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BEHAVIOUR OF MODULAR BUILDING INCORPORATING CFST COLUMNS SUBJECTED TO COMPARTMENT FIRE

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ABSTRACT

Modular construction has become increasingly popular in the construction industry due to its benefits such as cost-effectiveness, quicker construction time, reduced environmental impact, and flexibility in building sites with limited area. Adopting concrete-filled steel tubular (CFST) columns can further improve the performance of modular buildings. However, there is limited fire analysis research on modular buildings and none on composite modular buildings. This study delves into the fire behaviour of module compartments made of CFST columns and tubular beams, employing numerical analysis. The finite element (FE) model of CFST columns is validated against experiments, showing ability to capture temperature distribution, axial displacement, lateral displacement and failure modes. Then, a FE model of the composite module is developed to analyse the fire behaviour of the structural system. The design fire curve is determined using compartment zone models and considering a range of fuel loads, opening factors, and compartment properties. The thermal-structural response of the module structure is then analyzed. Parametric study is conducted to examine the influence of different sizes of columns, ceiling beams and floor beams on the fire behaviour of the module. The increase in member size in module can limit member expansion, delay global buckling formation and improve fire resistance. Based on the findings from the parametric analysis, recommendations are provided to improve the fire performance of the composite modules.

Keywords: CFST columns; compartment fire; composite building; modular building

1 INTRODUCTION

Modular construction involves the fabrication of prefabricated volumetric building units within factory settings. These units are then transported to the construction site and assembled to form the functioning structure[1]. Modular building has great potential because of its numerous advantages, such as structurally self-sufficient prefabricated units, adaptability to limited spaces, rapid on-site installation, high work safety and ability to be built for special applications requiring higher standards. Additionally, it offers minimal resource consumption, emissions, environmental impacts and ease of demolition [1].

Modular buildings have received a lot of attention from academics and engineers, who have been working to improve modular buildings' structural performance and construction effectiveness [2]. Due to

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advancements in manufacturing and material technology, several modular highrise buildings have been constructed in recent years; notably, Australia is credited with erecting four of the ten tallest modular buildings [3]. However, structural fire can have detrimental effects on the modular buildings. Structure fires, in fact, have a high total number of incidents and high associated costs. Several incidents occurred due to structural fires, such as the partial collapse of the First Interstate Bank Building in 1988, Windsor Tower, Spain in 2005 and the Faculty of Architecture Building, Netherlands in 2009 as well as the entire collapse of the World Trade Centre Building 1, 2 and 7 in 2001 and Plasco Shopping Center, Iran in 2017. In addition to statistics and actual instances of building damage, numerous theoretical studies have demonstrated that prolonged fire exposure can result in the progressive collapse of structures. During an extended fire, a significant risk of global collapse exists in the building due to the extensive thermal expansion and subsequent catenary action of the heated floor system[4]. Therefore, the fire safety of the building is essential to preventing and protecting against fire-related damage. It is one of the crucial aspects of structural design.

Plenty of research has explored the collapse behaviour of steel modular buildings [5][6]. However, steel exhibits several drawbacks, including susceptibility to corrosion, low thermal resistance, higher maintenance requirements, and vulnerability to local buckling. Therefore, utilising composite components that combine concrete and steel, such as CFST columns, offers an ideal solution to mitigate these issues. The use of concrete-filled steel tubular (CFST) columns in modular buildings can provide higher ultimate strength capacity and superior fire resistance than steel columns. Due to the presence of infill concrete in the CFST column, when fire occurs, the outer steel tube has a reduction in temperature rise as the concrete absorbs heat from the steel. Also, the infill concrete retard the local buckling formation on the tube.

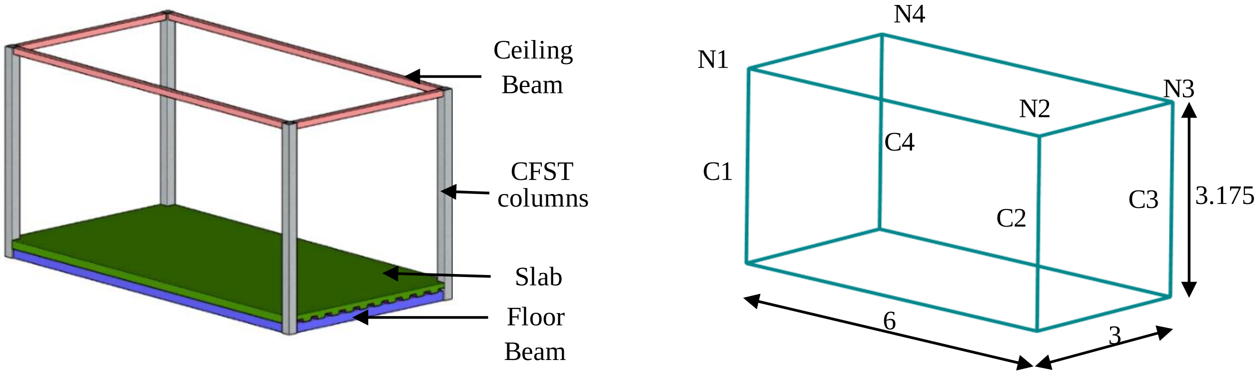
Few studies have been found on composite modular buildings. A study was conducted by Gaurav et al. [7] on 10-storey composite modular buildings to evaluate their progressive collapse behaviour. Various methods of module removal were considered, and different values of dynamic application factors were suggested based on the module removal location. In thinner gusset plates, failure was initiated by shear failure, while in thicker plates, global buckling of adjacent columns occurred before gusset plate shear failure.

Limited research has been conducted on steel buildings exposed to fire scenarios, with even fewer studies focusing specifically on steel modular buildings. Agarwal et al. [8] investigated the overall performance of steel buildings subjected to real fire conditions. This study utilised two ten-story buildings: one featuring a perimeter moment-resisting frame, while the other incorporated an interior RC shear wall core. The findings highlighted the significance of gravity columns in maintaining structural stability during a fire. Additionally, it was observed that the slab's reinforcement plays a crucial role in transferring axial loads from failed columns to adjacent columns, thereby reducing the risk of additional failures and progressive collapse. Gernay et al. [9] conducted a nonlinear analysis of the steel-framed building under fire using performance-based analysis. The study focused on the structural stability of the building during severe fire scenarios, including compartment fires, fire spread across compartments, and events involving column loss followed by fire. Findings revealed that the performance-based fire designs of structures could be completed using a rigorous and systematic methodology. Additionally, the benefits of utilising performance-based designs are highlighted. Shan et al. [10] evaluated the collapse performance of steel modular buildings under fire scenarios. The influence of various parameters on the global performance of the building is investigated using parameters such as load ratios, cross-sectional dimensions of columns, fire locations and fire types. Subsequently, a practical design method was developed based on the findings. There has not been any research on the study of the failure mechanism of the composite modular building. In addition, experimental testing is not only time-consuming and expensive but also impossible to test complicated and massive constructions due to the size restriction of the furnaces. Therefore, this research will focus on the nonlinear finite element analysis of composite modular buildings under fire. Thus, it is necessary to get in-depth knowledge of the CFST columns' fire performance.

This study explores the response of compartments equipped with CFST columns to fire exposure through the application of advanced numerical analysis techniques. Initially, finite element (FE) models of CFST columns are developed and validated against experimental data, considering key parameters such as temperature distribution, axial and lateral displacements, and failure modes. Following this, an FE composite module is constructed to conduct a comprehensive examination of the failure mechanisms inherent in different components. Through a sensitivity analysis, a design fire curve is selected, with parameters including fire loads, opening factors, and compartment properties being evaluated. Subsequently, parametric analysis is carried out with various sizes of columns, ceiling beams and floor beams. Based on the findings obtained from the parametric analysis, recommendations are developed to optimise both the design and performance of the module in fire scenarios.

2 DESIGN OF THE MODULE

A 10-storey modular building designed by Gaurav et al. [7] is utilised in this study. This building is designed for office building purposes according to the AS/NZS 2327 and is considered to be in Australia. The building features an overall plan dimension of 37m × 15.83m, with one direction comprising 6 bays and the other side accommodating 5 bays. Each module measures 6 m × 3 m and stands at a height of 3.3 m, aligning with the standard commercial sizes [3], as illustrated in Figure 1. The labels N and C in Figure 1(b) represent the node number and column number. The modules are made of CFST columns and steel beams. The dimensions of the components and the strengths of steel and concrete are adjusted slightly to align with the Australian Standard. The geometrical and mechanical characteristics of the ceiling beams, floor beams, slabs and columns are detailed in Table 1. The connections between the beams and columns are welded connections. The live load is taken as 2.5 kN/m^2 and the floor finish of 1kN/m^2 which is taken from the Eurocode. This model serves as the baseline for studying others.



(a) 3D view of module (b) FE model

Figure 1. Module

Table 1 Module components

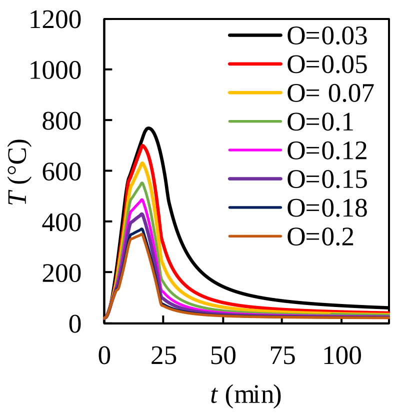
Components	Size (mm)	f_{ck} (MPa)	f_y (MPa)	Rebar mesh
Column	150×150×9	65	350	
Ceiling beam	100×100×5	-	350	
Floor beam	150×100×9	-	350	
Floor slab	6000×3000×130 & 0.9 (steel plate)	65	350	A252

3 FUEL LOAD AND DESIGN FIRES

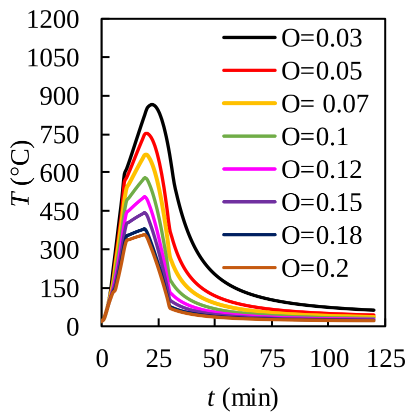
The fire exposure assessment involves the development of a fire time-temperature history to evaluate the thermal response of the structure. The fire rating for the components of the office building is 120 minutes according to the Australian Building Code. The characteristic fire load density used for all fire scenarios is set at 511 MJ/m^2, corresponding to the 80% fractile value of fuel load density for an office according to Eurocode 1-2 [11]. For the generation of fire curves, the Ozone software is employed. This building is assumed to be equipped with sprinklers and automatic fire detection by smoke, utilizing a fire load of 193 MJ/m^2.

The sensitivity analysis is conducted to evaluate the nature of the fire curve with respect to different parameters such as fuel loads, compartment properties and ventilation openings. This analysis considers three cases: buildings with both automatic water extinguishing systems and automatic fire detection by smoke, only the automatic water extinguisher system and those without all the above systems which resemble the three design fire loads such as 193 MJ/m², 264.3 MJ/m², and 433.3 MJ/m² respectively. It also encompasses the consideration of two scenarios regarding wall materials such as gypsum board and shear walls along with various opening factors opening factors ranging from 0.03 to 0.2, corresponding to areas from 2.15 m² to 14.4 m². The several fire curves generated are presented in Figure 2(a), Figure 2(b) and Figure 2(c).

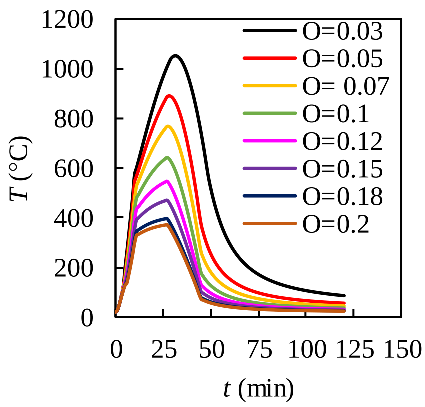
Overall, it can be observed that as the opening factor increases, the peak temperature decreases across all fire load cases. The maximum temperature fire curve, reaching 1050 °C, is illustrated in Figure 2(c) which also has a prolonged fire duration compared to other fire curves in Figure 2(a)-(c). In this study, the building has sprinklers and automatic fire detection by the smoke system (193 MJ/m²). In Figure 2(a), it is observed that the maximum fire temperature of 769 °C occurs for the 0.03 opening factor, hence it is selected for further investigation to account for the potential extreme fire scenario evolving within the compartment.



(a) Fire Load = 193 MJ/m²



(b) Fire Load = 264.3 MJ/m²



(c) Fire load = 433.3 MJ/m²

Figure 2. Design fire curves

4 PARAMETRIC ANALYSIS OF THE STRUCTURAL FIRE RESPONSE

The fire resistance of the modules can be enhanced by applying fire protection to the components and increasing the section sizes of the components. In this study, the fire resistance of the modules is investigated by altering the section sizes of different components. Initially, a module is developed with component details presented in Table 1. and thoroughly analyzed. This module is taken as the baseline for studying other modules. Subsequently, further study is conducted by changing parameters such as two different column sizes (200×200×6mm and 250×250×6mm), two ceiling beam sizes (150×150×5mm and 200×200×5mm) and a floor beam (200×100×9mm).

4.1 FE modelling

The thermal-structural response of the module is analysed through the nonlinear analysis software SAFIR. 2D thermal analysis is carried out for each component exposed to design fire curves. Fire is exposed to its surfaces. The floor beams are assumed to be at ambient temperature, as they are under the slab. Thermal and mechanical properties of the concrete and steel are taken from Eurocodes [12,13]. Siliceous concrete, featuring a convection coefficient of 25 W/m2K and an emissivity of 0.7 is employed. The concrete is characterised by a moisture content of 3% by weight and a density of 2300 kg/m3. The air gap of 0.005 Km²/W thermal resistance is adopted in this study. For the structural analysis, the FE modules are developed with 3D beam elements. The slab loads are directly applied to the beams. An initial sway imperfection of L/500 is applied to the compartment in the Y-direction. The connections between the beams and columns are rigid as they are welded together. The bottom sections of the columns have fixed boundary conditions to prevent displacement while permitting rotation along the X and Y directions. However, the top corner nodes remain unrestricted during analysis. During the loading, the fire incident combination DL+0.3LL is taken according to Eurocode1-1-2[11]. The vertical load at the top of the columns is 30% of the capacity of the columns. The columns' sectional capacity is calculated according to the Eurocode [14] and found to be 1716 kN.

4.2 Validation of the FE modelling of the CFST columns

Before analysing a full compartment analysis, validation of the CFST columns has been conducted. Several FE models were developed in SAFIR software and validated against the experimental data, covering fire resistance time, time temperature curves, axial displacement time and lateral displacement time curves. One of the validations of the column can be observed in Figure 3 and Figure 4, where the predicted FE curves closely align with the test curves. Moreover, the predicted fire resistance time of 41 min closely matches the tested time of 43 min, indicating the columns' capability to accurately predict outcomes. Further comprehensive validation details and parametric study are available in [15].

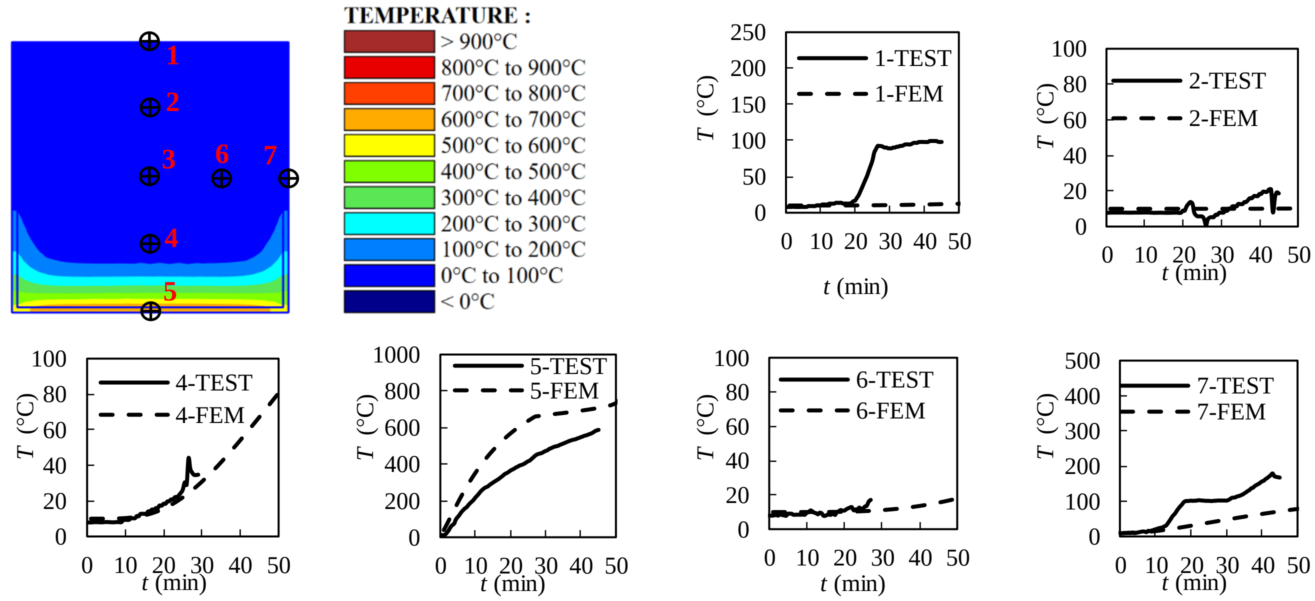
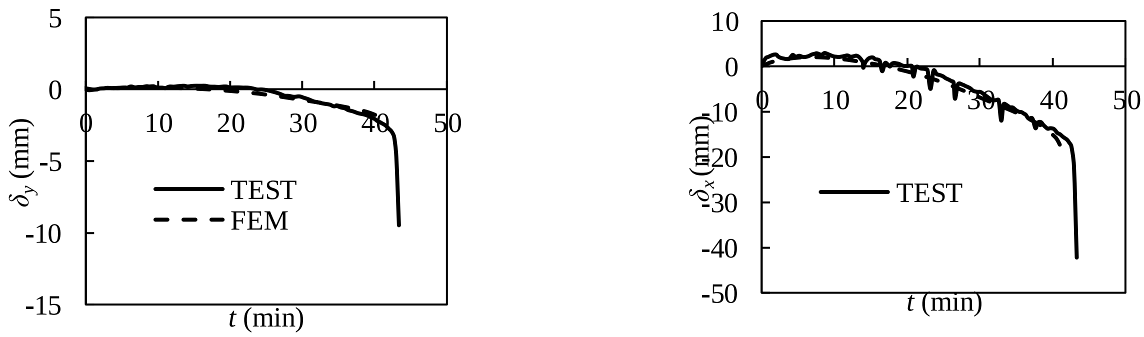


Figure 3. Comparison of predicted and measured time-temperature curves of specimen S5.

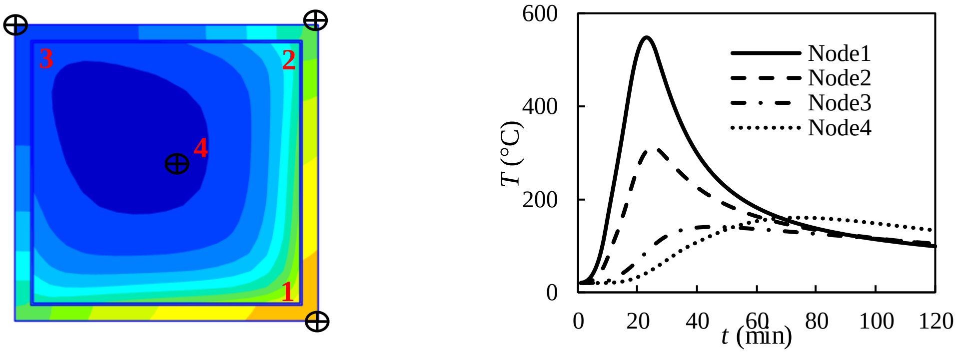


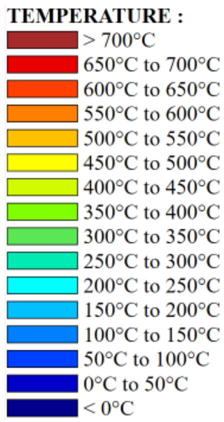
(a) Axial displacement (b) Lateral Displacement

Figure 4. Comparison of predicted and measured displacement-time curves of specimen S5.

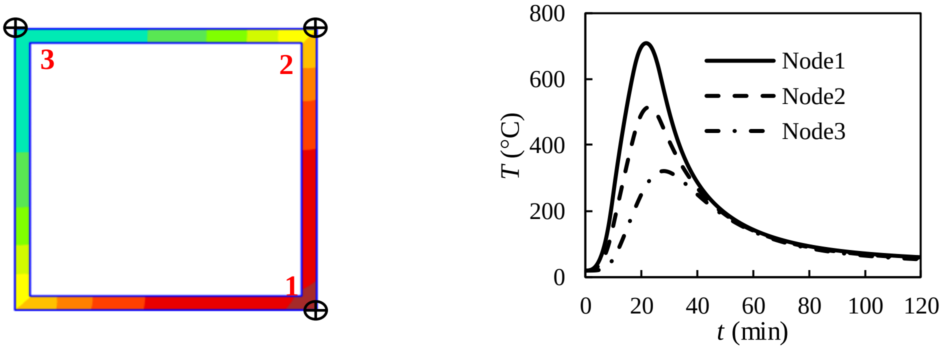
4.3 Thermal Analysis of the baseline module

The thermal analysis is conducted on the baseline module and the resulting temperature profile of the columns and ceiling beams exposed to two-sided fire are shown in Figure 5. Because of the presence of walls, both the column and the ceiling beam are assumed to be exposed to two-sided fire. In columns, the maximum temperature is observed at the exposed corner, with the minimum temperature found towards the opposite direction, as displayed in Figure 5(a). In a beam exposed to fire, the maximum temperature occurs at the exposed corner, with the minimum temperature found in the direction opposite corner to the maximum temperature in Figure 5(b). The maximum temperatures recorded are 533 °C and 707 °C while the minimum temperatures are 30 °C and 303 °C for columns and beams at 21 min respectively.





(a) CFST columns exposed to 2-s fire



(b) Beam

Figure 5. Temperature distribution and temperature curve at different locations of the section.

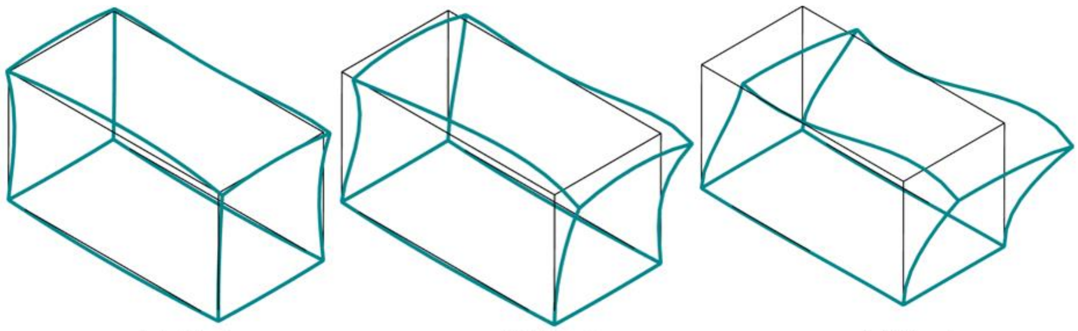
4.4 Structural analysis of baseline model

The failure mechanism of the module with the columns and beams exposed to 2-s fire exposure is analysed. The failure of the module exposed to fire occurs around 34 min. At the moment of failure, the maximum temperature observed on the column is 384 °C at the exposed corner, with a minimum temperature of 74 °C at the opposite corner. Similarly, the ceiling beam reaches a maximum temperature of around 383 °C, while the minimum temperature observed is 293 °C at the opposite corner. The displaced module at different phases, displacement of the different locations and axial force on different components of the compartments are depicted in Figure 6, Figure 7 and Figure 8.

At the beginning stage, the columns experience a thermal contraction due to the applied load as shown in Figure 7(c) with an axial compression force of 515 kN as presented in Figure 8(a). Subsequently, a continuous expansion is observed as the temperature rises, with rapid contraction when the temperature decreases in the module. Simultaneously, a horizontal shift towards X and Y direction is noticed in the top nodes of the columns N1, N2, N3 and N4 as represented in Figure 1(b) and Figure 7 (a) and (b). The buckling phenomena in columns start around 13 min when the sudden drop in axial compression force occurs as shown in Figure 8(a). Then the rapid swaying of the columns towards the Y direction similar to the initial sway direction is observed as shown in Figure 6(c) and Figure 7(b).

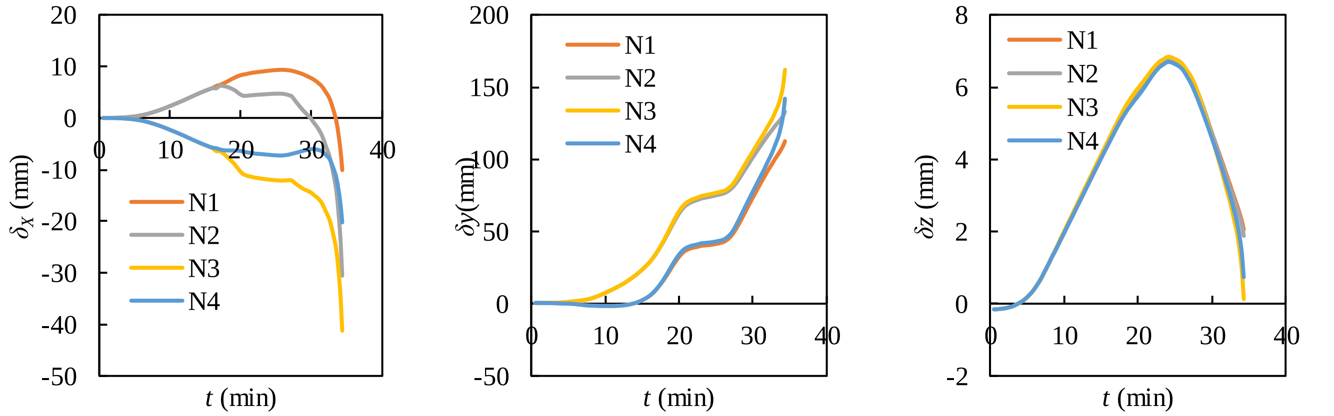
Ceiling beams, on the other hand, are noticeably impacted by temperature variations. Initially, the beams are slightly in compression. As the temperature rises, the axial force shifts to tension and the beams are displaced with the sway of the frames as shown in Figure 8(b) and Figure 7(b). In Figure 8, the labels LB and SB denote the long and short beams respectively and the attached numbers indicate the node numbers. Together with the expansion the beam bends towards the fire area as shown in Figure 6(a). As the fire progresses, the beams bend opposite to the fire and rapid axial expansion towards the Y direction can be observed inside the beams as displayed in Figure 6. The shorter beams expand along two opposite directions and with less value than the longer beams which only expand towards the positive Y direction as indicated in Figure 7(a) and (b). The initial sway direction and the longer beams higher expansion shift the upper parts toward the right direction. Conversely, the floor beams undergo gradual sagging, resulting in tensile axial forces.

Notably, the primary contributor to structural failure is the columns buckling as well as the ceiling beams, particularly the long beams as illustrated in Figure 6 and Figure 7. The heating and subsequent expansion of these beams lead to a pronounced expansion towards the Y direction, consistent with the initial sway direction as shown in Figure 6(c). This creates eccentricity in the columns. Then, the substantial moments generated at the bottom of columns due to the expansive sway of the ceiling beams and buckling of columns drag the upper structures towards the right, ultimately contributing to structural failure as displayed in Figure 6(c).



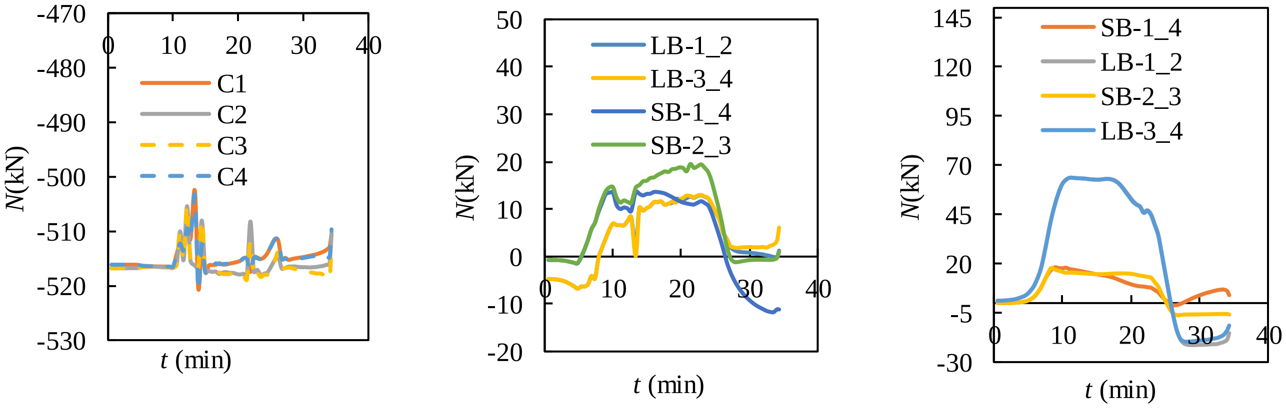
(a) 13 min (b) 25 min (c) 34 min

Figure 6. Different stages of module displacement (Scale: 10)



(a) Displacement along X-direction (b) Displacement along Y-direction (c) Displacement along Z-direction

Figure 7. Displacement of the nodes along X, Y and Z-directions



(a) Columns (b) Ceiling Beams (c) Floor Beams

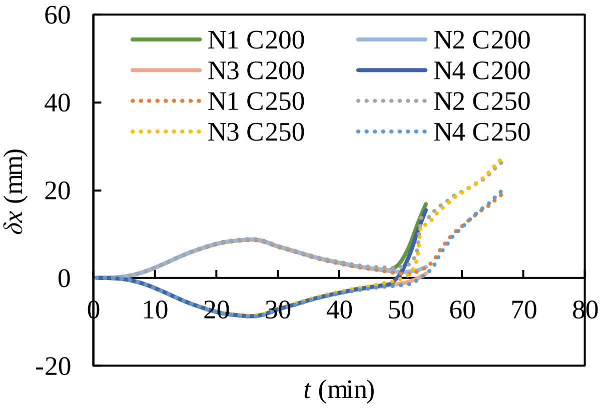
Figure 8. Axial force on different components

4.5 Parametric study

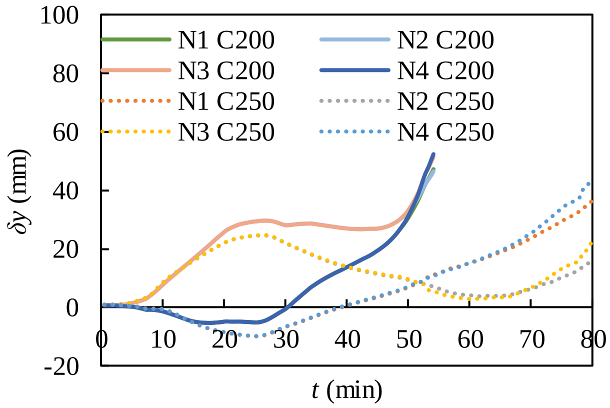
4.5.1 Effect of column sizes on fire resistance

The baseline model has a column size of 150×150×9. Subsequently, two different column sizes are selected for this study 200×200×6 and 250×250×6 and the module used these columns labelled as C200 and C250. As the column sizes increase, the fire resistance of the model is observed to increase from 34 min to 54 min and 80 min as illustrated in Figure 9.

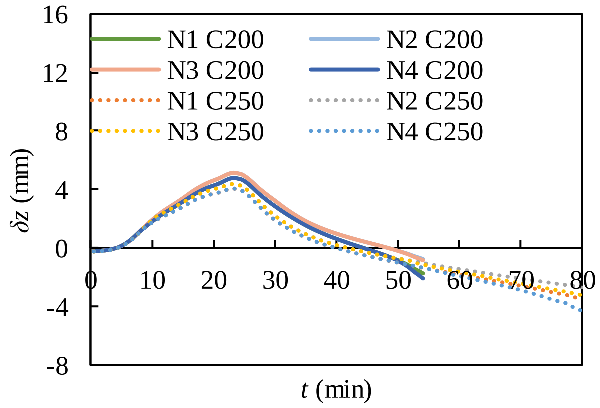
The displacement of nodes N1, N2, N3 and N4 along X, Y and Z directions are presented in Figure 1(b) and Figure 9. The displacement patterns are almost consistent across all three scenarios. The module C250 exhibits greater expansion in the x-direction compared to C200 at failure. Moreover, C250 demonstrates reduced sway in the Y direction, contributing to prolonged module stability. This reduced sway in C250 is advantageous for maintaining stability over an extended period. Additionally, C250 experiences more pronounced contraction during failure, attributed to its higher loading compared to the other configurations.



(a) Displacement along X-direction



(b) Displacement along Y-direction

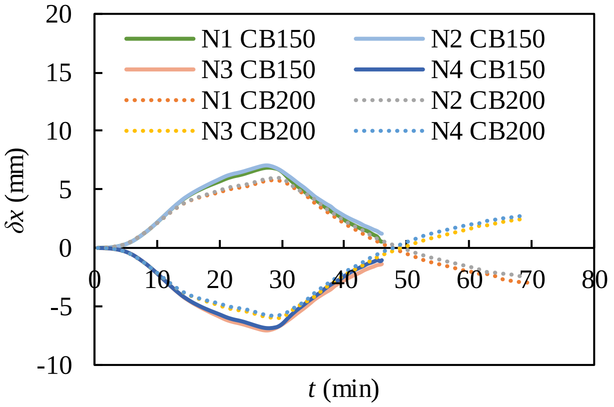


(c) Displacement along Z-direction

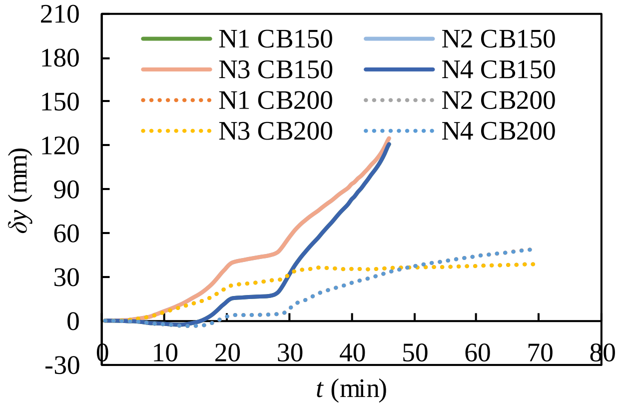
Figure 9. Displacement of the nodes along X, Y and Z-directions

4.5.2 Effect of ceiling beam sizes on fire resistance

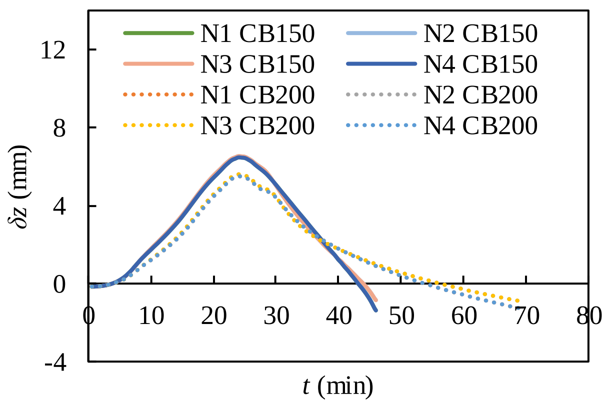
The baseline model, featuring ceiling beams measuring 100×100×5. Two variations in ceiling beam sizes are chosen for investigation such as 150×150×5 and 200×200×5 and labelled the modules with these ceiling beams as CB150 and CB200. When the ceiling beam sizes increase, the model's fire resistance is seen to progressively increase from 34 min to 46 min and 69 min respectively as shown in Figure 10. Figure 10 (b) demonstrates that the larger-sized ceiling beams experience decreased sway in the Y-direction. The slower lateral displacement along the Y-direction contributes to generating fewer moments at the bottom of columns, thus retraining overturning and increasing stability for a longer time.



(a) Displacement along X-direction



(b) Displacement along Y-direction

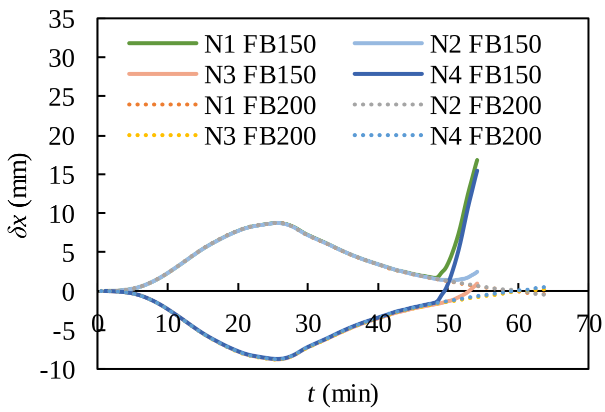


(c) Displacement along Z-direction

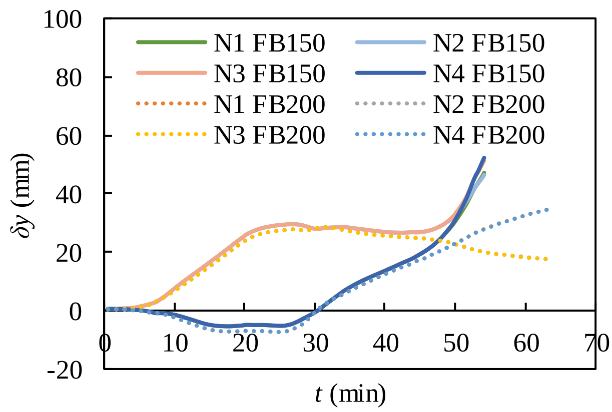
Figure 10. Displacement of the nodes along X, Y and Z-directions

4.5.3 *Effect of floor beam sizes on fire resistance*

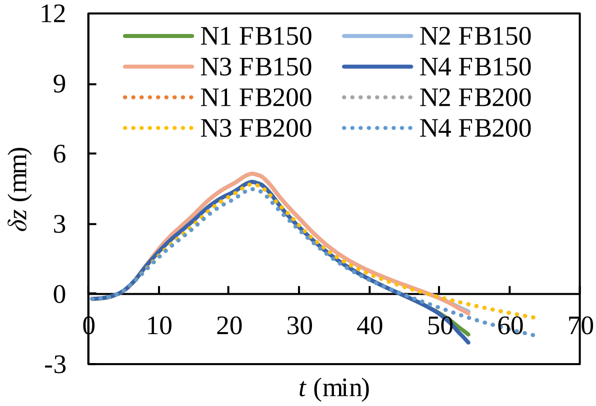
The baseline model is altered with columns measuring 200x200x6, thereby named FB150. Subsequently, the model is analyzed. Then, an additional change is implemented by replacing the 150x100x9mm floor beam of the baseline model with one measuring 200x100x9, resulting module labelled as FB200. With this increase in floor beam size, the fire resistance is increased from 54 minutes to 63 minutes, as depicted in Figure 11. The displacements at the top nodes of the module appear to be slightly minimized by the floor beams.



(a) Displacement along X-direction



(b) Displacement along Y-direction

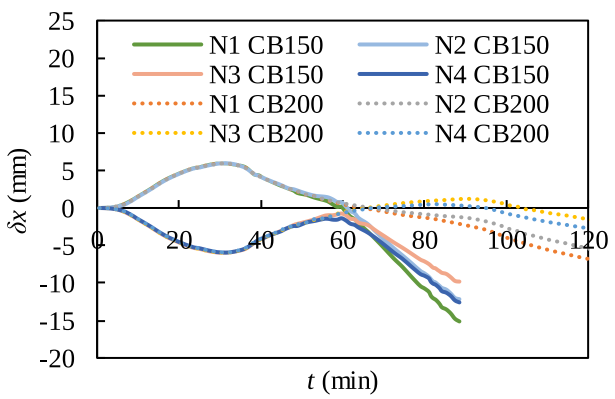


(c) Displacement along Z-direction

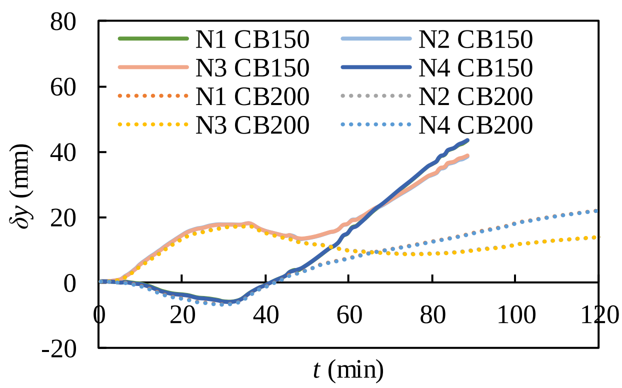
Figure 11. Displacement of the nodes along X, Y and Z-directions

4.5.4 *Effect of changes in multiple component sizes on fire resistance*

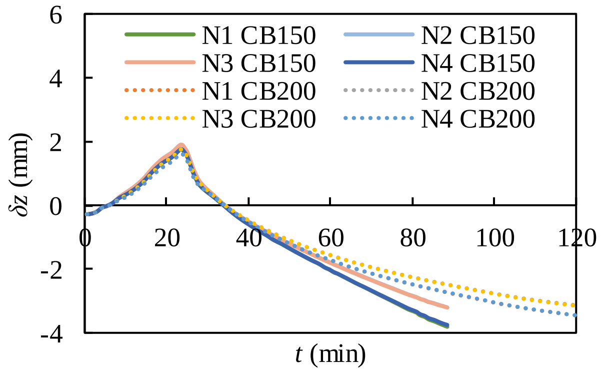
The investigation focuses on the effects of simultaneously altering multiple components within a module. Two distinct modules are examined: one characterized by increased column and ceiling beam dimensions compared to the baseline, with column and ceiling beam sizes of 250x250x6 and 200x200x9 respectively. In the other module, all component sizes are enlarged, including column (250x250x6), ceiling beam (200x200x9) and floor beam (200x100x9). These setups are labelled as CB150 and CB200 in Figure 12. Notably, fire resistance durations of 89 minutes and over 2 hours are exhibited by these configurations, respectively, highlighting the critical role of component sizes in ensuring structural stability during fire incidents. The comparatively low fire resistance of the baseline model can be significantly increased by altering the dimensions of columns, ceiling beams, and floor beams. The gradual increase in displacement can be observed in the CB200 module than others, as shown in Figure 12.



(a) Displacement along X-direction



(b) Displacement along Y-direction



(c) Displacement along Z-direction

Figure 12. Displacement of the nodes along X, Y and Z-directions

5 CONCLUSIONS

This study investigates the response of compartments equipped with CFST columns to fire exposure using advanced numerical analysis techniques. Initially, FE models of CFST columns are developed and validated against experimental data. Subsequently, an FE composite module is constructed to examine the failure mechanisms inherent in different components. Through sensitivity analysis, a design fire curve is selected. Parametric analyse is then conducted across a range of module configurations, varying in sizes of columns, ceiling beams and floor beams. The main conclusions are summarised as follows:

- The global buckling susceptibility is evident in columns when exposed to nonuniform fire exposure, higher in thinner size columns. Additionally, notable expansion of ceiling beams is observed.
- The increase in column size drastically enhances the fire resistance of the modules due to higher sectional capacity, low-temperature rise and observed delayed global buckling.
- The increase in ceiling beam size leads to an increase in fire resistance, with a significant delay in the instability by sway mode compared with the smaller beam size. The increase in floor beam size also improves fire resistance.
- The increment of multiple component sizes significantly enhances module fire resistance by collectively minimizing component expansion and maintaining structural integrity.

Future research will prioritize the integration of fire protection in modules to select the optimal module with reduced weight while ensuring high fire resistance and cost-effectiveness.

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