Northumbria
University
NEWCASTLE

Progressive collapse analysis of composite steel frames subject to fire following earthquake

Riza Suwondo, Lee Cunningham, Martin Gillie, Colin Bailey

Abstract

This paper presents three-dimensional progressive collapse analyses of composite steel frames exposed to fire following earthquake. The scenarios of heating columns located in various different fire compartments (internal, edge and corner bay) are first studied to investigate load redistribution paths and members' interactions within the composite frame. The results show that the loads previously supported by the heated columns are redistributed to adjacent columns along two horizontal directions, a phenomenon which cannot be captured in a 2D model. Then, the 3D model is adopted to investigate the effect of residual deformation after an earthquake on the progressive collapse behaviour of the composite building. It is found that neither the load redistribution path nor the fire resistance of the building is considerably affected by the residual deformation. A series of progressive collapse analyses subjected to travelling fires resulting from fire compartment damage is also performed. It is concluded that the survival of the building can be greatly affected by the spatial nature of the travelling fire as well as the inter-zone time delay.

1. Introduction

Fire following an earthquake is usually considered as a low-probability multi-hazard event. However, the damage associated with post-earthquake conflagration can be very severe. For instance, two of the most severe fire disasters of the 20th century, San Francisco 1906 and Tokyo 1923, were triggered by earthquakes and resulted in many fatalities and high levels of damage[1]. The 1995 Kobe earthquake caused huge damage due to both the earthquake itself and post-earthquake fire[2]. Although many recent strong earthquakes (1999 Izmit and 2005 Kashmir) were not followed by fire, the risk as mentioned above can be very high regarding the occupant safety and property damage.

In multi-hazard events such as fire following earthquake, building structures are required to maintain stability to guarantee an acceptable level of safety and to limit the financial costs of repair. Prescriptive fire safety design as often advocated by codes of practice is based on Standard Fire tests on isolated individual elements[3]. Thus, it is difficult for designers to estimate the level of risk associated with whole building structures subjected to real fire. Moreover, this approach is not realistic since boundary conditions and structural interaction may affect the behaviour of an element as part of the whole structure. A clear example of this sort of behaviour seen in real structures can be found in the Cardington fire tests[4,5]. These tests indicated that for a multi-storey composite steel frame, the composite floor system had greater fire resistance than the associated single elements in the standard fire test due to the development of tensile membrane action. In the case of fire following earthquake, the risk is increased due to limited fire-fighting availability, extreme traffic congestion, lack of water supply and other possible problems in the time period after the earthquake. Hence, the standard fire condition will be even less well representative of the actual behaviour of the structure. This shows the importance of studying the complete structure to understand and quantify the actual behaviour of structures in fire particularly under multi-hazard events such as fire following earthquake.

Progressive collapse occurs when local failure spreads from element to element resulting in collapse of a whole structure or large part of it. The collapse of the World Trade Centre towers in 2001 ultimately from a large uncontrolled fire has attracted renewed attention in understanding collapse of building structures in general. With substantial damage and losses in the case of collapse, it is essential to understand and quantify the performance of buildings under fire conditions to maintain their stability without disproportionate failure after local damage. A wide range of research has been produced on the progressive collapse analysis of steel structures under fire loading. Porcari et al.[6] presented a comprehensive literature review of fire-induced progressive collapse of multi-storey steel buildings. One of the key findings of Porcari et al. is that the application of performance-based fire design needs a useable and efficient tool to design and evaluate building structures for different fire protection configurations. Moreover, the designer needs to identify critical parts of the building that may be vulnerable under fire loading.

Sun et al.[7] developed a static-dynamic procedure adopting the software package Vulcan to assess the robustness of 2-D steel-framed structures in fire. The results of parametric studies showed that for unbraced frames, larger beams and lower loading ratios of columns can increase the failure temperature at which structural collapse occurs. The study also found that lateral stiffness via bracing is helpful to prevent initial local failure spreading from element to element. Talebi et al.[8] investigated the efficiency of buckling restrained brace systems (BRBs) to prevent the progressive collapse of a structural frame in fire. The results indicated that BRBs provide higher collapse temperature compared to Ordinary Concentrically Braced systems (OCBs). Jiang et al.[9] carried out dynamic explicit analyses to simulate progressive collapse resistance of a steel frame exposed to localised fire. The effects of strain rate and damping ratio were investigated. The results showed that the damping ratio can be neglected in progressive collapse assessment of steel frames. However, since the buckling of columns is usually sudden, high strain rates may be incurred which in turn require consideration of possible strain rate effects in the steel.

The studies mentioned above focussed on two-dimensional frames so that the models do not represent the overall structural behaviour accurately. Recently, some researchers have performed analysis of three-dimensional steel frame structures. Agarwal and Varma[10] investigated the importance of columns on the stability behaviour of a typical mid-rise (10-storey) steel building with composite floor systems subjected to corner compartment fires. It was found that when column failure occurs, the load will be redistributed to the adjacent columns to maintain stability. Jiang and Li [11] presented progressive collapse analysis of 3D steel frames in fire. An explicit dynamic analysis was performed to investigate the effect of loading ratio and heating location on the collapse mode and load redistribution path. Jiang and Lie found that the 3D model predicted a different collapse mode and load redistribution path compared to that predicted by the 2D model. The collapse resistance of the frame was mainly affected by uneven load transfer which cannot be captured in the 2D model.

From the above literature review, it is clear that a 3D model is required to perform a robustness analysis of structures. Although some critical issues can be captured by 2D models, they are not able to capture the load redistribution path in a realistic structure. It should also be noted that the majority of previous works have investigated the progressive collapse of the building under fire events only. The previous study by the present authors[12] adopted three-dimensional models to analyse the behaviour of earthquake damaged composite steel-frames in fire. The results showed that earthquake damage significantly reduces the fire resistance of the composite building. The study also revealed that failure of columns leads to collapse of the structure. However, no alternative load paths were observed in the study since all the columns were uniformly exposed to fire. Local heating scenarios (likely in large structures) were not studied. To bridge this knowledge gap, this paper presents progressive collapse analyses of a 3D composite building subjected to local fire following earthquake. The objective of the study is to identify global behaviour of building subject to local fire after earthquake damage. The remainder of this paper is structured as follows: the next section presents a description of the case study building, this is followed by a discussion of the modelling approach including validation with existing experimental data, then progressive collapse analysis of the case study frame is performed which in turn is followed by the conclusions.

2. Description of the Case Study Building

A five-storey office building was designed for high seismicity using a moment resisting frame with medium ductility according to Eurocode EN 1993-1-1[13], EN 1994-1-1[14] and EN 1998-1-1[15]. As shown in Figure 1, the frame has three bays of 9 m and 8 m with storey height of 4 m. A column size of UKC356x368x153 was adopted, while UKB457x191x74 and UKB356x127x39 sections were used for primary beam and secondary beams, respectively. The concrete slab acts compositely with the steel beams and has an overall depth of 130 mm. A rebar mesh was located in the middle of the concrete slab and consists of 6 mm diameter bars at 200 mm centres each way. The total design load at the fire limit state (1 x dead + 0.5 x live) according to Eurocode EN 1990[16] is taken as 5.5 kN/m^2. This load level results in a maximum load ratio at the internal columns and beams of 0.3 and 0.5, respectively.

The steel behaviour for columns and beams was assumed to be elastic-perfectly plastic with a yield stress of 355 MPa and Young's modulus of 210 GPa at ambient temperature. The compressive strength of concrete in the slab was 45 MPa, while the yield strength of the rebar was 450 MPa. Thermal expansion coefficients of steel and concrete are taken as 1.35×10^{-5} °C^{-1} and 13×10^{-6} °C^{-1}, respectively. The steel and concrete properties at elevated temperature were adopted according to Eurocode EN 1992-1-2[17] and EN 1993-1-2[18].

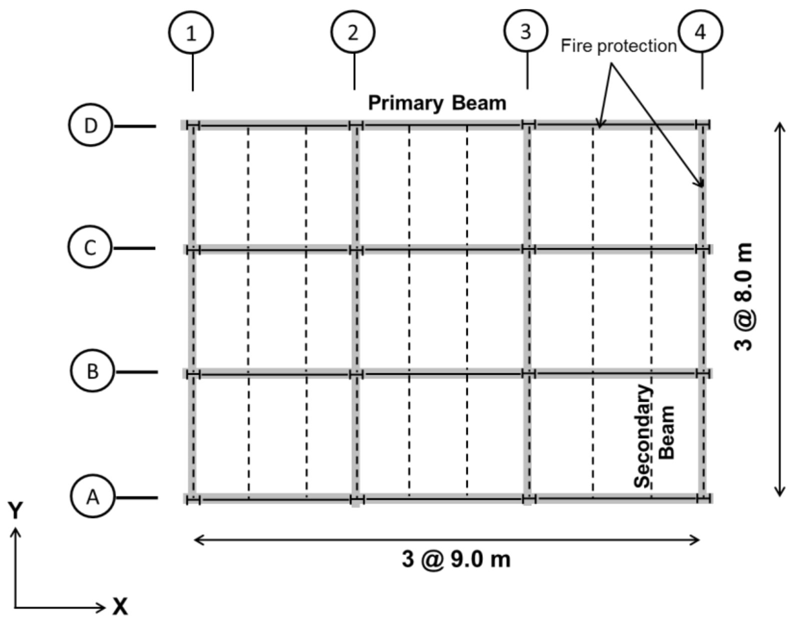


Figure 1: Case Study building plan

Two different fire curves, the ISO 834 Standard fire[19] and the Eurocode parametric fire[20] were used to simulate the fire event. The parametric fire is considered to give a comparison against the Standard fire which is commonly used, and to study the effect of the cooling phase of a fire. The parametric fire curve depends on several factors, such as area, ventilation and so on. In this study, it is assumed that the fire compartment is confined to the area of one bay (8m x 9m). The fire load of 511 MJ/m^2 was used according to Annex E of EN 1991-1-2[20], which is for office buildings. The ventilation factor and thermal inertia of compartments are taken as 0.06 $m^{1/2}$ and 1470 $Ws^{0.5}/m^2K$ respectively. The procedure in Eurocode EN 1993-1-2 was adopted to calculate the steel temperatures and a numerical heat transfer analysis using finite element analysis was used to calculate concrete slab temperatures through the thickness. Figure 2 shows the temperatures of both steel and concrete against time under the Standard Fire and the Parametric Fire.

This study investigates the potential failure of the columns that may cause collapse of the building. Accordingly, only the ground floor fire scenarios are investigated because the ground floor columns have the largest load ratios. It is believed that columns are always protected in practice and the Cardington Tests have demonstrated that the frame structure can resist collapse when all columns are fully protected. However, this study assumed that the fire loadings are applied to unprotected columns as an extreme scenario since there is a strong possibility that fire protection gets detached from the steel surface due to seismic actions. This study considers failure of columns as a key component leading to collapse of the building structure, therefore the steel columns are heated up to failure whether subject to the Parametric or Standard fire. The primary focus of this study is to investigate the robustness of the building after failure of the columns.

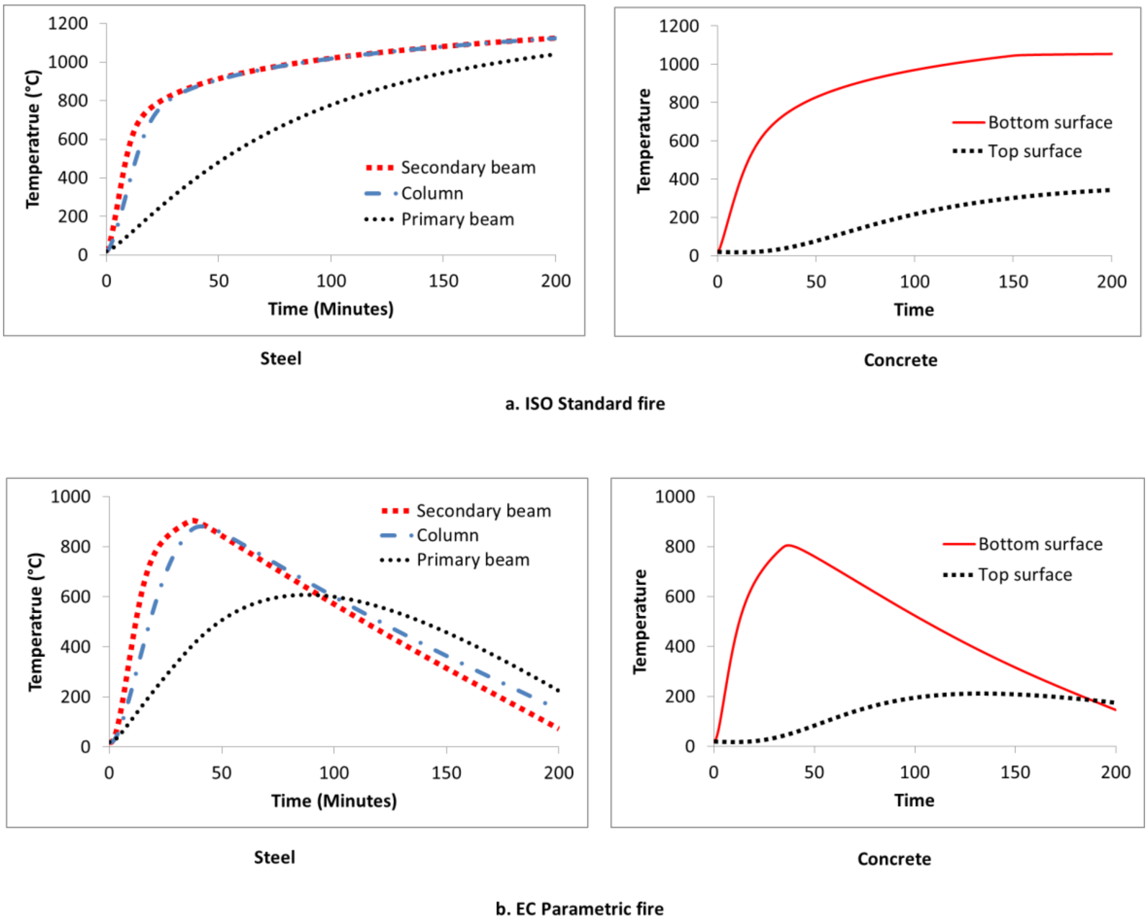


Figure 2: Time-temperature curves

A series of numerical analyses of three-dimensional composite buildings are carried out in which the progressive collapse of the buildings is investigated. Two sets of analyses are performed. In the first set of the progressive collapse analyses the undamaged composite building is studied. The collapse mode and load redistribution path of the building subjected to different fire locations are investigated. In the second set of analyses, two earthquake damage scenarios on the composite building are studied. The first damage scenario is lateral deformation that remains after the earthquake. Pushover analysis is performed to analyse the structure subjected to an earthquake. The second damage scenario is fire compartment damage that can lead to the fire travelling across the floor.

3. Numerical model

The finite element software ABAQUS v6.14[21] was used to model and analyse the structure. A three-dimensional model of the composite building was developed. Steel beams and columns were modelled using 1-D line elements (B31) which are 2-node linear beam elements in space. For simplicity, the steel beam-to-column and beam-to-beam connections were assumed to be ideally rigid and pinned respectively which approximates common practice for seismically designed steel frames. Thus, connection failures are not considered in this study. The rigid and pinned connections are modelled using the joint-rotation connector available in ABAQUS.

The concrete slabs were discretised using a four-node shell element with reduced integration (S4R), while rebar layers with appropriate steel material properties were defined in the shell elements to model the steel mesh. Tie constraints between the steel beam and the slab are applied to accommodate the composite action between the concrete slab and steel beam. The three-dimensional composite building frame is modelled as shown in Figure 3. A mesh size of 0.5m x 0.5m is used for the composite floor slab. Eight elements are meshed for all columns and 18 elements for the beams.

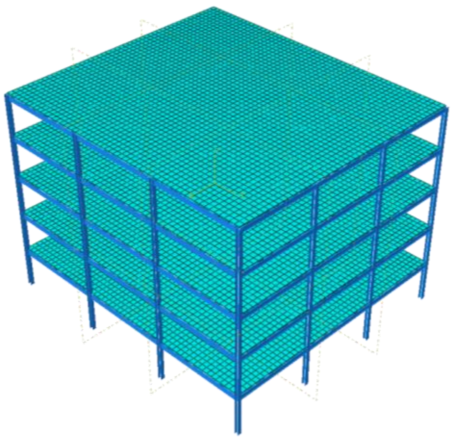


Figure 3: Three-dimensional finite element model

Elastic-perfectly plastic structural steel behaviour was assumed for both columns and beams, and temperature-dependant properties for steel were adopted according to Eurocode EN 1993-1-2[18]. For concrete, a damaged plasticity model was implemented in ABAQUS. The default values of parameters used to define the damaged plasticity model are: dilatation angle = 40°; eccentricity = 0.1; the ratio of equibiaxial compressive yield stress to uniaxial compressive yield stress, f_{b0}/f_{c0} = 1.16; the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian, K = 0.667; and viscosity parameter = 0. The uniaxial concrete material behaviour at ambient and elevated temperature was taken as defined by Eurocode EN 1992-1-2[17].

Due to its sudden nature, dynamic analysis is required to simulate the buckling and post-buckling behaviour of the heated column. Thus, dynamic explicit analysis is adopted in this study. To save computational cost, the real time is scaled down where the 1-h fire is scaled to 6 s. In the dynamic analysis, Raleigh damping is used with 5% viscous damping. When column buckling occurs, there is rapid deformation inducing large strain rate. Although the effect of strain rate on the yield strength of steel is relatively small, the dynamic yield strength of steel may be significantly larger than the static yield strength[22]. In this study, the Johnson-Cook[23] model available in ABAQUS[21] is adopted to model possible strain rate effects in the steel.

Two separate validation studies with existing experimental data were carried out to confirm the capability of the modelling technique used in this study as follows.

3.1. Progressive collapse analysis of steel moment frame in fire

To validate the model's capability of capturing progressive collapse in fire, the experimental study conducted by Jiang et al. [24] is simulated. These experiments were also modelled numerically by Jiang et al. [9] using shell element instead of beam elements in ABAQUS. Figure 4 shows the test frame. All columns and beams are rectangular hollow sections. The columns all had a section of 50x30x3. The yield strength and Young's modulus of column at ambient temperature were 380 MPa and 200 GPa, respectively. The beams had a section of 60x40x3. The yield strength and Young's modulus of the beam at ambient temperature were 306 MPa and 200 GPa, respectively. Fixed boundary conditions were applied to the columns bases and the steel beam-to-column connections were welded connections.The thermal expansion of steel was taken as zero for the temperature range from 750°C to 860°C, and 1.4×10^{-5} °C^{-1} at other temperatures.

Gravity loads were first applied to the test frame, and then the column was heated. The loads carried by the test frame are presented in Table 1. As seen in Figure 5, temperature of loading consists of heating the shaded area column and beams as indicated in Figure 4 to 800°C. Following heating, the structure is cooled to ambient temperature. The real running time of the analysis was scaled down to save computational cost. The influence of scaled time is shown in Figure 6. It can be seen that the scaled time has only a small influence on the buckling temperature of the column. A total time step of 9 s (scale 1-h to 6s) agreed well with the previous test data and previous numerical study as shown in Figure 6. Some minor oscillation can be observed in the later stage of the numerical output, similar oscillations were present in the numerical model output by Jiang et al [9] however these are not represented on the figure for clarity. It should be noted that further increase in the time step results in a reduction in the oscillation, however the computational effort increases greatly.

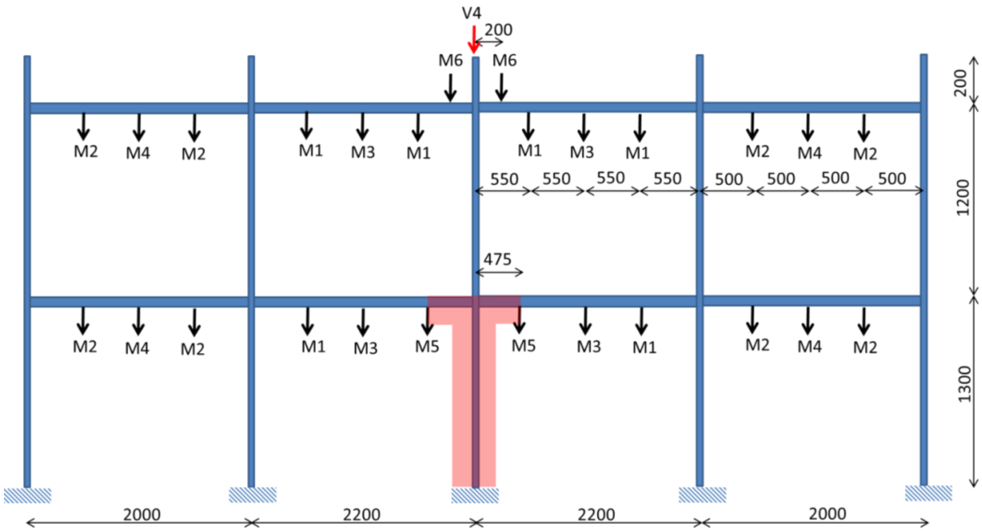


Figure 4: The test frame

Table 1: Loads carried by test frame

M1 (N)	M2 (N)	M3 (N)	M4 (N)	M5 (N)	M6 (N)
4667.3	2312.4	751.1	766.0	69.7	81.7

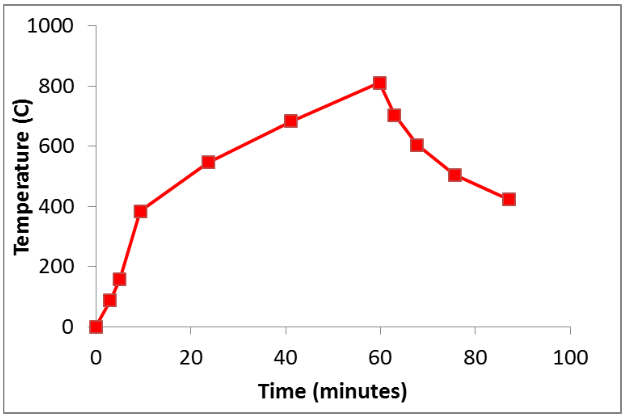


Figure 5: The test frame temperature

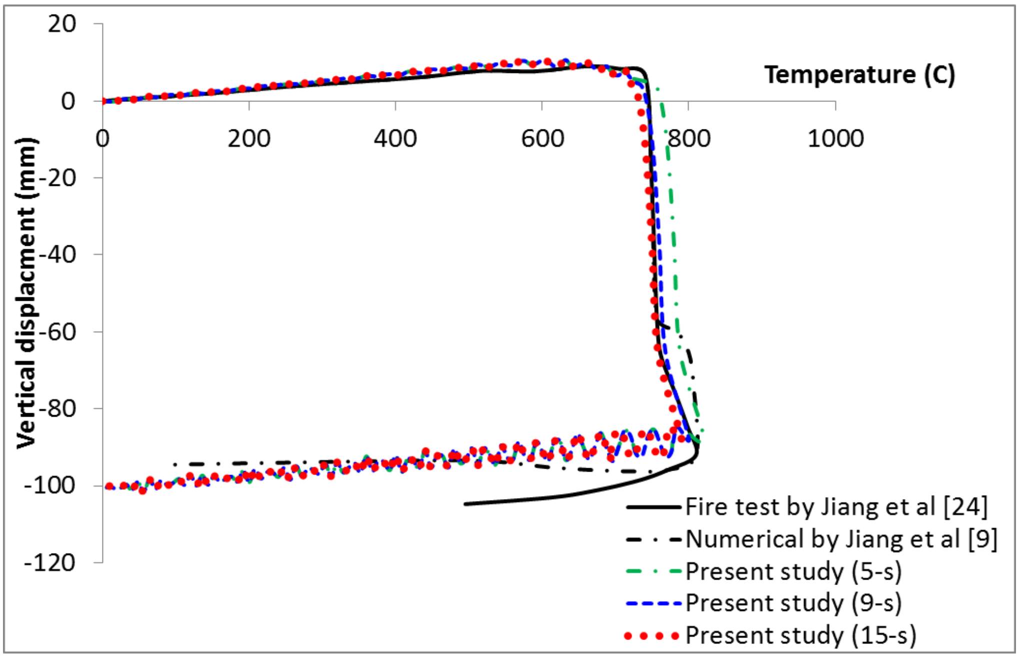


Figure 6: Comparison of previous studies and predicted axial displacement at the top of column V4 with different running times

3.2. Fire analysis of a composite steel frame building

A simplified version of the UK Cardington test[25] previously analysed by Gillie[26] was selected to validate the fire analysis on the composite building. Figure 7 shows the geometry of the composite frame. The Young's modulus and yield strength of the steel beams and columns was 210 GPa and 300 MPa respectively. The compressive strength of concrete in the slabs was taken as 47 MPa, and yield strength of rebar was 450 MPa. The concrete slab thickness was 130 mm. The rebar mesh consists of 6-mm diameter bars at 200-mm centres each way and the rebar was located at the mid-depth of slab. The thermal expansion of steel and concrete was taken as $1.35\text{x}10^{-5}$ °C^{-1} and $9\text{x}10^{-6}$ °C^{-1}, respectively. The concrete and steel material properties at elevated temperatures follow the recommendations in Eurocode EN 1992-1-2[17] and Eurocode EN 1993-1-2[18]. A total load of 5.48 kN/m^2 is applied all over the concrete slab and temperature loadings were applied on only the shaded area as shown in Figure 7. The secondary beam is heated to 800 °C, while the lower surface of the slab is heated to 600 °C with a linear gradient of 4.6 °C/mm. Then, the structure is cooled to ambient temperature. A running time of 10 s was used for this case. Figure 8 shows the mid-span deflection of the heated beam against temperature. The comparison shows that the results obtained from this study are in good agreement with existing experimental and previous analytical studies.

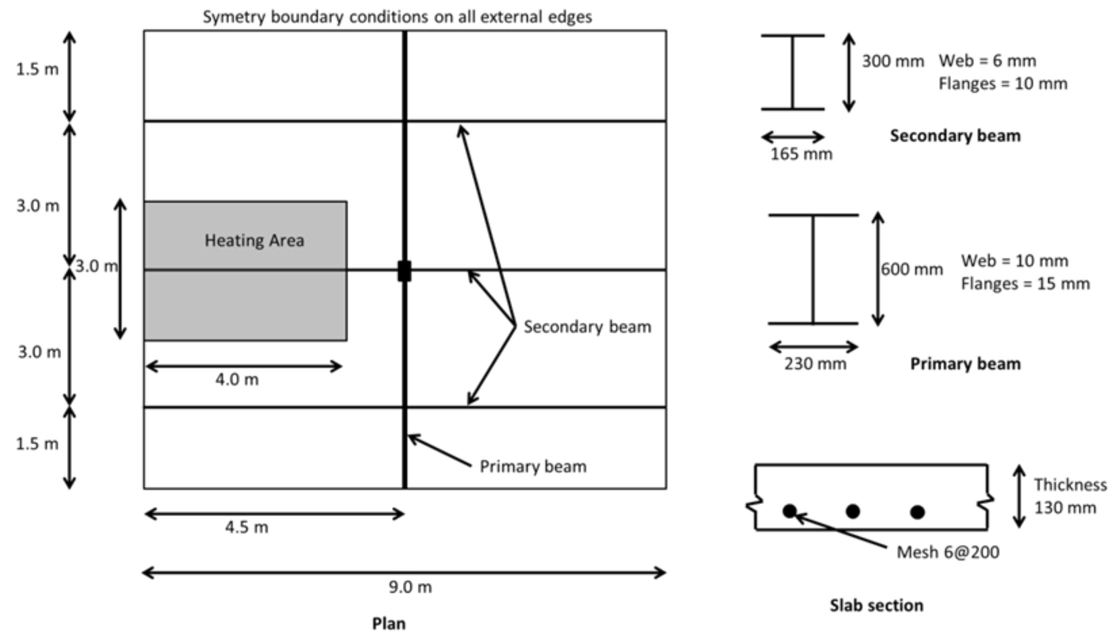


Figure 7: Geometry of simplified version of UK Cardington test[26]

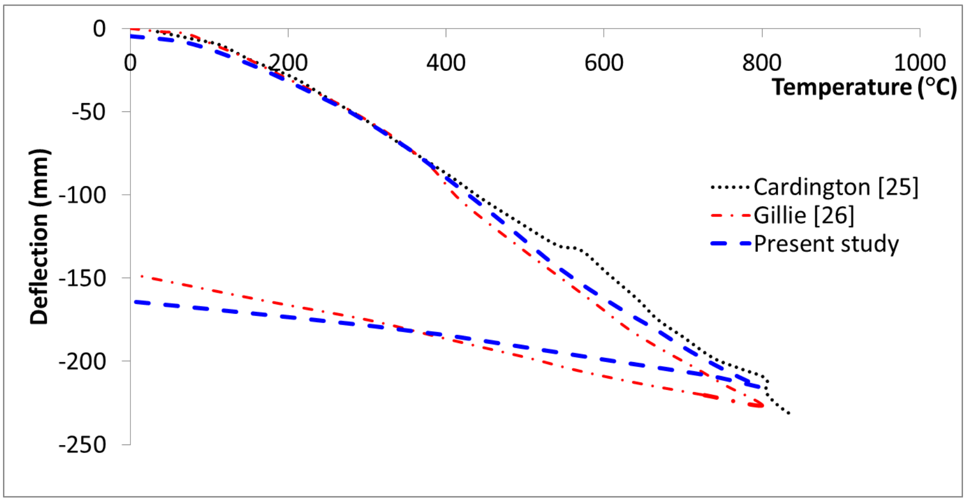


Figure 8: Mid-span deflection of the heated beam

4. Progressive collapse analysis of undamaged composite building

This section presents progressive collapse analyses of three-dimensional undamaged composite buildings subject to different fire locations using the validated model described above. This study is intended to give fundamental insight into load redistribution paths along two horizontal directions and the member interactions within the composite building frame.

4.1 Influence of location of fire compartment

A fire compartment located in the internal, edge and corner bays is considered in this study. Hence, the steel columns, steel beams and composite slab in one compartment are heated to failure at the same time as shown in Figure 9.

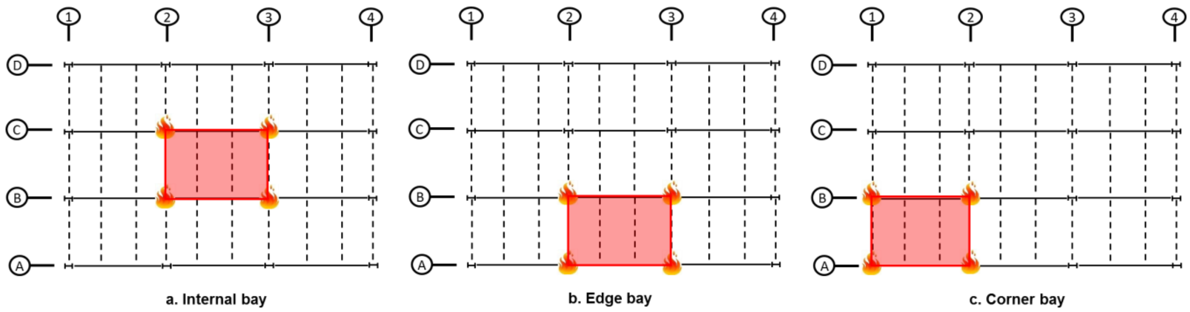


Figure 9: Fire compartment scenarios

Internal bay compartment

A fire compartment located in the internal bay is heated in this case. The top vertical displacement and axial forces of heated columns and adjacent columns under the standard fire and parametric fire are shown in Figure 10. The dashed line represents the heated column. It should be noticed that the building plan is symmetric so that only column A1, A2, B1 and heated column B2 are shown. The results are plotted against column temperature to determine at which temperature corresponding force and deflection occur.

For the case of standard fire, it can be seen from Figure 10 that the axial force of heated column B2 increases due to thermal expansion and the restraint provided by the surrounding structure. This causes a decrease in the axial force of adjacent columns (A2 and B1). Once the temperature reaches 520°C, the heated column B2 buckles and suddenly loses load bearing capacity. Then, the loads are transferred to the adjacent columns (A2 and B1). Thus, the axial forces of the adjacent columns start increasing. Meanwhile, column A1 is not affected by the heated columns. This indicates that the loads are redistributed dominantly through the protected steel beams along the column grid lines. This is confirmed by the distribution of stress in the steel reinforcement in the concrete slab as shown in Figure 11.

Similar behaviour is noticed for the case of the parametric fire. However, it is worth noting that the heated columns expand downward during the cooling phase, rather than revert to the initial position. This is due to the thermal contraction of the heated column during the cooling phase. Thus, the heated columns experience large tension so that there are additional loads transferred to the adjacent columns. This demonstrates that the cooling phase in the columns can be more critical than the heating phase.

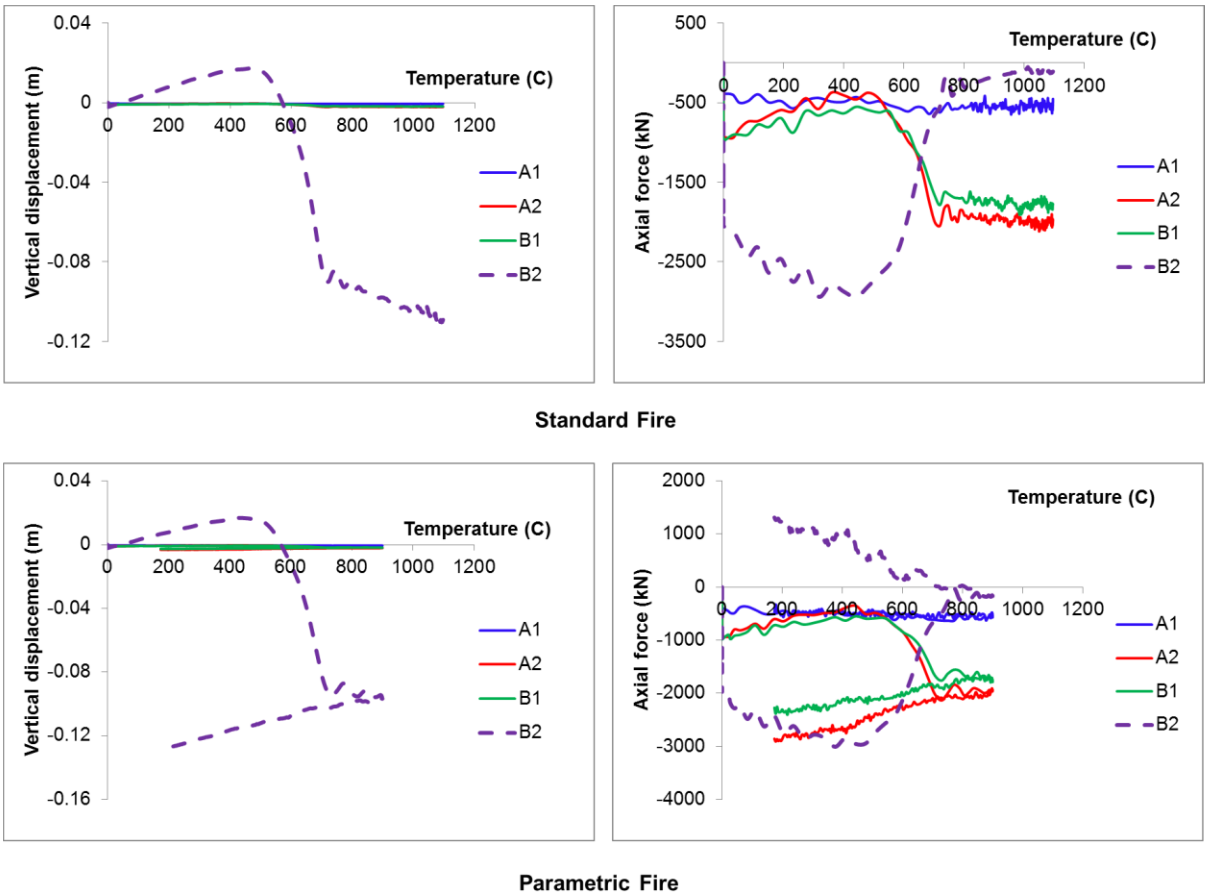


Figure 10: Vertical displacement and axial forces of heated columns and adjacent columns (internal bay scenario)

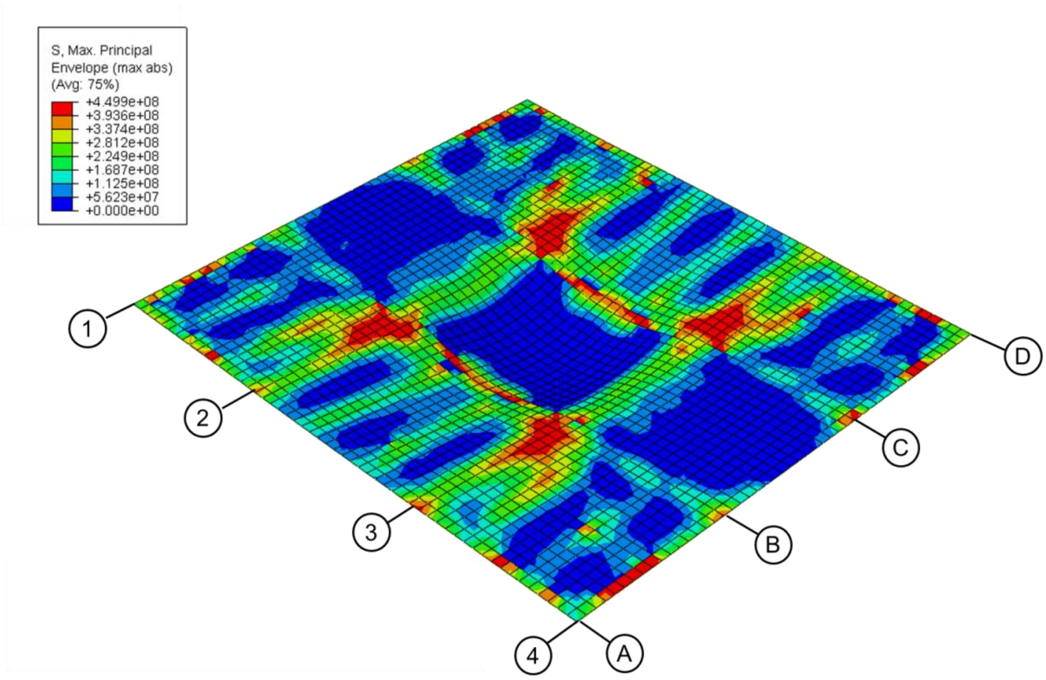


Figure 11: Maximum principal stress in the steel reinforcement layer for the concrete slab exposed to ISO standard fire at temperature 1125°C (unit in N/m^2)

Edge bay compartment

In this case (Figure 9b), a compartment located in the edge bay is heated. Figure 12 shows vertical displacement and axial forces of heated columns and adjacent columns under the standard fire and parametric fire. Similarly, the loads previously sustained by the heated columns are transferred to the surrounding column when buckling at the heated columns occurs. It can be seen that there is a different failure temperature of the heated column. The heated column B2 and A2 start to buckle when the temperature reaches 500°C and 590°C, respectively. This is due to the fact that the load ratio in the internal column B2 is higher than that of edge column A2. The sequence failure of the columns can also be observed. Once the heated column B2 buckles, there is a sudden increase in the axial force of the surrounding columns (A1, B1, and C2). Then the axial loads increase again when the heated column A2 buckles.

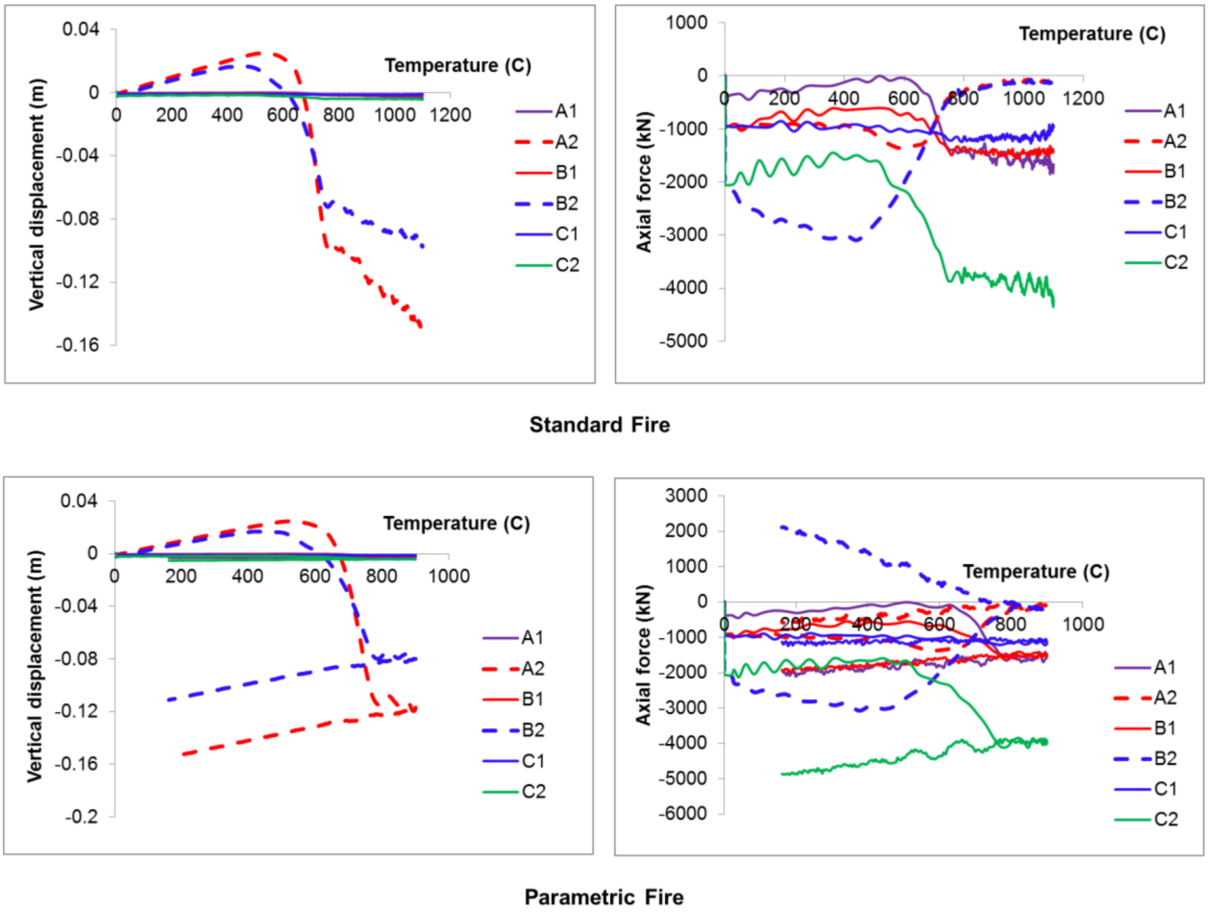


Figure 12: Vertical displacement and axial forces of heated columns and adjacent columns (edge bay scenario)

Corner bay compartment

Figure 13 shows vertical displacement and axial forces in heated column B2 and adjacent columns under standard fire and parametric fire. Similar to the previous analysis on the edge bay compartment, the heated internal column B2 buckles first followed by the heated edge column and the heated corner column. It can be seen that runaway failure (rapid increase in the rate of displacement) occurs when the temperature reaches 760°C. However, the frame can be re-stabilised eventually after the column failure since the structure is able to sustain the catenary force generated in the floor beams. Thus, the runaway failure disappears with increasing temperature.

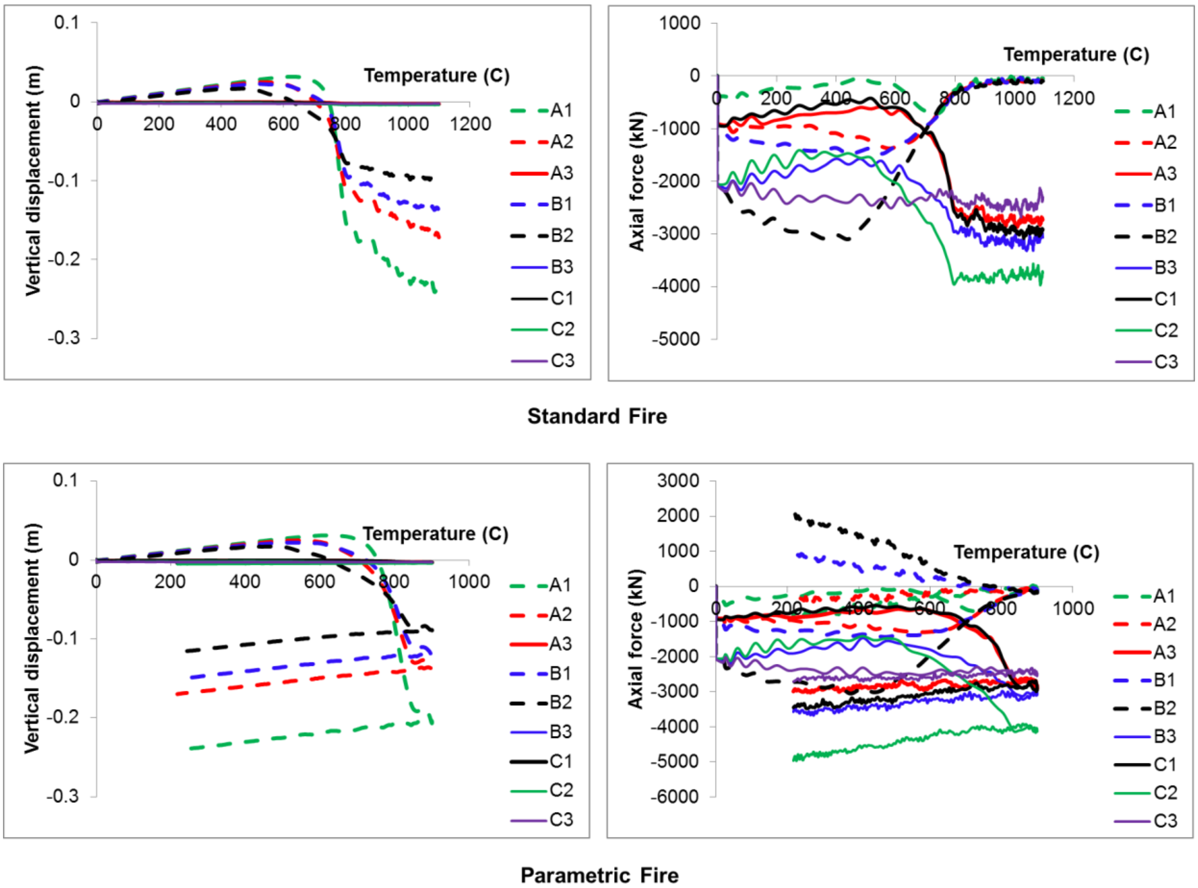
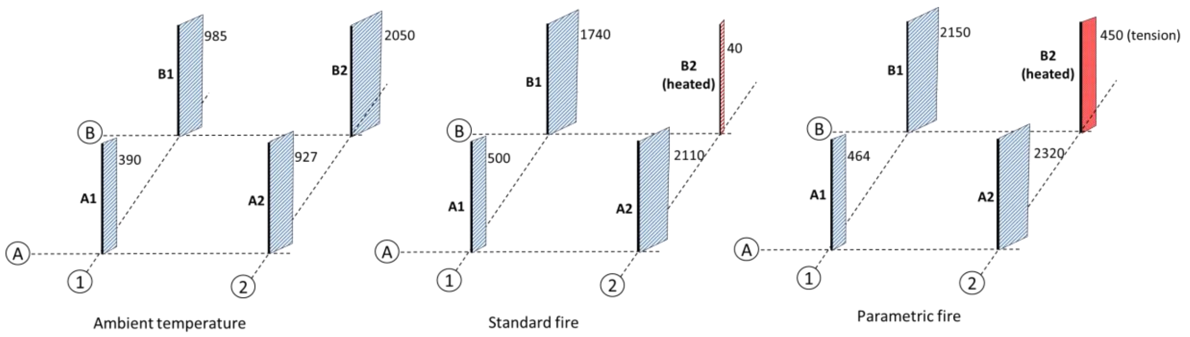


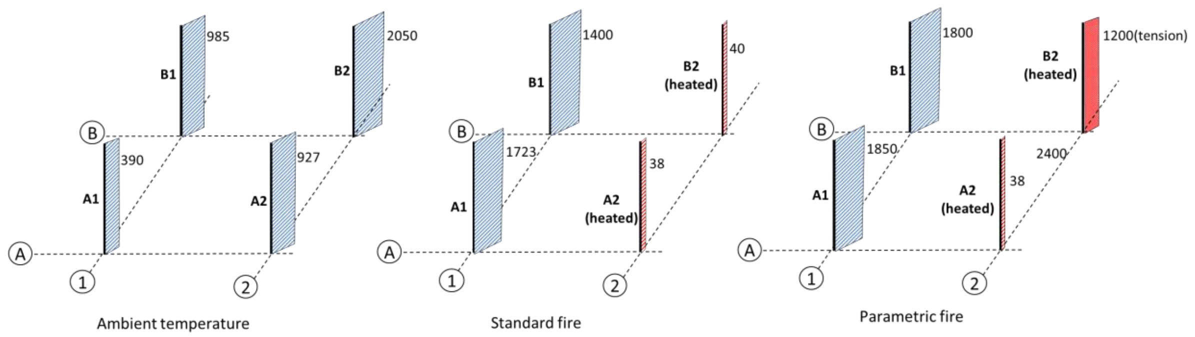
Figure 13: Vertical displacement and axial forces of heated columns and adjacent columns (corner bay scenario)

The analyses above show that the loads previously supported by the heated columns are redistributed to the adjacent columns along two horizontal directions which cannot be captured in a 2D model. Figure 14 shows the change in axial load value in the heated column and surrounded columns at ambient temperature and after heating. Table 2 shows the comparison of adjacent columns' axial forces and load ratio before and after heating. It is clear that the axial force and load ratios of the adjacent columns subject to the parametric fire are higher than that of the standard fire. Thus, the adjacent columns may buckle due to an increase in the axial force during the cooling phase. This demonstrates the importance of the cooling phase consideration in the progressive collapse analysis of the building.

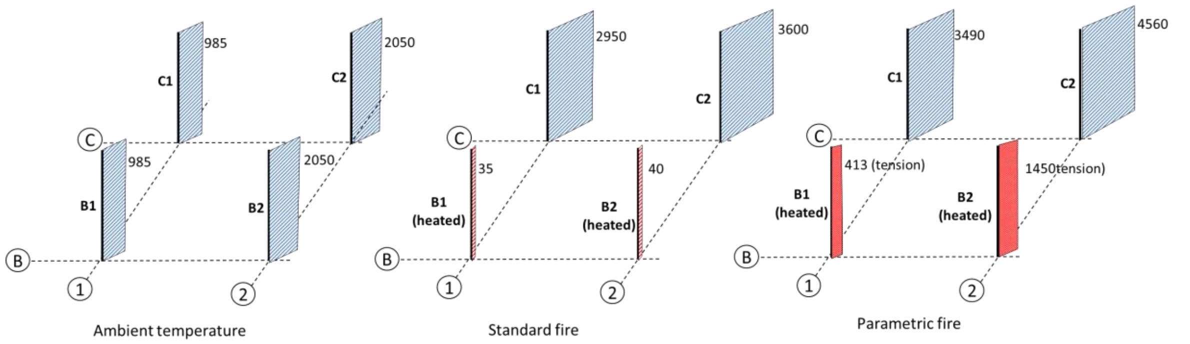
It also can be seen that there is no collapse of the building when four columns in one fire compartment are exposed to fire. This is due to the fact that the load ratios of the columns (0.3 for the internal column) are relatively low for the fire limit state design load according to Eurocode EN 1990[16]. Thus, the load previously sustained by the heated column can be safely transferred to the adjacent columns without causing buckling. This indicates that the failure of individual elements may be acceptable where overall structural collapse is prevented.



a. Internal bay



b. Edge bay



c. Corner bay

Figure 14: Axial forces diagram of heated columns and adjacent columns at ground floor before and after heating (kN)

Table 2: Comparison of adjacent columns load ratio before and after heating

Internal bay						
Column	Before		after			
			ISO		Parametric	
	Axial force (kN)	load ratio	Axial force (kN)	load ratio	Axial force (kN)	load ratio
A1	405	0.05	500	0.05	464	0.05
A2	902	0.13	2110	0.3	2320	0.33
B1	980	0.14	1740	0.27	2150	0.31
B2*	2168	0.3	40	buckling	450 (tension)	buckling
Edge bay						
Column	Before		after			
			ISO		Parametric	
	Axial force (kN)	load ratio	Axial force (kN)	load ratio	Axial force (kN)	load ratio
A1	405	0.05	1723	0.22	1850	0.24
A2*	902	0.13	38	buckling	38	buckling
B1	980	0.14	1400	0.23	1800	0.26
B2*	2168	0.3	40	buckling	1200 (tension)	buckling
C1	980	0.14	980	0.14	980	0.14
C2	2168	0.3	4216	0.58	4856	0.67
Corner bay						
Column	Before		after			
			ISO		Parametric	
	Axial force (kN)	load ratio	Axial force (kN)	load ratio	Axial force (kN)	load ratio
A1*	405	0.05	35	buckling	260	buckling
A2*	902	0.13	38	buckling	250	buckling
A3	902	0.13	2675	0.37	2820	0.39
B1*	980	0.14	35	buckling	413 (tension)	buckling
B2*	2168	0.3	40	buckling	1450 (tension)	buckling
B3	2168	0.14	3400	0.47	3690	0.51
C1	980	0.14	2950	0.42	3490	0.46
C2	2168	0.3	3600	0.57	4560	0.64
C3	2168	0.3	2385	0.33	2460	0.34

* heated column

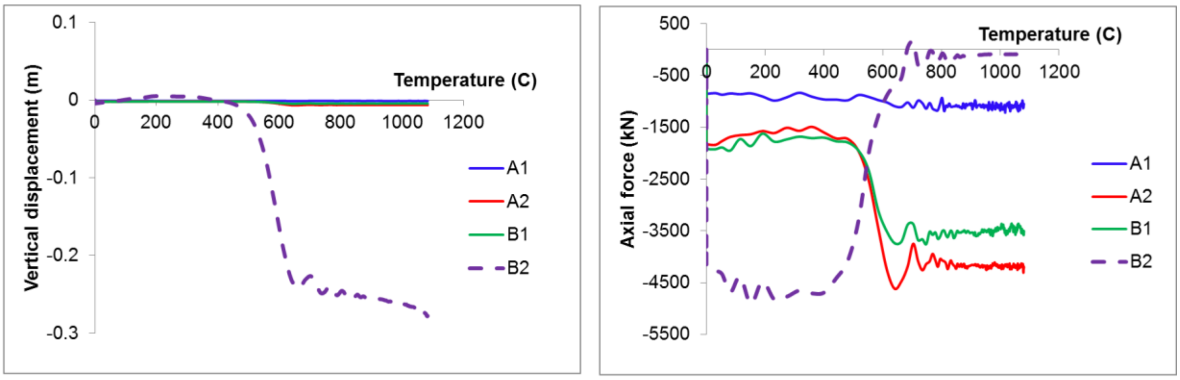
4.2 Influence of load ratio

As discussed above, the applied floor load 5.5 kN/m^2 (1 x dead x 0.5 live) taken for fire limit state design seems too low to cause collapse of the whole building. To investigate the effect of load ratio, the load was increased to 11 kN/m^2 (1.35 x dead + 1.5 x live) based on the Eurocode ultimate limit state for normal conditions. This led to a load ratio of 0.6 for the internal column. Three different compartment fire scenarios as shown in Figure 9 along with the standard fire curve are considered in this study.

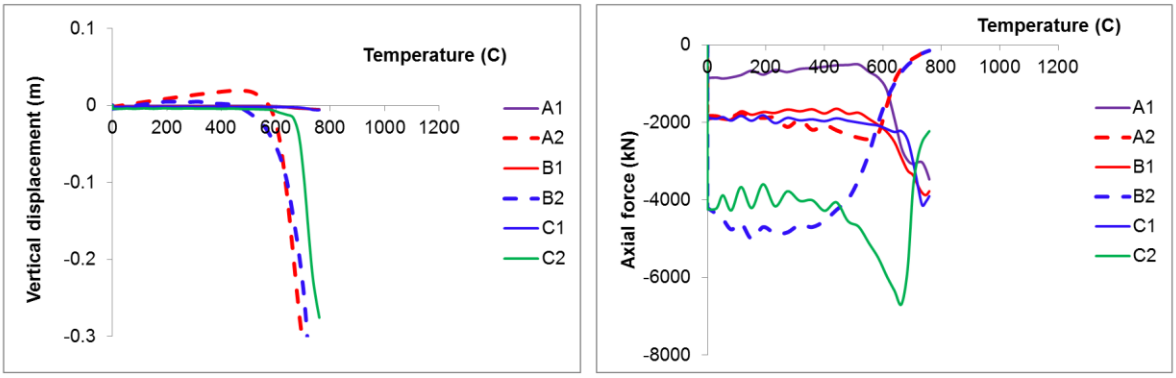
Figure 15 shows vertical displacement and axial forces of the heated columns and adjacent columns for all the scenarios. It shows that in the internal bay scenario, all heated columns buckle at 500°C, this is earlier in temperature terms compared to the edge bay and corner bay fire scenario. The earlier buckling is due to the higher load ratio of the heated columns at the internal bay. However, there is no collapse of the building subject to the internal bay compartment fire. This is because the adjacent columns still have enough capacity to accommodate the additional load previously sustained by the heated column as shown in Figure 15a. Moreover, the loads are uniformly re-distributed to the adjacent columns which avoid the concentration of axial loads in the adjacent columns. This behaviour leads to a relatively small extra load sustained by adjacent columns.

On the other hand, the fire scenarios at the edge bay and corner bay cause collapse of the building. The collapse can be identified when vertical displacement of the top columns continues with no re-stabilisation point. Figure 16 shows the collapse mode of these three scenarios. For edge bay fire, after buckling of the heated columns, significant deflection is noticed in the beams above the heated columns. Furthermore, catenary action of the beams occurs and induces lateral forces on the adjacent columns. Thus, the columns may fail due to the lateral force as well as the increased axial force.

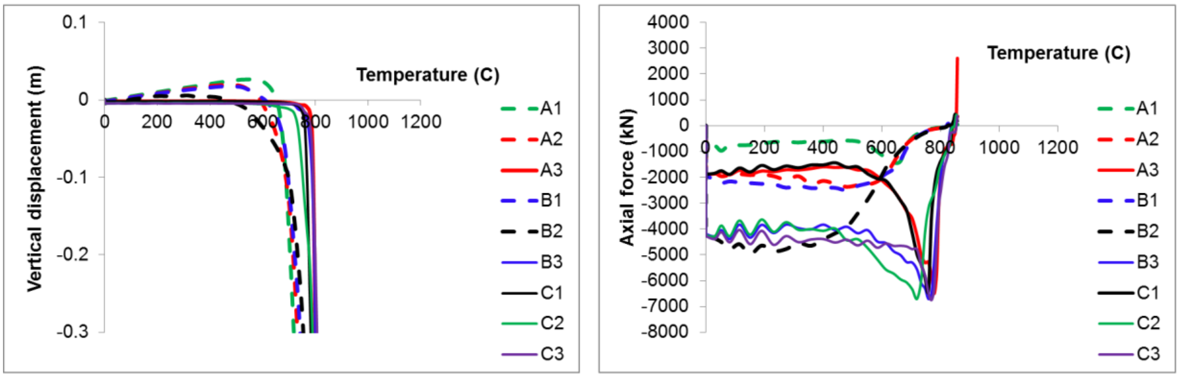
For the case of the corner bay fire scenario, it can be seen that the collapse is triggered by the buckling of the heated column B2 (500°C) followed by column A2 (630°C). After the buckling of the heated columns, the adjacent columns buckle due to the increased of compressive axial force as the load are transferred to them. Then, a domino effect of buckling of columns occur resulting total collapse of the building.



a. Internal bay



b. Edge bay



c. Corner bay

Figure 15: Vertical displacement and axial forces of heated columns and adjacent columns for load of 11 kN/m^2

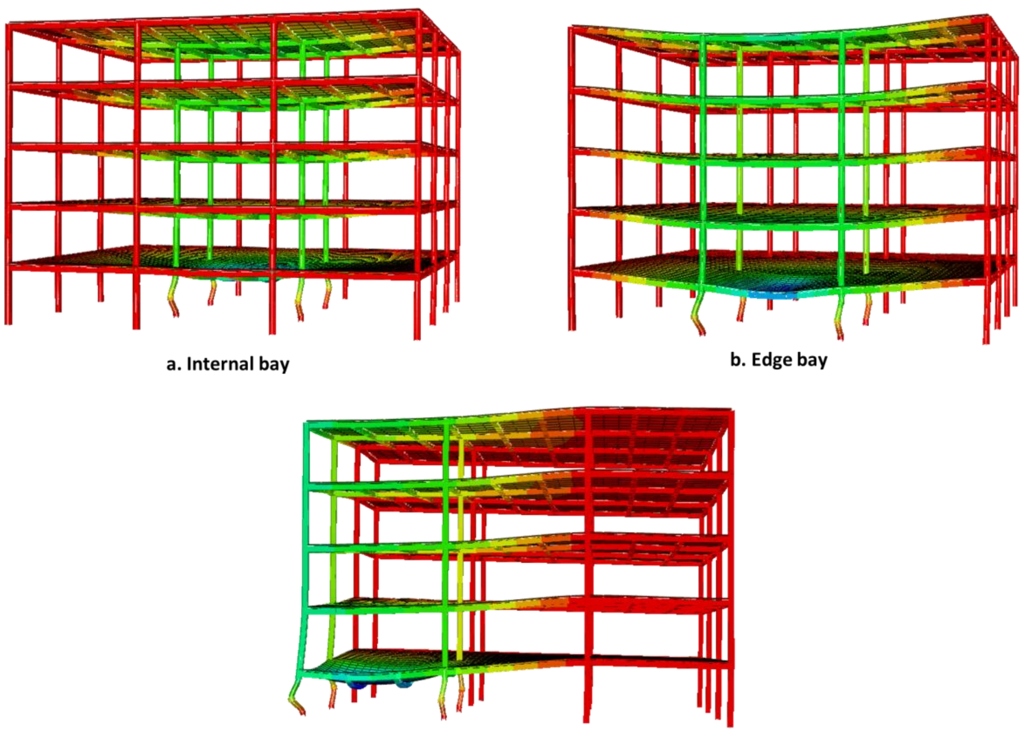


Figure 16: Collapse mode of building frame subject to compartment fire for load of 11 kN/m^2

5. Progressive collapse analysis of earthquake damaged composite building

In this section, two earthquake damage scenarios on the composite building are studied. The first damage scenario is lateral deformation that remains after the earthquake. The second damage scenario is fire compartment damage that can lead to the fire travelling across the floor.

5.1 Influence of residual deformation

This section presents progressive collapse analysis of a composite building with residual lateral deformation that remains after the earthquake. This can be done by performing a three-step procedure. First, the building is subjected to gravity load. Secondly, a non-linear pushover analysis is carried out to simulate the earthquake. In the pushover analysis, the building is pushed incrementally in the X direction (see Figure 1) using a specific lateral load to arrive at a target displacement, the load is then reduced to zero again. Lateral storey forces on the structure are applied in proportion to the product of storey mass and fundamental mode shape [27]. The target displacement is the expected displacement of the building when subjected to a design earthquake. According to ATC 40[27], the structural performance level of the building is divided into three categories, i.e., Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). These performance levels can be represented by the roof displacement at the centre mass as an indication of global stability of the structure. The lateral drift ratio value is less than 0.7%, 0.7% - 2.5% and 2.5% - 5% for a performance level of IO, LS and CP, respectively. In this study, the building is designed to satisfy the CP performance level which is the worst damage limitation.

Figure 17 shows lateral load versus top storey displacement. It can be seen that the building is pushed horizontally until the target displacement reaches 0.73 m (3.65 %), within the CP limit. This pushover analysis results in residual deformation on the top storey (0.48 m) that can be used as the initial step for the fire analysis. Finally, fire analysis of the frame with residual deformation is performed using dynamic explicit analysis. This section aims to investigate the effect of residual deformation on the progressive collapse analysis of the composite building. For the purpose of this study, only the fire scenario at the internal bay compartment and the uniformly distributed load of 5.5 kN/m^2 is considered.

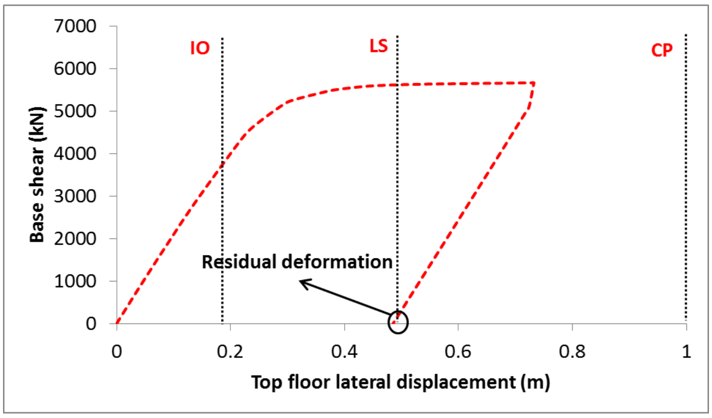


Figure 17: Pushover curve

Figure 18 shows a comparison of axial forces between undamaged columns and damaged columns under the standard fire and parametric fire. It can be seen that the load redistribution path in the damaged building is almost identical to that obtained from the undamaged building. This is due to the fact that the building satisfied the earthquake damage limitation and thus has relatively small permanent deformation. Therefore, neither the load redistribution path nor the fire resistance of the building are considerably affected by the earthquake damage in this particular case.

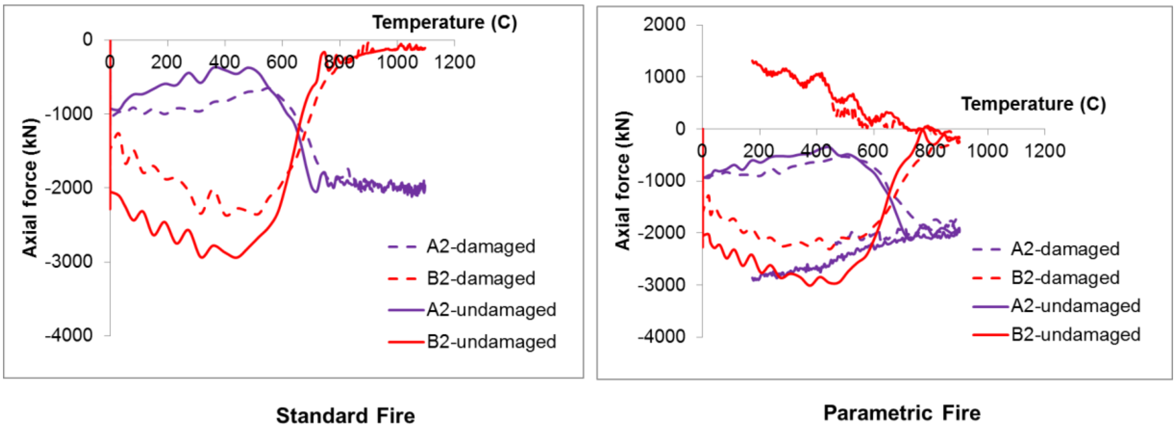


Figure 18: Comparison of axial forces of the undamaged and damaged column

5.2 Influence of travelling fire

This section presents the structural performance of the composite building subject to a fire that travels horizontally among defined zones across the floor. Travelling fire may occur during an earthquake since there is a high possibility of failure in fire containment such as doors and partitions etc. This can lead to fire spreading or traveling throughout the floor until it is successfully extinguished or contained by an intact fire compartment wall.

The floor is divided into nine zones (see Figure 19). From the previous study, it is noticed that fire at the corner bay and edge bay potentially cause collapse of the building. Therefore, the travelling fire in this study considers both fire compartment, corner bay and edge bay. Two different horizontal travelling fire scenarios using two inter-zone time delays are considered. In the first scenario, there are three phases. As seen in Figure 20, the four columns at zone 1 are initially exposed to a parametric fire, shown in Figure 21. After 50 minutes, the fire reaches the maximum temperature of 900°C and just starts to cool. At this point, the fire travels to Zone 2 (two columns heated). After a further 50 minutes, when Zone 2 reaches its maximum and starts to cool, the fire moves from Zone 2 to Zone 3 (two columns heated). The uniformly distributed floor load of 5.5 kN/m^2 is again used in this study.

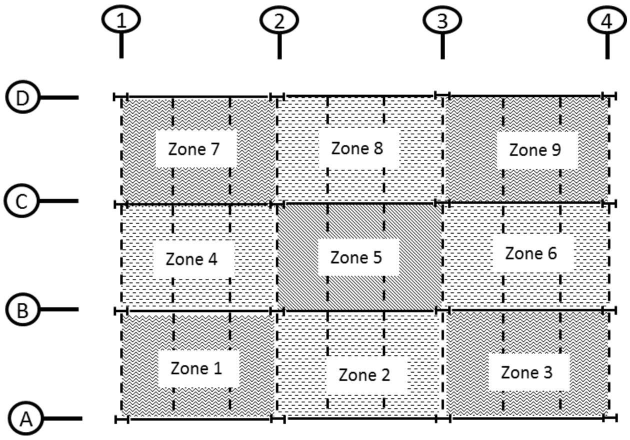


Figure 19: Zone classification considered for travelling fire scenarios

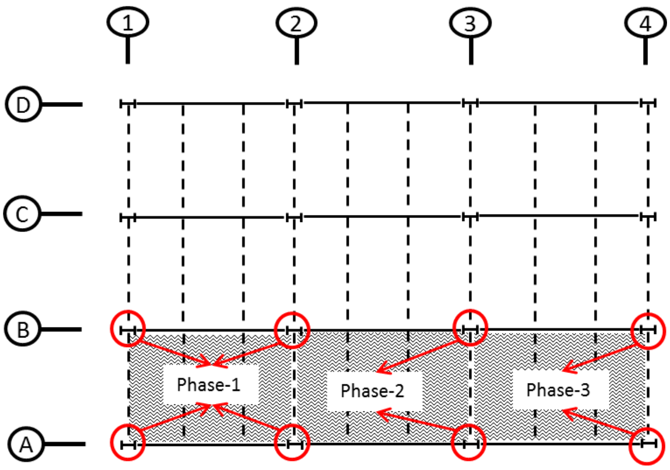


Figure 20: Travelling fire phase for scenario 1

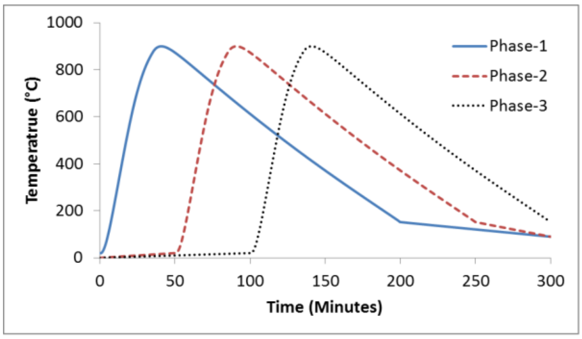


Figure 21: Parametric fire curve for 50 minutes inter-zone time delay

In the second travelling fire scenario, there are only two phases as shown in Figure 22. It is assumed that fire initiates in Zone 1. When Zone 1 reaches its maximum temperature (at 50 minutes), the fire starts from zone 1 then travels simultaneously into Zone 2 and Zone 4. The two travelling fire scenarios above are then repeated with a faster inter-zone time delay of 30 minutes, shown in Figure 23. It should be noted that parametric fires are assumed identical in each phase for simplicity. A different fire scenario with asymmetric distribution of fire is not taken into account in this study, such scenarios may have greater effect than those presented here..

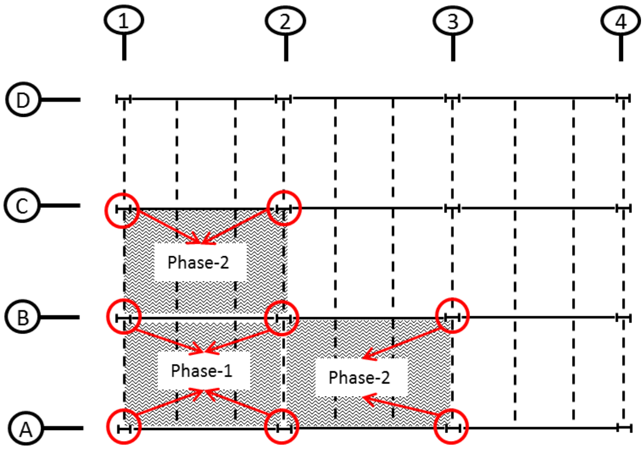


Figure 22: Travelling fire phase for scenario 2

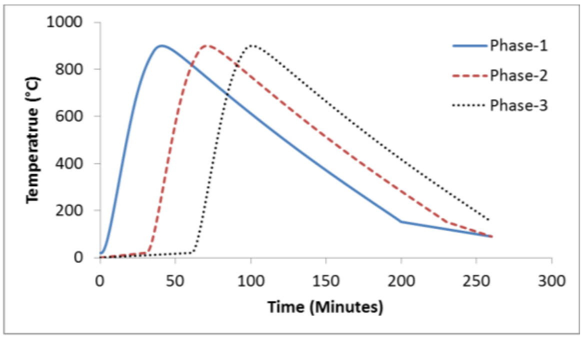


Figure 23: Parametric fire curve for 30 minutes inter-zone time delay

First scenario: Zone 1 – Zone 2 – Zone 3

Figure 24 shows vertical displacement of columns against time subject to the first travelling fire scenario. Only columns at grid line B and C are shown to represent the heated columns and the adjacent cool columns. For the case with inter-zone time delay of 50 minutes, it can be seen that there is transient instability due to buckling of some columns. However, the frame can resist the travelling fire because of its capacity to distribute loads carried by the failed columns to the neighbouring columns. Figure 25 shows the sequence of column failure for the first travelling fire case. When zone 1 is heated (Phase 1), the heated columns experience compression owing to restraints and this decreases axial loads carried by adjacent columns. When buckling of heated columns occurs, the majority of their loads are transferred to adjacent columns. Then a new stable stage is reached until Phase 2 starts at 50 minutes. Similar behaviour is noticed in Phase 2 and Phase 3. This behaviour can be associated with a moving localised fire, when at a certain point in time only one fire compartment is involved.

In contrast, total collapse occurs when the inter-zone time delay escalates to 30 minutes. As seen in Figure 24, the column failure spreads to the surrounding structure from bay to bay. Initially, the frame can be re-stabilised eventually during Phase-1 and Phase-2 as shown in Figure 24. However, there is no sign of re-stabilisation in Phase-3 after all heated columns buckle followed by adjacent columns thus leading to total collapse of the building.

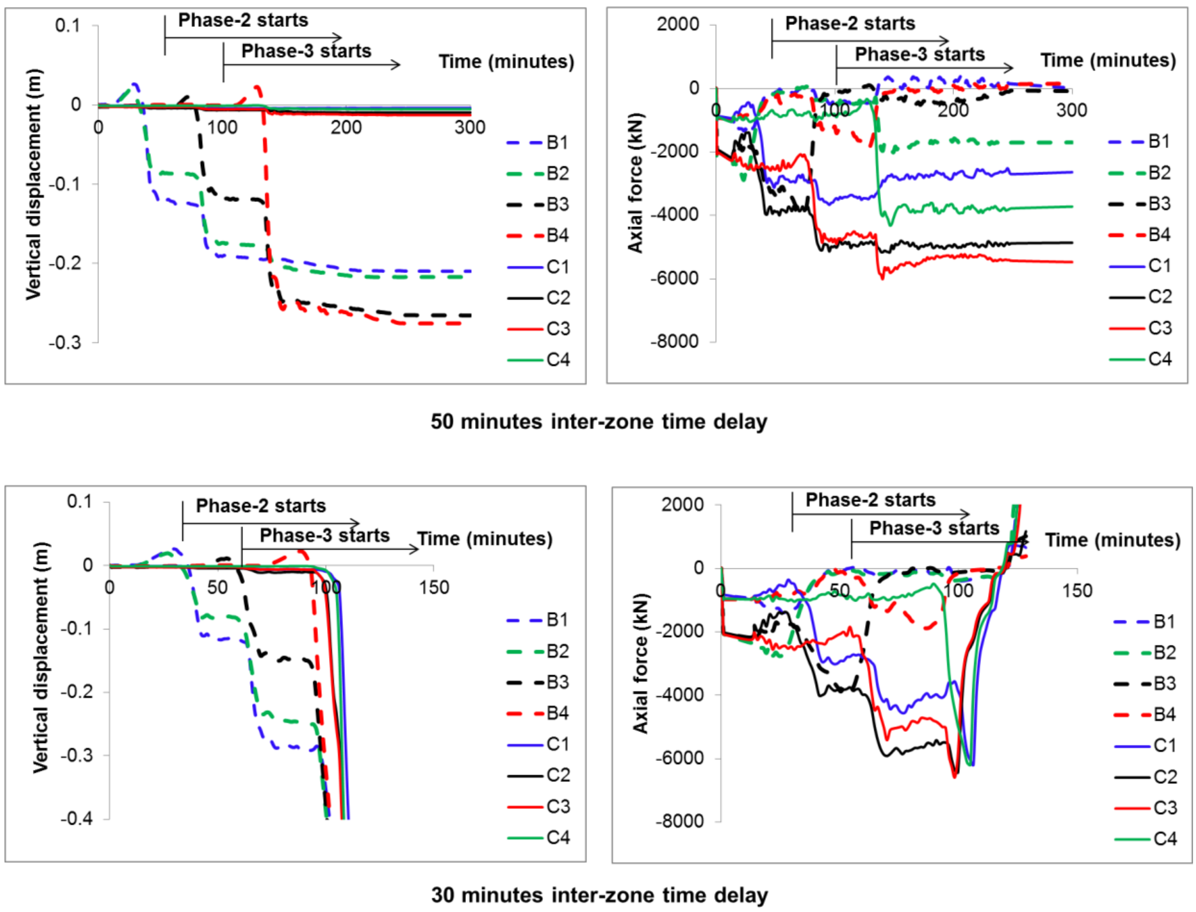


Figure 24: Vertical displacement and axial forces of heated columns and adjacent columns for travelling fire scenario 1

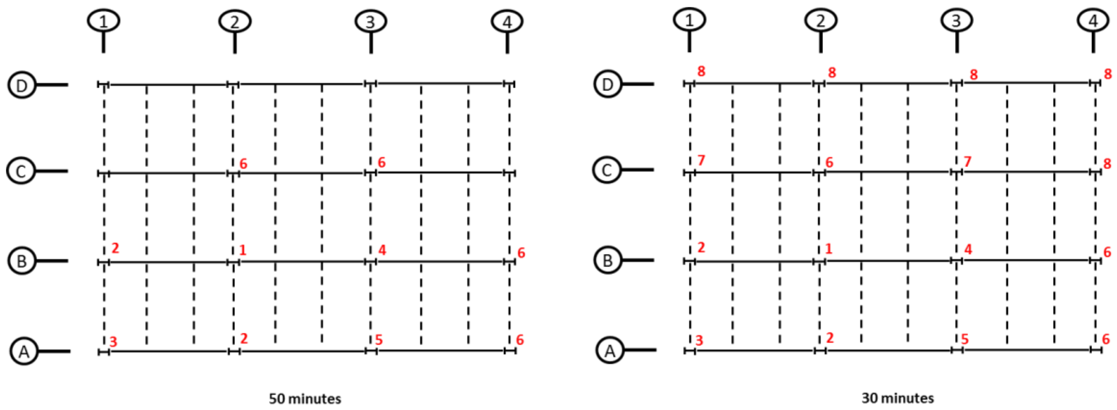


Figure 25: Sequence of failure columns of frames subjected to the travelling fire scenario 1

Second scenario: zone 1 – zone 2&4

Figure 26 shows vertical displacement of columns against time subject to the second travelling fire scenario. The similar behaviour is noticed for both scenarios of 50 minutes and 30 minutes of inter-zone time delay. Total collapse of the building occurs during Phase 2. Figure 27 shows the sequences of column failure for both scenarios. Interestingly, the sequences of column failure are different between the two scenarios. As seen in Figure 27, in the scenario with 30 minutes inter-zone time delay, the collapse occurs earlier (52 minutes) and more columns failed in the same time compared to that of the scenario with 50 minutes inter-zone time delay.

The study above showed that travelling fire and inter-zone time delay greatly affect the collapse resistance of the building. This demonstrates the importance of fire containment particularly during an extreme event such as an earthquake. The failure of fire containment can lead to the fire travelling across the floor resulting in wholesale collapse of the building. Therefore, a range of travelling fire scenarios must be considered based on the compartment condition to guarantee that the building can withstand the 'worst case scenario'.

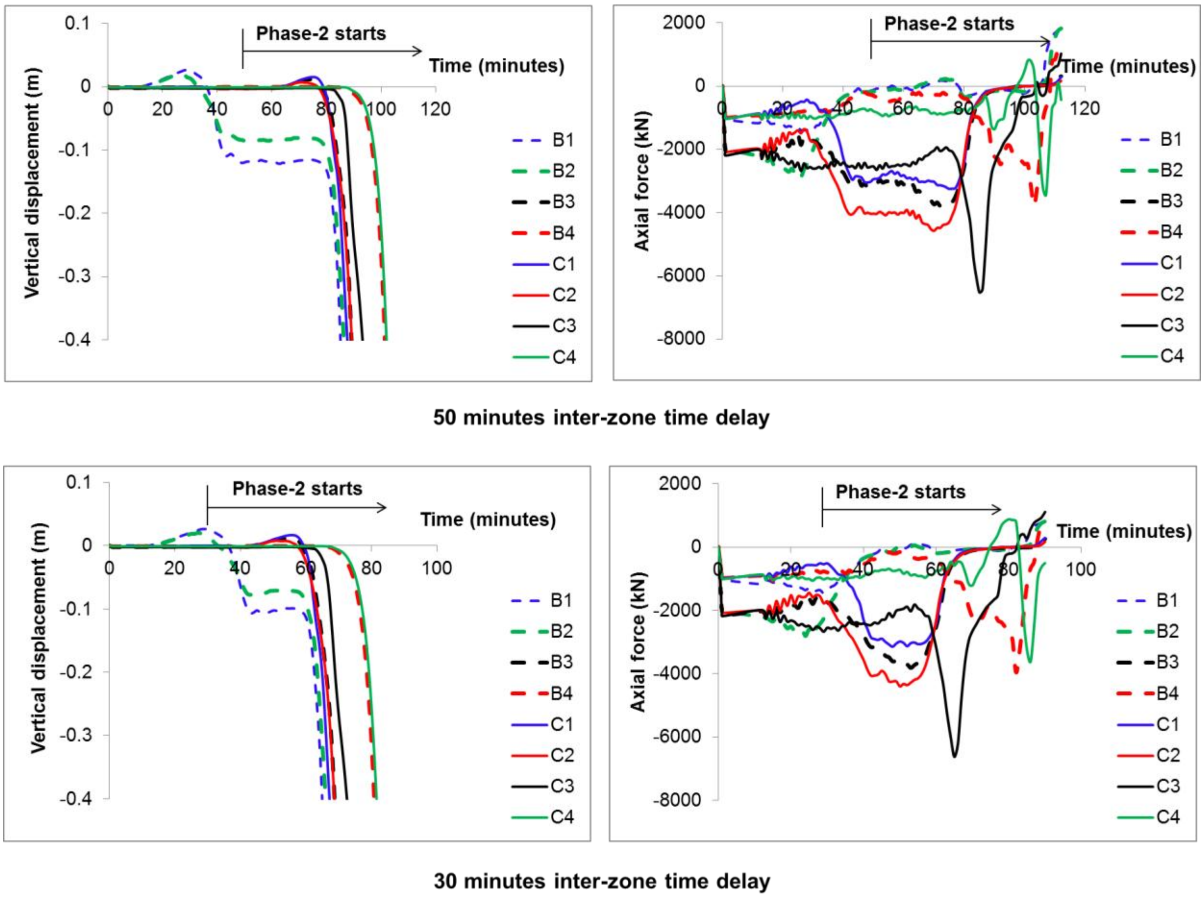


Figure 26: Vertical displacement and axial force of heated columns and adjacent columns for travelling fire scenario 2

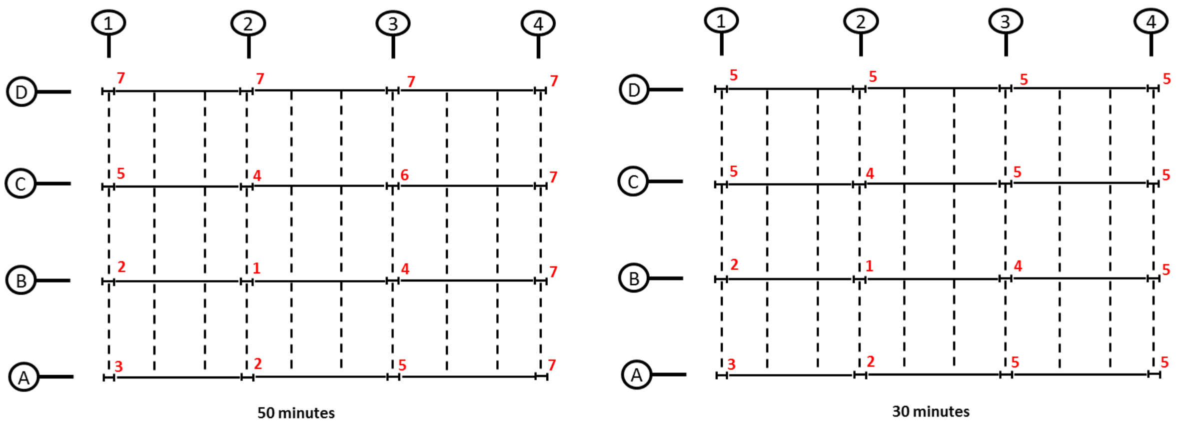


Figure 27: Sequence of column failure for frames subjected to the travelling fire scenario 2

6. Conclusions

This study presented the progressive collapse analysis of an earthquake damaged composite steel frame subjected to fire. The effects of fire location and earthquake damage on load redistribution paths and collapse modes were investigated. From the analyses, the following can be concluded:

- The 3D models described can simulate load redistribution between columns as a result of heating. The loads previously supported by the heated columns are redistributed to the adjacent columns along two horizontal directions. This cannot be accurately captured in 2D models as used in most previous studies.
- The load ratio plays an important role in the collapse resistance of the building. A lower load ratio (0.3) can prevent the building from collapse; no matter if the fire occurs at the internal, edge or corner bay of the frame. Although the heated columns suffer buckling, the loads can be resisted by the adjacent columns through the steel beams. The load ratios of the adjacent columns in this case increase up to 0.6 and do not suffer buckling. Thus, total collapse of the building can be prevented.
- The frame with a different location of heating can generate different collapse mechanisms. The frame with four columns heated in the internal bay has higher collapse resistance compared to four columns heated in the corner bay and edge bay. For the internal bay scenario, although the heated columns buckle earlier due to the higher load ratio, the load previously sustained by the heated column can be safely transferred to the adjacent columns (lower road ratio) without causing buckling. This demonstrates that the failure of individual elements may be acceptable as long as overall structural collapse can be prevented.
- During the cooling phase, the heated columns move downward due to thermal contraction resulting in further damage. There is a possibility that collapse may occur during the cooling phase as extra loads are transferred to adjacent columns. Hence, the cooling phase should be considered in the robustness analysis of the building (and also during fire-fighting or search operations).
- In the frame examined, residual deformation due to the design earthquake has a minor effect on the load redistribution or progressive collapse mechanism. This is due to the fact that the building satisfied the earthquake damage limitation and thus has relatively small permanent deformation.
- Based on the analysis results, it is shown that total collapse may occur when the fire travels horizontally across the floor. The travelling fire scenario and inter-zone time delay significantly affect the collapse resistance of the building. This shows the importance of fire containment to prevent building collapse in a multi-hazard event such as fire following an earthquake.

Acknowledgement

The authors would like to thank the Indonesia Endowment Fund for Education (LPDP) for funding this PhD research.

References

[1] C. Scawthorn, J. Eidinger, A. Schiff, Fire following earthquake, Tech. Counc. Lifeline Earthq. Eng. Monogr. No. 26. Am. Soc. Civ. Eng. (2005).

[2] B. Faggiano, Fire after earthquake, Proc. WG1 Meet. Urban Habitat Constr. Under Catrastophic Events. (2007).

[3] BS EN, BS5950 Part 8: Code of Practice for Fire Resistant Design, Br. Stand. Institute, London, UK. (1990).

[4] C.G. Bailey, T. Lennon, D.B. Moore, The behaviour of full-scale steel-framed buildings subjected to compartment fires, Struct. Eng. 77 (1999).

[5] M. Gillie, A.S. Usmani, J.M. Rotter, A structural analysis of the first Cardington test, J. Constr. Steel Res. 57 (2001) 581–601. doi:10.1016/S0143-974X(01)00004-9.

[6] G.L.F. Porcari, E. Zalok, W. Mekky, Fire induced progressive collapse of steel building structures: A review of the mechanisms, Eng. Struct. 82 (2015) 261–267. doi:10.1016/j.engstruct.2014.09.011.

[7] R. Sun, Z. Huang, I.W. Burgess, Progressive collapse analysis of steel structures under fire conditions, Eng. Struct. 34 (2012) 400–413. doi:10.1016/j.engstruct.2011.10.009.

[8] E. Talebi, M. Md Tahir, F. Zahmatkesh, A.B.H. Kueh, Comparative study on the behaviour of buckling restrained braced frames at fire, J. Constr. Steel Res. 102 (2014) 1–12. doi:10.1016/j.jcsr.2014.06.003.

[9] B. Jiang, G.Q. Li, L. Li, B. a. Izzuddin, Simulations on progressive collapse resistance of steel moment frames under localized fire, J. Constr. Steel Res. 138 (2017) 380–388. doi:10.1016/j.jcsr.2017.05.018.

[10] A. Agarwal, A.H. Varma, Fire Induced Progressive Collapse of Steel Building Structures: The Role of Interior Gravity Columns, Eng. Struct. 58 (2014) 129–140. doi:10.1016/j.engstruct.2013.09.020.

[11] J. Jiang, G. Li, Progressive collapse analysis of 3D steel frames with concrete slabs exposed to localized fire, Eng. Struct. (2016). doi:10.1016/j.engstruct.2016.07.041.

[12] R. Suwondo, M. Gillie, L. Cunningham, C. Bailey, Effect of earthquake damage on the behaviour of composite steel frames in fire, Adv. Struct. Eng. (2018). doi:10.1177/1369433218761138.

[13] CEN, Eurocode 3 Design of steel structures - Part 1-1: General rules and rules for buildings, Br. Stand. Institute, London, UK. (2005).

[14] CEN, Eurocode 4: Design of composite steel and concrete structures Part 1-1: General rules and rules for building, Br. Stand. Institute, London, UK. (2004).

[15] CEN, Eurocode 8 — Design of structures for earthquake resistance —Part 1: General rules, seismic actions and rules for building, Br. Stand. Institute, London, UK. (2004).

[16] CEN, Eurocode - Basis of structural design, Br. Stand. Institute, London, UK. 3 (2002) 89. doi:10.1680/cien.144.6.8.40609.

[17] CEN, Eurocode 2: Design of concrete structures - Part 1-2 General rules - Structural fire design, Br. Stand. Institute, London, UK. (2004).

[18] CEN, Eurocode 3 : Design of steel structures Part 1-2: General rules — Structural fire design, Br. Stand. Institute, London, UK. (2005).

[19] ISO, ISO 834-1975: Fire resistance test-elements of building construction, ISO 834-1975, 1975.

[20] CEN, Eurocode 1: Actions on structures. Part 1–2: General actions. Actions on structures exposed to fire, Br. Stand. Institute, London, UK. (2002).

[21] Abaqus, ABAQUS/Standard analysis user’s manual v14.2, SIMULIA. (2014).

[22] J.Y.R. Liew, Survivability of steel frame structures subject to blast and fire, J. Constr. Steel Res. 64 (2008) 854–866. doi:10.1016/j.jcsr.2007.12.013.

[23] G.R. Johnson, W.H. Cook, Fracture characteristics of three metals subjected to various strains, strain rates, temperatures and pressures, Eng. Fract. Mech. 21 (1985) 31–48. doi:10.1016/0013-7944(85)90052-9.

[24] B. Jiang, G.-Q. Li, L. Li, B. a. Izzuddin, Experimental Studies on Progressive Collapse Resistance of Steel Moment Frames under Localized Furnace Loading, J. Struct. Eng. 144 (2017) 1–10. doi:10.1061/(ASCE)ST.1943-541X.0001947.

[25] B.R. Kirby, The behaviour of multi-storey steel frame building in fire - Technical Report, Br. Steel. (1998).

[26] M. Gillie, Analysis of heated structures: Nature and modelling benchmarks, Fire Saf. J. 44 (2009) 673–680. doi:10.1016/j.firesaf.2009.01.003.

[27] ATC, ATC-40: Seismic evaluation and retrofit of concrete buildings, Appl. Technol. Counc. 1 (1996) 334. doi:10.1193/1.1586093.