Improving the robustness of steel frame structures under localised fire conditions

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Improving the robustness of steel frame structures under localised fire conditions

Abstract

Purpose: The robustness of building structures in a fire has recently drawn wide attention. This study presents the progressive collapse analysis of steel frame building structures under localised fire. The main objective of this study is to propose methods to enhance the structural collapse resistance of such structures in fire.

Design/methodology/approach: A modelling method was developed and validated against both experimental and analytical studies. Then, a series of robustness analyses were performed to investigate the interaction among the members and the pattern of load distribution within the structures. These analyses show that lateral resistance and load redistribution have a vital role in the robustness of the building. Thus, two approaches have been adopted to enhance the robustness of the focussed steel frame during a fire.

Findings: It is found that increased size of floor beams and vertical bracing systems are effective measures in preventing whole structure collapse. The larger beam section is able to prevent catenary action so that the load in the failed columns can safely transfer to the adjacent columns without buckling. On the other hand, the bracing system improves the lateral resistance that can accommodate the lateral force when catenary action occurs in the beam

Originality/value: All previous studies have focused on the collapse mechanism of steel frame structures. However, the parameters affecting the structural robustness in a fire have not yet been explored. To address this gap, this study adopted numerical modelling to undertake parametric studies to identify effective methods to improve the robustness of such structures under fire conditions.

Keywords: robustness, collapse resistance, progressive collapse, fire engineering, steel structures

1. Introduction

Standard design practice dictates that building structures are expected to maintain stability for a certain period during an accidental event such as fire, to assure an acceptable level of safety and to minimise economic loss. The ability of structures to sustain their stability without progressive collapse after local failure is often termed as robustness. The collapse of World Trade Center buildings in 2001 attracted attention from structural engineers since initial local damage followed by a fire triggered the collapse of the whole structure (Usmani, Chung and Torero, 2003). To date, steel framing is widely used in high-rise buildings since the steel structure possesses strength and ductility, which give advantages in regard to seismic performance for example. However, the material properties of steel reduce significantly during a fire. Thus, the robustness of steel frames under fire conditions needs particular attention.

In the traditional approach, the resistance of the structure in fire is evaluated based on the behaviour of an individual element under a standard fire test (BS EN, 1990). However, structural engineers are beginning to adopt the performance-based design, in which the entire structures are analysed integrally in structural safety design. The performance-based design requires an efficient tool to investigate and apply alternative fire protection strategies. Intensive research on the performance-based approach of structural robustness in a fire has been conducted, including experiment and analytical studies. Sun et al. (2012) adopted a static-dynamic method using Vulcan software to study the collapse resistance of steel frame structures during a fire. This method allows an analysis to continue further the temporary instabilities, which may trigger singularities in the static method.

Kim and Kim (2009) compared the linear static and the nonlinear dynamic analysis procedure using an alternate path method recommended by the US General Service Administration(GSA, 2003) and the Department of Defence(UFC-DoD, 2005) guidelines. The use of linear analysis produces relatively more conservative results for the progressive collapse of the structure. By contrast, more accurate results can be obtained from nonlinear dynamic analysis. Recent advancement in computer technology has made it possible to adopt nonlinear dynamic analysis for the evaluation of structural robustness without much difficulty.

Jiang et al. (2017) conducted experimental studies on the progressive collapse resistance of steel frame structures exposed to local fire. The results revealed that a steel column may fail progressively in a static way or suddenly in a dynamic way, depending on the restraint condition provided by beams in the frame and the load intensity. Based on this study, Jiang et al. (2017) performed a dynamic analysis to simulate the collapse mechanism of steel frames under local fire. The simulations showed that ignoring the dynamic effect may produce lower displacement of the structure and lead to less conservative design.

Suwondo et al. (2019) performed progressive collapse analysis of composite building under fire following an earthquake. Initially, the analysis of the undamaged building was performed to study load redistribution path and members' interaction. Then, the influence of residual deformation due to earthquake damage was investigated. This study showed that the residual deformation has a small influence on the load redistribution and members' interaction within the building.

All of the studies mentioned above have focused on the collapse mechanism of steel frame structures. However, the parameters affecting the structural robustness in a fire have not yet been explored. To address this gap, this study extends previous work by Suwondo et al. (2019) on modelling progressive failure of multi-storey steel moment frames exposed to fire. The modelling approach is adopted to undertake parametric studies to identify effective methods to improve the robustness of such structures under fire conditions. Thus, the main purposes of this study are:

1. Study the robustness of the steel frame structures subject to fire to investigate the mechanism of load redistribution and member interaction within the structures.
2. Propose methods of improvement to enhance the structural robustness of steel frame structures in fire.

2. Generic frame

A 2D frame representing a medium-rise building adopted from the SAC Steel Project, FEMA-355 (FEMA, 2000) with slight modifications forms the focus of this study. The steel frame is designed in accordance with the AISC Steel Construction Manual (AISC, 2015). The 2D frame is considered since a previous study (Quiel and Garlock, 2008) shows that the 2D frame represents the behaviour of steel frame in a fire with reasonable accuracy. However, when a large redistribution load is required, the 3D frame should be considered.

Figure 1 shows the configuration of the steel moment frame. For simplicity, W14x257 and W24x62 sections are selected for columns and beams for all floors, respectively. The columns have fixed connections at ground level. According to Eurocode 3(CEN, 2005b), the cross-sections of beams and columns can be classified as Class 1 so that the local buckling has a minor influence on the ultimate capacity of the member. In the present study, the concrete floor slab does not act compositely with the steel beams, this simulates the case of non-composite pre-cast slabs for example. Moreover, all beam-to-column connections are set as fixed to allow moment frame action. All structural members are assumed as unprotected in this study to represent the extreme scenario such as blast or earthquake.

A992 steel is used for both columns and beams. Elastic-perfectly plastic behaviour is considered for the steel material. The total gravity load at the fire limit state (1.0 dead load + 0.5 live load) is taken as 62.5 kN/m. This load level results in a load ratio for columns and beams of 0.4 and 0.8, respectively. The coefficient of thermal expansion is taken as 1.4 x 10^{-5} C^{-1}(BS EN, 1990). The reduction of steel properties at elevated temperature is adopted according to Eurocode 3(CEN, 2005a) model, as presented in Figure 2.

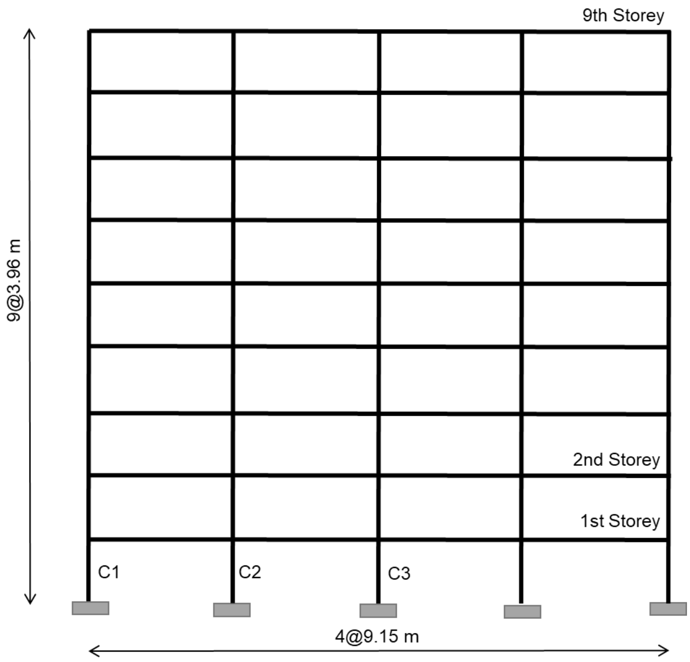


Figure 1: Configuration of the 2D steel moment frame

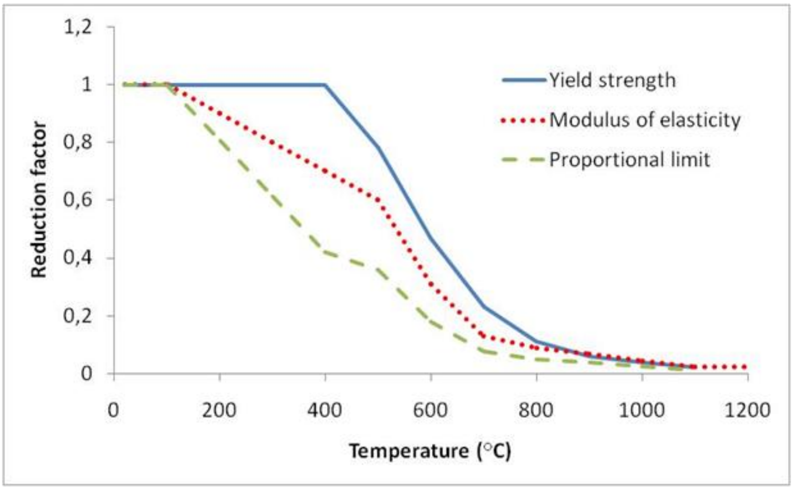


Figure 2: Reduction factors for steel at elevated temperature(CEN, 2005a)

3. Fire analysis

Recent design methods permit the development of finite-element models to analyse building performance under fire conditions. The models are exposed to temperature-time curves (design fire), and then the behaviour of the structure can be investigated. The Eurocode Parametric Fire (CEN, 2002), as shown in Figure 3, is selected to simulate the fire event in this study since it considers the heating as well as the cooling phase. The fire load of 511 MJ/m^2 is used, which is for office buildings, according to Eurocode 3 (CEN, 2002). The opening (ventilation) factor and thermal inertia of compartments are assumed as 0.06 m$^{0.5}$ and 1470 Ws$^{0.5}$/m^{2}K, respectively.

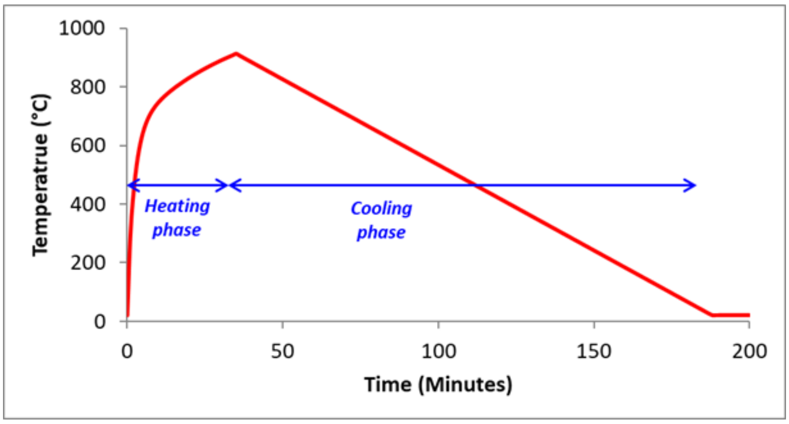


Figure 3: The Eurocode Parametric Fire curve

In any design of steel structure to resist fires, it is essential to determine the steel temperature. The calculation of steel temperature is handled differently depending on whether the steel is protected or not. For unprotected steel, it is possible to assume that the temperature of the steel structure is generally uniform due to thin steel sections. Thus, a simple energy approach can be used to determine the change in temperature of steel sections.

The method is generally based on the principle that the heating exposing the steel over the surface area in a small-time step Δt *(s)* is equivalent to the heat required to increase the temperature of the steel by ΔT_s *(C)* considering that the steel section is a lumped mass at the uniform temperature, so that

Heat entering = Heat to raise temperature

$$q''F\Delta t = \rho_s\, c_s\, V\, \Delta T_s \tag{1}$$

where ρ_s is the steel density (kg/m^3), c_s is the specific heat of steel (J/kgK), ΔT_s is the change in steel temperature in the time step and q'' is the heat transfer at the surface (W/m^2), given by

$$q'' = h_c(T_f - T_s) + \sigma\varepsilon(T_f^4 - T_s^4) \tag{2}$$

where h_c is the convective heat transfer coefficient (W/m^{2}K), T_f is the environment temperature during a fire and T_s is the steel temperature, σ is the Stefan-Boltzmann constant taken as 56.7 x 10^{-12} kW/m^{2}K^4, ε is the resultant emissivity.

These equations can be re-arranged to give:

$$\Delta T_s = \frac{F}{V}\frac{1}{\rho_s c_s}\{h_c(T_f - T_s) + \sigma\varepsilon(T_f^4 - T_s^4)\}\Delta t \tag{3}$$

The quantity of F/V is known as the section factor of the steel section and is given in standard section tables, or it can be calculated manually, where F is the surface area of a unit length of the member (m^2), and V is the volume of steel in a unit length of the member (m^3). As shown in Figure 4, in the case of a steel member supporting a concrete slab, the member can be assumed not to be heated through the surface that contacts the slab.

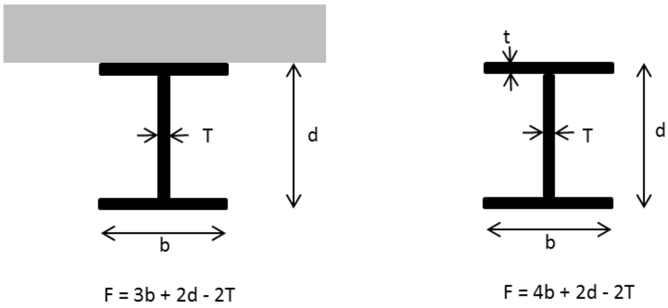


Figure 4: Typical perimeter of section factors

The equation above can be to calculate the temperature of the steel section for gas temperatures (standard fire or parametric fire) providing a sufficiently small time step is used. Eurocode 3 (CEN, 2005a) suggests a maximum time step of 30 s and the section factor *F/V* of no less than 10 m^{-1}. Previous studies (Gamble, 1989; Kay, Kirby and Preston, 1996; Rackauskaite, Kotsovinos and Rein, 2017; Suwondo *et al.*, 2021) have revealed that this method can offer a reliable prediction of steel beam temperatures. Figure 5 presents the temperatures at the steel structures exposed to gas temperature (parametric fire).

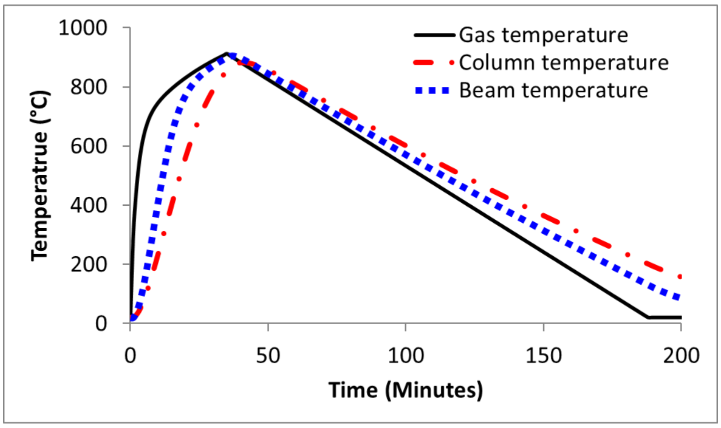


Figure 5: Steel temperature

4. Numerical model

The multi-purpose finite element software ABAQUS v6.14 is used to model and analyse the steel frame. ABAQUS provides a number of elements such as solid, shell and beam with different characteristics. The appropriate method of modelling structures should be determined based on the design objectives(Chen and Wang, 2012). For instance, a detailed solid element may be required to capture local buckling (Karami *et al.*, 2021; Naghsh *et al.*, 2021; Shishegaran *et al.*, 2021).

In this study, steel beams and columns are modelled using 1-D beam elements (ABAQUS library code B21) consisting of two-node linear beam elements in space with a mesh size of 0.5 m. The occurrence of local buckling is not expected since at least Class 3 cross-section with a smaller value of parameter ε is selected (Yang, Lee and Chan, 2006; Correia and Rodrigues, 2012). The beam-to-column connections are modelled as rigid.

There are two solution methods in ABAQUS that can be used in this simulation. The first solution is a general static analysis. This method is still applicable until the determinant of the stiffness matrix equals zero or is negative. However, the main disadvantage of this solution is that it is difficult to solve the non-convergence problem when complicated contacts are encountered. Moreover, dynamic effects may occur when the column rapidly buckles, as explained in the literature review. The second method is an explicit dynamic analysis. This solution can overcome the non-convergence problems due to complicated contact, but it is a very time-consuming process since very small-time steps are required. In the light of the above points, in this study, explicit dynamic analyses are adopted to investigate the dynamic effects during the failure mechanism of the columns and to prevent numerical convergence difficulties. The real running time of the analysis can be scaled down to save computational cost.

To consider the dynamic effects, in this study, Raleigh damping is taken as 5% viscous damping in the dynamic analysis. The Johnson-Cook damage model (1985) was implemented to represent possible strain rate effects in the steel, the equation of which is:

$$\varepsilon_d = [D_1 + D_2 exp(D_3\sigma^*)][1 + D_4 \, ln\, \varepsilon^*][1 + D_5 T^*] \qquad (5)$$

where ε_d is the equivalent plastic strain; the five constants D_1-D_5 are 0.05, 3.44, -2.12, 0.002, and 0.61, respectively; σ^* is a function of stress triaxiality defined as $\sigma^* = \sigma m/ \bar{\sigma}$; ε^* is strain rate; and T^* is homogenous temperature..

To validate the modelling approach used in this work, an experimental study is chosen (Figure 6), which has been previously conducted by Jiang et al.(2017a) and subsequently numerically modelled by Jiang et al.(2017b). Rectangular hollow steel sections of 50x30x3 mm and 60x40x3 mm are selected for columns and beams, respectively. The vertical displacement at the top of columns is investigated. The comparison in Figure 7 shows good agreement with the previous studies and confirms that the result produces an acceptable level of accuracy. It is also noticed that the scaled time influences the buckling temperature of the column. A running time of 9 s (scale 1-h to 6s) matched well with the previous experiment data as well as numerical study with approximately 0.5% error.

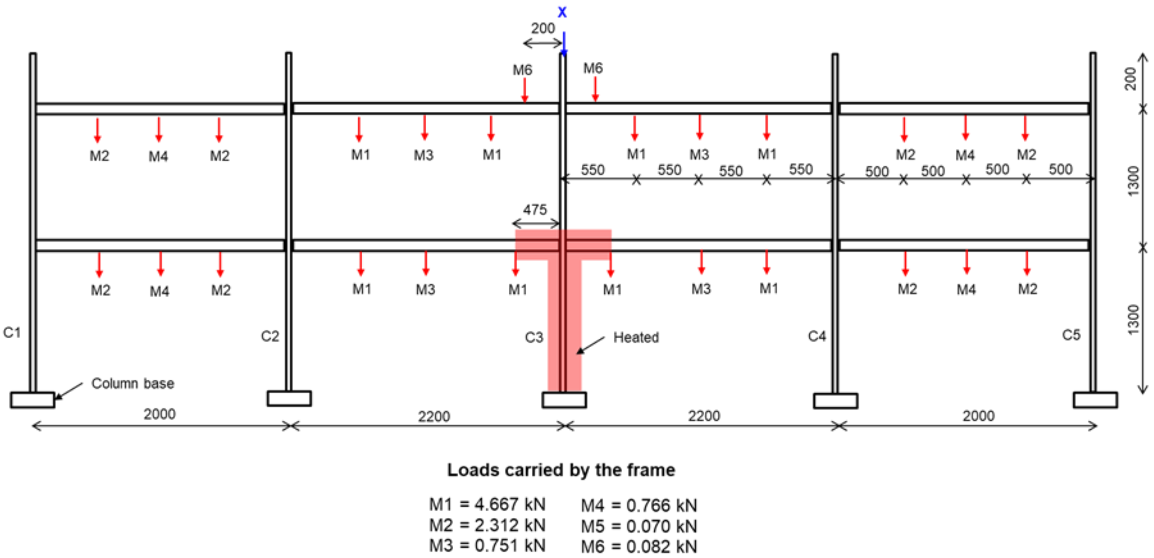


Figure 6: The test frame conducted by Jiang et al.(2017)

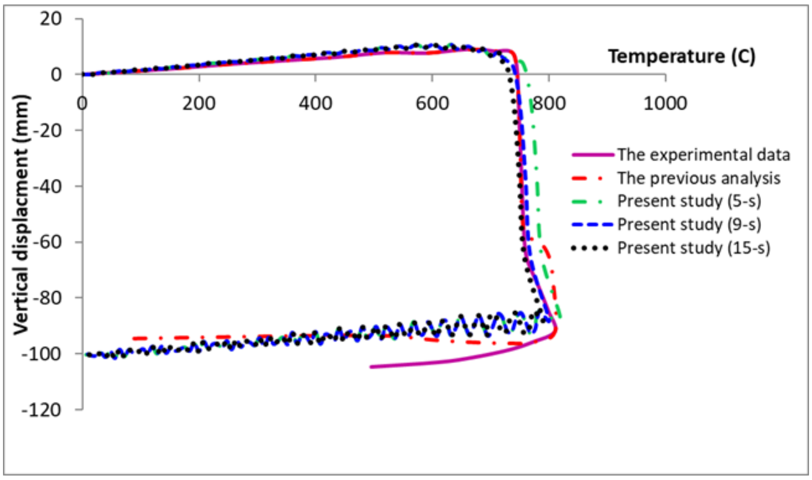


Figure 7: The vertical displacement at the top of column X

In addition, it is understood that mesh size is a key factor in the finite element analysis. Therefore, mesh sensitivity study is carried out with different mesh sizes (from 0.1 m, 0.3 m, to 0.5 m). Figure 8 shows the vertical displacement against mesh size. It can be seen that the vertical displacement is relatively not sensitive to the mesh sizes. This study confirms that the mesh size of 0.5 m can be used in the model with acceptable accuracy.

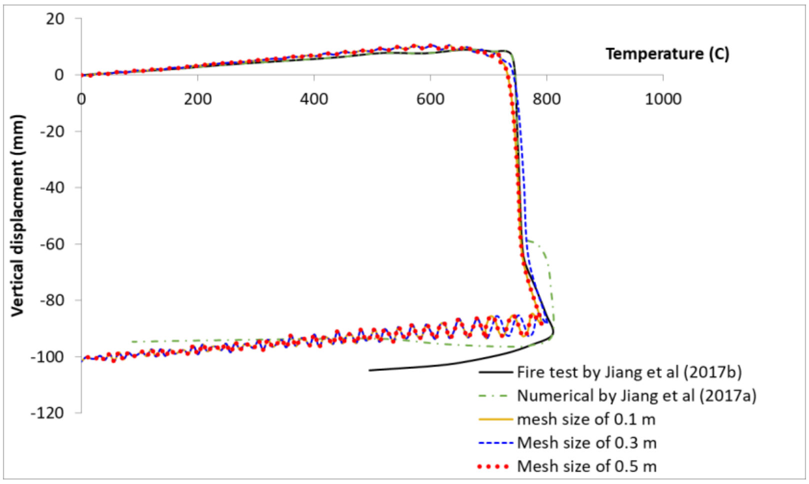


Figure 8: Comparison of vertical displacements with different mesh sizes

5. Robustness analysis of the focussed steel frame structure

In this section, a series of parametric analyses are conducted using the modelling approach described above. Firstly, the frame in Figure 1 with only one column heated is studied. Following this, two modifications to the frame are made. Firstly, the beam section sizes are increased, secondly, vertical diagonal bracing is introduced to investigate the impacts on the structural robustness of the frame under a localised fire situation.

The heated column C1, C2 and C3, as indicated in Figure 1 is considered in this study, with each column being heated up to failure. This study examines the failure of certain columns that potentially cause collapse of the steel frame. Therefore, only the ground floor fire scenarios are considered since the ground floor columns have the highest load ratios which is defined as the ratio of the applied load to the capacity of the structural member.

Column C1 heated

The axial displacements and forces occurring in the columns against temperature are presented in Figure 9. Initially, the heated column C1 enlarges upward owing to thermal expansion. Thus, the axial force at the column C1 increases gradually, resulting in reducing the axial force at the adjacent column C2. At 680°C, the heated column C1 starts to buckle and loses its bearing capacity. The load previously carried by the heated column C1 is distributed to the adjacent column C2, resulting in sudden increase of the axial force at column C2. As shown in Figure 10, the collapse occurs in the edge bay only. This is because the column C2 can safely receive the load from the heated column without buckling. Thus, the local failure can be controlled to prevent the total collapse.

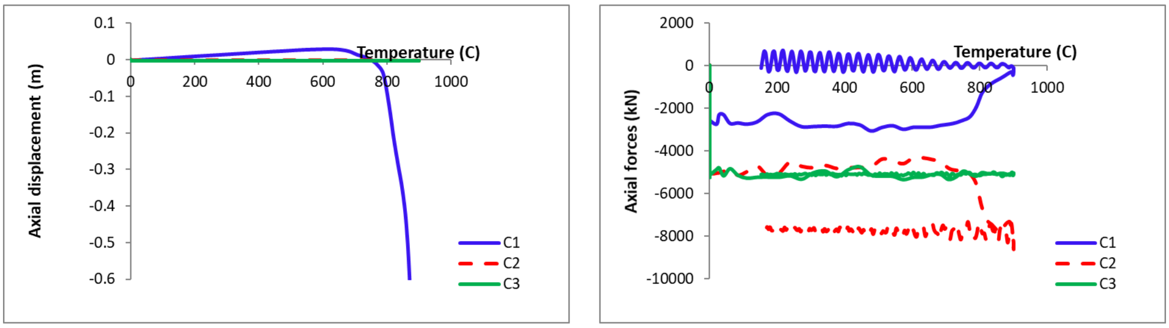


Figure 9: Vertical displacement and axial force of the columns for the case of column C1 heated

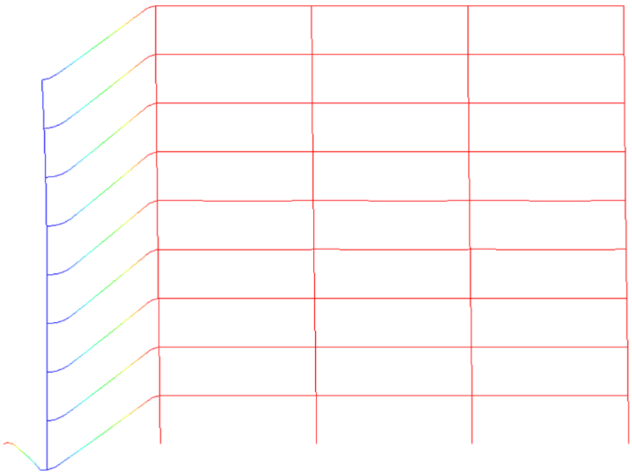
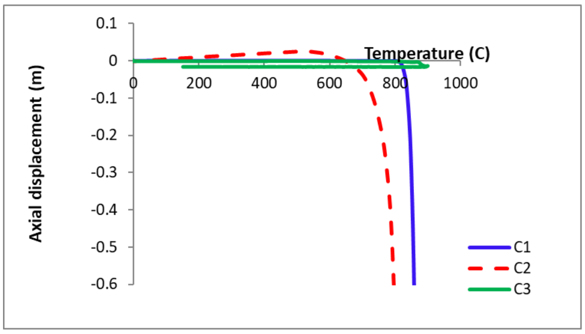


Figure 10: Failure mode for the case of column C1 heated

Column C2 heated

Similar behaviour can be observed for the case of column C2 heated. As shown in Figure 11, the heated column C2 buckles earlier (at 550°C) compared to the previous case. The load is then redistributed to the adjacent column C1 and C3. As illustrated in Figure 12, the failure at column C2 causes significant lateral and vertical forces to be transmitted to the adjacent columns. In this case, column C1 buckles due to lack of lateral resistance. On the contrary, column C3 does not buckle since the lateral support of column C3 via the floor beams to the right is sufficiently strong to sustain the huge catenary force in the beam to the left.



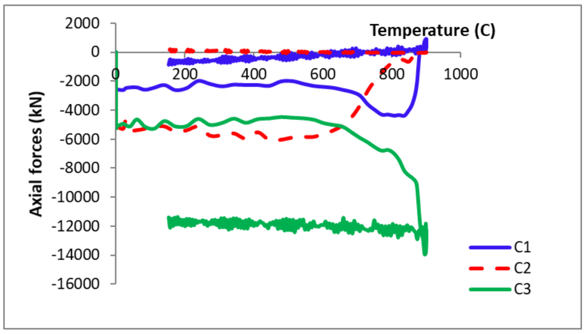


Figure 11: Vertical displacement and axial force of the columns for the case of column C2 heated

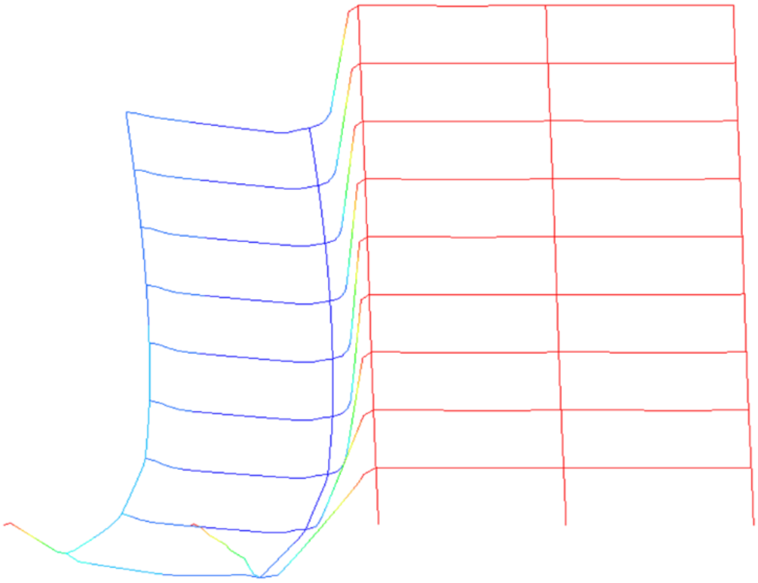
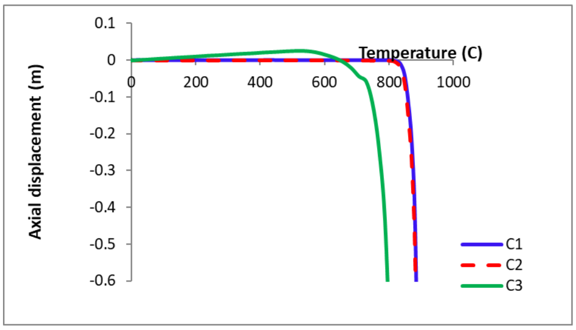


Figure 12: Failure mode for the case of column C2 heated

Column C3 heated

In this case, the internal column C3 is exposed to a fire. Similar to the case of column C2 heated, the heated column C3 buckles at about 550°C, as shown in Figure 13. The collapse mode shown in Figure 14 indicates that the catenary forces in the beam increase due to the large deformation. Therefore, since the adjacent columns do not have enough lateral support, the columns buckle progressively, which results in the total collapse of the steel frame.

This study shows that collapse mechanism is influenced by the location of heated columns. The heated column C1 and C2 generates collapse in part of the frame. Meanwhile, the heated column C3 is the worst scenario as the whole frame collapses. Furthermore, it can be observed that the catenary action in the beam causes a lateral force to the adjacent columns, resulting collapse of the building. This demonstrates that the lateral resistance has a vital role in the robustness of the building.



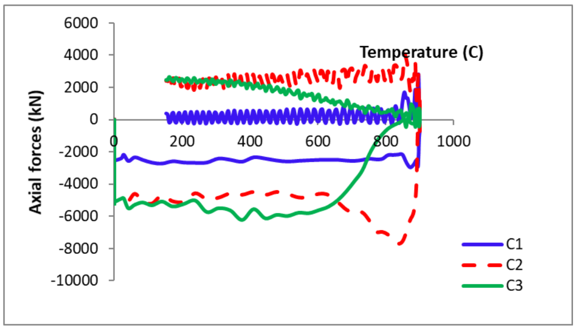


Figure 13: Vertical displacement and axial force of the columns for the case of column C3 heated

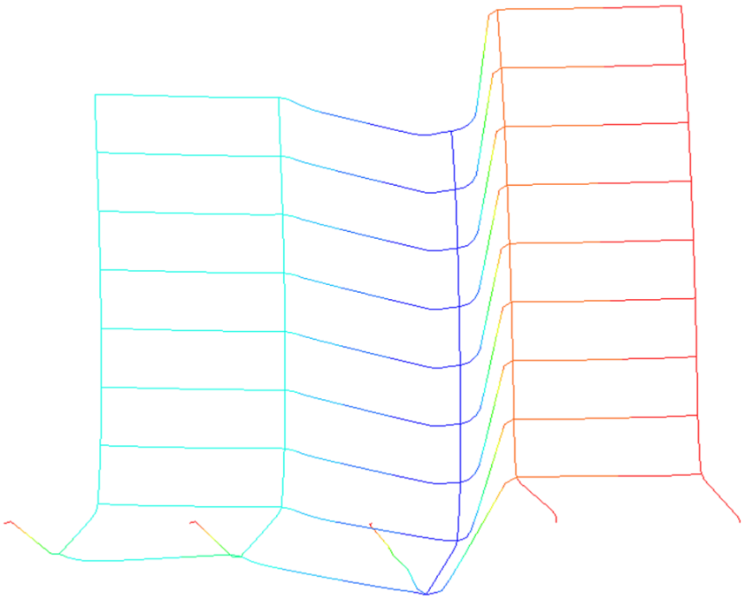


Figure 14: Failure mode for the case of column C3 heated

6. Methods of improving robustness of steel frame in a fire

This section presents the results of a parametric study on methods of improving robustness of the focussed steel frame during a fire. Two different methods have been considered in this study. First, the beam section is increased to prevent the catenary action in the beam element. Secondly vertical bracing is applied to improve the lateral stiffness of the frame. This study considers only the heated column C3 since it is the worst scenario as discussed above.

6.1 Using enhanced beam sections

To improve the robustness of the steel frame, the beam section of W24x62 is replaced with a section of W30x99, which change the load ratio of the beam from 0.8 to 0.4. Figure 15 shows the axial displacement and force of the columns against temperature. Similarly, after buckling of the heated column, the adjacent columns receive the load redistributed safely from the heated column. It should be noted that during the cooling phase, the heated column C3 experiences huge tension which can pull downward, resulting in additional axial force at the adjacent column C2. This demonstrates that the cooling phase is more critical than the heating phase. Figure 16 shows the collapse mode of the structure with enhanced beams. It is clear that the larger beam section is able to prevent catenary action so that the total collapse does not occur.

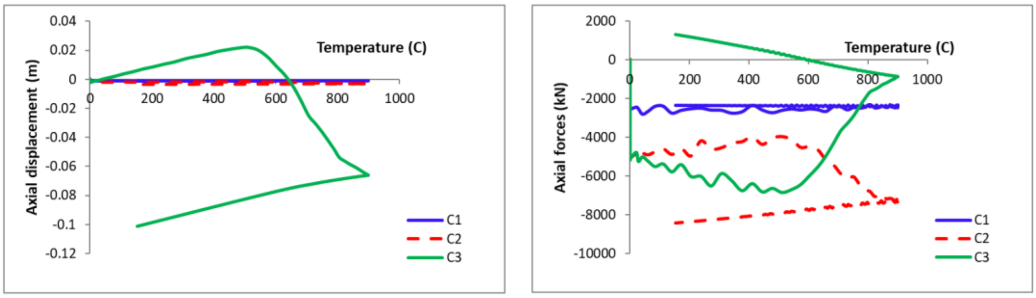


Figure 15: Vertical displacement and axial force of the columns with the enhanced beam section of W30x99

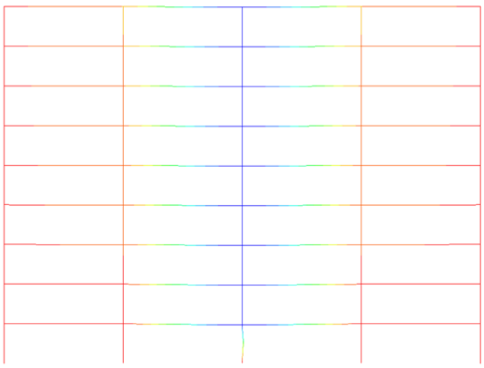


Figure 16: Failure modes of the frame with the enhanced beam section of W30x99

6.2 Using a vertical bracing system

As shown in the results of the previous section, the lateral support of the frame has a significant influence on the robustness the focussed steel frame during a fire. Under ambient conditions, the moment connections between beams and columns in this frame are adequate for lateral stability. In this following study, vertical K-bracing is applied to each edge bay. As indicated in Figure 17, the distribution of axial force at the columns is slightly different from the previous cases due to the presence of the bracing members. This shows that the catenary action occurs in the beam and pulls-in the adjacent column. However, the total collapse does not occur as the steel frame has enough capacity to accommodate the lateral forces.

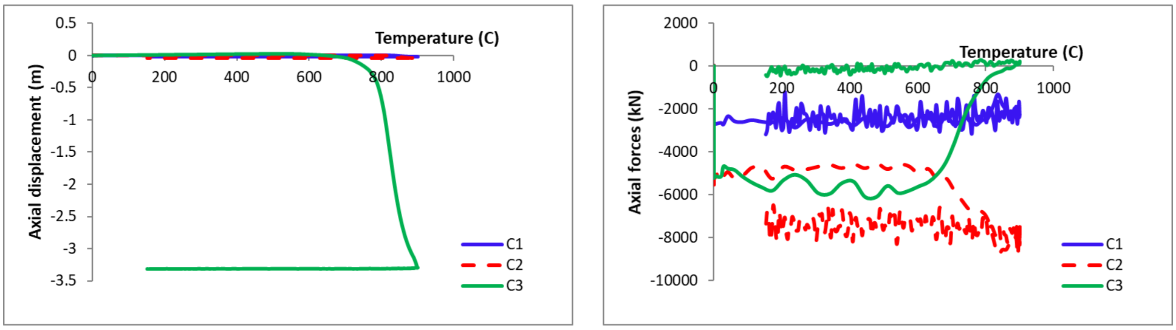


Figure 17: Vertical displacement and axial force of the columns with the vertical bracing

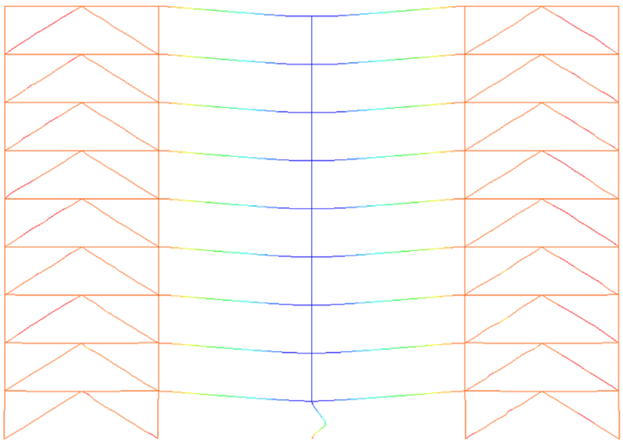


Figure 18: Failure modes of the frame with the vertical bracing

7. Conclusions

This paper presents a parametric study on the robustness of a typical steel frame during a fire. The finite element model was developed to analyse the structure. Validation analysis was carried out to confirm that the analysis produces an acceptable level of accuracy. Then, a series of robustness analyses were performed to investigate the interaction among the members and the pattern of load distribution within the structures. Based on the results, two different frame systems are investigated to enhance the collapse resistance of the steel frame. The following conclusions can be drawn from this study.

1. In performance-based fire design, the structures must be analysed integrally so that member interaction and the load redistribution path can be observed.
2. Based on parametric studies, the load in the failed element is transferred to the adjacent elements. In this case, the failure of the individual member may be still acceptable as long as the collapse of the frame as a whole can be prevented.
3. The steel frame with enhanced beam sections can significantly improve the robustness of the structure during a fire. This is because the larger beams can prevent catenary action, which in the case of this particular frame scenario is not beneficial to column stability, such that the load in the failed columns can safely transfer to the adjacent columns without buckling.
4. The addition of vertical K-bracing is beneficial as it helps to accommodate the lateral force when catenary action occurs in the beam. The additional bracing will enhance the lateral resistance of the moment frame which plays an important role in preventing the building from total collapse. However, altering the stiffness of the frame may have implications for the moment frame

performance under other load cases such as seismic loading and this should be considered in the design.

This study has achieved its main objective of providing methods to enhance the structural collapse resistance of such structures in fire. However, it is believed that the results have certain limitations. This is due to the fact that only a single symmetry plan has been considered in this study. Further investigation based on statistical data is required as statistical variations may have a significant role to obtain a reliability-based framework for this study.

Acknowledgements

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