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3D finite element analysis of the whole-building behavior of tall building in fire

Feng Fu*

School of Mathematics, Computer Science & Engineering, Department of Civil Engineering, City University of London, UK

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Abstract. In this paper, a methodology to simulate the whole-building behaviour of the tall building under fire is developed by the author using a 3-D nonlinear finite element method. The mechanical and thermal material nonlinearities of the structural members, such as the structural steel members, concrete slabs and reinforcing bars were included in the model. In order to closely simulate the real condition under the conventional fire incident, in the simulation, the fire temperature was applied on level 9, 10 and 11. Then, a numerical investigation on the whole-building response of the building in fire was made. The temperature distribution of the floor slabs, steel beams and columns were predicted. In addition, the behaviours of the structural members under fire such as beam force, column force and deflections were also investigated.

Keywords: tall building; fire; whole-building response; composite; steel; thermal behaviour

1. Introduction

After the event of 9.11, increasing research has been focused on the behavior of the tall building under extreme load conditions such as fire and blast loading and tries to find effective methods to prevent the collapse of the building. The collapse of World Trade Center building 7 (WTC7) , caused by the fire set by the debris falling from the World Trade Center, brought great attention to the researchers on the study of the structural behavior of tall building under fire condition. In the report of NIST (2005),(2008) detailed fire safety investigation on the world trade center and WTC7 was introduced. It discussed the Fire-Induced Progressive Damage of the Buildings in detail. In Europe, there is also design guidance available upon the structural fire design, such as Eurocode 3, Part 1-2 (1995) and Eurocode 4, Part 1-2 (2005) and BS5950 Part 8 (1990) and Eurocode 2 Part 1-2 (2004). However, most of above design methods are concentrated on the response of individual structural members without considering the interactions between the structural members during a fire, let alone the global behavior of the building.

In order to develop more rational design methods based on realistic behavior, some research has been conducted on the performance of building structures under fire conditions both through experimental and numerical modeling methods. Four fire tests were carried out in 1995 and 1996

*Corresponding author, Ph.D., E-mail: cenffu@yahoo.co.uk

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by the British Steel and Building Research Establishment (BRE). The tests undertaken were full-scale eight-storey structure with large building test facility at Cardington. Detailed description of the fire tests is shown in paper by Bailey *et al*. (1999), Wang *et al*. (1995, 2000) and many other technical papers as well. Based on the results of the fire tests at Cardington, some numerical modeling research was also conducted. Bailey *et al*. (1998) described the development of computer software capable of simulating the structural response of steel-framed buildings. He discussed the basic non-linear formulation of the program, which allows the three-dimensional response of steel members at elevated temperatures to be modeled. Bailey *et al*. (2000) discussed the structural behavior of steel frames with composite slab subjected to fire. They proposed a new design method for calculating the performance of steel framed buildings with composite flooring systems subject to fire. Foster *et al*. (2007) also investigated the thermal and structural behavior of a full-scale composite building subjected to a severe compartment fire. In their research, a single story building representing the Cardington test was modeled using the numerical software FPRCBC to analyze temperature distributions in slabs. Zehfuss *et al*. (2007) built a parametric natural fire model, which is derived on the basis of simulations with heat balance models for realistic natural design fires, taking into account the boundary conditions of typical compartments in residential and office buildings. Bailey *et al*. (2009) also proposed a 3D model of one-storey two bay concrete building with bonded post-tensioned concrete floor plates exposed to fire.

As mentioned above, although some numerical and analytical work have been done toward the structural behavior of steel composite frame building under fire conditions, most of them are using simplified methods. In addition, most models still concentrated on the localized behavior of the buildings and some are 2-D models rather than 3-D models, few investigations on the global thermal behaviors of an entire tall building has been done so far. However, it is important to consider not just the response of individual beams, columns but also to consider the whole-building response to the fire damage, especially in a 3-D modelling method. Therefore, it is imperative to investigate the whole-building behaviour of tall buildings under the fire condition. Due to the limitation of the numerical modelling method, the simulation of the whole-building behavior is difficult to achieve. To solve this problem, in this paper, a 3-D global model (as it shown in Fig. 1) was developed using finite element method to simulate the whole building behavior of the tall building under fire condition. A 20 storey steel composite frame building was modelled using the general-purpose program ABAQUS (2010).

In the simulation, a parametric fire scenario was chosen with the representative gravity loads (Dead+Live) for most offices buildings. The fire was set in storey 9, 10 and 11 to simulate the conventional fire incident. Based on this model, a numerical investigation of the thermal and structural behaviour of the building in fire was made. The measures to mitigate the progressive collapse of composite steel frame buildings subjected to fire attack is also recommended.

2. The prototype 3-D FE model

As it is not possible to set a complicate tall building directly in ABAQUS, the geometry of the entire tall building model is built in ETABS (2015), the model is imported to ABAQUS (2010) using the convertor program designed by the author, Fu (2009), using visual BASIC Language. The analysis was performed in ABAQUS with nonlinear material behavior as well as large displacements. The three-dimensional finite element model is shown in Fig 1. It replicated a conventional 20-storey steel composite building. A schematic plan view of the prototype building is shown in Fig. 2. The grid system in Fig. 2 describes the location of the structural elements on each floor. For an instance, Column C1 stands for the column at the junction of grid C and grid 1. Beam E1-D1 stand for the beam on grid 1 starting from grid E to grid D. The structure was designed with similar construction to the current conventional tall buildings. It has a level height of 3m on each floor. The grid space is 7.5m in both directions. The thickness for all the slabs is chosen as130mm. The reinforcement mesh of the slabs is A142 standard mesh. The columns are British universal column UC356X406X634 for ground floor to level 6, UC356X406X467 for level 7 to level 13, UC356X406X287 for level 14 to level 19. All the beams on each floor are British universal beam UB533X210X92. The lateral stability of the building is provided using the cross bracings which are British Circular Hollow section CHCF 273X12.5. The locations of these bracing are shown in Fig. 1.

3. Modeling techniques

In the 3-D model (as it shown in Fig. 1), all the beams and columns were simulated using beam element from ABAQUS library which is a two nodes 6 degree of freedom beam element. Beam element can model the one-dimensional stress state, including axial and flexural terms. Therefore, they can be used effectively to model columns and beams. The cross section of a structural member can be represented through integrated sub-sections across the cross-section at several points of the beam element allows any cross-sectional variation to be included. It is important to ensure that the numerical integration across the cross-section accurately models any variation in material and temperature. Although the beam element cannot predict the local buckling, as it shown in (1999) that, for the global behavior of the building this effect is not important. The slabs were simulated using the four nodes shell element from ABAQUS, including both membrane and flexural effects. The shell elements are integrating through the thickness allowing the variation of the properties to be included. The rib of the slab was not actually simulated, equivalent properties were used. Reinforcement inside the slabs was represented as a smeared layer in each shell element using the rebar element and was defined in both slab directions. The beam and shell elements were coupled together using rigid beam constraint equations to give the composite action between the beam elements and the concrete slabs.

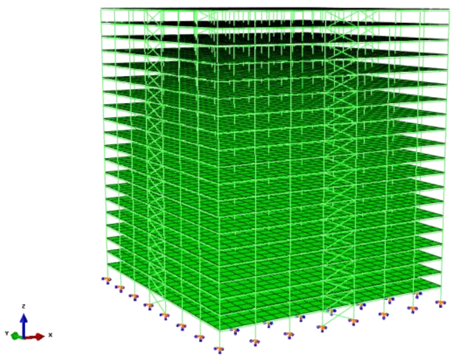


Fig. 1 3D Finite element model

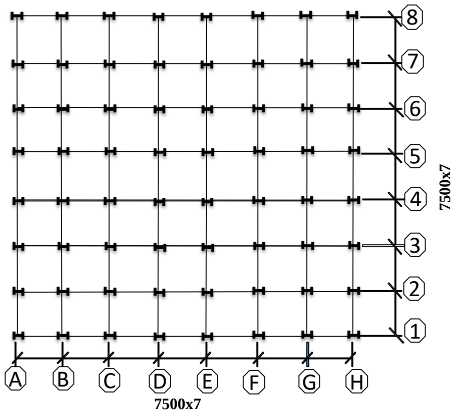


Fig. 2 Grid system describes the location of the structural elements

To simulate the typical semi-rigid connections in the construction projects, the steel beam to column connections is assumed to be fully pinned, however, the continuity across the connection is maintained by the composite slab acting across the top of the connection. Therefore, the beam to column connection is more or less like a semi-rigid composite connection which is to simulate the characteristic of the connections in normal construction practice. More detailed description of the modeling technique can be found in Fu (2009).

3.1 Material properties of concrete slab at elevated temperatures

Concrete material properties of all slabs are represented using a Concrete Damaged Plasticity model from ABAQUS, which is one of the constitutive models provided by ABAQUS to predict the constitutive behavior of concrete. It describes the constitutive behavior of concrete by introducing scalar damage variables. Material properties are assumed to vary with temperature using the relationships outlined in EN 1994-1-2 [4] (as shown in Fig. 3). Reinforcement meshes in the concrete slab are represented using the rebar function of the concrete shell elements.

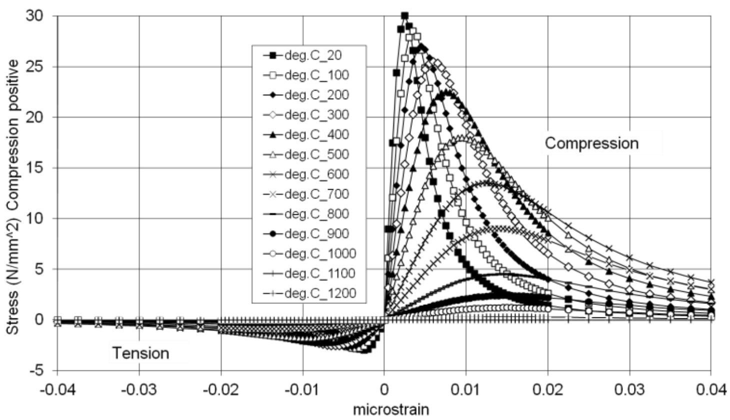
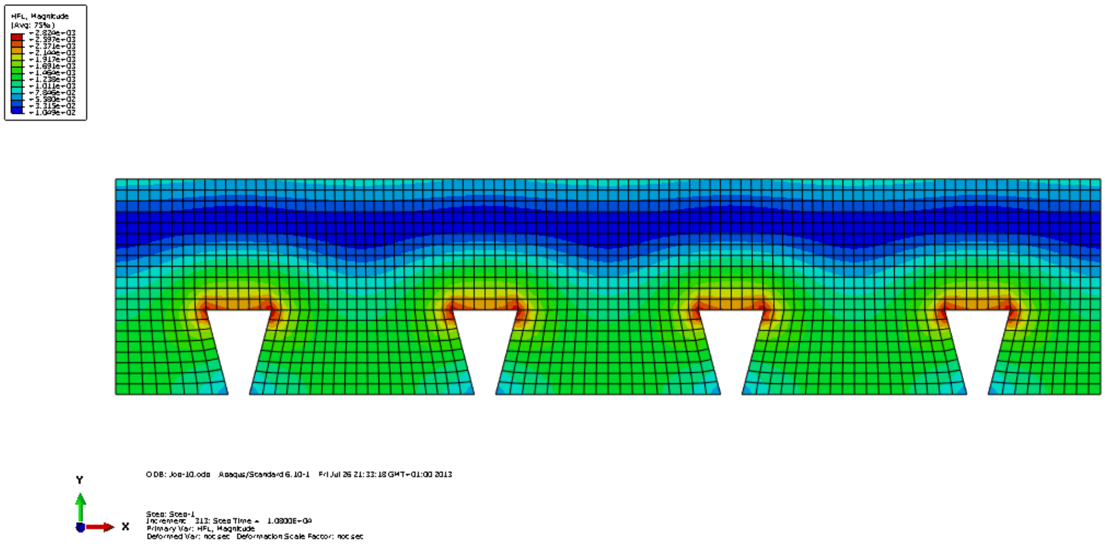


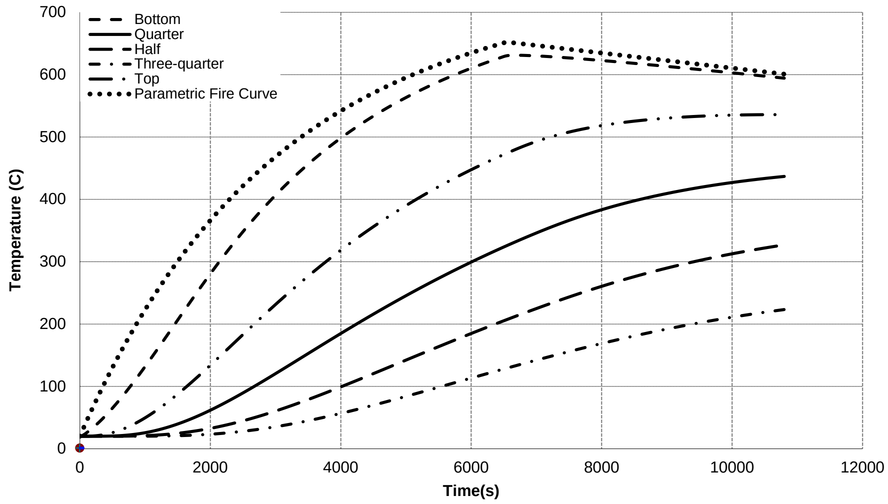
Fig. 3 Stress- Strain relationship of Concrete by EN1994-1-2 (2005)

The linear through depth temperature gradient is obtained through the heat transferring analysis in ABAQUS. Fig. 4(a) shows the result of the heat transferring analysis, and the temperature at the bottom, quarter, half Three quarter and Top was extracted and applied to the *Shell element at the correspondent layers as an idealization of the real temperature regime through the depth of the slab, the result is shown in Fig. 4(b).

The reinforcement is assumed to be located 30 mm from the top of the slab with a yield stress of 460 N/mm^2 and with material properties varying with temperature as outlined in BS EN 1993-1-2 [3], and shown in Fig. 5. The contribution of the structural metal deck is conservatively ignored.



(a)



(b)

Fig. 4 Heat transferring result of slab and parametric fire temperature curve

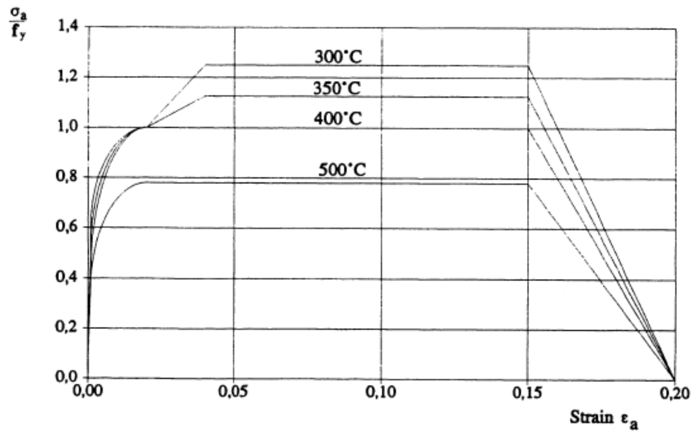


Fig. 5 Stress-strain relationship of structural steel at elevated temperature, allowing for strain hardening ENV 1993-1-2 (1995)

3.2 Material properties of structural steel members at elevated temperatures

Steel beam material properties are represented using an elasto-plastic material model with a yield stress at a room temperature of 355 N/mm^2. Material properties are assumed to vary with temperature using the relationships outlined in BS EN 1993-1-2 [3] (as it shown in Fig. 5). All the beams and columns are designed with intumescent protection to resist 2 hours fire. To simplify the analysis, different to the slabs, the unified temperature was applied to the beams and columns thought out the whole depth, this assumption provides enough accuracy, because the thermal conductivity steel materials, the variation of the temperature along the depth of the structural member can be ignored.

3.3 Gravity loading and boundary conditions

A gravity area load of 4 kN/m^2 representing the mean dead and live load used in the current construction practice was applied directly to each floor in the model. The deduction of the live load during the fire is also considered according to ENV 1993-1-2 (1995). The boundary condition is as pin supported at the ground column as it shown in Fig. 1.

Similar modelling technique to this paper was also used by other scholars such as Foster *et al*. (2007) and Zehfuss *et al*. (2007) to model simple composite buildings such as one storey or several bays of the buildings and has been validated against the fire tests of Cardington. However, the technique has not been extended to the tall buildings. The proposed method made it possible to simulate the whole building behavior of the structure.

3.4 Numerical modeling of fire loading

The fire load was applied as a fire temperature on the structural members in ABAQUS (2010) based on the parametric time-temperature curves (as it shown in Fig. 4(b)) outlined in the BS EN1993-1-2 (1995). In order to investigate the real behavior of tall buildings in fire, the fire was set on floor 9, 10 and 11 to simulate conventional fire duration in practice. This is because most of the fire only affected 2 or 3 storeys before it was put out.

4. Thermal response of the building during the fire

Through the numerical model described above, the behavior of the building during the entire period of the fire was investigated. There are two steps in the ABAQUS simulation, the static step and fire temperature step. In the first step, the model was loaded with the static load (Dead and Live) to catch the crack behavior of the concrete slab under gravity load and no temperature was added to the structural member in this step. The reactions due to dead and live loads at the base were double checked with the ETABS results to make sure the model was setting up correctly. In step 2, the temperature started to apply to each structural member based on the temperature profile shown in section 3. An analysis of the internal forces and deflection of the structure was performed here to provide a detailed understanding of the local and global behavior of the tall building as follows.

4.1 Behavior of beams

Figs. 6, 7 and 8 show the axial force, shear force and bending moment of beam B1A1 at end B on floor 10 during the fire. As it is shown in Fig. 6, the tensile axial force caused by the gravity loading was first observed in the Static Step, it increased from 0 to 74.657 kN. In step 2, fire temperature was applied, the temperature of each structural member started to increase against time. Due to thermal expansion and the restrain from the supporting columns, the axial force of the beam started to decrease into compression with the increase of the temperature. Due to the thermal expansion, as shown in Figs. 7 and 8, the shear force and bending moment of the beam changed from negative into positive value. This is because the thermal expansion produced the out-of-plan bending and shear in the beam.

It can be noticed that, in the event of fire, the behavior of the structural members are also governed by the thermal expansion, therefore, it is different to the normal gravity load scenarios.

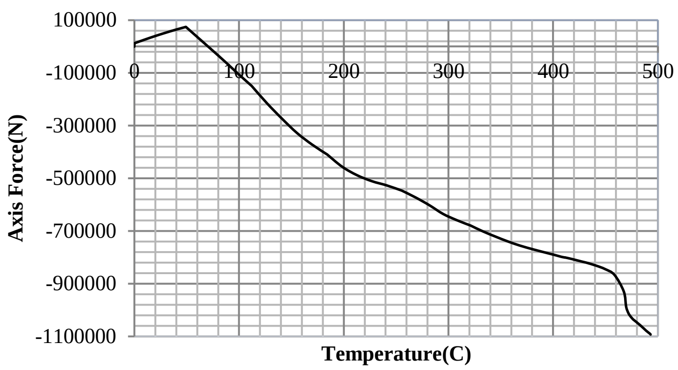


Fig. 6 Axial Force of Beam B1A1 at floor 10

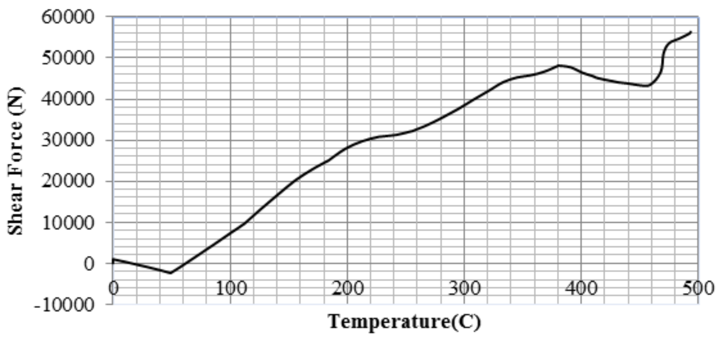


Fig. 7 Shear Force of Beam B1A1 at floor 10

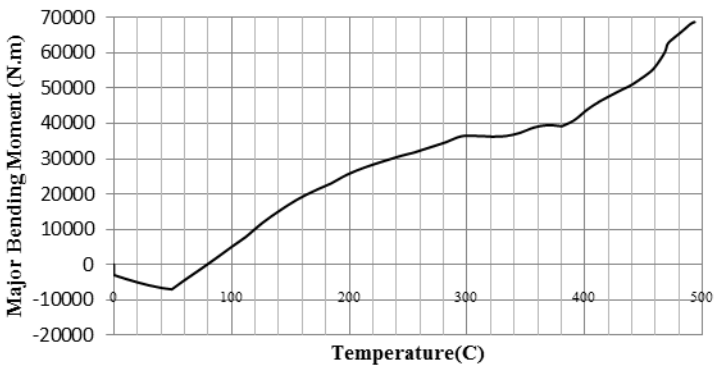


Fig. 8 Major Bending moment of Beam B1A1 at floor 10

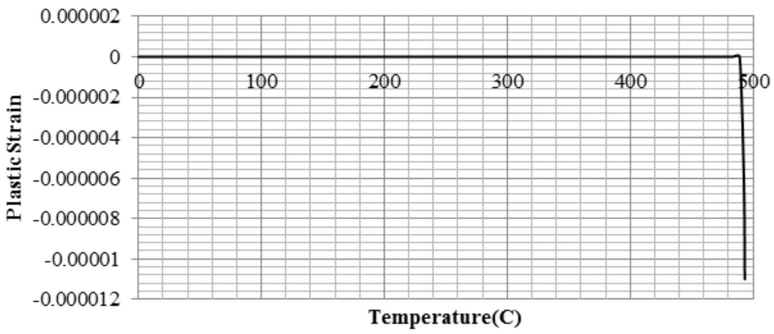


Fig. 9 Plastic strain in Beam B1A1 at floor 10

Fig. 9 is the plastic strain observed in beam B1A1 on floor 10 during under fire. It can be seen that, when the temperature is approaching 500°C, the plastic strain exceeded zero, which indicated the onset of yielding in the steel beam. When the yielding of the bottom and top fiber of the beam develop to a certain stage, the plastic hinge will form in the beam. If more than three hinges are formed, possible collapse of the beam will happen.

4.2 Behavior of column

As the corner columns are more venerable to fail due to their position, the conner column A1 on floor 10 is selected here for investigation. Fig. 10 is the contour of horizontal deflection of the building during the analysis. It can be seen that, the columns at the perimeter location deformed outward due to the thermal expansion which is different to the research outcome by other Usmani (2003) using a 2-D modeling method, where the column is deform inward due to the pulling force of the floor plate. This is because in the 2-D model, the slabs are not modeled; therefore, the deformation of the floor plate could not be presented. Therefore, the proposed 3-D method provides a more realistic structural behavior.

Fig. 11 shows the axial force of column A1 during the analysis. In the first static step, the column was loaded with the gravity load with no temperature load is applied. The load increased up to 700 kN at the end of the static step due to gravity load. It can be seen that, in this static step the mechanical stress in the column is in compression.

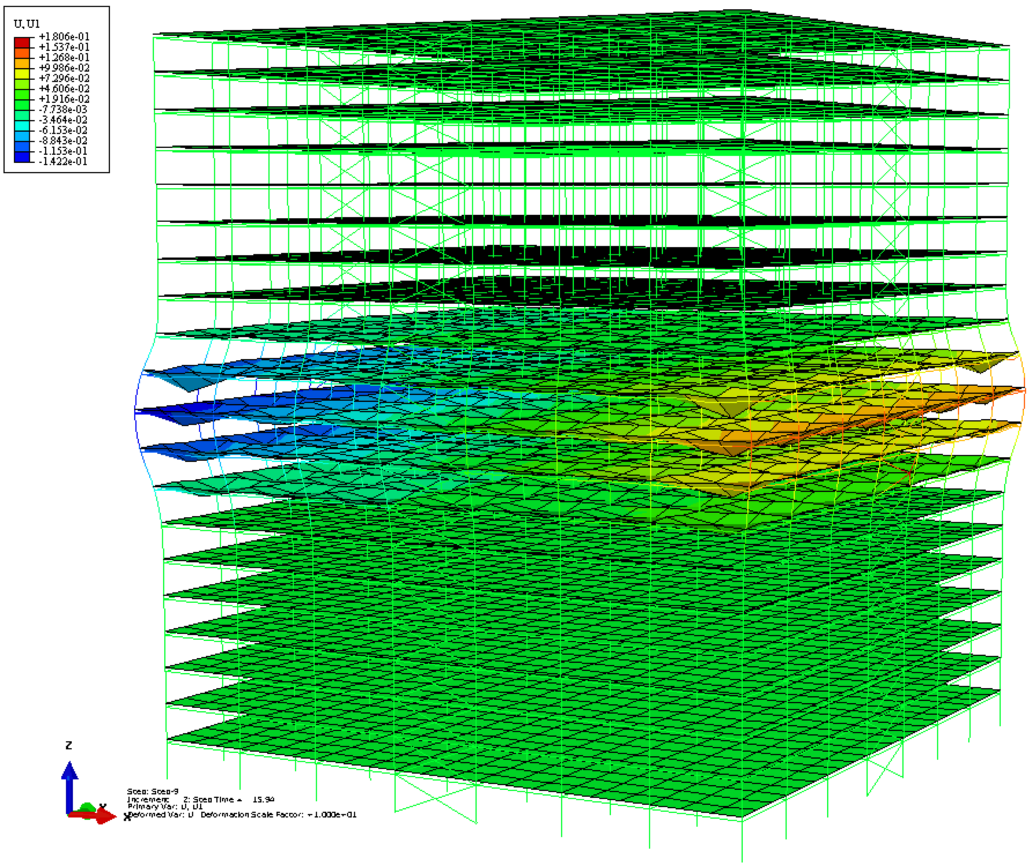


Fig. 10 Horizontal deflection of the model (deformation is amplified)

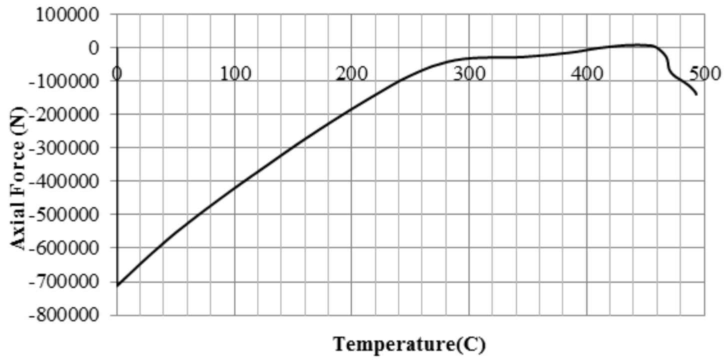


Fig. 11 Axial Force of Column A1 at floor 10

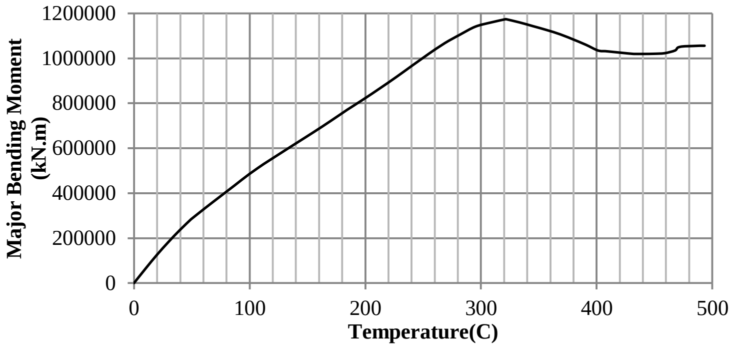


Fig. 12 Major Bending moment of Column A1 at floor 10

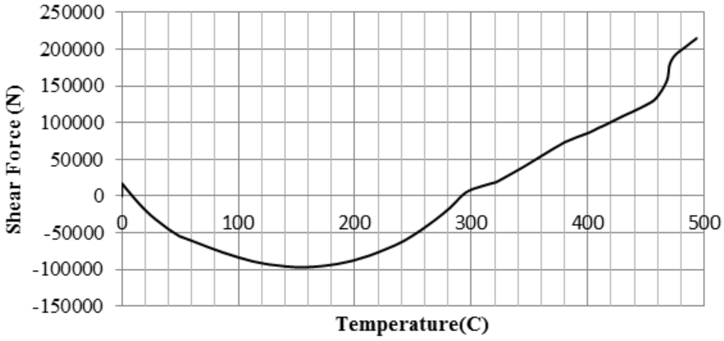


Fig.13 Shear Force of Column A1 at floor 10

In step 2, with the temperature started to increase, it can be noticed that the column load was decreasing. This is because the tensile thermal stress started to develop due to the elevated temperature. It is known that, the resultant force in the column is the summation of both the mechanical force and the thermal force, therefore, the column force started to decrease and at certain stage, becomes tension force in the column. In addition, we can also notice that, large

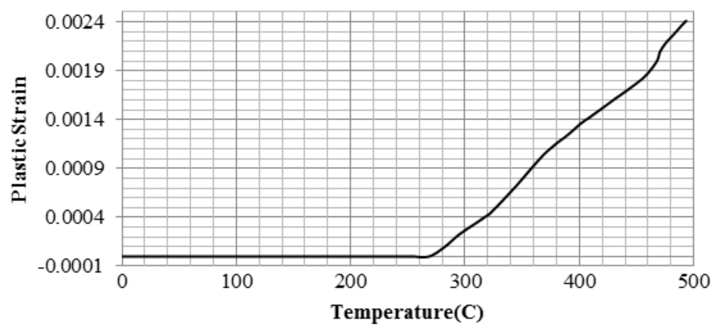


Fig. 14 Plastic strain in Column A1 at floor 10

bending moment and shear force due to thermal expansion is also observed as it shown in Figs. 12 and 13. With further increase of the temperature, we can notice that the bending moment started to drop at temperature around 300°C, this is because the plasticity are formed in the column as it shown in the plastic strain development in Fig. 14. This indicates the onset of yielding in the column. The column still have the reserve capacity due to the plasticity, when the yielding of the bottom and top fiber of the beam develop to a certain stage, the plastic hinge will form in the column, which will cause the failure of the column. If the plastic hinges are formed in all the columns on this floor, collapse mechanism will be formed. The building will start to collapse. Therefore, one of the effective methods to mitigate the collapse of the building in fire is to enhance the out-plan shear and bending capacity of the column.

From the comparison between Fig. 14 and Fig. 9, it is also noticed that, the onset of yielding in column is earlier than the beams. The column is at 300°C, however, the beam is at 500°C. Although in the tall building design, we intend to guarantee that the columns are stronger than the beams during the design. This is to prevent premature failure of the columns, therefore to prevent early collapse of the building. As beam failure won’t trigger the collapse, however, column does. However, due to excessive deformation in the fire, although larger column size are normally chosen in the conventional design, the columns are more vulnerable in fire, as they started yielding early than the beam due to the out plan bending and shear cause by thermal expansion and heavier column load due to the gravity load from the floors above. Therefore, the early failure of the columns should be prevented in the future design by further enhancement of the columns.

4.3 Behavior of braces

It is well known that, braces play an important role in the lateral stability resistance of the tall buildings. Therefore, the behavior of the braces is investigated in this section. Fig. 15 shows the axial force in bracing member at grid D1 on floor 10 during the analysis. It can be seen that with the increase of the temperature, the axial force in the brace increased gradually. Figs. 16 and 17 also shows the shear force and major bending moment in the bracing members, which are caused by the out-plan deformation due to the thermal expansion. At around 450°C, the plastic strain was developed as it is shown in Fig. 18, which indicates the onset of yielding of the steel brace. With the increasing of the temperature, it would further develop until full plasticity formed so that the plastic hinge would be formed in the brace.

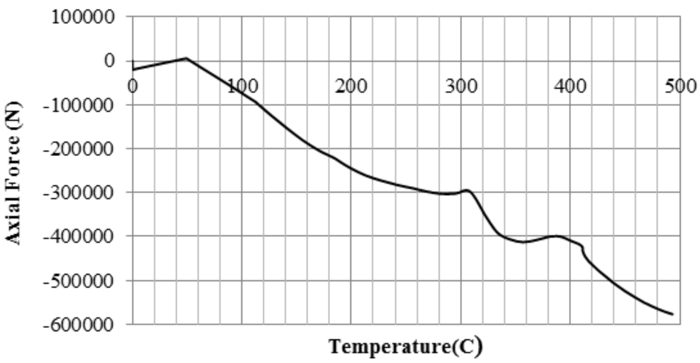


Fig. 15 Axial Force of Brace on Grid D1 at floor 10

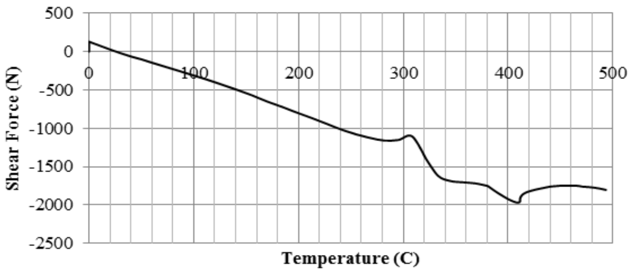


Fig. 16 Plastic strain in Brace on Grid D1 at floor 10

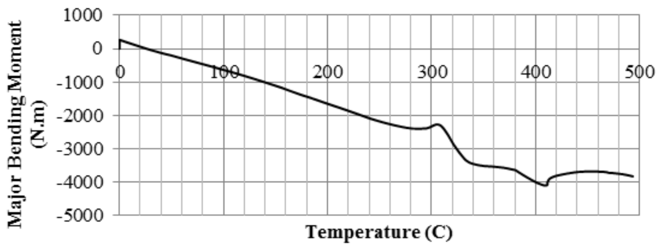


Fig. 17 Plastic strain in Brace on Grid D1 at floor 10

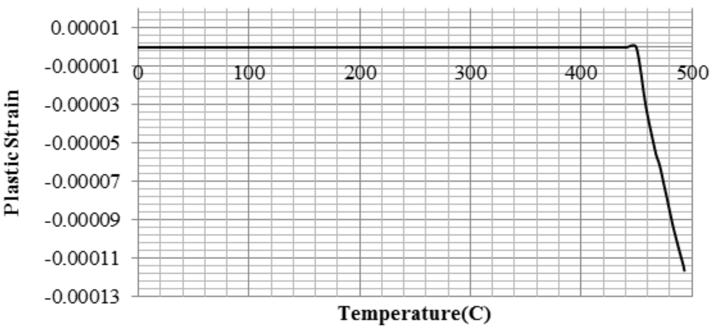


Fig. 18 Plastic strain in Brace on Grid D1 at floor 10

4.4 Behavior of slabs

Fig. 19 shows Contour of Vertical deflection of slabs on floor 9 and 10. It can be seen that, the slabs at the perimeter location of the floor shows larger displacements than the slabs in the center area of the floor. The largest displacement was observed at the four corners A1, A8, H1 and H8. This is due to the thermal expansion of the slabs which made the whole floor slabs deform toward the perimeter and these corners have less vertical stiffness than other locations of the slab due to less joint effect of the columns at those locations.

Fig. 20 is the tensor of plastic strain developed in the concrete slabs, indicating the formation of the crack in the concrete. This is the way ABAQUS represent the cracking. We can notice that most of the cracks are observed at the edge slabs, especially at the four corner of the floor.

Fig. 21 and Fig. 22 show the axial force of slab A3-A4-B4-B3 on floor 10 (the location of the slab is shown in Fig. 19) both at the center of the slab and the perimeter of the slab. It can be seen that, with the increasing of the temperature, the center of the slab is in tension and the perimeter of the slab is in compression due to the tensile membrane effect. This membrane action will enhance the load-carrying capacity of the slab. However, it is also noticed that, with the increasing of

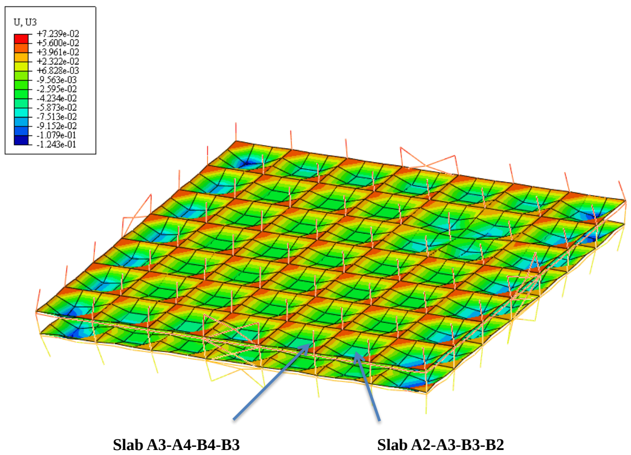


Fig. 19 Contour of Vertical deformation of slabs on floor 9, 10

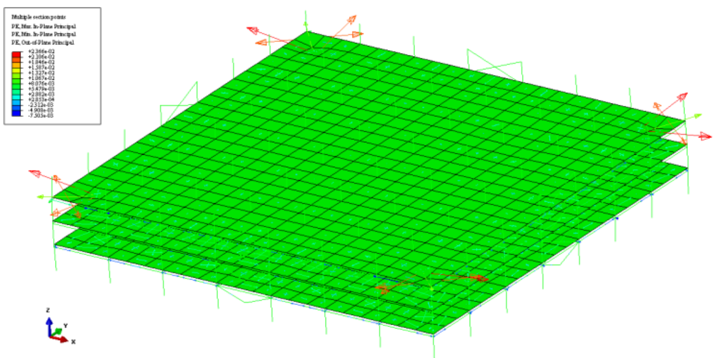


Fig. 20 Crack pattern (tensor of plastic strain) of the concrete slab on floor 9, 10 and 11

temperature, the axial force of the slab dropped, which is due to the large deformation of the supporting beams.

Figs. 23 and 24 show the axial force of slab A2-A3-B3-B2 at floor 10 (shown in Fig. 19). It can be seen that, different to slab A3-A4-B4-B3 the axial force at center and the perimeter of the slab were both in compression. It can be concluded that, due to the global deformation of the whole floor plate and the restrain from the supporting steel beams, tensile or compressive membrane force is developed in the different location of the floor which is similar to the full scale fire test result.

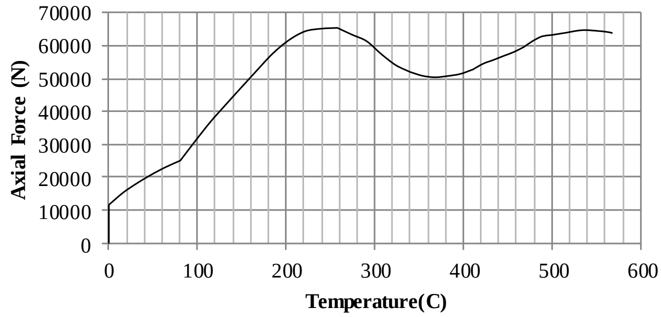


Fig. 21 Axial Force of at the center of slab A3-A4-B4-B3 at floor 10

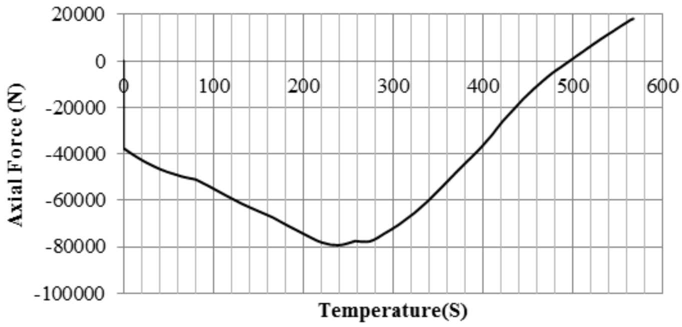


Fig. 22 Axial Force at the perimeter of slab A3-A4-B4-B3 at floor 10

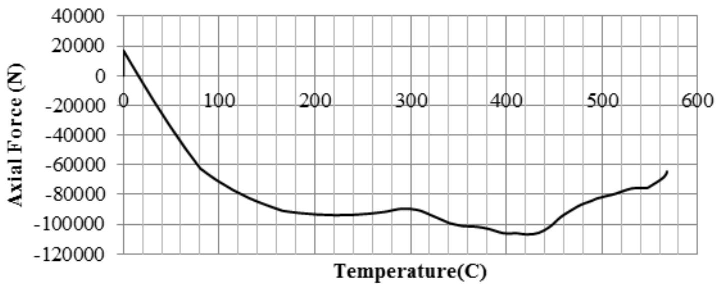


Fig. 23 Axial Force of at the center of slab A2-A3-B3-B2 at floor 10

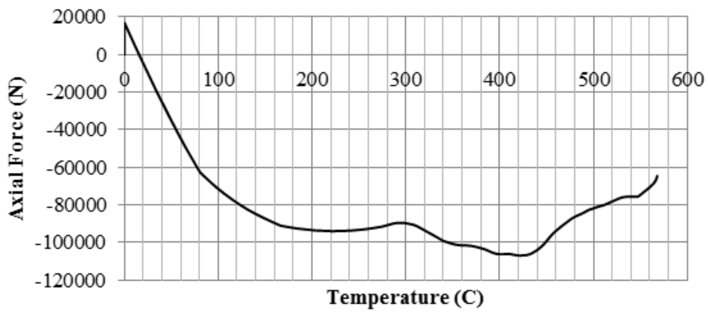


Fig. 24 Axial Force at the perimeter of slab A2-A3-B3-B2 at floor 10

5. Conclusions

In this paper, the methodology to simulate the whole-building behavior of the tall building under fire is first developed by the author using 3-D finite element method. The fire was set at floor 9, 10 and 11 to simulate the conventional fire scenario. Detailed thermal analysis was performed. The whole building behavior of the tall building was investigated. Based on the modeling observations and analysis of data, the following findings were made:

1. Strong column and weak beam is a design principle widely used in design practice to prevent early collapse of the buildings. In most loading scenarios such as earthquake, this design rule is efficient in preventing the collapse of the building, which enables the beams to fail earlier than columns. However, under the condition of fire, due to the thermal expansion and the strength degradation of the material, the columns are prone to fail earlier than the beams, so this design principle is not applicable.

2. Following conclusion 1, the effective ways to prevent the building collapse is to prevent the early failure of the columns. The possible methods are to increase the out-plan moment capacity and shear capacity of the column. This can be achieved by either increase the thickness of the fire-protection or increases the Flange and web thickness

3. Due to thermal expansion, the corner slabs are more vulnerable to fail than the slab at other locations.

4. Due to the global deformation of the whole floor plate and the restrain from the supporting steel beams, tensile membrane and compressive membrane is developed in the slab. This membrane action will enhance the load-carrying capacity of the slab. However, it is also noticed that, with the increasing of temperature, the axial force of the slab dropped, which is due to the large deformation of the supporting beams.

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