Shaking table test of a supertall building with hinged connections connecting a gravity load resisting system to a lateral force resisting system

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ABSTRACT
Considering a common practical engineering structure, namely, a concrete-filled steel tube (CFST)-steel frame-core tube structure, the rigid connections between the steel beams and the columns/core are modified into hinged connections. In this manner, a 1:40 reduced scale specimen representing a real supertall building that consists of 68 storeys and four underground storeys is constructed for a shaking table test to study the seismic performance of the structure. Numerical simulations are performed to analyse the time history responses of a CFST-steel frame-core tube structural system with three different connection configurations: hinged joints, rigid joints and strengthened layers (which are defined as storeys equipped with outrigger and belt trusses to limit inter-storey drift). The results indicate that the earthquake damage to the structure mainly occurs in the lower storeys at the connections and the corners of the core tube. The seismic response of the structure under the long-period ground motion is significant. The largest inter-storey drift ratio of the core tube reaches 1/26, which exceeds the threshold of the frame-core tube structure system in the Chinese Code by a factor of 3.8. Both the rigid joints and the strengthened layers can reduce the inter-storey drift ratio of the structure.

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1. Introduction
Hybrid structures, such as concrete-filled steel tube (CFST)-steel frame-core tube structures and steel frame-core tube structures, have been extensively applied to practical projects, especially in supertall buildings (Nie et al. 2010). Based on seismic safety specified by the Chinese Code (GB50011-2010 2010), the peripheral frame is required to undertake some of the seismic shear and serves as the second line of earthquake defence (JGJ 3-2010 2010). The internal force usually is transferred to the CFSTs or steel columns through the rigid joints of the steel beams. Furthermore, the cross-sectional dimensions of the beams and columns are increased in order to improve the stiffness of the peripheral frame. In this design method, extensive on-site welding work is required and the design and construction are complicated, which, to a certain extent, counteract the advantage of the prefabricated steel beam, and the “fat beams and columns” design restricts the column spacing of the peripheral frame while constraining the creation of...
new building and structural forms. Recently, bolted connections have been used in many projects (Wang, Tizani, and Wang 2010; Liu et al. 2018). The efficiency of the construction has been greatly increased by the development of the bolted prefabricated steel beam. The fundamental mode of the structural components is affected by the bolted prefabricated beams for connecting the peripheral columns and the central core tube. In this paper, bolted connections are applied to form hinged joints. The peripheral columns (hereafter called gravity columns) of the gravity load resisting system (GLRS) become a “gravity column” that bears only a vertical load (Hu, Zhao, and Qian 2015), thus decreasing the cross-sections of the columns and beams in the design, and the horizontal inertial force of an earthquake is completely supported by the central core tube, which constitutes the lateral force resisting system (LFRS). Therefore, the stresses of the different components are easily determined; there is no need to adjust the shear of the peripheral frame, and the design is therefore simplified. This simplification provides the basis for the creation of the structural form, and the lag of the peripheral frame construction relative to that of the core tube can be remedied. By installing equipment at hinge points, new connection retrofit forms can also be created. Although the structural redundancy is significantly reduced by the hinged joints, related research shows that the structure, decreasing the materials of the peripheral frame to increase the stiffness of the core tube, has an approximate seismic performance and a higher anti-collapse resistance (Lu et al. 2019).

Using hinged connections for the peripheral frame offers the following advantages. (1) The gravity columns do not resist the horizontal inertial force and sustain little damage under an earthquake. Hence, the stresses of the hinged joints are decreased. (2) The seismic performance of the hinged-connection structure is sufficient to prevent severe structural damage, and any damage can be quickly repaired by replacing partially damaged beams or slabs. (3) Using hinged connections to improve the construction efficiency of the peripheral frame significantly decreases the cost of the main structure.

With the development of the prefabricated structure, many studies have been conducted on the hinged joint in a seismic structural system. (Macrae, Clifton, and Mackinven 2010) proposed the slippage rotating “hinged safety box” joint in a frame structure system that is suitable for high-seismic-activity areas. (Psycharis and Mouzakis 2012) studied the seismic performance of a joint-hinged prefabricated concrete frame structure through a shaking table test and suggested that the hinged joints allow for larger rotational deformation without negatively affecting the shear load capacity. (Bournas, Negro, and Molina 2013) studied the seismic performance of a precast concrete structure through pseudo-dynamic experiments and showed that the beam-column hinged system can improve the seismic capability of the structure. (Reyes-Salazar, Cervantes-Lugo, and Baraza 2016) studied the seismic performances of gravity column-steel frame structures with different connection rigidities and found that the ideal hinged steel frame structure can reduce the seismic response and improve the seismic performance with a relatively good economy. (Ahmad and Masoudi 2020; Ahmad et al. 2019, 2017) studied the seismic performance of deficient reinforced concrete frames and highlighted the beam-column rigid connections in which the lateral strength and ductility of the structure are governed by the joint rotational resistance and deformability. Thus, a dual system based on local or global retrofit schemes can be used to enhance the stiffness and lateral strength of the structure. (Li, Hu, and Sun 2008) conducted seismic studies on a hinge-connected steel frame structure with buckling-restrained braces and noted that the new structure offers easy installation and post-seismic repair. The hinge-connected steel frame supports the vertical load, and the buckling-restrained braces resist the horizontal load. A simulation analysis indicated that this structure has good seismic performance. (Zhou et al. 2016) proposed a new structure that combines a steel frame with hinged beam-column connections and buckling-restrained steel plate shear walls. The steel column supports only the vertical load, whereas the lateral load is supported by the buckling-restrained steel plate shear wall. The force of the structure is easily determined, and the details of the connection are simple. A simulation analysis indicates that this new structure performs well under seismic conditions.

This paper is novel in the following ways. (1) The shaking table test for the entire structure, which requires more time and resources than most test methods, is often used to study complex buildings. This experimental method is one of the most effective tests for researching the seismic performance of a structure, and there is a lack of shaking table tests on supertall buildings (CFST-steel frame-core tube structure) with hinged connections, which are used at the ends of all steel beams. (2) Substructure and low-rise structure specimens are usually constructed to study the seismic performance of some structures (Casagrande et al. 2019; Tsampris et al. 2017, 2018; Scodeggio et al. 2016; Lee et al. 2016). However, it is difficult to consider the influence of the high order mode of building structures. Recent investigations have demonstrated that if higher-mode vibrations are neglected for a supertall building, the seismic response will be significantly underestimated (Guan et al. 2019; Behnamfar, Taherian, and Sahraei 2016). The entire structural test should
therefore be conducted to consider the higher-mode effect. (3) All steel beams installed by hinged connections for the entire structural system is a novel point in a shaking table test. The integrity of the structure's security using these connections can be faithfully reflected by the test phenomena of the scale model, representing the supertall building.

In this paper, through a case study of an actual project, shaking table tests are conducted to investigate the earthquake damage and failure behaviour for structures with hinged connections. Four different seismic intensities are used: PGA = 35 cm/s² (minor earthquake in the Intensity 7 Region, which is specified in the Chinese Code (GB50011-2010 2010)); PGA = 100 cm/s² (moderate earthquake in the Intensity 7 Region); PGA = 220 cm/s² (major earthquake in the Intensity 7 Region); and PGA = 400 cm/s² (mega earthquake, which is major earthquake in the Intensity 8 Region). Additionally, the probabilities of exceedance within 50 years were 63% (minor earthquake), 10% (moderate earthquake) and 2–3% (major and mega earthquakes). This project expands the understanding of the seismic performance of the structural system and provides the basis for verifying its seismic design and in-depth theoretical studies.

2. Design of the prototype structure

The prototype structure is a supertall building with four underground storeys, 68 aboveground storeys, and a main body height of 299.7 m. The building is located in an Intensity 7 Region, for which the design peak acceleration for ground motion is specified as 0.10 g in the Chinese Code (GB50011-2010 2010). According to the requirements of the Technical Specification for Concrete Structures of Tall Buildings (JGJ 3-2010 2010) and the Code for Seismic Design of Buildings (GB50011-2010 2010), the prototype structure was designed to meet the requirements for the seismic precautionary Intensity 7 level. The structure has a square plan containing 42 m × 42 m and 23.2 m × 23.2 m core tubes, as shown in Figure 1 (Fang, Wei, and Zhou 2014). The CFST-steel frame-core tube structure is equipped with two outriggers and belt trusses that transmit the overturning moment to the peripheral frame. The storeys equipped with outriggers and belt trusses are referred to as

![Figure 1. Typical design plans of the Guangxi financial plaza (mm): (a) typical floor; (b) belt trusses; (c) elevation layout.](image-url)
Table 1. Distribution of CFST columns and shear walls for the prototype structure (mm).

| Storey | CFST columns (diameter) | Interior wall | Outer wall |
|--------|-------------------------|---------------|-----------|
| 1 ~ 4  | 1800                    | 400           | 1000      |
| 5 ~ 10 | 1800                    | 400           | 900       |
| 11 ~ 14| 1800                    | 400           | 800       |
| 15 ~ 18| 1800                    | 400           | 700       |
| 19     | 1800                    | 400           | 600       |
| 20 ~ 25| 1600                    | 400           | 600       |
| 26 ~ 30| 1600                    | 400           | 500       |
| 31 ~ 32| 1600                    | 400           | 400       |
| 33 ~ 40| 1400                    | 400           | 400       |
| 41 ~ 42| 1400                    | 250           | 350       |
| 43 ~ 50| 1200                    | 250           | 350       |
| 51 ~ 55| 1200                    | 250           | 300       |
| 56 ~ 62| 1000                    | 250           | 300       |
| 63 ~ 68| 800                     | 250           | 300       |

The cross-sectional diameter of the columns gradually changes from 1800 mm at the bottom to 800 mm at the top. The interior wall thickness is 400 mm from the first to the 40th storey and 250 mm from the 41st storey onwards, and the outer wall thickness gradually changes from 1000 mm to 300 mm. The dimensions of the columns, walls and steel beams are detailed in Tables 1 and 2. Each side of a core tube has one 3100-mm-wide wall opening, and the heights of the wall openings change with the storey height, where the heights of the coupling beams are maintained at 800 mm. The storey heights going from the bottom to the top floor are as follows: 6000 mm (the first storey), 5400 mm (from the second to the 17th storey), 3600 mm (from the 18th to the 36th storey), 4200 mm (from the 37th to the 63rd storey), and 5100 mm (from the 64th to the 68th storey). An I-beam is employed in the peripheral frame, although a box girder is used for the outer beam at the strengthened layers. The concrete cubic compression is 60 MPa for the walls and the CFST, 30 MPa for the floor slabs, and 40 MPa for the floor slabs of the strengthened layers.

The building is composed of a reinforced concrete core tube, a peripheral frame and strengthened layers.

Table 2. Sections of steel beams for the prototype structure (mm).

| Storey         | Steel beam (I-beam: b \times h \times t \times t_e or box girder: b' \times h' \times t') |
|----------------|-----------------------------------------------------------------------------------------|
| 1 ~ 3          | 400 \times 1200 \times 25 \times 14                                                    |
| 4 ~ 13         | 300 \times 800 \times 25 \times 12                                                       |
| 14 ~ 16        | 250 \times 600 \times 22 \times 10                                                      |
| 17 ~ 18        | 320 \times 600 \times 25 \times 12                                                      |
| 19 ~ 33, 37 ~ 50, 54 ~ 68 | 280 \times 500 \times 25 \times 10                                                  |
| 34 ~ 36, 51 ~ 53 (strengthened layers) | 320 \times 600 \times 25 \times 12                                                      |
| 34 ~ 36, 51 ~ 53 (strengthened layers) | 800 \times 1000 \times 35                                                              |

b \times h \times t \times t_e: flange width \times web height \times flange thickness \times web thickness; b' \times h' \times t': width \times height \times thickness.

Figure 2. Shaking table test model (mm): (a) model structure; (b) typical floor of the model; (c) bolted connections.
Table 3. Distribution of CFST columns and shear walls for the model structure (mm).

| Storey | CFST columns (diameter) | Wall (thickness) |
|--------|-------------------------|-----------------|
|        |                         | Interior wall   | Outer wall |
| 1      | 1800                    | 550             | 1050       |
| 2      | 1800                    | 550             | 950        |
| 3 ~ 6  | 1800                    | 500             | 950        |
| 7 ~ 8  | 1800                    | 500             | 850        |
| 9 ~ 10 | 1800                    | 500             | 750        |
| 11 ~ 13| 1600                    | 400             | 650        |
| 14 ~ 16| 1600                    | 400             | 550        |
| 17     | 1600                    | 400             | 450        |
| 18 ~ 21| 1400                    | 400             | 450        |
| 22     | 1400                    | 400             | 400        |
| 23 ~ 26| 1200                    | 400             | 400        |
| 27 ~ 28| 1200                    | 400             | 350        |
| 29 ~ 30| 1000                    | 400             | 350        |
| 31 ~ 32| 1000                    | 300             | 350        |
| 33 ~ 37| 900                     | 300             | 350        |

Table 4. Sections of I-beams for the model structure (mm).

| Storey | Interior beam (b × h × t × t_w) | Outer beam (b × h × t × t_w) |
|--------|---------------------------------|-------------------------------|
| 1 ~ 2  | 300 × 1100 × 25 × 22            | 300 × 1100 × 25 × 22          |
| 3 ~ 10 | 300 × 750 × 40 × 14             |                               |
| 11 ~ 37| 300 × 650 × 40 × 12             |                               |

b × h × t × t_w: flange width × web height × flange thickness × web thickness.

To investigate the effect of hinged joints, two strengthened layers are removed here. All the rigid joints of the steel beams of the prototype structure are modified to bolted joints, which are assumed to be hinged joints. These strategies led to two structural subsystems, namely, the GLRS (floors system and gravity columns) and the LFRS (central core tube). We conducted extensive simulations and analyses on the structure with hinged joints, using the PKPM software, which was developed by the China Academy of Building Research. Some necessary conditions for the seismic design of the structural system are summarized as follows: (1) the central core resists 100% of the seismic design loads, and has the necessary lateral stiffness, flexural load capacity, shear load capacity, elastoplastic dissipated energy and anti-collapse resistance; and (2) the floor system is good and has a sufficient stiffness, shear load capacity and axial load capacity to ensure a coordinated deformation between the core tube and the peripheral frame.

3. Shaking table test

3.1. Design of the model structure

The prototype building is a structure that has 68 aboveground storeys. The main features of the prototype’s structure are maintained, and the necessary modifications of the layout plan are made. For instance, secondary beams and secondary elements are neglected in Figure 1(a). The detail modifications are illustrated in the differences between Figures 1(a) and 2(b). After the modification, the two strengthened layers of the prototype structure are removed, and every two adjacent floors are combined into one floor (except for the tall storeys) (Lu et al. 2016). As a result, the 68-storey prototype structure is converted to a 37-storey model structure and the loads of two adjacent floors are applied to one combined floor. The model structure is redesigned using PKPM software, and the cross-sectional dimensions of the columns, walls and beams are given in Tables 3 and Tables 4. The models are established by considering the reduced stiffness and mass matrix (Paz 1989). According to the limitations of the experimental conditions, the distributions of the mass and stiffness do not fully satisfy the ideal model. The simplified framework and core tube are shown in Figure 2(a,b), and the designs of the hinged connection are shown in Figures 2(c) and 3. According to the seismic conditions of precautionary Intensity 7 and the second group of the site soil class III field in the Chinese Code (DBJ/T 15-128-2017 2017), a seismic design analysis is conducted with a fundamental period of $T_1 = 8.486$ s.

Considering the factors of the shaking table size, maximum load, and material properties, the similarity relation of the model is presented in Table 5 (Xu, Xu, and Xiao 2013). Cement mortar, galvanised iron wire and stainless steel are adopted to simulate the
Table 5. Similarity coefficients between the model and the prototype.

| Physical parameters | Similarity constant |
|---------------------|---------------------|
| Stress              | 1/3                 |
| Strain              | 1                   |
| Elastic modulus     | 1/3                 |
| Density             | 4.04                |
| Length              | 1/40                |
| Angular displacement| 1                   |
| Concentrated force  | $1.89 \times 10^{-4}$|
| Bending moment      | $4.73 \times 10^{-4}$|
| Mass                | $6.31 \times 10^{-5}$|
| Frequency           | 10.95               |
| Horizontal acceleration | 3                |

concrete, rebar, steel beam and steel tube. The properties of these materials are listed in Table 6. The horizontal response is the main effect on this structure so the similarity coefficient of vertical acceleration can be ignored in this paper. The hinged joint is simulated by the connection of four bolts, as shown in Figure 2(c). The cantilever segments are installed on the gravity columns and the core tube beforehand, and the steel beams are connected to the cantilever segments using bolts. The bolted joints of the steel beams have considerable rotational capabilities, such that the joints are approximately ideal hinged joints. The thickness of the floor and the additional weight are increased to achieve the mass similarity coefficient. The model is shown in Figure 2.

3.2. Seismic recording and testing work conditions

The ground motion selection principle from a related study (Lu et al. 2015a) is used to select one artificial accelerogram (Q1) and three actual accelerograms (Q2-Q4), which are used widely in the shaking table test, as the input excitations. The similarity coefficients are used to scale the input ground motion waves, and the full-scale waves are shown in Table 7 and Figure 4 for clarity. Q1, Q2, and Q4 have long-period components. In particular, the long-period component of Q4 is rich, and the predominant frequency is less than 1.0 Hz. Q3 is a classic El Centro accelerogram, but the long-period component below 0.2 Hz is absent. During the long-period segment (3.5 s–10 s) in the acceleration response spectra, the Q1–Q3 response spectra are similar to the standard response spectrum, and the smallest and largest responses are the Q3 and Q4, respectively. According to the similarity relation, the peak ground accelerations (PGA) of the input accelerograms are 0.11 g, 0.31 g, 0.67 g and 1.22 g, which are three times greater than the minor, moderate, major and mega earthquake intensities, respectively. Additionally, the earthquake duration is 0.09 times the original duration. Only the X direction and both the X and Y directions are considered in each type of input accelerogram. After each ground motion case, a white noise test is executed to observe the damage occurred. The cases are arranged based on the results for the damage states under a selected accelerogram. Generally, the accelerometers that produce more severe damage should be placed further down in the list. There is a total of 36 cases in the shaking table test (Table 8). 83 acceleration transducers, six displacement transducers and 15 resistance strain gauges were deployed to collect the experimental data.

Long-period ground motion can severely damage high-rise buildings (Koketsu and Miyake 2008), and large responses can be observed in the displacement response spectrum of these buildings.

Figure 4(e,f) show the elastic response spectrums of natural seismic waves, where the natural period of the prototype structure can be seen to be 8.486 s. In the acceleration response spectrum (Figure 4(e)), the ratio of the acceleration at 8.486 s to the maximum acceleration is only 0.53% to 8.67%. However, the large displacement response ranges from 1.09 cm to 5.76 cm at 8.486 s, which is 35.01% to 80.70% of the maximum displacement response in the ground motion spectra. Figure 8 displays the maximum lateral displacement of the model structure obtained from the shaking table test results. Subjecting the structure to the ChiChi wave resulted in a top-story displacement of up to 20 cm, which is 2.5 times larger than 5.76 cm. The displacement response is clearly the

Table 6. Parameters of cement mortar, galvanized iron wire and stainless steel.

| Cement mortar | Galvanized iron wire | Stainless steel |
|---------------|----------------------|-----------------|
| Axial compressive strength (MPa) | 12.3 | 11,423 | 280 |
| Elastic modulus (MPa) | 2 $\times 10^5$ | 300 | 2 $\times 10^3$ |
| Yield strength (MPa) | 280 | 2 $\times 10^5$ | 300 |
| Elastic modulus (MPa) | 2 $\times 10^3$ | 300 | 2 $\times 10^3$ |

Table 7. Ground motions used in the shaking table test.

| Number | Site and time       | Magnitude | Station and direction | Acceleration level | Available low frequency/Hz |
|-------|---------------------|-----------|-----------------------|-------------------|----------------------------|
| Q1    | RGW (Artificial wave) | 6.69      | SOA225                | 0.0643            | 0.2                         |
| Q2    | Northridge, USA1994 | 6.69      | West Covina           | SOA315            | 0.075                       |
| Q3    | Imperial Valley, USA1940 | 6.95 | ELC270                | 0.2107            | 0.2                         |
| Q4    | ChiChi, Taiwan 1999 | 6.2       | CHY056                | 0.0388            | 0.0485                      |
principal factor for the seismic performance of a high-rise building and cannot be neglected, which validates the selected input ground motions.

4. Experimental results and discussion

4.1. Seismic damage

After the impact of minor earthquakes, cracks and deformation were not obvious, and the fundamental frequency was decreased by only 3.82% (1.31 Hz to 1.26 Hz). These results indicated that the structure was generally in the elastic stage, which satisfied the performance requirement of “undamaged under minor earthquakes”. During moderate earthquakes, tiny cracks can be observed at the slab-column connections and slab-core tube connections (Figure 5(a)). These cracks are mainly distributed on storeys 4–10. Under major earthquakes many new cracks are generated on the lower floors. Localized cracking appeared at the connection between the floor bottom and the steel beam, and obvious diagonal cracks appeared on the core tube of the fifth storey (Figure 5(b)). The original cracks in the floor spread out and extended, and the concrete peeled off at certain corners (Figure 5(c)). Deformation occurred in the mid-spans of certain steel beams, but there was no obvious damage to the bolted joints on the ends of the steel beam. Under the impact of mega earthquakes, the original cracks in the floor and the core tube were further developed. The concrete of the core tube corner was crushed at many places on the lower floors, and there was extensive destruction on all floors (Figure 5(d)). However, the surface strain of the gravity columns was always below 1400 με, which indicated that the columns appeared to be essentially undamaged. Although the magnitude of the lateral vibrations under mega earthquakes was very high, the model did not collapse.

In the experimental process, the damage occurred in the following sequence: slab-column connections, slab-core tube connections, slab-beam connections, and the concrete corners of the core tube. The analysis above demonstrates that the floor and the core tube are the main components of the structural damage. Based on the damage of the model structure, the design targets of “undamaged under minor earthquakes, repairable under moderate earthquakes, and not collapsed under major earthquakes” are reached.

4.2. Dynamic characteristics

The data obtained via a white noise scan are analysed using the MEscope software to identify the modal parameters of the system, as reported in Table 9.
Table 8. The detailed testing scheme.

| Case | Type of input accelerogram | PGA (g) | X | Y | Intensity | Intensity | Type of input accelerogram | PGA (g) | X | Y | Intensity | Intensity |
|------|-----------------------------|---------|---|---|-----------|-----------|-----------------------------|---------|---|---|-----------|-----------|
| 1    | White noise scan            | 1.04    | 0.57| 0.31| 32        | 0.11      | White noise scan            | 1.04    | 0.57| 0.31| 32        | 0.11      |
| 2    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 3    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 4    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 5    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 6    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 7    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 8    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |
| 9    | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      | White noise scan            | 1.22    | 0.11| 0.31| 29        | 0.11      |

MEscope software (Vibrant 1991) is a commercial modal analysis program that was developed by Dr. Mark Richardson’s research team and can be used to analyse a variety of noise and vibration problems in machinery and structures. The polyreference complex exponential (PRCE) method is applied to acquire the frequency and viscous damping. As the seismic intensity increased, the natural frequency of the translational modes for the model’s structure gradually decreased, the damping ratio gradually increased, and the damage of the structure constantly accumulated. In particular, the damage from major earthquakes have the largest influence on the structural frequency, followed by that of mega earthquakes, and the influence of minor earthquakes is the smallest. From the “before vibration” (period is 0.76 s) to the “after mega earthquake” (period is 2.94 s), the period of the model structure was elongated by 387%. Based on the dynamic analysis and the experimental observations, the cracks occurring at the slab-core tube connections and the crushing of the corner concrete are the main reasons for the natural frequency decreasing. The damage to the slab-core tube connections results in a sharp decrease in the stiffness. Nevertheless, floors system only suffer from gravity loads after the connection damage. The floor connections are similar to the energy dissipation devices so that the model structure have no obvious collapse phenomenon. During moderate earthquakes, the structural damage in the Y direction is significantly greater than that in the X direction, which implies that there are some different bearing capacities or stiffnesses between the X and Y directions. Both the natural frequency of the torsional modes and damping ratio are steady under different earthquake intensities, and the torsion stiffness of the entire structure is generally degraded.

4.3. Peak strain

Because the seismic response is the largest in the ChiChi accelerogram, measurement sites S14 (corner of the core tube on the fifth storey) and S15 (the gravity column on the fifth storey) under the ChiChi accelerogram were used to realize the strain responses (the strain-time history curve is shown in Figure 6). In Figure 6(a), the concrete in the core tube corner exhibited the maximum strain under the major earthquakes. When the core tube corner was crushed by the mega earthquakes, the concrete had tensile strain (negative value), and the measurement sites failed. The results shown in Figure 6(b) indicate that the gravity column is essentially in an elastic working state.

4.4. Acceleration responses

The acceleration response data obtained from the different cases must be processed for baseline correction and filtering to acquire the available data.
SeismoSignal (SeismoSoft 2015) software is adopted to carry out this processing step and obtain the floor acceleration, displacement and inter-storey drift ratio. With PGA as the reference, the peak acceleration amplification factors of the different storeys are calculated. The maximum results of the data collected for

Table 9. Vibration frequency, damping ratio and mode shape of model structure.

| Dynamic characteristics | Mode 1 | Mode 2 | Mode 3 |
|--------------------------|--------|--------|--------|
| Before vibration         |        |        |        |
| Frequency/Hz             | 1.31   | 1.34   | 4.09   |
| Damping ratio            | 0.0574 | 0.0463 | 0.0347 |
| Mode shape               | Translation in X direction | Translation in Y direction | Torsion |
| After minor earthquake   |        |        |        |
| Frequency/Hz             | 1.26   | 1.27   | 4.12   |
| Damping ratio            | 0.0478 | 0.0437 | 0.0384 |
| Mode shape               | Translation in Y direction | Translation in X direction | Torsion |
| After moderate earthquake|        |        |        |
| Frequency/Hz             | 1.03   | 1.21   | 4.14   |
| Damping ratio            | 0.0492 | 0.0442 | 0.0384 |
| Mode shape               | Translation in Y direction | Translation in X direction | Torsion |
| After major earthquake   |        |        |        |
| Frequency/Hz             | 0.55   | 0.66   | 4.07   |
| Damping ratio            | 0.0635 | 0.0689 | 0.0323 |
| Mode shape               | Translation in Y direction | Translation in X direction | Torsion |
| After mega earthquake    |        |        |        |
| Frequency/Hz             | 0.34   | 0.46   | 3.55   |
| Damping ratio            | 0.0916 | 0.1050 | 0.0451 |
| Mode shape               | Translation in Y direction | Translation in X direction | Torsion |

Figure 5. Local structural damage: (a) tiny cracks at the slab-column connection; (b) diagonal cracks on the core tube; (c) concrete spalling; (d) damaged core tube.
the four earthquakes are taken in the \( X \) and \( Y \) directions (Figure 7). As the structural damages are accumulated, the natural periods increase and the acceleration amplification factors of the different storeys generally decrease. For example, after mega earthquakes the fundamental period is \( T_1 = 2.941 \text{ s} \), but when amplified to the period of the prototype structure, it reached \( T_1 = 32.206 \text{ s} \). In theory, the acceleration response of a long-period structure is similar to the acceleration of the ground motion. In addition, the influence of the high order mode is significant in the long-period structure, which causes the average acceleration amplification factor to be less than one in many storeys during the mega earthquakes. At the top storeys of the structure, due to the whipping effect, the acceleration amplification factor is sharply increased. The cross-sectional dimensions of the core tube are smallest at the top storeys and hinged connections are adopted at the ends of the beams (bolted connections). It remarkably decreases the stiffness of the entire structure, especially the top storeys. It is thus necessary to reduce the weight of the roof and improve the stiffness and ductility of the top storeys.

4.5. Lateral displacement

By subtracting the table surface displacement from the absolute displacement, the relative lateral displacements of the different storeys are obtained. According to the characteristics of the maximum lateral displacement curves (as shown in Figure 8), the curves for moderate earthquakes and mega earthquakes are selected as representative cases. The lateral displacement increases as the earthquake intensity increases, and the displacement response is dominated by the fundamental mode. Due to the rich long-period pulse component of the ChiChi accelerogram, the lateral displacement of the structure is significantly larger than the displacements under the other accelerograms. High order modes should be considered in

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Figure 6. Strain-time history curves under the ChiChi accelerogram: (a) measured point S14; (b) measured point S15.

Figure 7. Acceleration amplification factor.
the supertall building, as it has an obvious effect on the lateral displacement under the mega earthquake. As Figure 8(b) shows, the trend of the displacement curve under the ChiChi accelerometer is different from that under the other accelerograms in the Y direction and its lateral displacement rises more quickly at the top storeys. The structural damage leading to the structure’s period increase and the high order mode becomes a significant factor for the lateral displacement. The natural frequency is also an important factor affecting the lateral displacement. As reported in Table 9, after a minor earthquake the Y direction and X direction frequencies of the specimen are 1.26 Hz and 1.27 Hz, respectively, which are declined to 1.03 Hz and 1.21 Hz after a moderate earthquake, respectively. These frequencies of the specimen thus decline by 18% and 5%, respectively. The corresponding largest lateral displacement in the Y direction is four times that in the X direction. This result implies that the lateral displacement will increase as the natural frequency decreases.

4.6. Inter-storey drift ratio

The inter-storey drift ratio is defined as the relative lateral displacement between adjacent measuring sites divided by the relative height. The maximum inter-storey drift ratios under the different earthquakes are listed in Table 10. Similar to the lateral displacement, the long-period component of the ChiChi accelerometer resulted in the inter-storey drift ratios being larger than those of the other accelerometers. The inter-storey drift ratios in the ChiChi accelerometer exceeded the design thresholds as defined by the Chinese Code (1/500 in a minor earthquake and 1/100 in a major earthquake). After major earthquakes, considerable damage is observed in the lower storeys, and the largest inter-storey drift ratio under mega earthquakes reached 1/26 in storey 29. Because the inter-storey drift ratio under the Chi-Chi accelerometer is remarkable, it is selected for analysis.

The inter-storey drift ratios of the GLRS and the LFRS are then extracted for comparison, which are used to investigate the interaction between them. According to the measured data, the curves of the inter-storey drift ratio versus storey height are plotted in Figure 9. The inter-storey drift profiles of the core tube and frame are the independent envelopes at their peak value moment. Under moderate earthquakes, the curves of the GLRS and the LFRS are similar. The slab-core tube connections are not damaged, and the floor system has enough stiffness and bearing capacity to transfer the horizontal inertial force, ensuring a deformation compatibility between the LFRS and the GLRS. Under both major and mega earthquakes, the plastic deformation and the torsional effect are obvious. This finding suggests that the slab-core tube connections are cracked, and the inter-storey drift ratio between the GLRS and the LFRS is incompatible at the middle and upper storeys.

Figure 9(a) shows a gradual increase in the inter-storey drift ratio going from the bottom to the upper floors under the lower acceleration. Plastic structural damage at the fifth storey (as shown in Figure 5(b)) results in a higher inter-storey drift ratio for the lower storeys than the upper storeys (as shown in Figure 9(b)). The damage phenomenon and the variation in the inter-storey drift ratio of the lower storeys shows that plastic damage causes severe deformation. The stiffness of the lower storeys may be sufficiently large that these storeys are subjected to a higher seismic force than the upper storeys and are damaged first. As the upper storeys are in the elastic stage at this time,

![Figure 8. Lateral displacement responses: (a) moderate earthquake; (b) mega earthquake.](image-url)

Table 10. Maximum inter-storey drift ratios for earthquakes with different intensities.

| Earthquake name | Minor earthquake | Moderate earthquake | Major earthquake | Mega earthquake |
|-----------------|------------------|---------------------|------------------|----------------|
| RGW             | 1/2506           | 1/384               | 1/138            | 1/49           |
| Northridge      | 1/1043           | 1/310               | 1/143            | 1/47           |
| El Centro       | 1/872            | 1/349               | 1/83             | 1/45           |
| ChiChi          | 1/452            | 1/82                | 1/38             | 1/26           |
decreasing the stiffness of the lower storeys would increase the inter-storey drift ratio.

Figure 9 shows the envelope diagrams of the maximum inter-storey drift ratios for the GLRS and the LFRS. During minor, moderate, major and mega earthquakes, the maximum inter-storey drift ratios of the LFRS are 1/521, 1/82, 1/45, and 1/26, respectively, and that of the GLRS are 1/452, 1/84, 1/38, and 1/28, respectively. The maximum value, 1/26, exceeded the threshold defined by the Chinese Code (JGJ 3-2010 2010) by a factor of 3.8. However, the structure did not collapse, which demonstrates that the threshold is too strict.

Figure 10 shows the maximum inter-storey drift ratios consistently appeared in the middle and upper storeys, but the main damage to the
structure occurred in the lower storeys. This result indicates that it is difficult for the inter-storey drift ratio to exactly describe the damaged locations of the floors in the structure. The bending moment cannot be transferred by the steel beams, and the inter-storey drift ratio of the CFSTs is dominated by a harmless inter-storey drift, leading to the result that the appearance of CFSTs have no obvious plastic deformation. This analysis demonstrates the fact that the largest inter-storey drift ratio cannot accurately reflect the damage and collapse of the structures. This point is also verified by a recent relevant study, certifying the hinged joint allowing for a larger rotational deformation of the entire structure (Psycharis and Mouzakis 2012).

4.7. Torsion

According to the lateral displacements on the two sides of the LFRS and the GLRS on the same floor, the torsion angle is calculated. The torsion angle increases as the earthquake intensity increases. When the structure is subjected to minor, moderate, major and mega ChiChi earthquakes, the maximum torsion angles are 1/433, 1/116, 1/67, and 1/49, respectively, which are all greater than those of the other earthquakes. In Figure 11, under the tests for minor and moderate earthquakes, the curves for the LFRS and the GLRS are essentially synchronous, whereas under the mega earthquakes there are large differences between them. The torsion angle of the LFRS peaks at the middle storeys, and that of the GLRS changes abruptly at the lower storeys. In addition, the torsion angle of the LFRS is greater than that of the GLRS under most cases, and the torsional response of the structure is mainly dominated by the LFRS. Due to the small torsion stiffness of the GLRS, a sufficient stiffness is necessary for the LFRS to resist torsion. The floor is thus the key factor in stably transmitting the torque.

4.8. Inertial force and storey shear

The average maximum accelerations and storey mass for different storeys are calculated in order to obtain the seismic inertial force (Zhou and Lu 2016). The seismic acceleration is different in the $X$ and $Y$ directions. The direction with the larger acceleration is defined as the principal direction. The inertial force is usually greater in the principal direction, so it should be the focus of the study. As seen in Figure 12, the ChiChi earthquake is selected as the representative due to the similar test results for the individual earthquakes. The curve of the inertial force shows that first, the seismic inertial force is higher on the lower storeys and the highest at the fifth storey. The reason is that the masses of the lower storeys are much higher than those of the other storeys, which result in the seismic energy dissipation being concentrated in the lower storeys, and the main damage phenomenon appeared in storeys 4–10. Second, although the masses of the top storey are low, the inertial force increased sharply due to the whipping effect. Third, the long-period component of an earthquake has a small influence on the inertial force, with more influence on the inter-storey drift.

The storey shear is the superposition of the seismic inertial forces at higher storeys, and the highest base shear moment is selected to be analysed. The magnitude of the shear stress, complying with the variation in the inertial force, is increased as the earthquake intensity increases. In the middle and upper storeys, the variation in the shear is stable, and the increment rate of the shear in the lower storey first increases and then decreases. The increment rate of shear is the highest at storeys 4–10. The magnitudes of the base shear have the following order: RGW earthquake < Northridge earthquake < ChiChi earthquake < El Centro earthquake. Generally, for mega earthquakes (Figure 13) the earthquake corresponding to the curve of the largest storey shear is the El Centro earthquake, which lacks a long-period

![Figure 11. Comparison of the maximum torsion angle between the GLRS and the LFRS: (a) minor earthquake; (b) moderate earthquake; (c) major earthquake; (d) mega earthquake.](image-url)
component. This result indicates that the storey shear is affected by the long-period component of the earthquakes (such as in the ChiChi earthquake) more obviously than the short-period component. The selection of the earthquake motions is an influencing factor for the storey shear, but it has a more significant influence on the displacement response.

4.9. Ductility demand

It is frequently difficult to meet the ductility requirements of high-rise structures, and Figure 14 indicates how the skeleton curve can be used to estimate the structural ductility from dynamic test results. Figure 15 shows the ductility of each storey. These basic data can be used to analyse the requirements for the overall ductility and the component ductilities for a multi-degree-of-freedom structure.

Due to the influence of the structural torsion, the hysteresis loop exhibits a migration deviating from an ellipse (Figure 14). In this figure, the 25th-storey hysteresis loop under a moderate earthquake is used as an example. The outer contour points are selected along the overall direction of the curve and connected with a smooth line to form a skeleton curve. The ratio between the inter-storey drift of apex and yielding deformation, which is determined by the energy equivalence method, is regarded as the storey ductility demand.

To exclude the interaction effect among earthquakes, only cases with the first earthquake under moderate and major earthquakes are taken as the ductility demands. Figure 15 shows the average storey
Figure 14. Hysteresis loop and skeleton curve of the 25th storey.

Figure 15. Distribution of the inter-storey displacement ductility demand.
ductility demand $\mu$ along different storeys. The ductility demand of the top storey is relatively small, and the maximum ductility demands for moderate and major earthquakes are 2.2 and 3.1, respectively. Under moderate earthquakes, the ductility demand is generally stable for different storeys, with a large ductility required in the lower storeys and the middle and upper storeys. However, as a result of the obvious stiffness difference of the structure between the $X$ and $Y$ directions, the torsional effect and the higher-mode influence become more remarkable under major earthquakes. The average ductility demand of the structure changed abruptly in the middle and upper storeys, and individual storeys exhibited the phenomena of energy concentration.

5. Simulation and analysis

The CANNY (Li 2006) software package, a nonlinear finite element analysis software developed by Li Kang-Ning, was adopted to establish a full-scale numerical model that simulated seismic responses under different cases. The CANNY package has been applied widely to analyse the nonlinear behaviour of buildings, and its reliability has been validated (Li, Tetsuokubo, and Yang 2000; Chong et al. 2013; Cai et al. 2011; Zhou et al. 2013). The dynamic equations are solved by the Newmark-$\beta$ method (Kontoe, Zdravkovic, and Potts 2008; Krause and Walloth 2012). It is difficult to consider the damping ratios in the different levels of input ground motions. The damping ratio is assumed to be uniform value in this simulation. In accordance with the measured values and the Chinese Code (JGJ 3-2010 2010), the classical Rayleigh damping model with a damping ratio of 5% is adopted. The final results are discussed to evaluate the effectiveness of the hinged connections in comparison with the performance of the same structural system but with rigid connections or strengthened layers between LFRS and GLRS.

5.1. Simulation method

Figure 16 shows the element types adopted by the steel beam and the coupling beam, which include bending, shearing, and tensile compressed models. The plastic deformation is not obvious in the steel beams, and only the axial deformation of the beam is considered due to the hinged joints at the ends. Stiffness degradation after yielding is considered, while neglecting the unloading and reloading stiffness degradation. As in the simplification above, the bilinear hysteretic model BL2 is applied for the steel beam. By neglecting the axial deformation of the coupling beams, which are subjected to shear stress and bending moments, the trilinear hysteretic models CP3 and CP7 are used to describe the bending moment and shear performance, respectively. Unlike CP3, the pinching effect of the hysteresis loop is considered in the CP7. The column is idealized by the MS model and MSS model (Li and Otani 1993) (Figure 17). The MS model is a series of three elements: the two-end MS elements and the line element. It has bidirectional bending and shear and axial deformations (tension/compression and torsion). According to the plane-section assumption, the coupling between the bending moment and the axial force is automatically considered, similar to the fibre model. The MSS model is composed of many shear springs and adopted to consider the shear force in the horizontal plane, which represents the shear performance of the column elements. For the excellent bidirectional seismic capacity of the cross-shaped walls inside the core tube, these walls should be regarded as the column model. The peripheral and corner walls of the core tube can be decomposed into multiple planar simplified models, considering only the in-plane capabilities, including bending, shear and axial behaviour. The tops and bottoms of the elements are set as rigid beams, and thus the model is referred to as the "Rigid wall-beam model" (Figure 18).

![Figure 16. Uniaxial spring model.](image-url)
Figure 17. Column element model.

Figure 18. Rigid wall-beam model: (a) wall model with a rigid wall-beam; (b) nodal rotations equal to the wall rotation.

Figure 19. Material constitutive model: (a) steel; (b) concrete.
The steel constitutive model adopted the trilinear hysteretic SS3 model. Figure 19(a) shows the hysteretic SS3 model, and the values of the tensile and compressive strengths $f_{ty}$ and $f_{cy}$ for the steel material are set according to specifications (GB 50010-2010 2010).

The unloading stiffness of the SS3 model is determined by the following equation:

$$K_{su} = \begin{cases} K_c d_m \leq 1.5d_{cy}, d_m \geq 1.5d_{cy} \\ K_c \left( \frac{1.5d_{cy}}{d_{sm} - d_m} \right)^\gamma d_m \geq 1.5d_{cy}, d_m' \leq 1.5d_{cy} \end{cases}$$

(1)

where $K_c$ and $K_{su}$ are the initial stiffness and unloading stiffness, respectively, which are adjusted with the exponential coefficient $\gamma$ (0.5 $\geq$ $\gamma$ $\geq$ 0, and $\gamma$ = 0.4); $d_{cy}$ and $d_{cy}'$ are the displacements corresponding to $f_{ty}$ and $f_{cy}$, respectively; and $d_m$ and $d_m'$ are the displacements corresponding to unloading points $M$ and $M'$, respectively.

The measured stress-strain curve of concrete, experiencing cracking, yielding, and damage stages, shows that there is no obvious cracking point or yield point in the compressed ascending stage. To ensure the convergence stability, the ascending stage adopted two line segments, and the yielding is presumed to be ideally plastic. In the tensile stage, the concrete exhibited brittle behaviour, where the tensile strength is always zero after compressive or tensile failure. The unloading stiffness formula of concrete referred to the Clough model, (Clough 1966) as shown in Equation (2). Figure 19(b) shows the trilinear hysteretic CS3 model, which is similar to Kent-Scott-Park model (Xiao et al. 2014). $\lambda$ = 0 for unconstrained concrete, $\lambda$ = 1 for constrained concrete, and the concrete strength grade is assigned a standard value of $\sigma_c$=36.9 MPa and a tensile strength of $\sigma_t$=2.85 MPa.

The unloading stiffness of the CS3 model is determined by the following equation:

$$K_{cu} = \begin{cases} K_c d_m' \leq d_c \\ K_c \left( \frac{d_c}{d_m'} \right) d_m' > d_c \end{cases}$$

where $K_c$ is the initial stiffness, $K_{cu}$ is the unloading stiffness (we also used $\gamma$ to adjust $K_{cu} (\gamma = 0.4)$). $d_c$ is the displacement corresponding to the compression strength, and $d_m'$ is the displacement corresponding to the unloading point $M$.

5.2. Comparison between the simulation and experimental results

Comparing the results between the simulation and experiment shows that the simulated natural periods are greater than those of the experimental results, and the relative error is 1.64%-5.75% (Table 11). The main reasons for the error are as follows: (1) the material of the specimen is not completely consistent with that of the numerical simulation, and the material of the specimen is difficult to uniformly distribute; (2) the numerical model adopted an ideal hinged joint that is not fully representative of the practical connection; and (3) the specimen is bidirectionally asymmetric, but the numerical model is an ideally symmetric model. The test data for the X principal direction under a representative bidirectional Northridge accelerogram are selected for comparison. Figure 20 shows the comparison of the displacement results at the top storey of the structure. The simulation reflects the variation tendency of displacement precisely, but the extreme points do not completely coincide with the experimental results at certain times. According to the statistics, the average error is 20.6% and the simplification of damping may result in a maximum error of 93.3% after the ground motion is over. However, we are concerned about the peak value of displacement in the time history. The errors of maximum displacement between the simulation and the experiment are 2.3%-9.5%, which already satisfies the accuracy of actual engineering applications. Because the local accumulated damage of the structure, subjected to a mega earthquake, is severe at storeys 4–5, the numerical model is not considered here.

Table 11. Comparison of natural periods.

| Vibration mode | 1     | 2     | 3     | 4     | 5     |
|----------------|-------|-------|-------|-------|-------|
| Simulation results (s) | 8.49  | 8.49  | 2.83  | 2.03  | 2.03  |
| Experiment results (s) | 8.36  | 8.17  | 2.68  | 1.94  | 1.94  |
| Relative error (%)     | 1.56  | 3.95  | 5.75  | 4.61  | 4.35  |

Figure 20. Displacement time history of the top storey under the Northridge accelerogram: (a) minor earthquake; (b) moderate earthquake; (c) major earthquake.
6. Influence of the connection constraints

Three types of structures with different connection constraints are established: the CFST-steel frame-core tube structure with either hinged connections, rigid connections or hinged connections with added strengthened layers, as shown in Figure 21. The strengthened layers of the hinged-connection structure are applied at storeys 18–19 and storeys 27–28.

6.1. Inter-storey drift ratio

Figure 22 shows that under minor and moderate earthquakes, the inter-storey drift ratios of the hinged-connection structure are higher than the other connections at the middle and upper storeys, and the curves of the rigid-connection structure are consistent with those of the strengthened-layer structure. It is shown that under an earthquake with a relatively low intensity, the rigid connections and strengthened layers have similar effects on the inter-storey drift ratio, whereas the hinged connections cannot effectively reduce the inter-storey drift ratio. Under major earthquakes, considerable damage occurred on the middle and lower storeys. The majority of rigid joints are so seriously damaged as to form plastic hinge with degraded strength. For the rigid connections and the hinged connections, the inter-storey drift ratio above storey 15 changed rapidly. In contrast, the seismic force is distributed to the GLRS by the strengthened layers, such that the inter-storey drift ratio changed slowly.

In summary, the trends of the inter-storey drift ratio curve for structures with different connection constraints are similar and essentially overlapped with the lower storeys. The highest inter-storey drift ratio is found for the hinged-connection structure, and the curve of the strengthened-layer structure, which can better control the inter-storey drift ratio, is generally lower than that of the other structures. The inter-storey drift ratio of the strengthened-layer structure is 0.62–0.89 times, on average, that of the hinged-connection structure. Table 12 presents thresholds for different damage levels of the hinged-connection structure based on technical specifications and the literature (JGJ 3-2010 2010; Deng 2006). The structure with

![Figure 21. Three types of connections.](image1)

![Figure 22. Distribution of the average inter-storey drift ratio under each earthquake level.](image2)
rigid connections, strengthened layers and hinged connections satisfies the requirement that the inter-storey drift ratio be smaller than 1/500 under minor earthquakes and smaller than 1/150 under major earthquakes. Figure 22 shows the hinged connections that are adopted in the strengthened-layer structure, thereby reducing the inter-storey drift ratio of the resulting structure to below that of the rigid-connection structure.

6.2. Inertial force and storey shear

The inertial force of the strengthened-layer structure is generally slightly lower than that of structures with the other two types of connection constraints. However, an obvious mutation of the inertial force appeared at the storeys with a strengthened layer, as shown in Figure 23. The distribution patterns of the inertial force for the structure with rigid connections and hinged connections are essentially the same.

Figure 24 shows the distributions of the shear stress among different storeys. Under minor and moderate earthquakes, the shear stress distribution of the strengthened-layer structure is lower in the structures, and the difference among them reaches a maximum at the bottom layer of the structure. Under major earthquakes, the distribution of the shear stress is essentially the same for the three types of connection structures. Based on the inertial force and storey shear analysis, the connection constraints did not have a large influence on the distribution of the inertial force or shear stress, and the structure with a strengthened layer have a slightly smaller inertial force and storey shear for most storeys.

6.3. Overturning moment

The overturning moments of the strengthened-layer structure are approximately linearly distributed among the storeys. For the rigid connections and the hinged connections, the overturning moments quickly increase on the middle storeys and slowly on the lower storeys. On the bottom layer, the overturning moments are similar values for the different connection constrains (Figure 25). During major and mega earthquakes, there are obvious differences among the three structures on the middle and lower storeys.
Figure 25. Distribution of the average overturning moment under each earthquake level.

Figure 26. Distribution of the peripheral frame’s overturning moment ratio.

Figure 26 shows the overturning moment ratio of the peripheral frame to the entire structure. For the hinged connections, the LFRS cannot transfer the bending moment to the GLRS through the bolted joints, which leads to the GLRS being unable to accommodate the overturning moment. The peripheral frame with rigid joints steadily maintains the overturning moment, which accounts for approximately 40% to 50% of the total overturning moment on most storeys and sharply increases on the top storey. From the bottom layer to storey 17 and from storey 18 to storey 26, the overturning moments for the strengthened-layer structure increase approximately linearly, reaching 100% at the strengthened storeys. Above storey 27, the overturning moment decreases to approximately 3.4% of the total overturning moment on average. The strengthened layer can increase the overturning moment of the peripheral frame for the lower storeys.

In conclusion, the GLRS withstands a small proportion of the seismic force, making this structure an approximate pure shear wall structure. The strengthened-layer structure enhanced the connection between the core tube and peripheral frame. The peripheral frame below the strengthened storeys can support a certain ratio of the overturning moment. The peripheral frame adopted rigid connections and can stably transfer the bending moment under the earthquakes, which accommodates a large fraction of the overturning moment.

6.4. Discussion

Compared to the other connections, the hinged-connection structure has a similar inertial force and storey shear but a different inter-storey drift ratio, which nevertheless still meets the appropriate performance level under minor, moderate and major earthquakes. The distribution of the inter-storey drift ratio and the overturning moment show that adding strengthened layers to the hinged-connection structure can effectively reduce the inter-storey drift ratio and increase the overturning moment of the peripheral frame. This result shows that the weaker seismic performance of the hinged-connection structure compared to the other connections can be improved by adding strengthened layers or other connections (such as energy dissipation devices). The structure design can be optimized based on the simulation data.

Compared to the upper storeys, the lower storeys have a higher inertial force that makes the joints at
these storeys more vulnerable to damage. This conclusion is consistent with the damage phenomenon observed during the test. Thus, reliable seismic measures should be implemented for the connections of members at the lower storeys to prevent excessive deformation and improve overall structural safety.

As the strengthened layers sharply increase the inertial force of the structure, the use of flexible connections is recommended to reduce the variation in the stiffness and the magnitude of the internal force for the strengthened layers of the hinged-connection structure. Further study is required to ensure that the strengthened layers are sufficiently stable to transfer the overturning moment and decrease the displacement.

Although the hinged connection can reduce the seismic force of the peripheral frame, the storey shear and overturning moment of the overall structure are not significantly reduced. Therefore, the bearing capacity of the core tube still needs to be enhanced to guarantee the seismic performance of the overall structure, and a core tube design that can maintain the reliability of the overall structure needs to be developed.

7. Conclusions
An explorative study on the seismic performance of a CFST-steel frame-core tube hinged-connection structure, which are used at the ends of all steel beams, is presented. There is a lack of research on hinged connections for all steel beams in supertall buildings (CFST-steel frame-core tube structures). In this paper, a supertall building is adopted to investigate the influence of hinged joints and a 1:40 reduced scale specimen is constructed for a shaking table test. Although most of the conclusions obtained are only applicable to the specific cases described in this paper, general observations can be derived from the experimental and numerical studies presented herein, as follows:

(1) The damage to the structure mainly occurs in the lower storeys at the concrete slab-column connections, the concrete slab-core tube connections, the concrete slab-steel beam connections, and the corners of the core tube.

(2) The lateral stiffness is relatively low in high-rise structures with bolted joints, and the natural period is greater than that of general frame-core tube structures. The acceleration amplification factor is decreased, whereas the long-period ground motion response is relatively significant. The maximum lateral displacement and inter-storey drift ratio of the structure are significantly increased.

(3) The hinged connection has a very large rotating capability and does not transfer the bending moment to the gravity column of each storey. The maximum inter-storey drift ratio of the core tube is 3.8 times that of the current threshold for the frame-core tube structure system, but the structure does not collapse.

(4) The torsion stiffness of the GLRS is very low, and the torsional response of the structure is mainly dominated by the core tube. Under major earthquakes, the difference in the torsion angle between the LFRC and the GLRS is large for individual storeys. It is thus necessary that the core tube has a sufficient torsion stiffness to resist the entire torsional response, and the entire concrete floor becomes the key factor in stably transmitting the torsion.

(5) The long-period component has less of an effect on the inertial force and storey shear than the inter-storey drift, whereas the short-period seismic component affects the inertial force and storey shear more significantly.

(6) The objective of this study is to present an overview that is relevant to a structural engineer and provides important guidance in selecting optimal measures for a hinged-connection structure. The simulation data will be used to carry out future studies on the substructure and joints.

(7) Insufficient experience is available for the hinged connections investigated here to design high-rise buildings. In practical structure design, it is necessary to select accelerograms that match the design spectrum in accordance with the specifications to carry out nonlinear time history analysis.

(8) Representative accelerograms were selected for experimental and numerical analyses to make it convenient for researchers to compare the effects of the three connections on high-rise structures and to verify the feasibility of structures with hinged connections. By comparing the results under different accelerograms, the long-period component was found to have a great influence on the displacement response of structures.

(9) There is no need to adjust the shear of the peripheral frame in structures with hinged connections, which simplifies the design. The hinged connections used in structures can reduce human costs and improve the efficiency of construction. This research contributes to the creation of new peripheral frame designs.

This study has some limitations. For instance, the significant scale produces a scale effect for the overall model test. Moreover, the materials used do not completely conform to the similarity requirements. Seismic performance tests on full-size substructures and members should be carried out to acquire more accurate data for correcting
the numerical model. The results of the aforementioned studies should be used to further improve the seismic performance of hinged-connection structures. Some examples of future research directions are hinged-connection design, guaranteeing the reliability of the core tube, and the design of the connection between the floor slab and the other components.

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