Earthquake-Induced Collapse Simulation of a Super-Tall Mega-Braced Frame-Core Tube Building

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Abstract: Research on earthquake-induced collapse simulation has a great practical significance for super-tall buildings. Although mega-braced frame-core tube buildings are one of the common high-rise structural systems in high seismic intensity regions, the failure mode and collapse mechanism of such a building under earthquake events are rarely studied. This paper thus aims to investigate the collapse behavior of a super-tall mega-braced frame-core tube building ($H = 550$ m) to be built in China in the high risk seismic zone with the maximum spectral acceleration of 0.9 g (g represents the gravity acceleration). A finite element (FE) model of this building is constructed based on the fiber-beam and multi-layer shell models. The dynamic characteristics of the building are analyzed and the earthquake-induced collapse simulation is performed. Finally, the failure mode and mechanism of earthquake-induced collapse are discussed in some detail. This study will serve as a reference for the collapse-resistance design of super-tall buildings of similar type.

Keywords: *super-tall building; collapse simulation; finite element model; fiber-beam model;*

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multi-layer shell model.

1. Introduction

Tall buildings, in particular super-tall buildings, have become important symbols of a country's economic prosperity. The Council on Tall Buildings and Urban Habitat (CTBUH) defines “super-tall” as a building over 300 m in height [1]. Super-tall buildings have become increasingly popular and have large impact on the economy and society. Such buildings have a complicated structural system consisting of hundreds of different components, including those with complex features and large dimensions. To ensure safe and economic design, construction and operation of super-tall buildings under various loading conditions in particular earthquake events, detailed studies are required to examine the seismic performance of super-tall buildings.

The scaled shaking table tests have been widely adopted as a traditional research tool to understand the seismic performance of super-tall buildings. In 2001, Xu *et al.* [2] conducted a 1:25 scaled shaking table test of a reinforced concrete (RC) frame core-tube building of 218 m in height. The dynamic properties and seismic responses of the building under different seismic intensities were evaluated. In 2006, Li *et al.* [3] performed a 1:20 scaled shaking table test of a thirty-four-story RC structure with a 2.7 m height transfer story, by which the seismic responses under small to extreme earthquakes in medium-intensity areas were investigated. In 2006 and 2010, respectively, Lu and his colleagues [4, 5] conducted 1:50 scaled shaking table tests of Shanghai World Financial Center (492 m) and Shanghai Tower (632 m) models. The dynamic properties and seismic responses of these two super-tall buildings under different seismic intensities, according to their located seismic zone, were also evaluated. A similar test

with a 1:20 scale was conducted by Zhao *et al.* [6] in 2008 on the Beijing New Poly Plaza (105.2 m) model. In all the above reported scaled shaking table tests, no collapses were observed in the structural models. Therefore, these tests cannot facilitate further investigation of the collapse mechanisms of super-tall buildings.

On the other hand, the shaking table test is also gradually adopted to study the collapse behavior of structures in recent years. In 2006, Wu *et al.* [7] carried out a 1:3 scaled shaking table collapse test of a one-story, single-bay and three-span RC frame structure. In 2007, a full-scale collapse test of a three-story RC structure with a flexible foundation was performed by Toshikazu *et al.* [8] on the E-Defense shake table. In the following year, a similar test of a four-story steel frame structure was conducted by Yamada and his colleagues [9, 10] and a progressive collapse test of a four-span three-story RC plane frame was conducted by Yi *et al.* [11]. Despite these experimental efforts, all the existing collapse tests were focused on single- or multi-story building models, primarily due to limited capacity of the testing facilities, such as the shaking table scales. Note that the size effect has a significant influence on the collapse test results. This requires a much larger scale (ranging from full scale to 1:4 scale) for conducting collapse tests on a shaking table than that of a regular shaking table tests (ranging from 1:20 to 1:50 scale, e.g., the work done by Li *et al.*, Lu *et al.* and Zhao *et al.* [3-6]). As a consequence, for super-tall buildings of several hundred meters high, studying seismic collapse resistance by means of shaking table tests is both difficult and impractical.

To investigate the seismic performance and collapse mechanisms of buildings, numerical simulation has been shown to be an important alternative research tool. Using LS-DYNA, Lu and Jiang [12] proposed in 2002 a finite element (FE) model of the World Trade Center to

simulate the collapse process induced by aircraft impact and explored the main reasons of progressive collapse. Similarly, in 2002, Quan and Birnbaum [13] simulated the collapse process of the South Tower of the World Trade Center with AUTODYN-3D. In 2004, Luccioni *et al.* [14] simulated the entire collapse process of a seven-story RC structure under blast loads, and the analysis results showed a good agreement with the actual collapse process. In 2007, Huang *et al.* [15] simulated the earthquake-induced collapse of a 115 m high reinforced concrete chimney by using 3-D pushover analysis. In 2009, Fan *et al.* [16] constructed a refined FE model of Taipei 101 with ANSYS by which the seismic performance of the building under different seismic intensities was evaluated. This work illustrated that super-tall buildings with mega-frames have sufficient safety margins and satisfy the design requirements for the Maximum Considered Earthquake (MCE) ground motion specified in the design code.

However, only limited research has been reported on the earthquake-induced collapse simulation of actual super-tall buildings. With the development of structural materials and construction technology, super-tall building construction has entered into a period of vigorous development. Such buildings are rapidly increasing in number and structural height. Therefore, fundamental understanding of the seismic performance and collapse resistance of super-tall buildings will become an important research frontier in earthquake engineering.

This paper presents an earthquake-induced collapse simulation of a super-tall building to be built in China in a high risk seismic region with a maximum spectral acceleration of 0.9 g. A FE model of this building is constructed based on the fiber-beam and multi-layer shell models. The dynamic characteristics of the building are analyzed and the earthquake-induced

collapse simulation is performed. Finally, the failure mode and mechanism of earthquake-induced collapse are discussed in some detail. This study will serve as a reference for the collapse-resistance design of super-tall buildings of similar type.

2. Overview of the super-tall building

The research object is a multi-functional super-tall office building to be built in a region of an 8 degree seismic intensity. The corresponding PGA (peak ground acceleration) value of the design earthquake (i.e., exceedance probability of 10% in 50 years) is 200 cm/s^2 . 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-braced/frame-core tube/outrigger” (Figure 1) is adopted. Details of this system are described as follows:

(1) According to the architectural and fire-safety requirements, eight strengthened 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, with its bottom size of $68 \text{ m} \times 68 \text{ m}$. The size of the square decreases linearly with the building height until reaching a minimum of $50 \text{ m} \times 50 \text{ m}$ in Zone 7, and then increases to $60 \text{ m} \times 60 \text{ m}$ at the top of the building.

(2) The RC core tube is of $30 \text{ m} \times 30 \text{ m}$ in square. For the core tube, the compressive strength of concrete is 38.5 MPa and it remains constant along the height of the core tube. The yield strength of the steel plate embedded in the shear wall is 390 MPa; and the yield strength of the reinforcement in the wall is 335 MPa. The thickness of the flange wall of the core tube

is 1.1 m at the bottom. It decreases gradually with the height of the building down to 0.5 m at the top of the tube. The thickness of the web wall of the core tube, on the other hand, changes little with the height, being 0.4 m in the lower four zones and 0.3 m in the upper four zones.

(3) 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 function as the mega-columns, and their maximum cross-sectional dimension is 6500 mm $\times$ 6500 mm at the bottom. The compressive strengths of concrete in the CFST are 44.5 MPa and 38.5 MPa, respectively, in the lower four zones, and in Zones 5 and 6. For Zones 7 and 8, the compressive strength reduces to 32.4 MPa. The corresponding thickness of the steel tube is 80 mm with a yield strength of 390 MPa. All the mega-columns extend from the bottom to the top of the building. The cross-sectional size decreases gradually to 2000 mm $\times$ 2000 mm, and the thickness of the tube decreases to 40 mm. The mega-braces are constructed from welded steel box beams, with a maximum dimension of 1800 mm (height) $\times$ 900 mm (width) $\times$ 110 mm (thickness of the web) $\times$ 110 mm (thickness of the flange). The closely spaced perimeter columns in Zones 5 to 8 are also in form of steel box with a sectional dimension of 700 mm (height) $\times$ 700 mm (width) $\times$ 30 mm (thickness of the web) $\times$ 30 mm (thickness of the flange). The yield strength of steel in the mega brace is 390 MPa.

(4) There are a total of 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 made of H-shaped or box-shaped steel beams with a

yield strength of 390 MPa.

(5) According to the Loading Code for the Design of Building Structures (GB5009-2001) [17], the wind pressure is about 0.45 kN/m^2 with a 50-year return period and 0.5 kN/m^2 with a 100-year return period. Given the objectives for a higher design performance, the 100-year return period wind pressure is adopted herein to design the strength of the structural components; while the 50-year return period wind pressure is used to assess the horizontal displacement under the wind load. Note that this super high-rise building is located in the high risk seismic zone, therefore the maximum story drift ratio subjected to the serviceability seismic load (i.e., a 25-year return period) is about $1/570$, which is much larger than the maximum story drift ratio subjected to the wind load (i.e., $1/940$). It is evident that these two story drift ratios satisfy the acceptable criteria of the maximum story drift ratio of $1/500$ specified in the Chinese design code, i.e., Code for Seismic Design of Buildings [18]. Thus, the design of this super high-rise building is governed by the seismic load instead of the wind load.

3. Finite element model

Collapse simulation of a super-tall building is a challenging task, which consists of modeling complex structural components, solving highly nonlinear differential equations and performing large-scale computations. Based on the general-purpose finite element code MSC.Marc [19], which has proven performance record in nonlinear computation, a 3D FE model of the super-tall mega-braced frame-core tube structure is constructed with the proposed material constitutive laws, element models and elemental failure criteria. The details

of the FE modeling are described as follows.

3.1 Constitutive material models

Adopted in the FE model are the fiber-beam elements in conjunction with the multi-layer shell elements. These two models have been successfully used in the collapse simulation of a number of high-rise buildings; and detailed mechanisms and validation of these models are given in Lu *et al.* [20]. In the fiber beam element, the cross section of the beam or column is divided into a number of fibers and each fiber exhibits different constitutive material models. In the multi-layer shell element, each element is divided into a number of layers along the thickness direction. The horizontal and the vertical reinforcement of the wall or the embedded steel plate are treated as the equivalent steel layers. When using these elements, the macro-scale elemental behavior (e.g., axial force, bending moment, displacement, rotation etc.) are directly linked to the micro-scale material constitutive laws (e.g., stress, strain etc.). This facilitates accurate representation of the nonlinear behavior and failure process of the structural components under complex stress states (i.e., coupled axial forces, bending moments and shear forces). The building materials used in this super-tall building are concrete and steel. Therefore, three types of constitutive material 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 frames.

The shear walls and coupling beams are modeled with multi-layer shell elements (with details presented in the next section). The elasto-plastic-fracture concrete constitutive model provided by MSC.Marc [19, 21] is adopted. The rationality of this material model is validated

by Miao *et al.* [21] where the numerical results are in a good agreement with the experimental data in simulating the mechanical behavior of reinforced concrete members under complex stress states.

The mega-columns are constructed of CFST, in which the mechanical behavior of the confinement effect of core concrete is a key component in the modeling. In this study, the confined concrete constitutive model for CFST as proposed by Han *et al.* [22] is adopted. The backbone curve of this model can be calculated by Eqs. (1) and (2) below, and a typical stress-strain curve for confined concrete is also presented in Figure 2 [22]. Through comparison with numerous test results, Han *et al.* [23, 24] has proven that this model can accurately represent the nonlinear behavior of CFST.

$$y = 2 \cdot x - x^2 \quad (x \leq 1) \quad (1)$$

and
$$y = \frac{x}{\beta \cdot (x-1)^\eta + x} \quad (x > 1) \quad (2)$$

where
$$x = \frac{\varepsilon}{\varepsilon_0} \quad \text{and} \quad y = \frac{\sigma}{\sigma_0} \quad (3)$$

$$\beta = \begin{cases} \frac{(f'_c)^{0.1}}{1.35\sqrt{1+\xi}} & (\xi \leq 3.0) \\ \frac{(f'_c)^{0.1}}{1.35\sqrt{1+\xi \cdot (\xi-2)^2}} & (\xi > 3.0) \end{cases} \quad (4)$$

$$\eta = 1.6 + 1.5 / x \quad (5)$$

In the above equations, σ_0 and ε_0 are respectively the peak compressive stress and the corresponding peak strain of the core concrete, which are expressed as:

$$\sigma_0 = [1 + (-0.0135 \cdot \xi^2 + 0.1 \cdot \xi) \cdot (\frac{24}{f'_c})^{0.45}] \cdot f'_c \quad (6)$$

$$\varepsilon_0 = (1300 + 12.5 \cdot f'_c) + [1330 + 760 \cdot (\frac{f'_c}{24} - 1)] \cdot \xi^{0.2} \quad (\mu\varepsilon) \quad (7)$$

where ξ is the confinement factor $\xi = (A_s / A_c) \cdot (f_y / f_{ck})$, which reflects the confinement effect of the steel tube, the larger the ξ is, the stronger the confinement effect is; f'_c is the cylinder axial compressive strength of concrete; A_s is the sectional area of the steel tube; A_c is the area of the concrete in the tube; f_y is the yield stress of steel and f_{ck} is the prismoidal compressive strength of concrete, which equals $0.96 f'_c$. The strain corresponding to the $10\% \sigma_0$ is adopted as the ultimate strain ε_u for the confined concrete in the tube.

The von Mises yield criterion-based plastic constitutive model [21] is used for steel. The stress-strain backbone curve exhibits four stages, including elastic, yield, hardening and post-necking. The key points of the steel backbone curve and their corresponding values are shown in Figure 3.

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 elements proposed by Miao *et al.* [21]. A schematic diagram of the element is shown in Figure 4. This type of element is based on the principles of composite material mechanics. The shell is divided into several layers over its thickness and each layer has either concrete or steel constitutive model. The multi-layer shell model performs well in simulating the complex nonlinear behavior of shear walls by considering the coupling effect of bending and both in-plane and out-plane shear. Lu *et al.* [20] and Miao *et al.* [21] have verified the accuracy and efficiency of the multi-layer shell element model for shear walls and coupling beams. According to the actual reinforcement arrangement in the shear wall, a total of 21 layers are adopted in every multi-layer shell element. The FE models of the typical core tubes along the

height of the building are shown in Figure 5. Note that the colors in the figure represent different wall thicknesses, t , in different zones.

3.3 FE model for the outrigger and mega-brace

In this super-tall building, all components of secondary steel frame, mega-braces, outriggers and closely spaced perimeter columns, except the mega-columns, are constructed of H-shaped or welded box-shaped steel beams. The fiber-beam model provided by MSC.Marc [19] 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 fibers. In total, there are 27 fibers in the H-shaped section and 32 fibers in the box-shaped section, as shown in Figure 6. In the refined FE model, the mega-braces are meshed with very fine elements, with more than 5 elements for each structural component. Thus the global buckling of these structural components can be simulated in the collapse analysis. However no global buckling of the mega-braces is observed in the following collapse analysis due to the slab restraints provided to the mega-braces in each floor. Note that the effect of local buckling is not considered in this study in simulating the behaviors of the outriggers and mega-braces. This is because the width-thickness ratio of the web or the flange of the sections satisfies the requirement specified in the Code for Design of Steel Structures [29]. Therefore local buckling of the outriggers or the mega-braces can be effectively prevented. The fiber-beam element model has been widely used in the elasto-plastic analysis of earthquake-induced failure behavior of structures, by which the accuracy of model was verified [25-28].

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 is subdivided into two sub-columns. The maximum cross section of the CFST columns is shown in Figure 7, which is approximately 40 m² with a steel ratio of 4.86%.

Note that the external steel tube provides a strong confinement for the core concrete. To replicate the confinement effect, the mega-column is modeled with a fiber-beam element, in which the section of the CFST column is divided into 100 fibers, including 64 concrete fibers for the core concrete and 36 steel fibers for the external steel tube. The distribution of the fibers in a typical cross-section is shown in Figure 8. The concrete and steel constitutive models for CFST, as described in Section 3.1, are adopted for concrete and steel fibers, respectively. Note that the fiber-beam model has been widely used to study the mechanical behavior of the CFST columns [30-34] and has been demonstrated to perform well in replicating the actual behavior of CFST.

The local buckling behavior and the biaxial stress states of the steel tubes are two important issues in CFST research which has lead to various analytical methods [35-37]. It should be noted that this study places more focus on the global structural seismic behaviors. Note also that for the concerned super-tall building, the cross-section of the CFST columns is considerably large in dimension which requires sufficient shear keys and diaphragms to be welded in the inner steel tube. This offers the ability for concrete to restrain the inward and

outward displacements of the tube thereby preventing the occurrence of local buckling. Hence, for global analysis of the structure, the effect of local buckling of the steel tubes can be neglected in the current fiber-beam element model. In addition, the biaxial stress effect can also be neglected in the finite element model due to its insignificance in global analysis. Similar studies [38, 39] have also demonstrated the applicability and reliability of the fiber-beam element model in predicting the global seismic responses of the CFST structure without considering the effects of local buckling and biaxial stress states.

3.5 Elemental failure criteria

During the process of collapse, the structural components either crush or break into fragments. This phenomenon is simulated with element-deactivation technology. When a specified element-failure criterion is reached, the element is “deactivated” and a small value is set for the stiffness matrix and mass matrix of the corresponding element. In this study, each multi-layer shell element is divided into 21 layers and each section of the fiber-beam element is divided into 27~100 concrete or steel fibers (Figures 6 and 8). If the principal compressive strain in a concrete layer/fiber 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/fiber exceeds the fracture strain of steel, the stress and the stiffness of this layer/fiber are deactivated, meaning that this layer/fiber no longer contributes to the computation of the entire structure. If all the layers of a shell element or all the fibers in a fiber-beam element are deactivated, the element is considered fully deactivated from the model [20, 40].

Generally, confined concrete (e.g., concrete-filled square steel tube columns) exhibits much higher ductility than its unconfined counterpart (e.g., concrete in the cover layer), therefore, different failure criteria for concrete crushing are adopted, as well as different failure criteria for different steel. Details of the failure criteria for concrete and steel are summarized in Table 3.

4. Structural collapse process and failure mechanisms

In general, a super-tall building possesses a sufficient safety margin to resist the MCE ground motion specified in the design code. In the present study, to fully understand the collapse process and failure mechanisms of super-tall buildings, the ground-motion intensity is scaled up until the structure collapses. Although an earthquake of the scaled magnitude is unlikely to occur, the ability to understand the mechanical properties of super-tall buildings based on the predicted collapse modes and mechanisms will be helpful.

4.1 Basic dynamic characteristics

To obtain the basic dynamic properties of this super-tall building, a dynamic modal analysis is performed before the collapse simulation. The total gravity load of the building is 7.534×10^5 tons. The first nine vibration periods and the corresponding modal properties are shown in Table 1 and Figure 9. The translational modal shapes of the building in the Y-direction are similar to those in the X-direction because the planar dimension of the building is a square and the building has a symmetrical layout. These translational modal shapes are common for tall buildings. The fundamental period of this building is

approximately 7.69 s in the Y-direction and 7.44 s in the X-direction, which are beyond the maximum vibration period specified in the design response spectrum in the Chinese Code for the Seismic Design of Buildings (i.e., 6.0 s) [18].

4.2 Elasto-plastic analysis of model subjected to the MCE ground motion

The widely used ground motion recorded at the El-Centro station in the USA in 1940 (referred to as “El-Centro” hereafter) [42] is selected as a typical example of ground motion input. The normalized acceleration time history of the east-west component of the El-Centro ground motion and its elastic response acceleration spectrum with a 5% damping ratio are shown in Figure 10. The PGA is scaled to 400 cm/s^2 and 510 cm/s^2 , which correspond to the MCE ground motion in seismic design intensity 8 and 8.5 regions, respectively [18]. The ground motion input is applied to the Y-direction of the building. The distribution of plastic zones for this super-tall building under the abovementioned two seismic intensities is shown in Figure 11.

Figure 11 indicates that when $\text{PGA} = 400 \text{ cm/s}^2$, most of the plastic zones occur in columns and beams in the secondary steel frame in Zones 2 and 3. When $\text{PGA} = 510 \text{ cm/s}^2$, the plastic zones expand in the secondary steel frame in Zones 2, 3 and 8. These plastic zones are developed as a result of the complicated interaction between the much stiffer mega-braces and the adjoining weaker secondary steel frame. Note that both the inner core tube and the external mega-columns and mega-braces constitute the main lateral-load-resisting system of the super-tall building. Such an arrangement causes excessive axial loads in the mega-braces under seismic loading, which in turn leads to yielding of the secondary steel frame at the

adjoining location to the mega-braces. Despite the existence of the plastic zones, most of the building components remain elastic. It can be concluded that this super-tall building has sufficient seismic resistance to the MCE specified in the design code. The maximal story drift ratio of the building subject to $\text{PGA} = 510 \text{ cm/s}^2$ seismic load is 1/110, which is smaller than the plastic story drift ratio limitation specified in the Technical Specification for Concrete Structures for Tall Building (i.e., 1/100) [43].

4.3 Seismic collapse simulation and analysis

4.3.1 Seismic collapse simulation subjected to El-Centro ground motion

The El-Centro ground motion is also selected as a typical input in the Y-direction to perform the collapse simulation. The intensity of the ground motion is scaled up step by step, and the structure starts to collapse when $\text{PGA} = 2940 \text{ cm/s}^2$, 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^2). Due to the lack of damping ratio data for super-tall buildings subjected to strong earthquakes, a 5% damping ratio suggested in Section 5.3.4 of the Specification for the Design of Steel-Concrete Hybrid Structures in Tall Buildings (CECS 230 : 2008) [44] is adopted in this analysis. The damping effect is simulated with Rayleigh damping model. A typical collapse mode of this building under the El-Centro ground motion is shown in Figure 12. Distribution of the failed elements (i.e., deactivated elements) during the structural seismic collapse is displayed in Figure 13.

The overall and detailed collapse processes are shown in Figures 14 and 15, respectively.

At the initial stage when $t = 1.461$ s (Figure 15a), the shear wall at the bottom of the building begins to fail due to concrete crushing as a result of large compressive forces. The failed shear walls are mainly located at the edge of the core tube. When $t = 1.585$ s (Figure 15b), many shear wall elements at the bottom of Zone 1 are destroyed, and the coupling beams located in Zones 6 and 7 begin to fail due to shear. Subsequently when $t = 2.433$ s (Figure 15c), more than 50% of the shear walls at the bottom of Zone 1 collapsed, and the internal forces are redistributed to other structural 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 (Figure 15d), 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 (Figure 15e), the mega-columns at the bottom of Zone 1 and half of the mega-columns in Zone 2 are destroyed, and the core tube at the bottom of Zone 1 is severely damaged. All these failures contribute to the local collapse at the junction of Zones 1 and 2 which in turn have a significant impact on the entire building. From the collapse process described above, 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.

The roof displacement time history in the Y- and vertical directions when the building is subjected to El-Centro ground motion is shown in Figure 16. The distribution of horizontal displacement along the structural height in the Y-direction of the super-tall building is shown in Figure 17. In the figure, the envelop values refer to as the maximum absolute values

obtained through time-history analysis. It can be concluded from Figure 10 that, due to the long translational periods (first- and second-order) and small magnitude of the corresponding seismic loads, failure of this super-tall building is dominated by higher-order vibration modes, particularly the third translational vibration mode (shown in Figure 9g). Therefore, as the building approaches collapse, the deformation mode resembles a higher-order vibration mode. Figure 16 indicates that the vertical displacement is much larger than the horizontal counterpart at the stage of collapse. Figure 17 shows that the mass center of the structure above the failure region does not undergo significant displacement. Therefore, the main collapse mode of this super-tall mega-braced frame-core tube structure is a vertical “pancake”-type collapse, rather than lateral overturning.

The above analysis illustrates that when this building is subjected to the El-Centro ground motion, severe damage occurs mainly in the lower zones of the building, 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 health monitoring or field inspection to detect earthquake-induced damage.

Figures 11 and 15 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, plastic deformation is mainly concentrated in Zones 2, 3 and 8. However, collapse occurs at the bottom of the building. Therefore, to discover the actual critical area of the structure, collapse analysis is highly important.

4.3.2 Seismic collapse simulation subjected to Kobe ground motion

A similar failure mode and collapse process of the same super-tall building can be observed under other ground-motion inputs. For example, the KOBE-SHI000 (referred to as “KOBE” hereafter) [42] is selected as the input in the Y-direction of the building for a collapse simulation. The ground motion is also scaled up step by step to $PGA = 1764 \text{ cm/s}^2$ until the structure collapses. The normalized acceleration time history of the east-west component of KOBE ground motion and its elastic response acceleration spectrum with a 5% damping ratio are shown in Figure 18.

The overall collapse process is shown in Figure 19. At the initial stage of $t = 12.310 \text{ s}$, the shear wall at the bottom of the building begins to fail due to concrete crushing, and the failure region expands rapidly. When $t = 12.410 \text{ s}$, the coupling beams located in higher zones begin to fail due to shear. Next, when $t = 12.810 \text{ s}$, 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 mega-columns in Zones 1 and 2 begin to fail under combined over-turning moment and compression. When $t = 13.500 \text{ s}$, most of the mega-columns and shell walls at the bottom of Zone 1 are destroyed and the mega-columns in Zone 2 are severely damaged. All these failures lead to the collapse of the entire building.

The roof displacement time history in the Y- and vertical directions when the building is subjected to KOBE ground motion is shown in Figure 20, which indicates that the vertical displacement is much larger than the horizontal one at the stage of collapse. Figure 20 shows

that the mass center of the structure above the failure region does not have significant displacement. Therefore, it can be further confirmed that the main collapse mode of this super-tall mega-braced frame-core tube structure is in the form of vertical “pancake” rather than lateral overturning.

5. Conclusions

Worldwide competitions have rapidly increased in the design and construction of super-tall buildings. The collapse resistance study of these buildings has become a research frontier in earthquake engineering. By using the fiber-beam elements, multi-layer shell elements and element-deactivation technology, the earthquake-induced collapse simulation of an actual super-tall mega-braced frame-core tube building ($H = 550$ m) to be built in China is successfully conducted in this work. Both the El-Centro and KOBE ground motions are selected and scaled up as input to induce collapse of the building. The overall and detailed collapse processes, the critical collapse regions and the corresponding structural responses are reported in some detail. The simulation reveals that the main collapse mode of this super-tall building is of vertical “pancake” type. Furthermore, the actual collapse regions do not necessarily coincide with the initial plastic zones predicted by the traditional nonlinear time-history analysis. Therefore, the collapse simulation and analysis are highly important to help identify the actual critical areas of the structures. This study has provided a feasible methodology for the collapse simulation of super-tall buildings of similar type. It can also serve as a reference for the collapse-resistance design of this type of buildings.

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List of Tables

- Table 1. The first nine vibration modes of the super-tall building.
- Table 2. The relevant parameters for the concrete modeling of the mega-columns
- Table 3. The failure criteria for concrete and steel

List of Figures

- Figure 1. The FE model of the super-tall building..
- Figure 2. Typical stress-strain curves for confined concrete.
- Figure 3. The stress-strain backbone curve of the steel.
- Figure 4. The schematic diagram of the multi-layer shell element.
- Figure 5. The FE models of typical core-tubes.
- Figure 6. The fiber-beam element model for H-shaped or welded box-shaped steel beams.
- Figure 7. Typical cross section of the CFST columns (unit: mm).
- Figure 8. Fiber distributions in a section of CFST column.
- Figure 9. The first nine vibration modes of the super-tall building.
- Figure 10. Dynamic characteristics of El-Centro ground motion.
- Figure 11. Distribution of plastic zones under two different seismic intensities.
- Figure 12. Typical collapse mode of the super-tall building subjected to El-Centro ground motion ($PGA = 2940 \text{ cm/s}^2$).
- Figure 13. Distributions of the failure elements.
- Figure 14. Overall collapse process of the super-tall building subjected to El-Centro ground motion in the Y-direction ($PGA = 2940 \text{ cm/s}^2$).
- Figure 15. Collapse details of the super-tall building subjected to El-Centro

ground motion in the Y-direction ($\text{PGA} = 2940 \text{ cm/s}^2$).

Figure 16. The vertical and horizontal roof displacements of the super-tall building subjected to El-Centro ground motion ($\text{PGA} = 2940 \text{ cm/s}^2$).

Figure 17. Distribution of horizontal displacement along the structural height in Y-direction of the super-tall building subjected to El-Centro ground motion ($\text{PGA} = 2940 \text{ cm/s}^2$).

Figure 18. Dynamic characteristics of KOBE ground motion.

Figure 19. Overall collapse process of the super-tall building subjected to KOBE ground motion in the Y-direction ($\text{PGA} = 1764 \text{ cm/s}^2$).

Figure 20. The vertical and horizontal roof displacement of the super-tall building subjected to KOBE ground motion ($\text{PGA} = 1764 \text{ cm/s}^2$).

Figure 1 The FE model of the super-tall building. (a) Three dimensional view; (b) planar layout of different zones.

Figure 2 Typical stress-strain curves for confined concrete.

Figure 3 The stress-strain backbone curve of the steel.

Figure 4 The schematic diagram of the multi-layer shell element.

Figure 5 The FE models of typical core-tubes. (a) Zone 1; (b) Zones 3-4 junction; (c) Zone 8.

Figure 6 The fiber-beam element model for H-shaped or welded box-shaped steel beams. (a) H-shaped; (b) welded box-shaped.

Figure 7 Typical cross section of the CFST columns (unit: mm).

Figure 8 Fiber distributions in a section of CFST column.

Figure 9 The first nine vibration modes of the super-tall building.

(a) first-order translation in Y-direction; (b) first-order translation in X-direction; (c) first-order torsion; (d) second-order translation in Y-direction; (e) second-order translation in X-direction; (f) second-order torsion; (g) third-order translation in Y-direction; (h) third-order translation in X-direction; (i) third-order torsion.

(a)

(b)

Figure 10 Dynamic characteristics of El-Centro ground motion. (a) Normalized acceleration time history of east-west component; (b) elastic response spectrum with 5% damping ratio.

Figure 11 Distribution of plastic zones under two different seismic intensities. (a) $\text{PGA} = 400 \text{ cm/s}^2$; (b) $\text{PGA} = 510 \text{ cm/s}^2$.

Figure12 Typical collapse mode of the super-tall building subjected to El-Centro ground motion ($\text{PGA} = 2940 \text{ cm/s}^2$).

Figure 14 Overall collapse process of the super-tall building subjected to El-Centro ground motion in the Y-direction ($PGA = 2940 \text{ cm/s}^2$).

Figure 15 Collapse details of the super-tall building subjected to El-Centro ground motion in the Y-direction ($\text{PGA} = 2940 \text{ cm/s}^2$).

Figure 16 The vertical and horizontal roof displacements of the super-tall building subjected to El-Centro ground motion ($PGA = 2940 \text{ cm/s}^2$).

Figure 17 Distribution of horizontal displacement along the structural height in Y-direction of the super-tall building subjected to El-Centro ground motion ($PGA = 2940 \text{ cm/s}^2$).

(a)

(b)

Figure 18 Dynamic characteristics of KOBE ground motion. (a) Normalized acceleration time history of the east-west component; (b) elastic response spectrum with 5% damping ratio.

Figure 19 Overall collapse process of the super-tall building subjected to KOBE ground motion in the Y-direction ($PGA = 1764 \text{ cm/s}^2$).

Figure 20 The vertical and horizontal roof displacement of the super-tall building subjected to KOBE ground motion ($PGA=1764 \text{ cm/s}^2$).