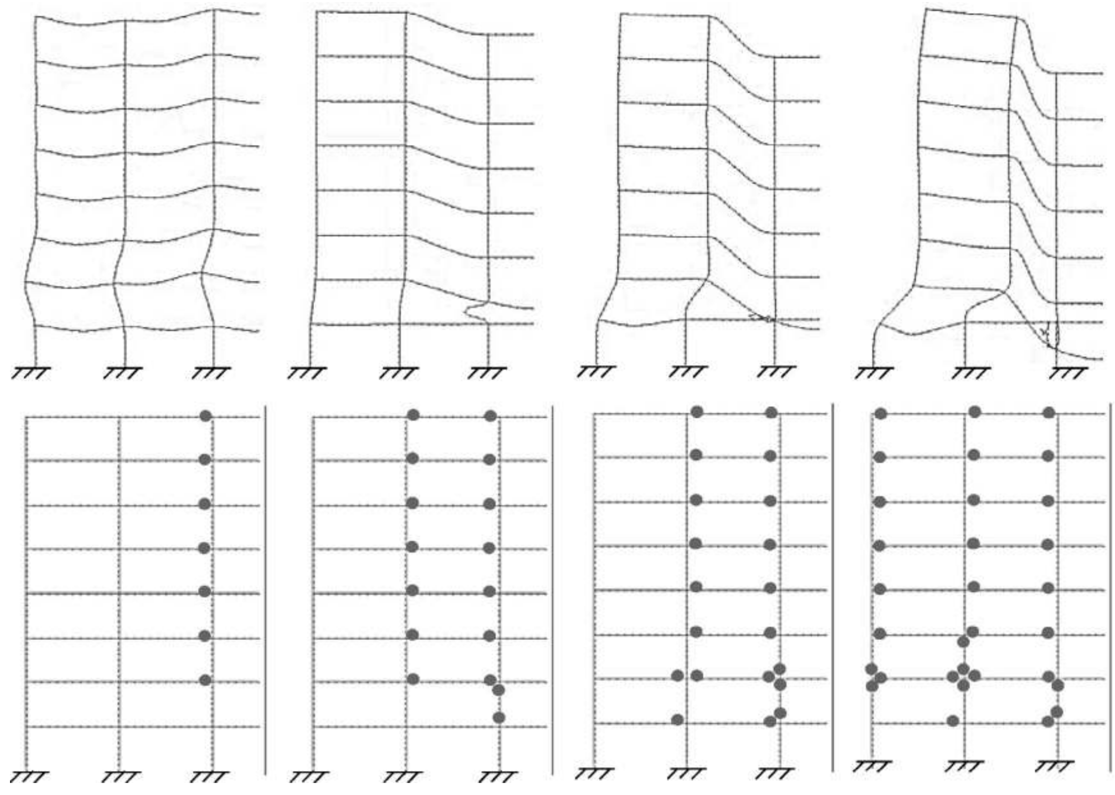


Figure 7. Collapse process of the frame in Fire 1 under UDL of 50 kN/m

restrained thermal expansion by the surrounding cool structure. Meanwhile, the material properties of steel are degraded as temperature rises. Once the compression in the column exceeds its critical buckling load (given by Euler's Formula $F_{cr} = EI\pi^2/L^2_{ef}$), the column buckles at around 530ºC as shown in Figure 7(b). After that, the load sustained by this buckled column has to be transferred to the adjacent columns. The redistribution of the load aggravates the deformation of the adjacent frame where tension force can be generated in the beams just above the buckled column, i.e. catenary action, due to their large deflection. The tension force in the beam then pulls in the upper frame when the temperature reaches 650 ºC as shown in Figure 7(c). Finally, subsequent buckling of columns on the second floor of the frame occurs, leading to the collapse of the whole structure. During the collapse of the frame, the columns on the ground floor keep stable. Figure 7 also illustrates the corresponding development of plastic hinges in the frame. As temperature increases, the plastic hinges in the beams and columns propagates from the middle bay of the fame to the edge bay while the frame is pulled inwards and fell down. This is because of the sequent buckling of the columns and corresponding load redistribution. Figure 8 and 9 show the displacements and axial forces in the columns on the ground two floors, respectively, where the development of the four collapse stages can be seen clearly. For the Stage and Stage, the upward thermal expansion of the heated column C32 continues with increasing compression force developed in the column until about 530ºC, and then starts to buckle downward. The previous loadings in the column C32 are then redistributed in the column

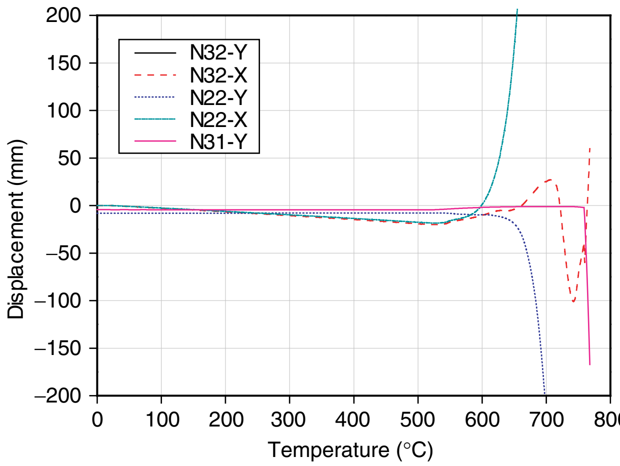


Figure 8. Displacements of the top of the columns under UDL of 50 kN/m

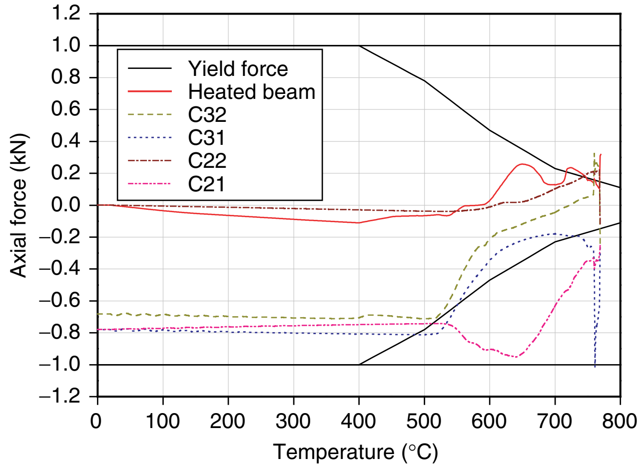


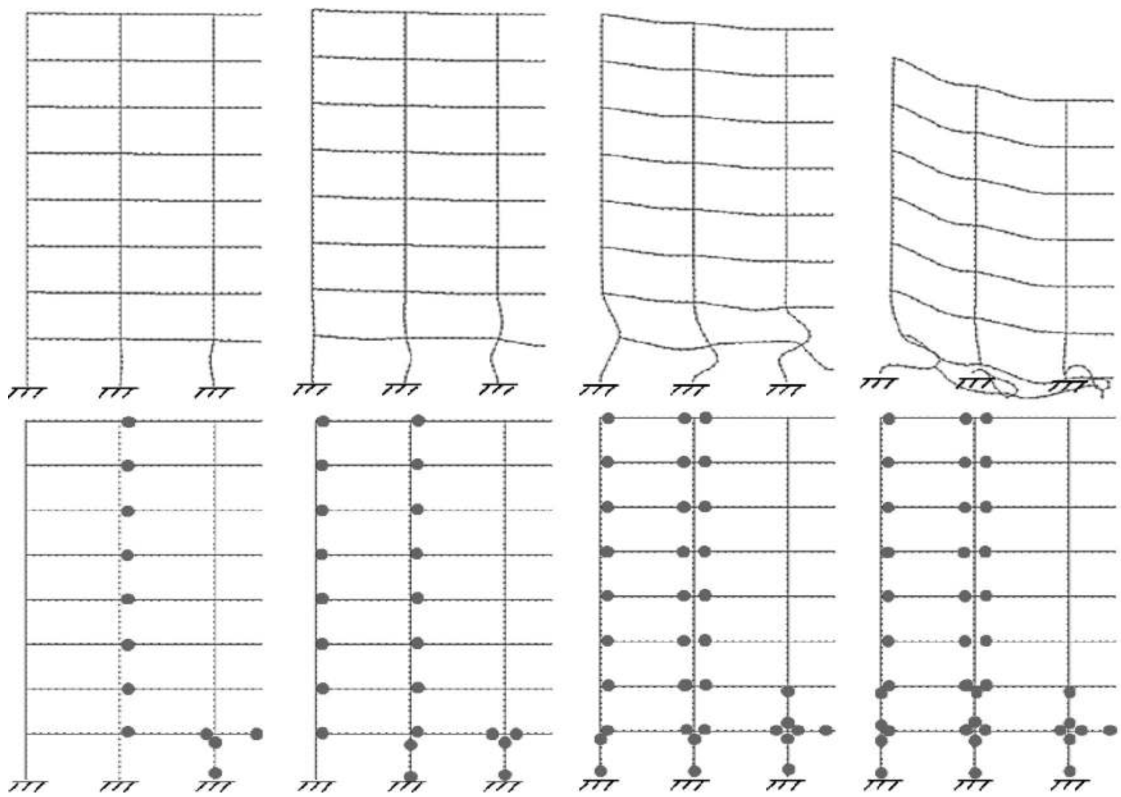
Figure 9. Axial forces in the columns under UDL of 50 kN/m

C21 until about 650ºC. After that, the buckling of the column C31 precipitates the collapse of the whole frame after about 750ºC.

(2) Case 2 Steel frames in Fire case 1 under UDL of 65 kN/m

The collapse procedure of the steel frame under the UDL of 65 kN/m is shown in Figure 10. The nodal displacements and column forces against temperature are shown in Figure 11 and 12, respectively. Different from the Case 1, the failure mode in this case is the downward collapse of the whole frame. The column C31 just below the heated beam on the ground floor buckles first as early as 100 ºC followed by the buckling of its adjacent column C21. After the buckling of the columns on the ground floor, the subsequent buckling of columns on the second heated floor of the frame occurs. There is no obvious load redistribution effect in columns as shown in Figure 12. The frame starts to collapse at a very early heating stage about 250ºC. The plastic hinges first form in the second bay of the frame due to the premature buckling of the columns along it. As the temperature increases, the distribution of plastic hinges in the beams develops to the edge bay. It is noted that there are no plastic hinges formed in the central bay of the frame during the heating. This is because that the beams at the central bay have small rotation due to their falling down together with adjacent bays after the buckling of the bottom columns.

Figure 13 and 14 show the comparison of the displacement and axial force of heated members of the frame under different levels of UDLs, respectively. The heated beam at the central bay on the second floor of the frame with UDL of 65 kN/m experiences large compression force after 200ºC which is due to the pull-in of the first floor where large compression generated



(a) Stage I (100°C) (b) Stage II (150°C) (b) Stage III (185°C) (b) Stage IV (250°C)

Figure 10. Collapse process of the steel frame in Fire 1 under UDL of 65 kN/m

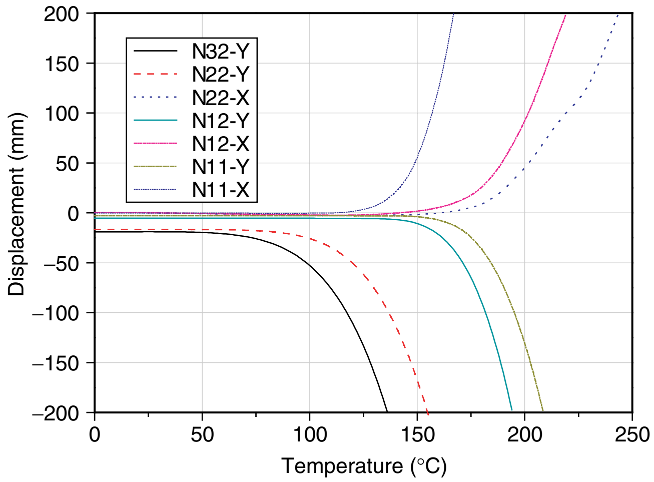


Figure 11. Displacements of the key nodes of the frame under UDL of 65 kN/m

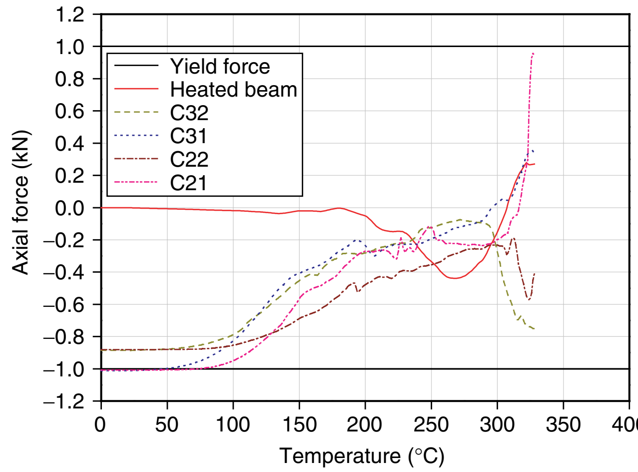


Figure 12. Axial forces in the columns of the frame under UDL of 65 kN/m

in the beams on the second floor as shown in Figure 10(c).

4.2.2. Influence of beam sections

Previous sections have presented collapse mechanisms of the steel frame exposed to fire varying with load levels applied on the structure. In this section, the influence of the strength of beams on the collapse behavior of the frame is studied. Three types of beam sections (UB610 × 229 × 125, UB305 × 165 × 40 and UB203 × 102 × 23) were chosen to represent the strong, medium and weak beam. The UDL is taken as 50 kN/m. Figure 15 shows the collapse modes of steel frames with various beam sizes. The collapse mode of the frame with strong beams of UB 610 × 229 × 125 is so-called column

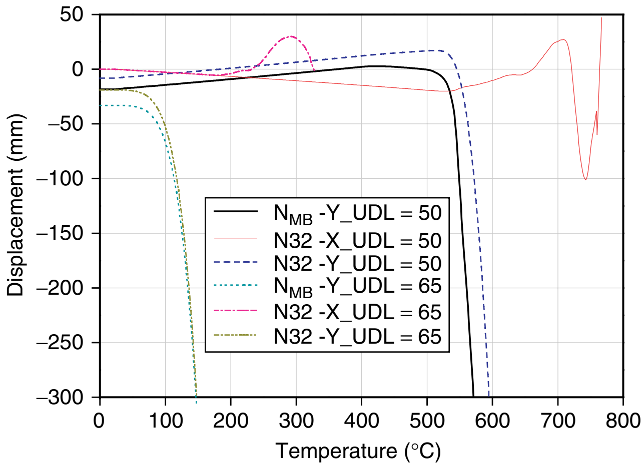


Figure 13. Comparison of displacement at the top of the columns under different UDL

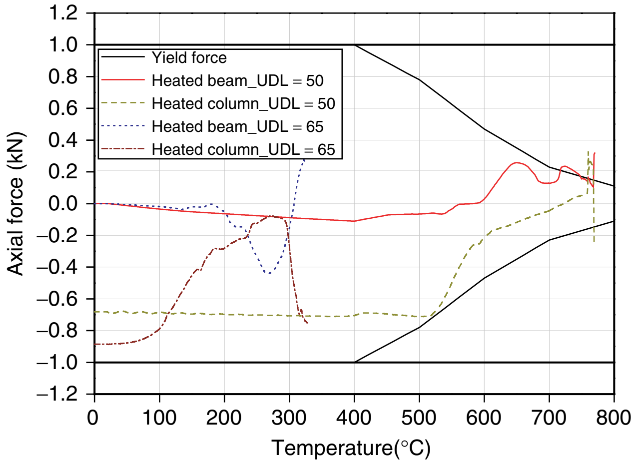
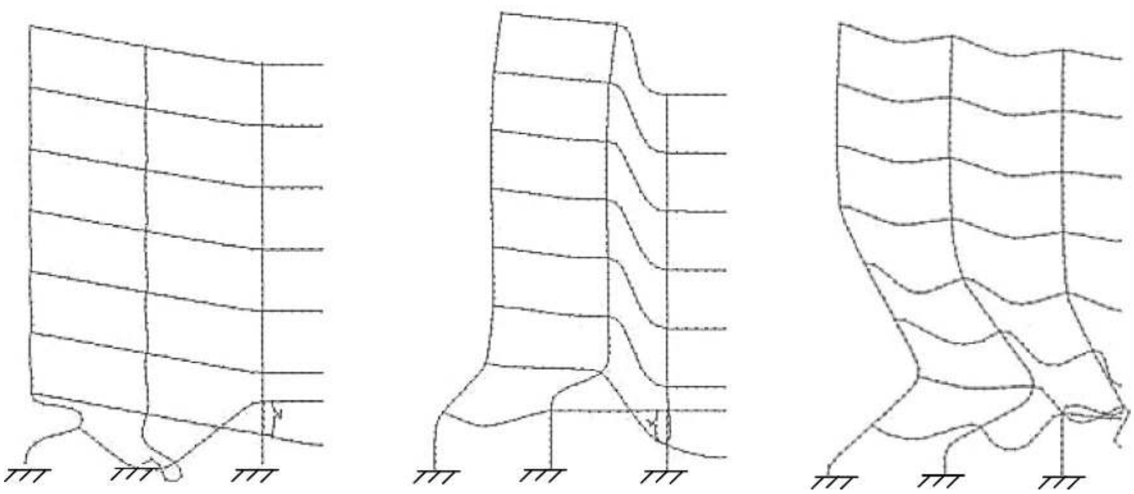


Figure 14. Comparison of the axial forces in heated members under different UDL

failure mechanism represented by the downward collapse of the whole structure. As for the frame with weak beams of UB203 × 102 × 23, the collapse is caused by the horizontal movement of the second storey driven by the large deflection of the beams on this storey. It can be named beam failure mechanism. The procedure of each collapse mechanism is presented in the following sections in comparison with plastic hinges formed in the frame.

(1) Case 1 Steel frames in Fire case 1 with UB 610 × 229 × 125

Figure 16 shows the column failure mechanism of the steel frame with UB610 × 229 × 125. Figure 17 and 18 show the nodal displacements and axial forces in the bottom columns, respectively. It can be seen that the beam is strong enough that only the buckling of columns happens, first in the heated column C32 at about 470 ºC and then in the column C21 at 520 ºC. From the development of plastic hinges shown in Figure 16, it is clear that the failure spreads to the adjacent spans after the buckling of the heated columns. The load sustained by the buckled heated column is first transferred to the column C21 at the adjacent bay on the ground floor where plastic hinges form at its ends at about 520ºC. After that the column just above it buckles with development of plastic hinges at its ends. The same procedure starts to develop at the edge bay at 600ºC. It is noted that there is a short plateau for the displacements of the top of the columns at elevated temperature. This represents the load redistribution process from the buckled columns to the rest of the structure which can be seen more clearly in Figure 18. The additional load was first sustained by the columns near the heated column along the second bay and then by the edge columns.



(a) UB610 × 229 × 125 (920°C) (b) UB305 × 165 × 40 (750°C) (c) UB205 × 102 × 23 (690°C)

Figure 15. Collapse mechanisms of steel frames in Fire 1 with different beam sections

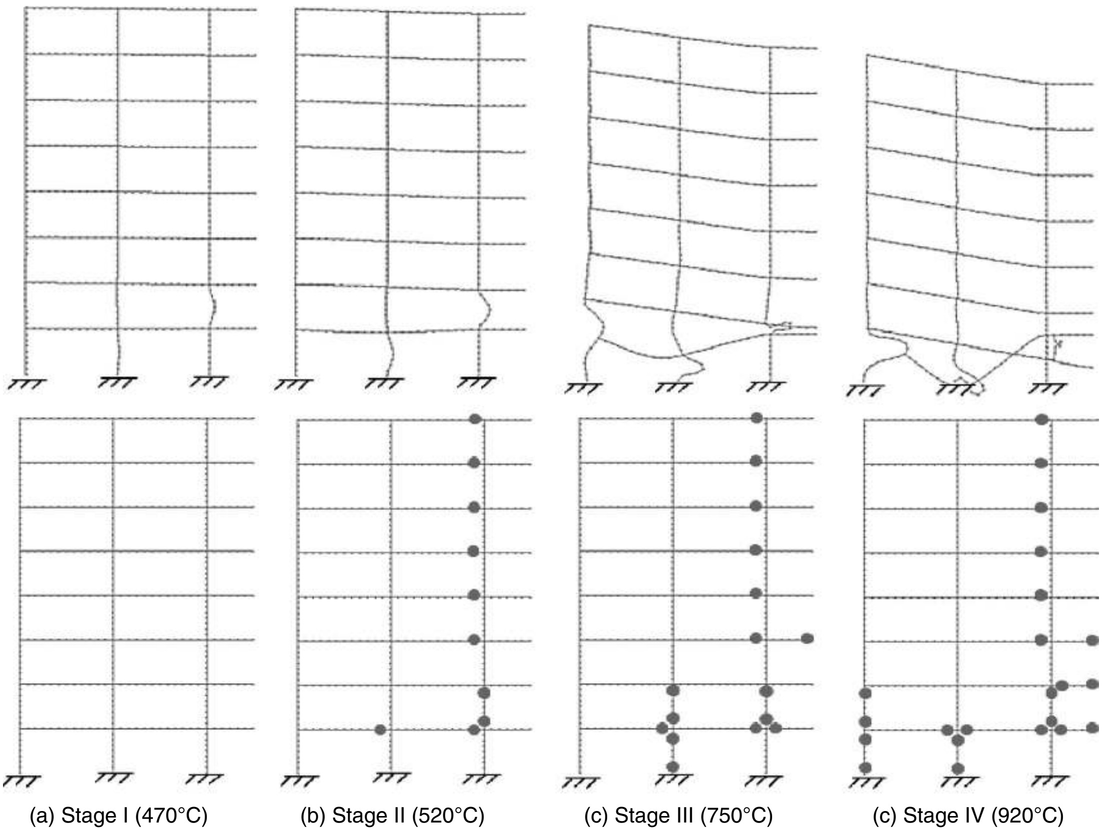


Figure 16. Collapse process of the steel frame in Fire 1 with UB610 × 229 × 125

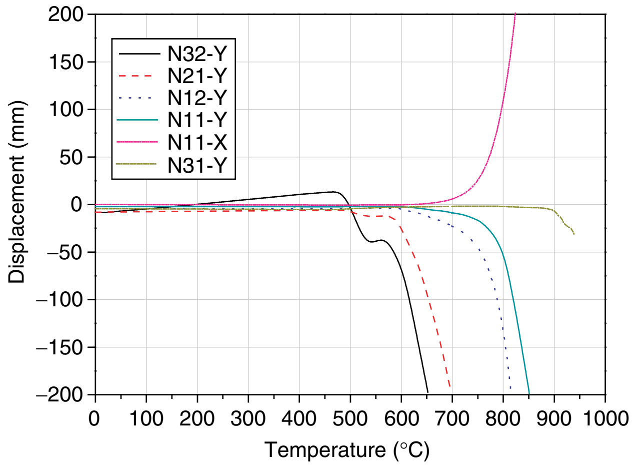


Figure 17. Displacements of the key nodes of the frame with UB610 × 229 × 125

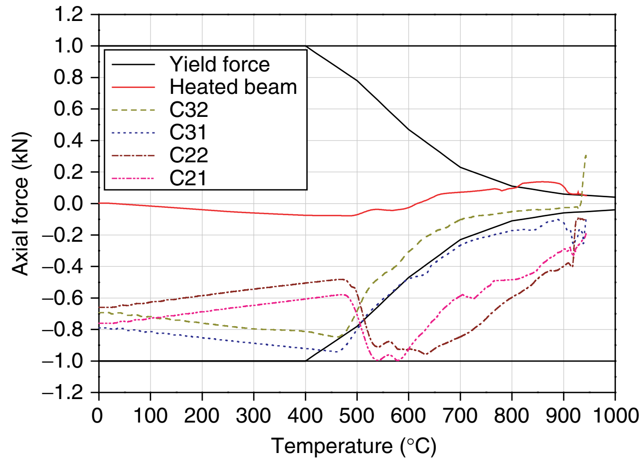


Figure 18. Axial forces in the columns of the frame with UB6 10 × 229 × 125

(2) Case 2 Steel frames in Fire case 1 with UB203 × 102 × 23

The failure process of the steel frame with UB203 × 102 × 23 is shown in Figure 19. The nodal displacements and column forces are shown in Figures 20 and 21, respectively. The beams in the frame are so weak that the plastic hinges are formed at their ends under UDL alone at ambient temperatures as shown in Figure 19(a). The premature development of plastic hinges in beams leads to the large deflection of the heated beam at the early stage of the heating as shown in Figure 19(b). The overwhelming deformation of the heated beam generates tension force in it which drives the second storey moving in after 500 ºC. The bottom columns below the heated column except the column C31 buckles almost at the same time and there is no sign of load redistribution in the columns as shown in Figure 21.

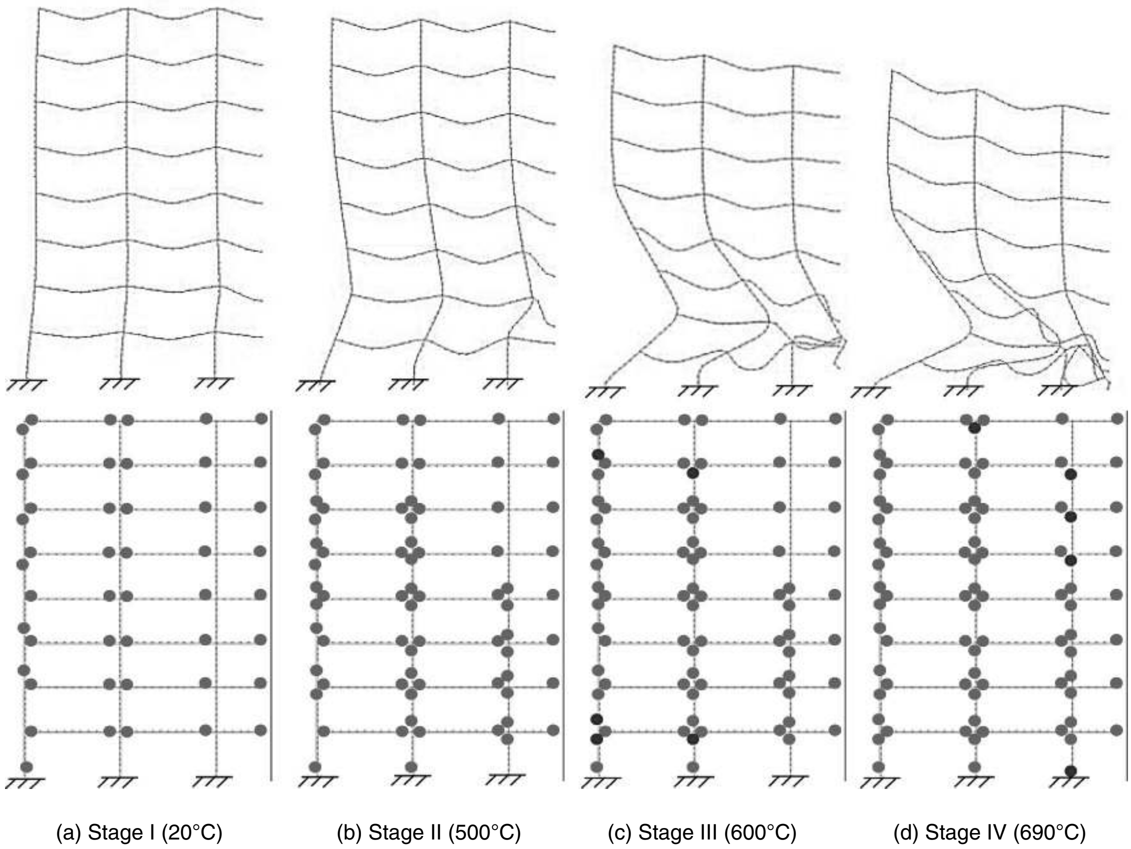


Figure 19. Collapse process of the steel frame in Fire 1 with UB203 ×102 × 23

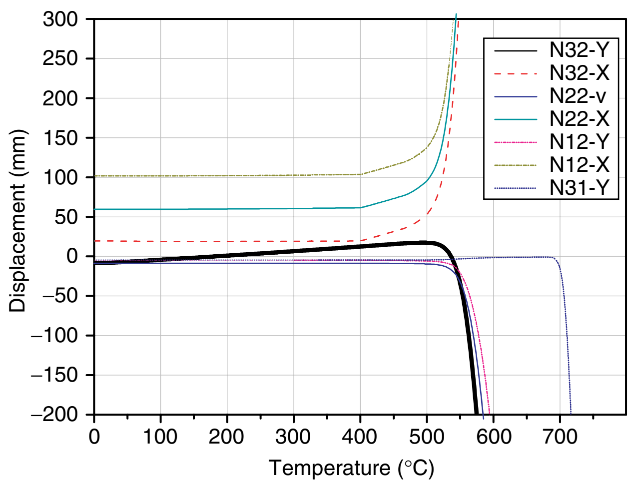


Figure 20. Displacements of the key nodes of the frame with UB203 × 102 × 23

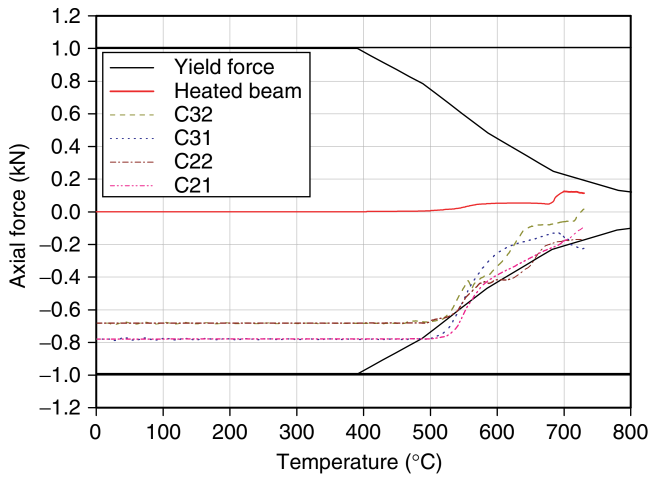
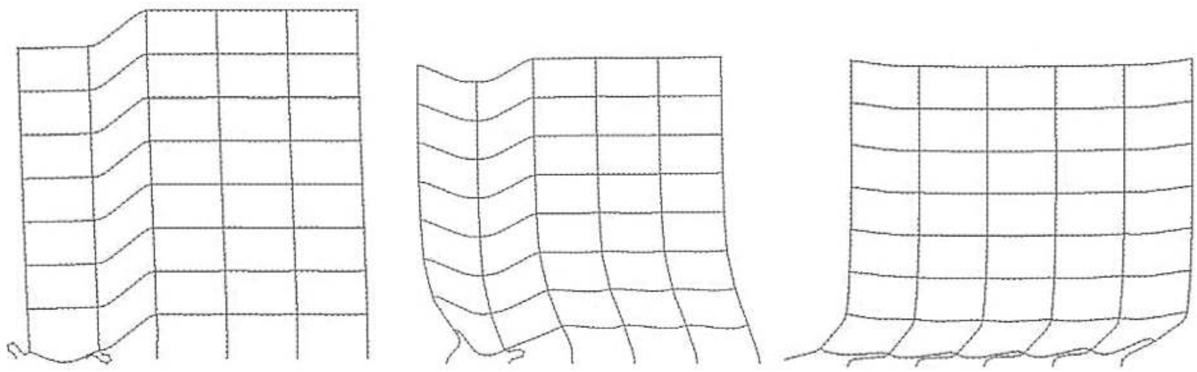


Figure 21. Axial forces in the columns of the frame with UB203 × 102 × 23

4.3. Edge Bay Fire (Fire Case 2)

4.3.1. Influence of the loadings

In this section the collapse behavior of steel frames subjected to Fire case 2 (edge bay fire on the ground floor) are investigated and the influence of loads and beam sections is discussed. Three uniformly distributed loads (UDL) (30 kN/m, 50 kN/m and 60 kN/m) were applied vertically on all the beams of the steel frame. The beams is taken as UB 305 × 165 × 40. Figure 22 shows the collapse modes of steel frames under the three UDLs and corresponding upper limit of temperature. Similar to the cases under central bay fire, it is found that, for smaller UDL of 30 kN/m and 50 kN/m, the collapses of frames under the Fire case 2 are initiated by



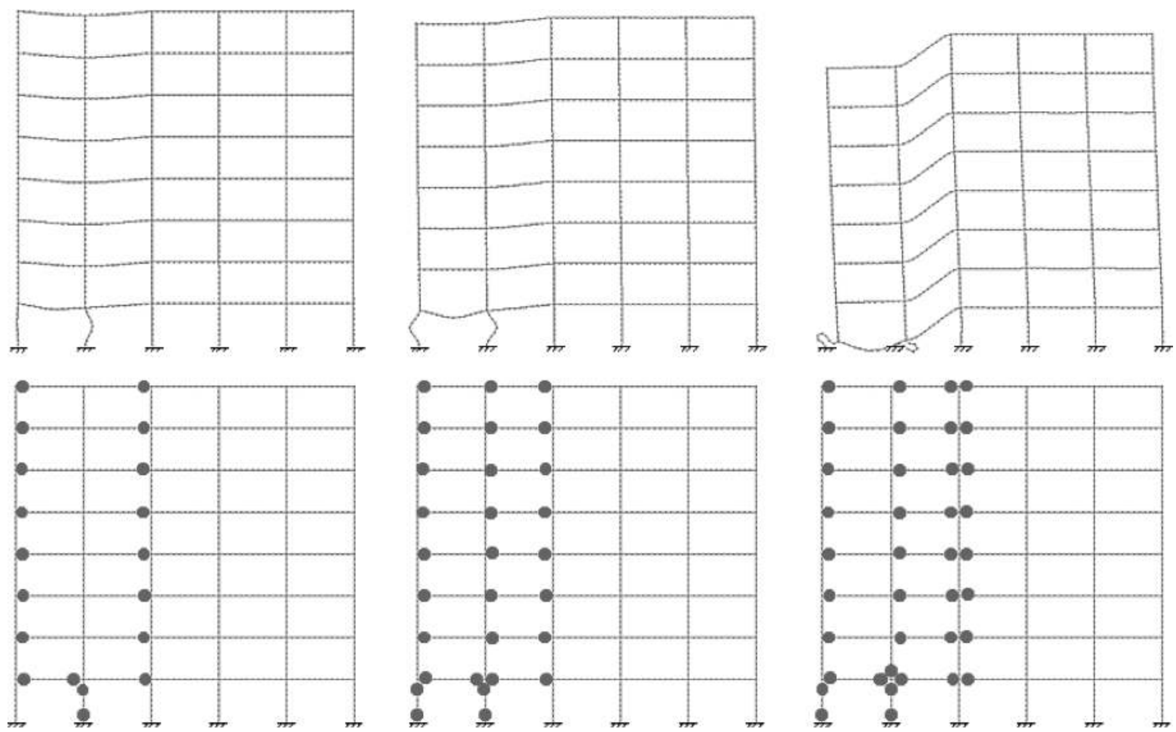
(a) UDL = 30kN/m (900°C) (b) UDL = 50kN/m (680°C) (c) UDL = 60kN/m (480°C)

Figure 22. Collapse mechanisms of steel frames in Fire 2 under different loadings

the buckling of the heated column. For a higher UDL of 60 kN/m, an obvious horizontal drift of the whole frame occurs before the buckling of the heated columns as shown in Figure 22(c). The cause and sequence of each collapse mechanism are presented in details in the following sections in comparison with the formation of plastic hinges in the frame.

(1) Case 1 Steel frames in Fire case 2 under UDL of 30 kN/m

The collapse process of the steel frame subjected to Fire case 2 under UDL of 30 kN/m is shown in Figure 23. At the temperature of 600°C, the inside heated column C21 buckles first followed by the buckling of the edge heated column C11 100 °C later. After this point the frame between the first and second bay starts to collapse downward while the rest of the frame keeps nearly static with little lateral drifts. This phenomenon is also shown in Figures 24 and 25. It is noted that the forces supported by the buckled columns (C11 and C21) are transferred to the adjacent column C31 after 600 °C as shown in Figure 25. The column C41 as well as other adjacent columns contributes little for this force redistribution. The



(a) Stage I (600°C) (b) Stage II (700°C) (c) Stage III (900°C)

Figure 23. Collapse process of the frame in Fire 2 under UDL of 30 kN/m

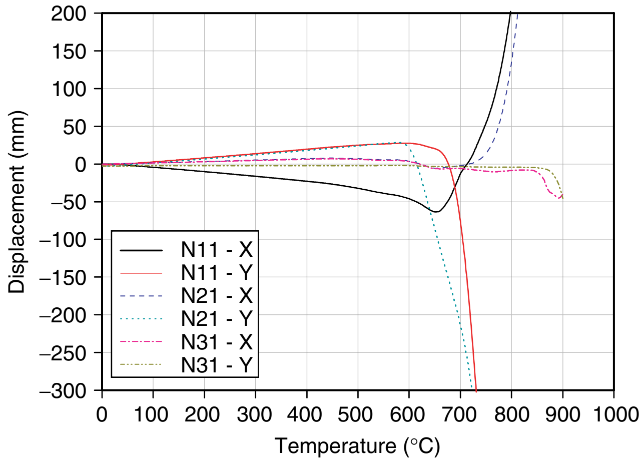


Figure 24. Displacements of the top of the columns of frame with Fire 2 under UDL of 30 kN/m

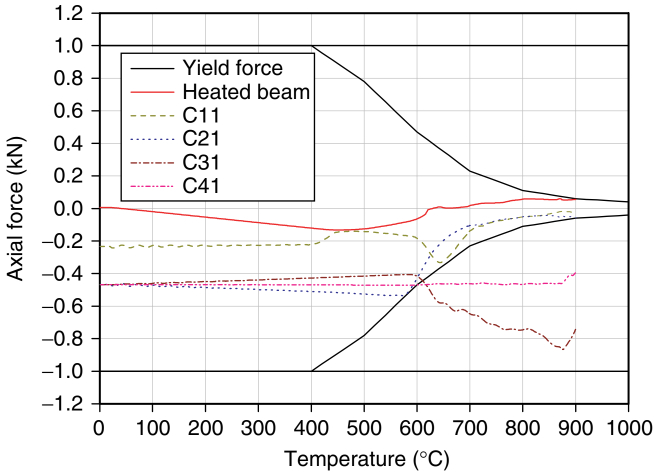


Figure 25. Axial forces in the columns of frame with Fire 2 under UDL of 30 kN/m

development of plastic hinges is confined in the left two spans as shown in Figure 23(c).

(2) Case 2 Steel frames in Fire case 2 under UDL of 50 kN/m

The collapse procedure of the steel frame subjected to Fire case 2 under UDL of 50 kN/m is depicted in Figure 26. The collapse of the frame is triggered by the buckling of the inside heated column at about 500ºC as shown in Figure 27. Without the support of the column, the deflection of the beams above the column on the second floor accelerates (at about 550ºC) 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 650 ºC as shown in Figure 26(b). 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. The forces sustained by the heated columns are sequentially transferred to the adjacent columns, from C31 to C61 as shown in Figure 28. This is different from the load redistribution scheme for the case with UDL of 30 kN/m where the additional loadings are sustained by column 31 alone.

(3) Case 3 Steel frames in Fire case 2 under UDL of 60kN/m

Figure 29 shows the collapse procedure of the steel frame subjected to Fire case 2 under UDL of 60 kN/m. Figures 30 and 31 show the displacements and axial forces in the columns on the ground floor, respectively.

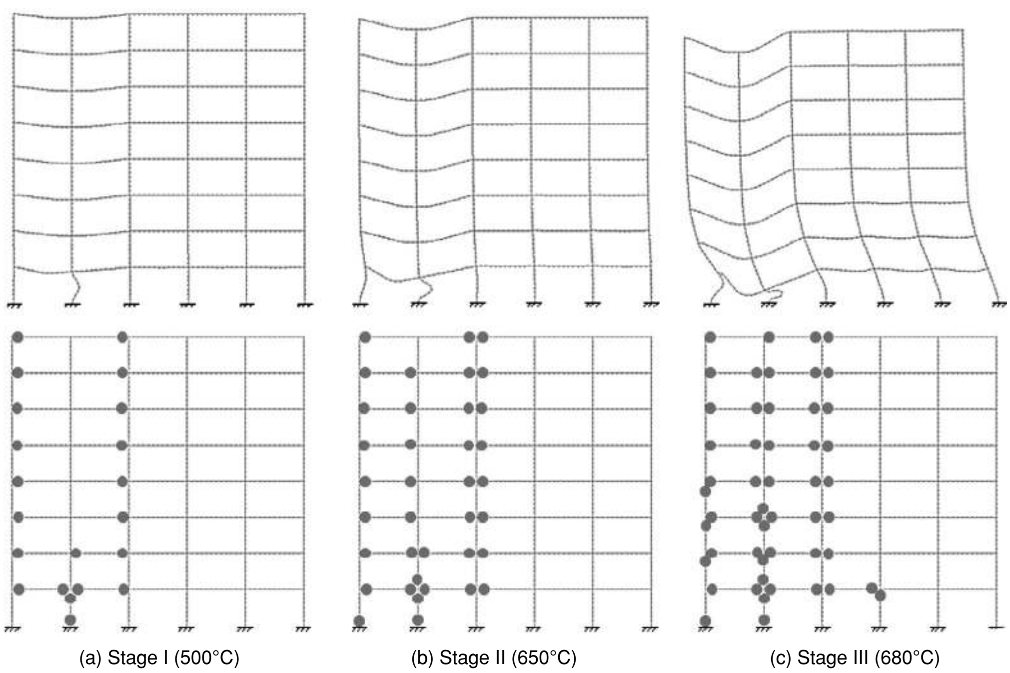


Figure 26. Collapse process of the frame in Fire 2 under UDL of 50 kN/m

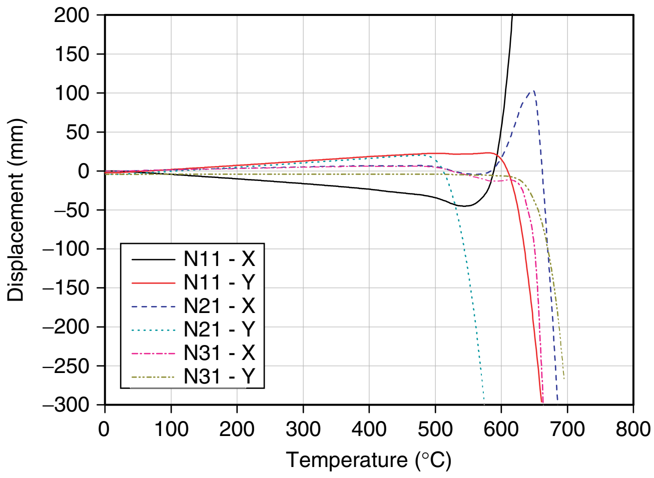


Figure 27. Displacements of the top of the columns of frame with Fire 2 under UDL of 50 kN/m

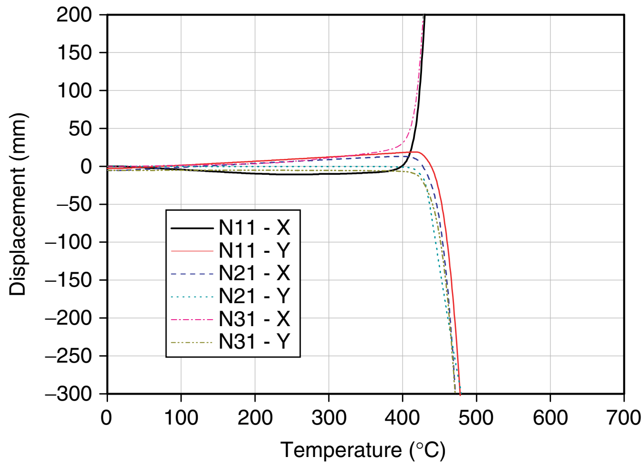


Figure 30. Displacements of the top of the columns of frame with Fire 2 under UDL of 60 kN/m

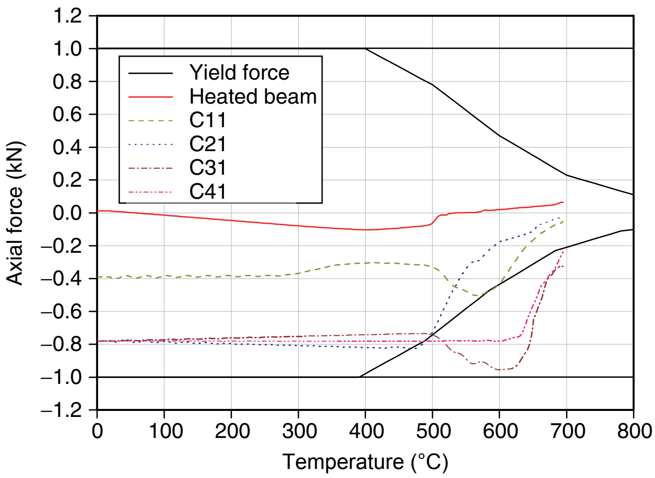


Figure 28. Axial forces in the columns of frame with Fire 2 under UDL of 50 kN/m

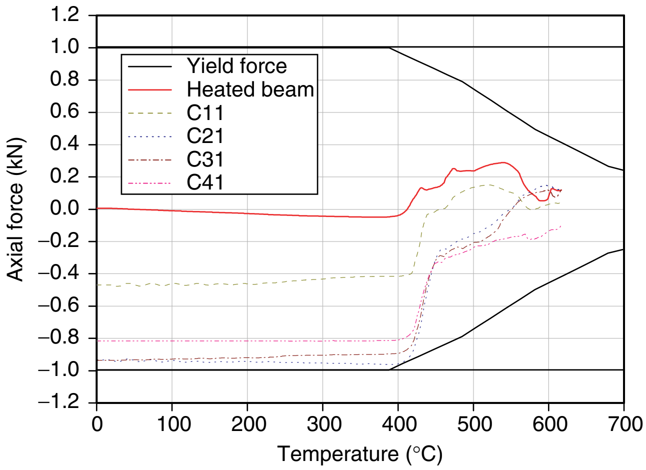
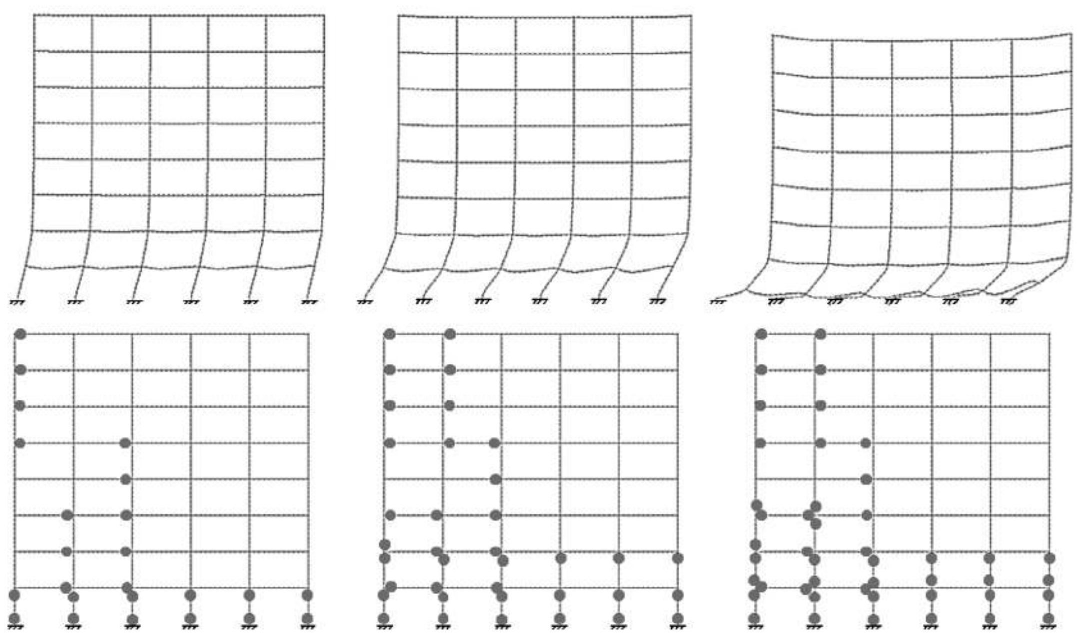


Figure 31. Axial forces in the columns of frame with Fire 2 under UDL of 60 kN/m



(a) Stage I (400°C) (b) Stage II (440°C) (c) Stage III (480°C)

Figure 29. Collapse process of the frame in Fire 2 under UDL of 60 kN/m

Different from the previous two cases, plastic hinges start to form at the ends of all the columns on the ground floor at the early stage of the heating about 400ºC as shown in Figure 29(a). 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-δ 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, leading to the lateral collapse of the whole frame.

4.3.2. Influence of beam sections

In this case, two types of beam sections (UB610 × 229 × 125, UB305 × 165 × 40) were chosen to study the influence of beam sections on the collapse mechanisms of steel frames exposed to edge bay fire. The UDL is taken as 50 kN/m. The collapse procedure of the steel frame with UB305 × 165 × 40 is shown in Figures 26 × 28. The collapse behavior of the frame with UB610 × 229 × 125 is shown in Figures 32–34. It can be seen that the collapse of the frame is triggered by the buckling of the heated columns followed by the sequent buckling of the other columns on the ground floor. There is no plastic hinges formed in the beams while in the columns the plastic hinges develop from the heated compartment to the other cool edge.

In summary, the collapse mechanisms and critical temperatures of steel frames exposed to fire are concluded in Table 1. It is found that the most

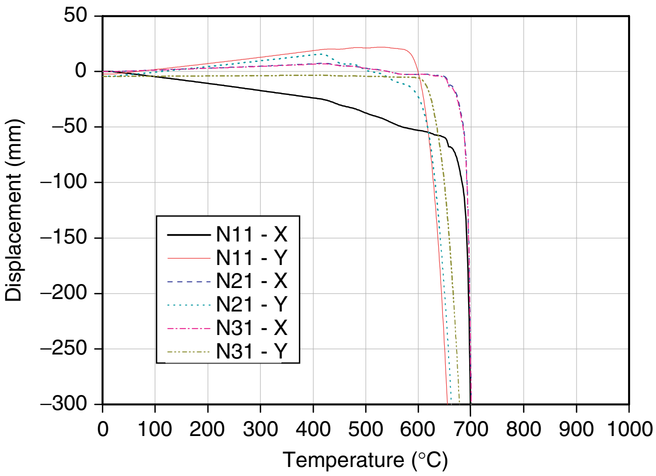


Figure 33. Displacements of the top of the columns of frame with Fire 2 under UDL of 60 kN/m

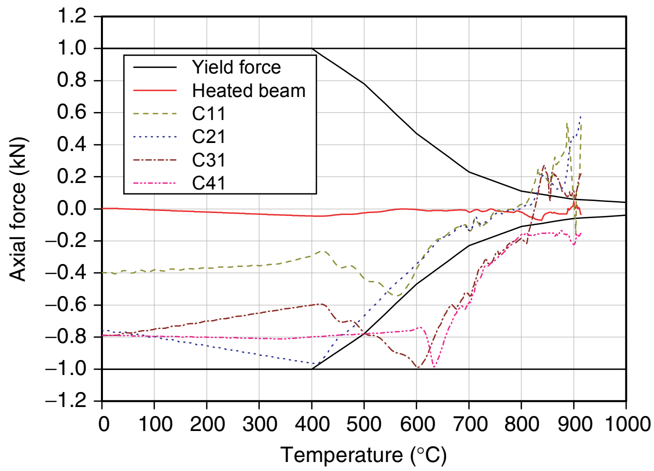


Figure 34. Axial forces in the columns of frame with Fire 2 under UDL of 60 kN/m

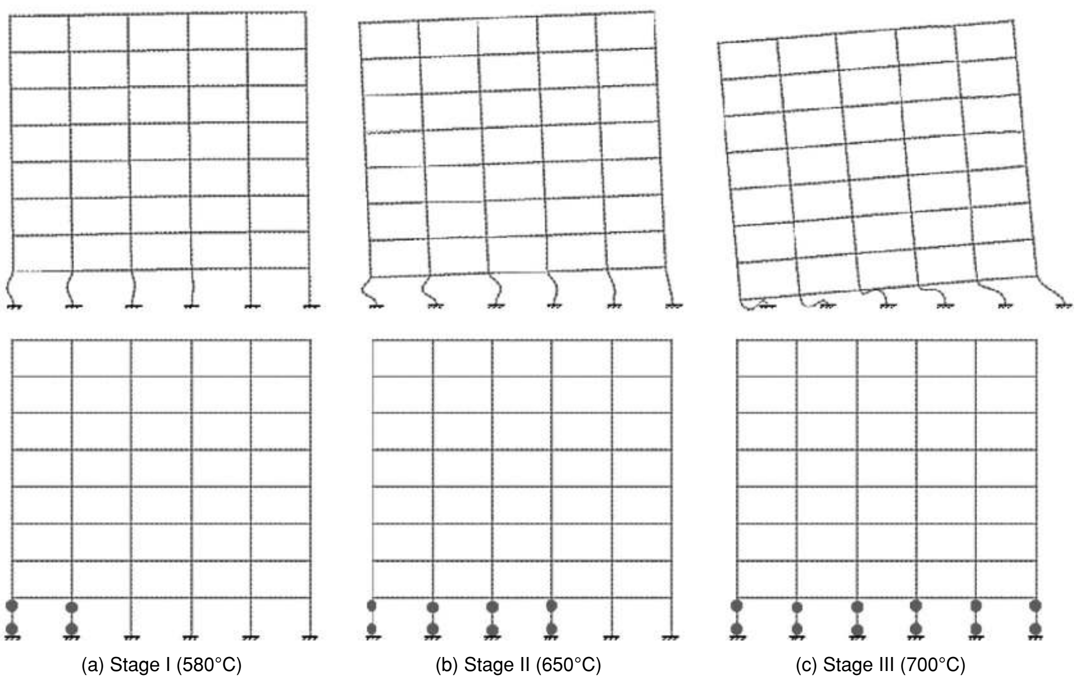


Figure 32. Collapse process of the frame in Fire 2 with UB 610 × 229 × 125

Table 1. Collapse mechanisms and critical temperatures of steel frames in fire

Mechanisms	Characteristics	Causes	Critical temperature
1. General	Heated bay collapses and upper frame above heated floor drifts laterally	Moderate load and beam in (1) Central bay fire (Figure 8) (2) Edge bay fire (Figure 27)	(1) 750 ºC (2) 680 ºC
2.Global downward collapse	Collapse downward due to buckling of columns	(1) High load in central bay fire (Figure 11) (2) Strong beam in central bay fire (Figure 17)	(1) 250 ºC (2) 920 ºC
3.Local lateral drift	Lateral drift of the heated floor	Weak beam (Figure 20)	690 ºC
4.Global lateral collapse	Collapse laterally due to buckling of ground floors	In edge bay fire when (1) High load (Figure 30) (2) Strong beam (Figure 33)	(1) 480ºC (2) 700ºC

dangerous case is the frame under high loads subjected to the central bay fire (fails at 250 ºC) followed by that in the edge bay fire (fails at 480 ºC) where the collapse resistance is enhanced by relatively larger stiffness provided by the surrounding parts of the frame.

5. CONCLUSIONS

This paper presents the collapse mechanisms of steel frames under fire conditions for various loads, beam strength and fire scenarios. 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 sequent buckling of the columns at the same storey of the heated column or below. 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) The collapse mechanisms of steel frames under fire conditions vary with the loadings. For small load levels applied on the structure, there occurs horizontal movement of the frame before the collapse of the frame where the collapse is generally confined to the storey above the heated floor. However, as the load increases, this period vanishes and instead the collapse is triggered directly by the sequent buckling of the bottom columns. The collapse mode for high loadings, in the form of downward collapse of the whole structure, may occur as early as about 250 ºC.

(3) The collapse behavior of steel frames is also dependent on the beam strength. As the size of the beam section increases, the collapse mechanism transforms from the beam failure mechanism to column failure mechanism. In the beam failure mechanism, the beams are so weak that the failure is initiated by the premature development of plastic hinges in the beams at early stage of heating, even under UDL at ambient temperature. In contrast, in the column failure mechanism, the beam is strong enough that the collapse is due to the buckling of the columns below the heated column.

(4) Generally, the edge bay fire is more prone to induce progressive collapse of structures in fire than the central bay fire. The collapse mode is either local inclined collapse toward the fire compartment or global downward collapse. The former occurs in the frame under relatively small loads and the latter is for the frame with strong beams.

(5) For the steel frame subjected to central bay fire under large loads, the heated beam experiences large deflection to some extent that catenary action is generated. The tension forces developed in the beam will pull the adjacent frame inward and lead to the inclined collapse. On the other hand, for the steel frame exposed to edge bay fire under large loads, the thermal expansion of the heated beam at early stage of heating causes asymmetric deformation of the frame which makes premature formation of plastic hinges in the bottom columns due to the P-Δ effects, leading to the lateral collapse of the whole frame.

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