Case Studies in Fire Safety 4 (2015) 28–38

Contents lists available at ScienceDirect

Case Studies in Fire Safety

journal homepage: www.elsevier.com/locate/csfs

# Fire safety assessment of super tall buildings: A case study on Shanghai Tower

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ARTICLE INFO

*Article history:*
Received 21 February 2015
Received in revised form 11 June 2015
Accepted 27 June 2015
Available online 9 July 2015

*Keywords:*
Fire safety
Case study
Shanghai Tower
Super high-rise
Progressive collapse

ABSTRACT

Shanghai Tower is an existing super high-rise building composed of mega frame-core-outrigger lateral resisting systems. Its structural safety in fire has been given great attention. This paper presents an independent review of the performance of Shanghai Tower in case of fire. Two fire scenarios: standard fires and parametric fires have been considered. The fire resistance of key component, including the concrete core, mega columns, the composite floor, outrigger trusses and belt trusses were examined first. Their real fire resistance periods proved to be far beyond the design fire resistance. The components with weak fire resistance such as peripheral steel columns and web members of belt trusses were then removed to study the resistance of the residual structure against progressive collapse. The results show that Shanghai Tower has a minimum of 3 h fire resistance against fire-induced progressive collapse. The concrete components have smaller residual displacements compared to the steel components. It is recommended, for the design of other similar structures, that effective fire protection should be provided for the outrigger trusses to guarantee the connection between the core and mega columns.


## 1. Introduction

Steel–concrete composite structures combine the advantages of steel and concrete structures which makes them particularly suitable for application in high-rise and super tall buildings. However, steel structures are not inherently fire resistant because much of the strength of steel is lost when its temperature reaches 600 °C or above during a fire. Concrete may suffer spalling at high temperature which may cause premature exposure of reinforcement to fire, leading to severe damage of concrete structures. The likelihood of fire incidents is low. However, due to the high-rise nature of such buildings, the probability of them being subjected to longer duration fire is high, e.g., a terrorist attack. When such an incident occurs, despite fire protection, the likelihood of some members losing their local load-bearing capacity is very high due to a combination of feasible reasons such as more severe fire exposure than designed, loss of fire protection due to impact (the case of World Trade Centre) or lack of durability. Should a structure have low resistance against progressive collapse after

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http://dx.doi.org/10.1016/j.csfs.2015.06.001

local failure of some components, consequent catastrophic progressive collapse could take place, causing tremendous tragedy as a result of loss of lives and property and immeasurable societal impact.

The progressive collapse of structures is defined as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it” [1]. The assessment of collapse performance of structures and measures for the mitigation of disproportionate collapse can be found in various design codes [1–3]. They propose three main design methods such as tie force method, alternate path method and specific local resistance method among which the alternate path method is the most popular one. Progressive collapse is a relatively rare event as it requires both an abnormal loading to initiate the local damage and a structure that lacks adequate continuity, ductility and redundancy to resist the spread of failure. Since the Broadgate Phase 8 fire in London and the subsequent Cardington fire tests, researchers have began to investigate and understand the behavior of whole steel-framed structures in fire. Especially since the collapse of the Word Trade Tower (WTC) under terrorist attack on September 11, 2001, there has been considerable interest in understanding the collapse of tall buildings in fire. Usmani et al. [4–6] carried out a 2D numerical modeling of the WTC tower subjected to fire alone, regardless of the damage caused by the terrorist attack. A possible progressive collapse mechanism for tall frames such as the WTC twin towers was proposed. It showed that the failure of columns played a key role in the collapse of the tower. Ali et al. [7] studied the collapse mode and lateral displacement of single-storey steel-framed buildings exposed to fire. The results showed that the lateral displacement of frames increased with the increase of the spatial extent of fire and roof weight which may affect the minimum clearance between frames and firewalls. Fang et al. [8] conducted a realistic modeling of a multi-storey car park under a vehicle fire scenario. Three failure modes such as single-span failure, double-span failure and shear failure were proposed. Simplified robustness assessment methods of car parks under localized fire were proposed [9,10]. Lange et al. [11] proposed two collapse mechanisms of tall buildings subjected to fire on multiple floors, namely, a weak floor failure mechanism and a strong floor failure mechanism. A simple design assessment methodology was proposed. Sun et al. [12] carried out static-dynamic analyses of progressive collapse of steel structures under fire conditions using Vulcan. The influences of load ratios, beam size and horizontal restraint on the collapse mechanisms were discussed. The same procedure was then used to study the collapse mechanisms of bracing steel frames under fire conditions [13]. Jiang et al. investigated the influence of load ratio, fire scenarios, bracing layout, beam/column stiffness on the resistance of steel framed structures in fire [14–18]. The results showed that the progressive collapse of structures was triggered by buckling of heated columns. The bracing system can effectively enhance the resistance of structures against collapses. Horizontally distributed multi-compartment fires the most dangerous cases.

This paper investigates the performance of the Shanghai Tower against fire-induced progressive collapse. The fire resistance of key components such as the core, mega columns, composite beams and truss systems has been examined in the context of standard and real fire scenarios. The alternative path method is used to study the progressive collapse resistance of residual frame after removing the peripheral steel columns and web members of belt trusses.

## 2. Structural layout of Shanghai Tower

The Shanghai Tower is a mega-tall skyscraper in Lujiazui, Pudong, Shanghai. The building stands approximately 632 m high (structural height is 580 m) and has 124 stories, with a total floor area of 380,000 $m^2$. It is composed of a core-outrigger-mega frame lateral system. The tower structure takes the form of nine functional zones including a business zone at the bottom levels, five office zones, two hotel/apartment zones, and sightseeing floors at the top as shown in Fig. 1. The floor plate

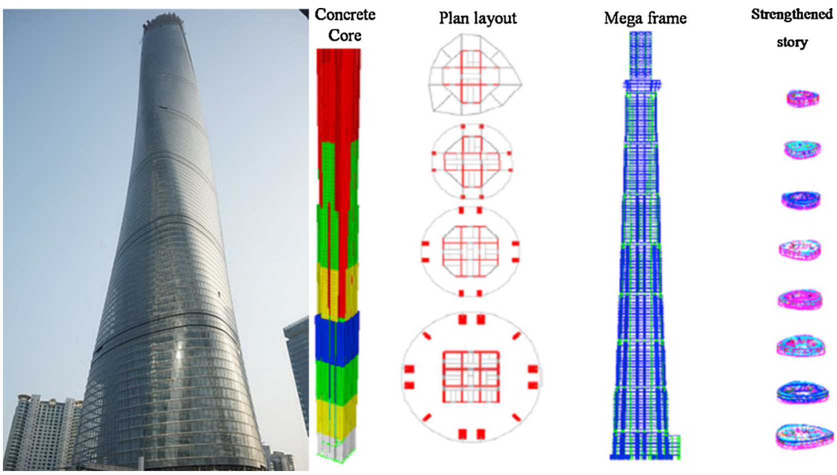



**Fig. 1.** Layout of Shanghai Tower.

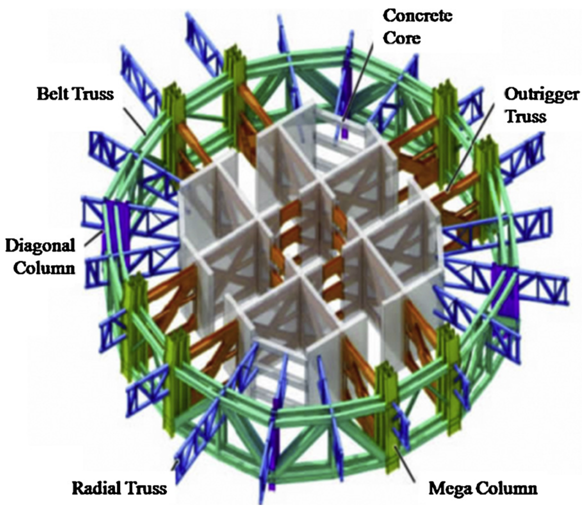



**Fig. 2.** Key components of Shanghai Tower.

diameter varies by zones, from 82.2 m at Zone 1 to 46.5 m at Zone 8. The Shanghai Tower is currently the tallest building in China and the second-tallest in the world, surpassed only by the Burj Khalifa in Dubai.

The lateral system consists of three parts (Fig. 2): concrete composite core, exterior mega frame (super columns, diagonal column and double belt trusses) and outrigger trusses. The exterior mega frame resists about 55% of the base shear and 75% of the overturning moment at the base level under wind and seismic loads.

## 3. Thermal and mechanical loads

The ISO834 standard fire (Eq. (1)) was used in this study. To study the behavior of structures in real fire, a parameterized temperature–time curve was adopted, according to the Eurocodes, based on the compartment size and ventilation conditions as shown in Fig. 3. This parametric fire was chosen to represent a typical office fire.

$$T(t) = T_0 + 345\log_{10}(8t + 1) \tag{1}$$

where $T_0$ is the initial ambient temperature, $t$ is time.

This parametric curve was applied to each of the structural elements in fire, as an office fire is a scenario to which all elements may be exposed.

As a small probability event, fire is generally taken as an accidental load, and an accidental load combination in Eq. (2) was adopted in this study.

$$S = 1.0G_k + 0.7Q_k + 0.3W_k \tag{2}$$

where $G_k$,$Q_k$, $W_k$ are the effects due to the characteristic values of permanent, live and wind load, respectively.

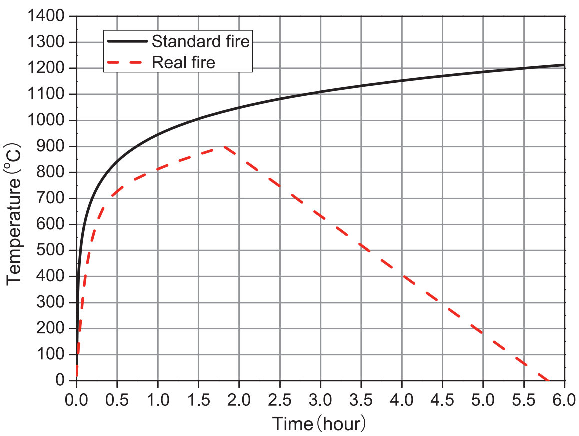



**Fig. 3.** Standard and real fire scenarios adopted.

## 4. Fire resistance of key components

This section focuses on the numerical analysis of the fire resistance of key components under standard fire and parametric fire scenarios using ABAQUS. These components involve steel–concrete composite beams, concrete core, steel reinforced concrete mega columns, outrigger trusses and belt trusses.

### *4.1. Composite beam*

The dimensions of composite beams in Zones 2, 5, 7 are listed in Table 1. The thickness of a typical composite slab was 155 mm (75 mm for steel sheet and 80 mm for flat concrete plate). The characteristic values of concrete compressive strength and steel yield strength were 23.4 MPa (C35 in China) and 345 MPa, respectively. The composite beams taken from the Zones 2, 5, 7 were modeled in ABAQUS as shown in Fig. 4. The steel beam and concrete slab were modeled by solid elements of DC3D8 for heat transfer analysis and C3D8R for mechanical analysis. The steel deck is modeled by shell elements of DS4 for heat transfer analysis and S4R for mechanical analysis. A surface to surface contact is used for the connection between steel beams and steel decks with a "Hard" contact in the normal direction and "Penalty" contact with a friction coefficient of 0.1 in the tangential direction. The same contact is used for that between concrete slabs and steel decks except a friction coefficient of 0.5.

Heat transfer analysis was conducted first and the temperature distribution of beams is shown in Fig. 5. The structural responses of composite beams exposed to the standard fire and real fire were then determined based on the heat transfer results as shown in Fig 6. It can be seen that the beam experiences small midspan deflection (about 100 mm) within the design fire resistance of 3 h. The deflection sharply increased to failure after 4 h fire exposure. For real fire scenarios, the maximum deflection at midspan of beams in the three zones was no more than 20 mm. The residual deformation of the three beams after fire were 15 mm, 6 mm and 2 mm, respectively. The real fire resistance of composite beams was one hour beyond the design value at least. The differences in the deflection of beams in the three zones was controlled by their spans. The enhancement in the fire resistance of composite beams was due to the relatively low temperature distribution in the upper concrete slab.

### *4.2. Concrete core*

The floor systems of the Shanghai Tower transforms from a square at the bottom to a square with truncated corners in the middle, and to the cross shape at the top as shown in Fig. 7. This study considered the shear wall components of cores in Zones 2 and 5 for example. The load level, defined as the ratio of the load applied at elevated temperature to the capacity at ambient temperature was about 0.4. The finite element models and dimensions of shear walls are shown in Fig. 8. The shear walls were modeled by solid elements of DC3D8 for heat transfer analysis and C3D8R for mechanical analysis. A tie contact was used between the surface of concrete wall and steel reinforced members. The height of shear walls was taken as 27 m. The temperature distribution from heat transfer analysis is shown in Fig. 9. The axial displacement of shear walls in the standard fire is shown in Fig. 10. There was no obvious damage in the shear wall and its axial displacement was only 15 mm even after a 6 h fire exposure which shows excellent fire resistance. The super high fire resistance of shear walls was due to its

**Table 1**
Dimensions of composite beams in different zones.

| Zone | Steel beam | Slab height (mm) | Rib height (mm) | Effective slab width (mm) | Beam span (mm) |
|---|---|---|---|---|---|
| Zone 2 | H 606 × 201 × 12 × 20 | 155 | 75 | 2060 | 15,139 |
| Zone 5 | H 396 × 199 × 7 × 11 | 155 | 75 | 2060 | 8600 |
| Zone 7 | H 300 × 150 × 6.5 × 9 | 155 | 75 | 1870 | 5378 |

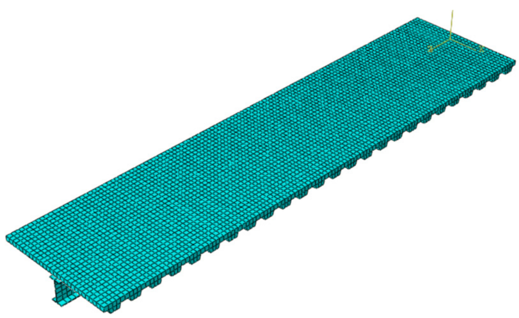
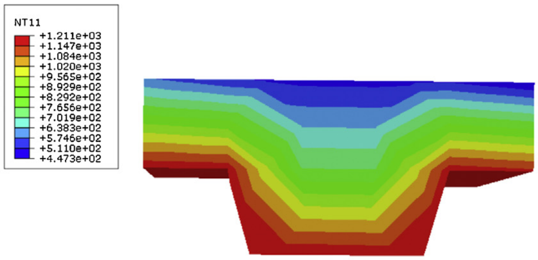
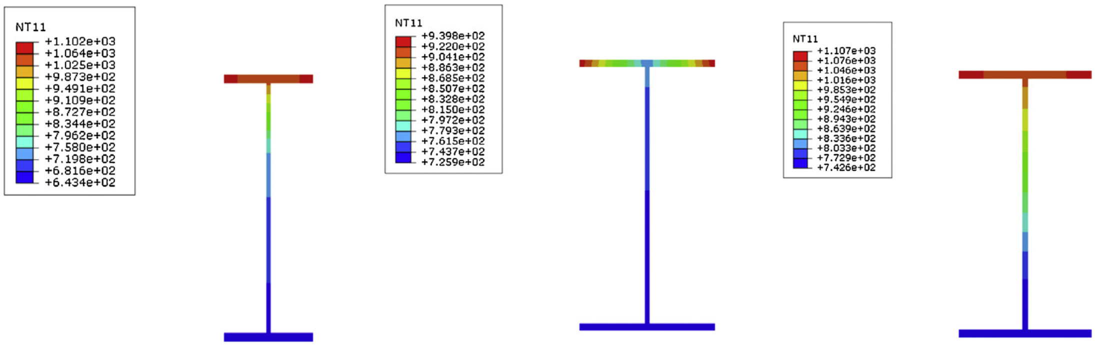


**Fig. 4.** Finite element model of composite beams.




(a) Concrete plate



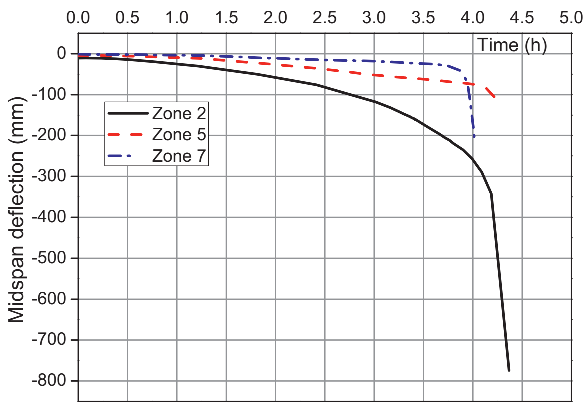

(b) steel beam

**Fig. 5.** Maximum temperature distribution of beams from heat transfer analysis.

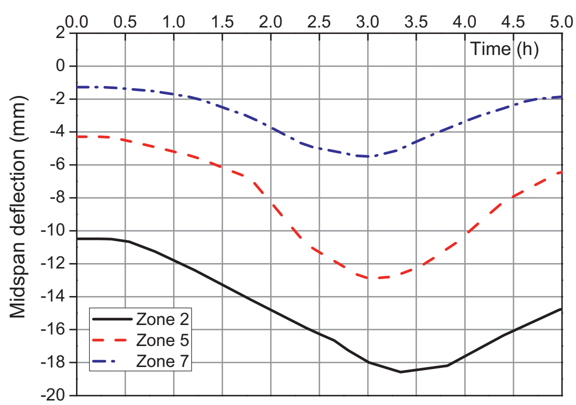
large thickness. It leads to a relatively low temperature in the core of shear walls (about 100 °C) as shown in Fig. 9, meaning full strength of concrete was maintained.

### *4.3. Mega column*

As the key components in the Shanghai Tower, the mega columns resist both vertical and lateral loads and play key roles in the structural stiffness. There are two types of columns as shown in Fig. 11. The finite element model is shown in Fig. 12. The same element and contact types were used as those of concrete core. The heat transfer analysis shows that the temperature of the steel reinforced elements was below 50 °C due to the protection of exterior concrete which greatly strengthened the load-bearing capacity of columns. Fig. 13 shows the axial displacement at the top of columns. It can be seen

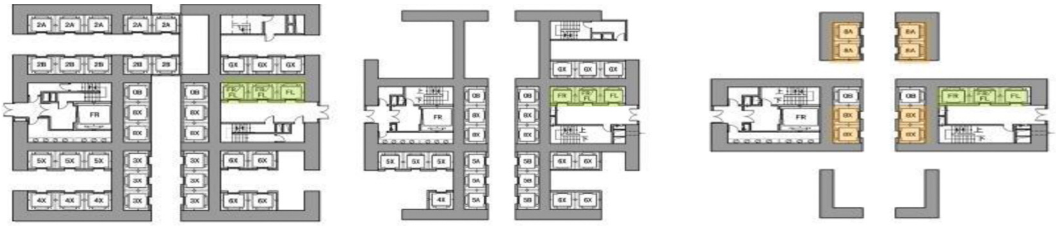



(a) standard fire



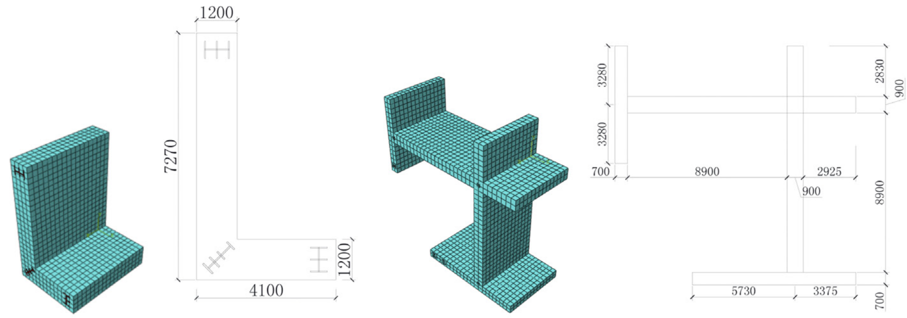

(b) parametric fire

**Fig. 6.** Midspan deflection of composite beams in standard and real fire.



(a) 1-4 zones (b) 5-6 zones (c) 7-8 zones

**Fig. 7.** Typical plan layout of floors.



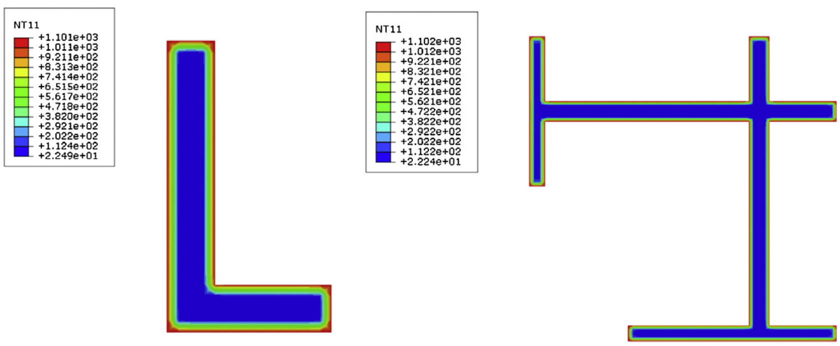

(a) Zone 2 (b) Zone 5

**Fig. 8.** Finite element model of shear wall of concrete core.

that the column moved upward at the early stage of heating due to its thermal expansion and reduced due to material degradation. When these two effects balanced the displacement stayed stable at high temperature. It is interesting to see that there was an ascending trend of axial displacements of columns in the standard fire as shown in Fig. 13a. This may be due to the thermal elongation of columns, overwhelming the material softening. The displacement of columns in the Zones 2 and 7 became stable after 1.5 h and 2 h, respectively, which is smaller than the design fire resistance of 3 h. The fire resistance of the mega columns was enhanced by the reinforced steel members being embedded in the column. They had low temperatures due to their protection of concrete cover. The inner concrete and steel members together contributed to the resistance of mega columns against buckling.

### 4.4. Outrigger truss

Six outrigger truss systems of two storey height are arranged in the Zones 2 and 4–8, respectively as shown in Fig. 14a. The outriggers connect the concrete core and mega columns to reduce the lateral deformation. The web and chord members take




(a) Zone 2 (b) Zone 5

**Fig. 9.** Temperature distribution of shear walls from heat transfer analysis.

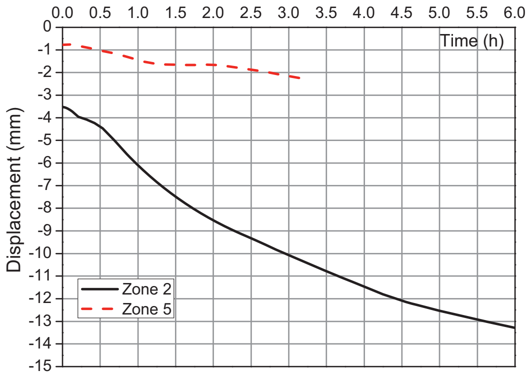
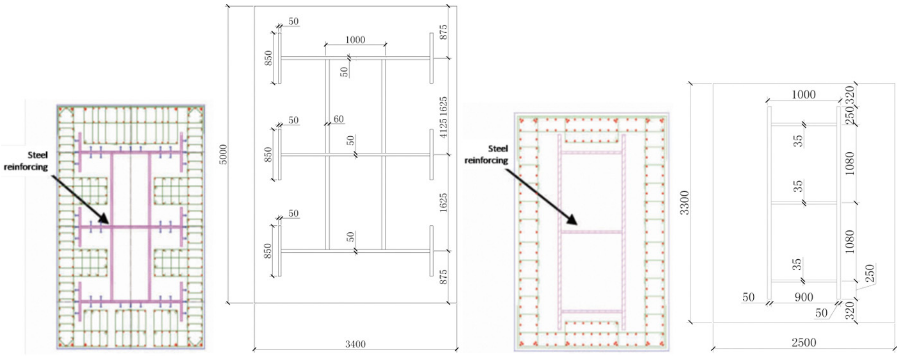


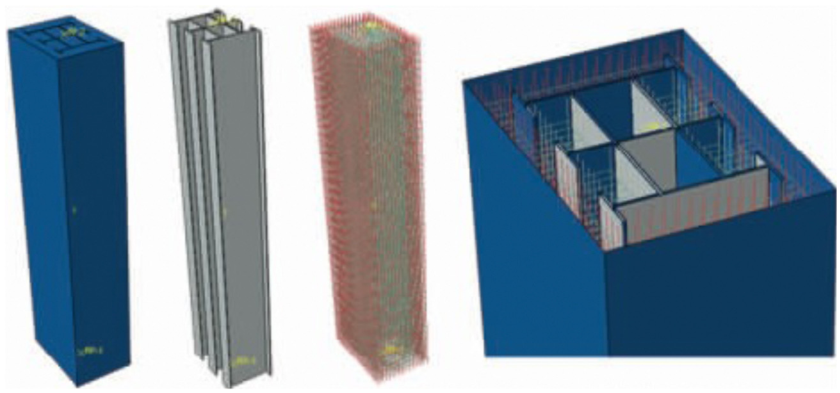

**Fig. 10.** Axial displacement of shear walls in standard fire.




(a) 1-6 zones (b) 7-8 zones

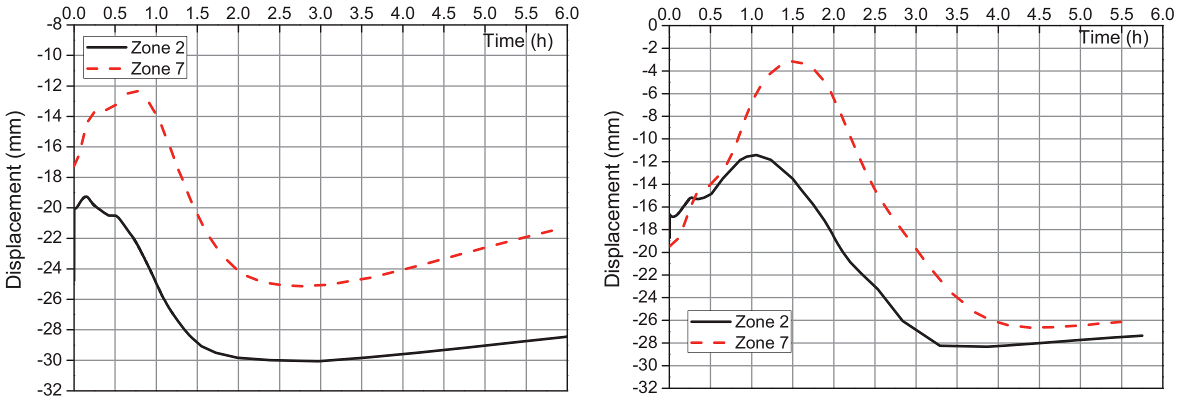
**Fig. 11.** Typical layout of mega columns.

sections of H1000 × 1700 × 100 × 100 and H1000 × 1000 × 90 × 90, respectively. The design yield strength of steel was 325 N/mm$^2$. The outriggers in the Zones 5 and 7 (with a span of 8.6 m and 5.4 m, respectively and the same height of 9.8 m) were studied (Fig. 14b). The detailed dimensions of the outrigger trusses are listed in Table 2. The steel members were modeled by T3D2. The temperature distribution in the trusses was calculated according to EC3. Due to the design fire protection measures, the maximum temperature of outrigger trusses was found to be within the range of 350–550 °C. The midspan deflection of trusses in both the standard and real fires are shown in Fig. 15. There is the same plateau stage of the displacement in the real fire as the mega columns. The outrigger trusses have a fire resistance as high as 6 h due to their heavy fire protection. At ambient temperature, the outrigger trusses, as connections between the internal concrete core and



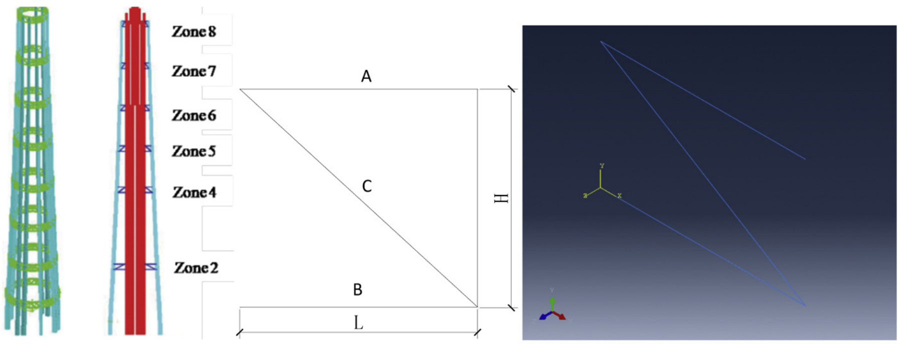
(a) concrete (b) steel reinforcing (c) reinforcements (d) composite columns

**Fig. 12.** Finite element model of mega columns.




(a) standard fire (b) parametric fire

**Fig. 13.** Axial displacement of mega columns exposed to fire.

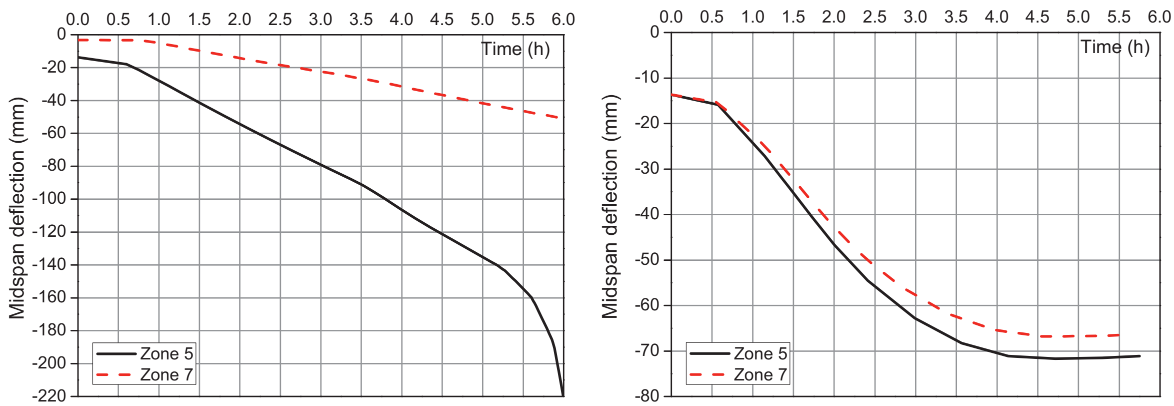



(a) vertical layout (b) finite element model

**Fig. 14.** Finite element model of outrigger trusses.

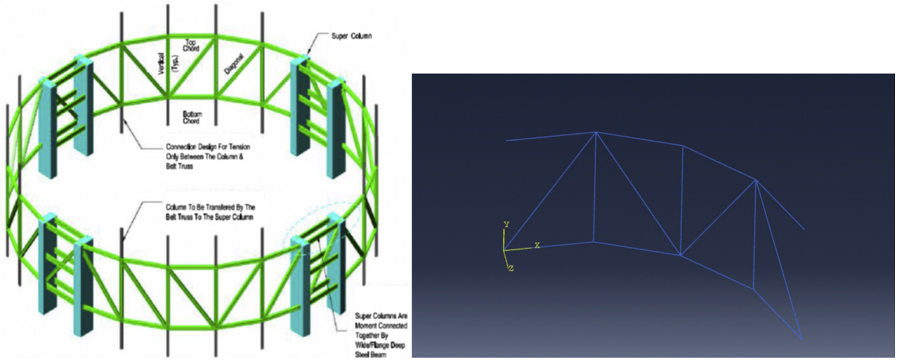
**Table 2**
Dimensions of outrigger trusses in Zones 5 and 7 (A, B, C as shown in Fig. 14).

| | A | B | C |
|---|---|---|---|
| Zone 5 | H 1000 × 1000 × 50 × 55 | H 1000 × 1000 × 60 × 60 | H 1000 × 1600 × 100 × 100 |
| Zone 7 | H 1000 × 1000 × 60 × 60 | H 1000 × 1000 × 80 × 80 | H 1000 × 1000 × 100 × 100 |



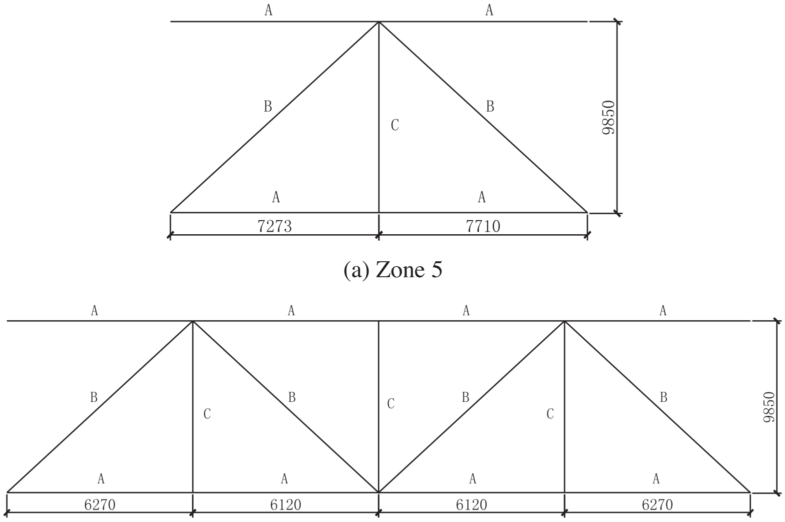

(a) standard fire (b) parametric fire

**Fig. 15.** Midspan deflection of outriggers exposed to fire.




(a) structural layout (b) finite element model

**Fig. 16.** Model of belt trusses.




(a) Zone 5

(b) Zone 7

**Fig. 17.** Dimensions of belt trusses.

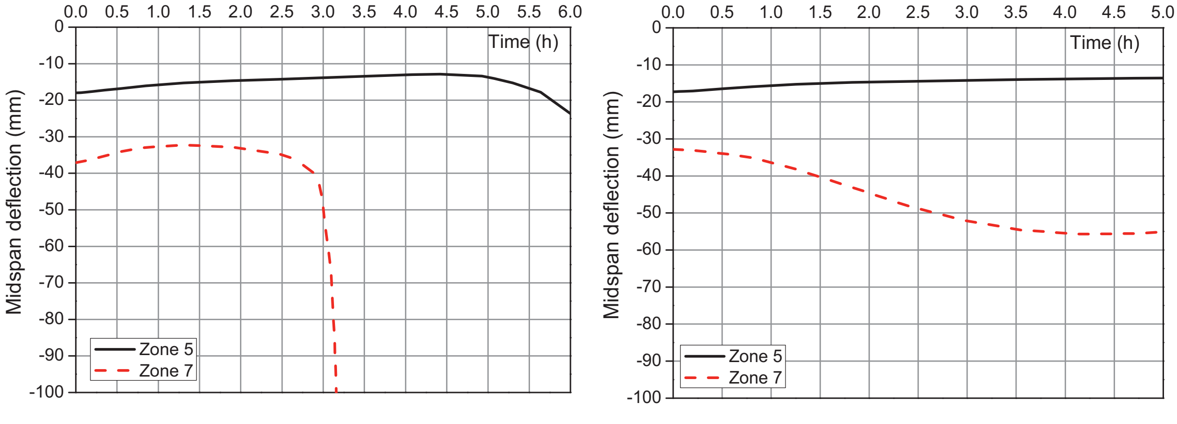
external steel frames, transfer lateral forces at ambient temperature. In fire, however, they act as the horizontal restraint for the mega column and influence its effective length. From the observation and analysis of the collapse of World Trade Center tower in 911, the failure of columns was triggered by their weakened buckling resistance with increasing effective length due to the loss of lateral restraint provided by the floor truss system. This is why the outrigger trusses have been paid additional attention in terms of heavy fire protection.

### *4.5. Belt truss*

The belt trusses in a two-storey height act as an important part of lateral resisting system (Fig. 16) and provide load transfer paths for the periphery of the frame. The belt trusses in the bottom five zones connect the mega columns and diagonal columns with spans varying from 22.4 m in the first zone to 15 m in the fifth zone. The detailed dimensions of belt trusses are listed in Fig. 17 and Table 3. The finite element of T3D2 was used to model the belt trusses. The temperature

**Table 3**
Dimensions of belt trusses in Zones 5 and 7 (A, B, C as shown in Fig. 17)

| | A | B | C |
|---|---|---|---|
| Zone 5 | H 1000 × 550 × 65 × 65 | H 1200 × 550 × 100 × 100 | H 700 × 550 × 35 × 35 |
| Zone 7 | H 1000 × 500 × 30 × 35 | H 850 × 550 × 90 × 90 | H 700 × 500 × 90 × 90 |




(a) standard fire (b) parametric fire

**Fig. 18.** Midspan deflection of belt trusses exposed to fire.

distribution in the trusses was calculated according to EC3. For upper zones, the exclusion of diagonal columns made the truss span increase to 22.3–26.7 m. The midspan deflection of belt trusses is shown in Fig. 18. The displacement of the belt truss in Zone 7 experienced a sharp reduction after a fire exposure of 3 h which should be paid more attention.

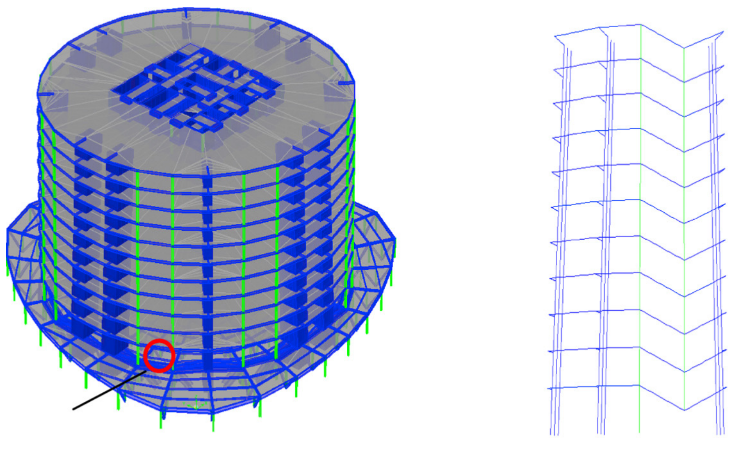
## 5. Progressive collapse of tower

The performance of Shanghai Tower against fire-induced progressive collapse was investigated. The results, from the previous sections, have demonstrated that the key components have satisfactory fire resistance, this is to say, these components will not suffer local failure in the event of fire. Therefore, this study looked to remove the steel column in the peripheral frame and web members in the belt trusses, using ETABS software.

The peripheral steel column on the ninth floor was removed and the deformation of the residual frame is shown in Fig. 19. The residual frame suffered obvious downward displacement but not collapse. Fig. 20 shows the deformation of the belt trusses after removal of a web member. Similarly, no collapse occurred.

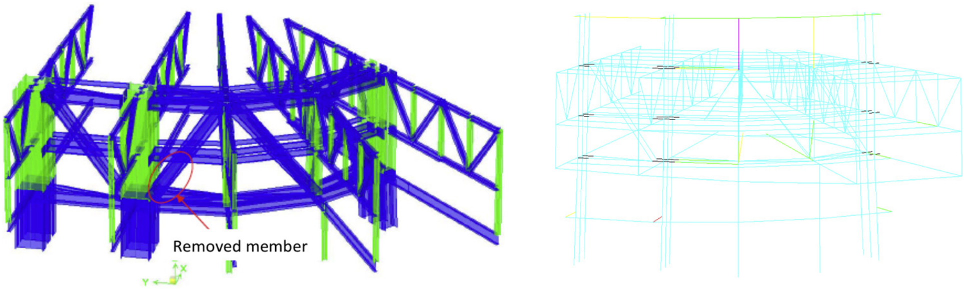
## 6. Conclusions

The fire resistance of the Shanghai Tower against progressive collapse was studied in this paper, as an independent review of the design. The fire resistance of the key components of Shanghai Tower is listed in Table 4. They indicate that their real fire resistances are far beyond their design values (3 h). The concrete components show better fire resistance and smaller residual displacements than the steel components. The high fire resistance of the concrete members is due to the reinforced



(a) removing location (b) deformation after element removal

**Fig. 19.** Progressive collapse analysis of Shanghai Tower by removing peripheral steel column.




(a) removing location (b) deformation after element removal

**Fig. 20.** Progressive collapse analysis of Shanghai Tower by removing a web member in belt trusses.

**Table 4**
Quantitative estimation of fire resistance of key components of Shanghai Tower.

| Key components | Real fire resistance (h) | Residual deformation (mm) |
|---|---|---|
| Composite beam | 4 | 15 |
| Concrete core | >6 | 25 |
| Mega column | >6 | 24 |
| Outrigger truss | 5.5 | 70 |
| Belt truss | 5 | 90 |

steel members being embedded, and the core of concrete was at a low temperature because of the large thickness of concrete cover. It is recommended that effective fire protection should be provided and guaranteed for the outrigger and belt trusses to maintain the connections between the core and mega columns.

## Acknowledgement

The work presented in this paper was supported by the National Natural Science Foundation of China with grants 51408418 and 51120185001.

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