# **Collapse Simulation for a Super-Tall Mega-braced Frame Core-tube Building**

# W.K. Zhang, X. Lu, X.Z. Lu, L.P. Ye & J.R. Qian

Department of Civil Engineering, China Key Laboratory of Civil Engineering Safety and Durability of China Education Ministry, Tsinghua University, Beijing, China.



### SUMMARY:

Earthquake-induced collapse simulation has great significance for the research of super-tall buildings. In this paper, a refined finite element (FE) model of a proposed super-tall mega-braced frame core-tube building (H = 550 m) located in a seismic fortification intensity 8 region in China is established by using fibre-beam model and multi-layer shell model. The earthquake-induced collapse simulation of the building is performed, and the failure mode and mechanism of the earthquake-induced collapse are discussed. This study will provide reference for the collapse-resistant design of super-tall buildings.

Keywords: super-tall building, collapse simulation, fibre-beam model, multi-layer shell model

# **1. INSTRUCTIONS**

With the development of structural materials and construction technology, super-tall building construction has entered into a period of vigorous development. The buildings are rapidly increasing in number and structural height. Super-tall buildings always have complicated structural systems consisting of hundreds of different components, including certain special components with large dimensions. Because super-tall buildings have a large impact on the economy and society, special studies are required on the seismic performance of super-tall buildings. However, only limited research has been reported on the earthquake-induced collapse simulation of a proposed super-tall buildings. Therefore, this paper presents the earthquake-induced collapse simulation of a proposed super-tall building. The research outcome will provide reference for the collapse-resistant design.

# 2. BASIC INFORMATION ABOUT THE SUPER-TALL BUILDING

The research object is a multi-function super-tall office building located in China's seismic fortification intensity region 8. The corresponding PGA (peak ground acceleration) value of the design earthquake (i.e., exceedance probability of 10% in 50 years) is 200 cm/s<sup>2</sup>. The building has 119 stories above the ground with a total height of 550 m. A hybrid lateral-load-resisting system known as the "mega-brace/frame/core-tube/outrigger" is adopted, as shown in Fig. 2.1. The details of this system are described as follows.

Eight reinforced stories (refuge stories) are constructed every 13 to 15 stories, which divide the entire structure into eight zones from the bottom to the top. The planar dimension of the building is a square. The size at the bottom is 68 m  $\times$  68 m. The size of the square first decreases and then increases slightly with the structural height. The minimum size of the square is 50 m  $\times$  50 m in Zone 7, and the planar dimension increases to 60 m  $\times$  60 m at the building's top.

The core tube is a 30 m  $\times$  30 m square RC tube. The thickness of the flange wall of the core tube at the bottom is 1.1 m, and the thickness decreases gradually to 0.5 m at the top with the height of the

building. Compared with the flange wall, the thickness of the core tube's web wall changes little with the height. The thickness of the web wall is 0.4 m in the lower four zones, while the thickness reduces to 0.3 m in the upper four zones.

The mega-braced frame system consists of mega-columns at the four corners. The mega-braces are located in Zones 1 to 4. The closely spaced perimeter columns are located in Zones 5 to 8. The concrete-filled square steel tube (CFST) columns are adopted as the mega-columns, and the maximum cross-sectional dimension is 6500 mm  $\times$  6500 mm at the bottom. The corresponding thickness of the steel tube is 80 mm. All the mega-columns extend from the bottom to the top of the building. The section size decreases gradually to 2000 mm  $\times$  2000 mm, and the thickness of the steel tube decreases to 40 mm. The mega-braces are constructed with welded steel box beams, and the maximum sectional dimension is 1800 mm  $\times$  900 mm  $\times$  110 mm  $\times$  110 mm. The closely spaced perimeter columns in Zones 5 to 8 are steel box columns with a sectional dimension of 700 mm  $\times$  700 mm  $\times$  30 mm  $\times$  30 mm.

There are eight perimeter outriggers from the bottom to the top of the building. Radial outriggers are installed in Zones 5 to 8 to connect the outside frame system and the inner core tube system. The height of the outrigger is 9 m, and all the components of the perimeter and radial outriggers are H-shaped or box-shaped steel beams.



Figure 2.1. The FE model of the super-tall building.

# **3. FINITE ELEMENT MODEL**

The collapse simulation of a super-tall building is a challenging task, which consists of complex structural modelling, solving extremely nonlinear equations and large-scale computation. Based on the general-purpose finite element code MSC.Marc, which has an excellent performance in nonlinear computation, a refined 3D FE model of the super-tall mega-brace frame core-tube structure is constructed with the proposed material constitutive laws, element models and elemental failure criteria. The details of the FE model are described as follows.

# **3.1.** Material constitutive models

Because the fibre-beam element and the multi-layer shell element are adopted in the refined FE model, the macro-scale elemental behaviours (e.g., axial force, bending moment, displacement, rotation etc.)

are directly connected with the micro-scale material constitutive laws (e.g., stress, strain etc.). Therefore, the nonlinear behaviour and failure process of the structural components under complex stress (i.e., the coupled axial force, bending moment and shear force) can be accurately represented. The building materials used in this super-tall building are concrete and steel. Therefore, three types of material constitutive models are adopted in this analysis, including elasto-plastic-fracture concrete constitutive models for the shear walls and coupling beams, a confined concrete constitutive model for the CFST columns, and the elasto-plastic steel constitutive law for the steel reinforcement, steel tubes and steel elements.

The shear walls and coupling beams are modelled with multi-layer shell elements (which will be introduced in detail in the next section). The elasto-plastic-fracture concrete constitutive model provided by MSC.Marc (Miao *et al.* 2011) is adopted. The rationality of this material model is validated by Miao *et al.* (2011), and the numerical results are in good agreement with the experimental results in simulating the mechanical behaviour of reinforced concrete members under complex stress.

The mega-columns are constructed of CFST, in which the confinement effect of the core concrete is special and will have great influence on the mechanical behaviour. Therefore, the confined concrete constitutive model for CFST proposed by Han *et al.* (2001) is adopted. By comparing numerous test results, Han *et al.* (2007) prove that this model can accurately represent the nonlinear behaviour of CFST.

The von Mises yield criterion-based plastic constitutive model (Miao *et al.* 2011) is used for steel. The backbone curve of  $\sigma$ - $\varepsilon$  has four stages, including elastic, yield, hardening and post-capping.

# **3.2. FE model for the core tube**

The shear walls and coupling beams in the core tube are simulated by the multi-layer shell element proposed by Miao *et al.* (2011). The element's schematic diagram is shown in Fig. 3.1. Along the thickness of the shell, it is divided into several layers, and each layer has a different constitutive model including concrete and steel. The multi-layer shell model performs well in simulating the complex nonlinear behaviour of shear walls by considering the bending and shear coupling both in-plane and out-plane. According to the actual reinforcement in the shear wall, in this work 21 layers are adopted in every multi-layer shell element. Miao *et al.* (2011) verified the accuracy and efficiency of the multi-layer shell element model for the shear walls and coupling beams.



Figure 3.1. The schematic diagram of the multi-layer shell element

# 3.3. FE model for the outrigger and mega-brace

In this super-tall building, all the components of external frame, mega-braces, outriggers and closely spaced perimeter columns are constructed with H-shaped or welded box-shaped steel beams except the mega-columns. The fibre-beam model provided by MSC.Marc is used to model these components. To ensure computational accuracy, each segment of the cross section (i.e., the flange and web) is divided into 9 fibres. In total, there are 27 fibres in the H-shaped section and 32 fibres in the box-shaped section, as shown in Fig. 3.2. The fibre-beam element model has been widely used in the elasto-plastic

analysis of earthquake engineering, and the model's accuracy has been verified by much literature (Spacone *et al.* 1996; Tang *et al.* 2011; Li *et al.* 2011; Lu *et al.* 2012).



Figure 3.2. The fibre-beam element model for steel beams. (a) H-shaped; (b) welded box-shaped.

#### 3.4. FE model for the mega-columns

A special component in this super-tall building is the mega-columns located at the four corners. In Zone 1, the mega-column system consists of four CFST columns. From Zone 2, each CFST column has been subdivided into two columns. The maximum cross section of the CFST columns is shown in Fig. 3.3, and the section area is approximately 40 m<sup>2</sup> with a steel ratio of 4.86%.

Because the external steel tube will provide strong confinement for the core concrete, the mega-column is modelled with a fibre-beam element, in which the section of the CFST column is completely divided into 100 fibres, including 64 concrete fibres for the core concrete and 36 steel fibres for the external steel tube. The distribution of fibres in the section is shown in Fig. 3.4. The CFST concrete constitutive model and steel constitutive model described in section 3.1 are adopted for concrete and steel fibres, respectively. The fibre-beam model has been widely used to study the mechanical behaviour of CFST columns (Hajjar *et al.* 1996; Zhang *et al.* 1999; Varma *et al.* 2002; Aval *et al.* 2002; Tiziano *et al.* 2008), and the existing literature indicates that the fibre-beam model performs well in replicating the mechanical behaviour of CFST.



y Concrete fiber Steel fiber

Figure 3.3. Typical cross section of the CFST columns (unit: mm).



#### 3.5. Elemental failure criteria

During collapse, the structure's components will crush or break into many fragments. This phenomenon is simulated by element-deactivation technology. When a specified element-failure criterion is reached, the element will be "deactivated" and removed from the FE model. For the multi-layer shell model, each element has 21 layers. Similarly, for the fibre-beam model, each section has 27~100 concrete or steel fibres, as shown in Figs. 3.2 and 3.4. If the principal compressive strain in a concrete layer/fibre exceeds the crushing strain of concrete (i.e., the softening branch of the concrete approaches zero) or the principal tensile strain in a steel layer/fibre exceeds the fracture strain of steel, the stress and the stiffness of this layer/fibre are deactivated, meaning that this layer/fibre no

longer contributes to the computation of the entire structure. If all the layers of a shell element or all the fibres in a fibre-beam element are deactivated, the element is considered fully deactivated from the model (Lu *et al.* 2008). A small value is set for the stiffness matrix and mass matrix of the corresponding element.

### 4. STRUCTURAL COLLAPSE PROCESS AND MECHANISM

Generally, a super-tall building has a sufficient safety margin to resist the Maximum Considered Earthquake (MCE) ground motion specified in the design code. In the present study, to fully understand the collapse process and mechanism of super-tall buildings, the ground-motion intensity is scaled until the structure collapses. Although an earthquake of the modelled magnitude is unlikely to occur, the ability to understand the mechanical properties of super-tall buildings based on the predicted collapse modes and mechanism will be helpful.

#### 4.1. Elasto-plastic analysis of model subjected to the MCE ground motion

The widely used ground motion recorded at El-Centro station in the USA in 1940 (referred to as "El-Centro" hereafter) was selected as a typical example of ground motion input. The normalized acceleration time history of the east-west components of El-Centro ground motion and its elastic response acceleration spectrum with a 5% damping ratio are shown in Fig. 4.1. The peak ground acceleration (PGA) is scaled to 400 cm/s<sup>2</sup> and 510 cm/s<sup>2</sup>, which correspond to the MCE ground motion in seismic fortification intensity regions 8 and 8.5, respectively. The ground motion is input along the building's Y direction. The distribution of plastic zones for this super-tall building under the above two seismic intensities is shown in Fig. 4.2.



**Figure 4.1.** Normalized acceleration time history and elastic response spectrum with a 5% damping ratio of El-Centro. (a) Normalized acceleration time history of east-west component; (b) elastic response spectrum.



Figure 4.2. The distribution of plastic zones under two different seismic intensities.

Fig. 4.2 indicates that the lateral-load-resisting system of the building remains elastic when subjected to El-Centro ground motion with  $PGA = 400 \text{ cm/s}^2$  and  $510 \text{ cm/s}^2$ . When the  $PGA = 400 \text{ cm/s}^2$ , most plastic zones occur in columns and beams of the external frame in Zones 2 and 3. When the  $PGA = 510 \text{ cm/s}^2$ , more plastic zones expand in the external frame in Zones 2, 3 and 8. Nevertheless, most of the building remains elastic. It can be concluded that this super-tall building has sufficient seismic resistance to resist the earthquake specified in the design code.

# 4.2. Collapse simulation and analysis

The El-Centro ground motion is selected as the typical input in the Y direction to perform the collapse simulation. The intensity of the ground motion is scaled step by step, and the structure starts to collapse when the PGA = 2940 cm/s<sup>2</sup>, which is 9.6 times larger than the actual ground-motion intensity (the actual PGA of the El-Centro ground motion was approximately 307 cm/s<sup>2</sup>). Due to the lack of damping ratio data for super-tall buildings subjected to strong earthquakes, the 5% damping ratio suggested in the Specification for the Design of Steel-Concrete Hybrid Structures in Tall Buildings (CECS 230 : 2008, 2008) is adopted in the analysis. The damping effect is simulated with Rayleigh damping model. The typical collapse model of this building under El-Centro ground motion is shown in Fig. 4.3.

The details of the collapse process are shown in Figs. 4.4 and 4.5. First, when t = 1.461 s (Fig. 4.5a), the shear wall at the bottom of the building begins to fail due to concrete crushing. The failed shear walls are mainly located at the edge of the core tube. The reason for such failure is due to the compressive force. When t = 1.585 s (Fig. 4.5b), most of the shear walls at the bottom of Zone 1 are destroyed, and the coupling beams located in Zones 6 and 7 began to fail due to shear. Next, when t =2.433 s (Fig. 4.5c), more than 50% of the shear walls at the bottom of Zone 1 are destroyed, and the internal forces are redistributed to other components. The vertical and horizontal loads in the mega columns increase gradually and reach their load-carrying capacities. Then, the mega columns in Zones 1 and 2 begin to fail under combined over-turning moment and compression. When t = 3.5 s (Fig. 4.5d), the shear walls at the junction of Zones 6 and 7 are severely damaged and most of the coupling beams in these two zones fail due to shear. Finally, when t = 4.5 s (Fig. 4.5e), the mega columns at the bottom of Zone 1 are destroyed, the core tube at the bottom of Zone 1 is severely damaged and half of the mega-columns in Zone 2 are destroyed. All these failures contribute to the local collapse at the junction of Zones 1 and 2 and then affect the entire building. From the above process, the general structural failure sequence proceeds as follows: from the core tube at the bottom, to the shear walls and coupling beams in the higher zones, and finally to the mega-columns in Zones 1 and 2.







Figure 4.4. Brief collapse process when the super-tall building is subjected to El-Centro ground motion in Y direction



(a) t = 1.461 s, shell walls at the bottom of Zone1 begin to fail



(b) t = 1.585 s, most of the shell walls at the bottom of Zone1 are destroyed



(c) t = 2.433s, mega-columns of Zones 1 and 2 begin to fail



(d) t = 3.500 s, shell walls in Zone 7 begin to fail



(e) t = 4.500 s, all the mega-columns and most of the shell walls at the bottom of Zone 1 are destroyed and structural members at the bottom of Zone 2 are severely destroyed

Figure 4.5. Collapse details when the super-tall building is subjected to El-Centro ground motion in Y direction

The roof displacement time history in the Y and vertical directions when the building is subjected to El-Centro ground motion is shown in Fig. 4.6. The drift ratio between the outriggers is shown in Fig. 4.7. It can be concluded from Fig. 4.1 that because the first- and second-order translational periods are long and the corresponding seismic loads are small, the failure of this super-tall building is dominated by high-order vibration modes. Therefore, as the building approaches collapse, the deformation mode resembles a high-order vibration mode. Fig. 4.6 indicates that the vertical displacement is much larger than the horizontal displacement at the stage of collapse. Fig. 4.7 shows that the mass centre of the structure above the failure region does not undergo significant displacement. Therefore, the main collapse mode of this super-tall mega-brace frame core-tube structure is vertical "pan-cake" collapse, rather than lateral overturning.



Figure 4.6. The vertical and horizontal roof displacement of the super-tall building subjected to El-Centro ground motion



**Figure 4.7.** The distribution of the story drift in direction Y of the super-tall building subjected to El-Centro ground motion. (a) Distribution of drift ratio between adjacent outriggers; (b) distribution of horizontal displacement along the height

The above analysis illustrates that when this building is subjected to El-Centro ground motion, serious damage occurs mainly in the building's lower zones, particularly in Zones 1 and 2. Finally, local collapse occurs at the junction of Zones 1 and 2 and spreads to the entire building. In addition, severe damage occurs at the junction of Zones 6 and 7. These areas are particularly weak zones to structural collapse and more attention should be paid to these areas during inspection to detect earthquake-induced damage.

Figs. 4.2 and 4.7 indicate that the initial plastic zones revealed by traditional nonlinear dynamic analysis may not coincide with the actual collapse regions. In the conventional elasto-plastic analysis under the MCE ground motion, the deformation is mainly concentrated in Zone 6. However, the collapse occurs at the bottom of the building. Therefore, to reveal the structure's actual critical area,

collapse analysis is highly important.

# **5. CONCLUSIONS**

Around the world, a competition in the design and construction of super-tall buildings has started, and the collapse resistance of super-tall buildings has become a research frontier in earthquake engineering. In this paper, a refined finite element (FE) model of a proposed super-tall mega-brace frame core-tube building (H = 550 m) located in the seismic fortification intensity 8 region in China is constructed with the general-purpose finite element software MSC.Marc. The earthquake-induced collapse simulation of the building is successfully implemented with the fibre-beam element, multi-layer shell element and element-deactivation technology proposed by the authors. The research outcome could provide reference for the collapse-resistant design of super-tall buildings.

#### AKCNOWLEDGEMENT

The authors are grateful for the financial support received from the National Nature Science Foundation of China (No. 90815025, 51178249), the National Key Technologies R&D Program (No. 2012BAJ07B012), Tsinghua University Initiative Scientific Research Program (No.2010THZ02-1, 2011THZ03) and the Fok Ying Dong Education Foundation (No. 131071).

#### REFERENCES

- Miao, Z.W., Ye, L.P., Guan, H., Lu, X.Z. (2011). Evaluation of modal and traditional pushover analyses in frame-shear-wall structureures. *Advances in Structural Engineering* **14:5**,815-836.
- Han, L.H., Zhao, X.L., Tao, Z. (2001). Tests and mechanics model of concrete-filled SHS stub columns, columns and beam-columns. *Steel and Composite Structures* **1**:1,51-74.
- Han, L.H. (2007). Concrete-filled Steel Tubular Structureures: Theory and Practice. (in Chinese) Beijing, Science Press, China.
- Spacone, E., Filippou, F., Taucer, F. (1996). Fiber beam-column modeling for non-linear analysis of R/C frames. *Earthquake Engineering and Structural Dynamics* **25**:**7**,711-725.
- Tang, B.X., Lu, X.Z., Ye, L.P., Shi, W. (2011). Evaluation of collapse resistance of RC frame structureures for Chinese schools in seismic design categories B and C. *Earthquake Engineering and Engineering Vibration* 10:3,369-377.
- Li, Y., Lu, X.Z., Guan, H., Ye, L.P.. (2011). An improved tie force method for progressive collapse resistance design of reinforced concrete frame structures. *Engineering Structures* **33:10**,2931-2942.
- Lu, X.Z., Ye, L.P., Ma, Y.H., Tang, D.Y.. (2012). Lessons from the collapse of typical RC frames in Xuankou School during the great Wenchuan Earthquake. *Advances in Structural Engineering* **15:1**,139-154.
- Hajjar, J.F., Gourley, B.C.. (1996). Representation of concrete-filled steel tube cross-section strength. *Journal of Structural Engineering-ASCE* **122:11**,1327-1336.
- Zhang, W.Z., Bahram, M.S. (1999). Comparison between ACI and AISC for concrete-filled tubular columns. *Journal of Structural Engineering-ASCE* **125:11**,1213-1223.
- Varma, A.H., Ricles, J.M., Sause, R. (2002). Seismic behavior and modeling of high-strength composite concrete-filled steel tube (CFT) beam-columns. *Journal of Constructional Steel Research* 58:5-8, 725-758.
- Aval, S.B.B., Saadeghvaziri, M., Golafshani, A.A.. (2002). Comprehensive composite inelastic fiber element for cyclic analysis of concrete-filled steel tube columns. *Journal of Engineering Mechanics* 128:4, 428-437.
- Tiziano, P., Roberto, L.. (2008). Behavior of composite CFT beam-columns based on nonlinear fiber element analysis. *Composite Construction in Steel and Concrete VI*, 237-251.
- Lu, X.Z., Lin, X.C., Ma, Y.H., Li, Y., Ye, L.P.. (2008). Numerical simulation for the progressive collapse of concrete building due to earthquake. *the 14th World Conference on Earthquake Engineering*, CDROM.
- CECS 230 : 2008. (2008). Specification for design of steel-concrete mixed structureure of tall buildings. (in Chinese) Beijing: China Institute of Building Standard Design & Research.