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# Experimental Study on the Seismic Performance of Four-Tower High-rise with an isolated Sky-Corridor on the top

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## *Abstract*

A four-tower high-rise with an isolated sky-corridor on the top has been designed in the seismic region of China. The 300 meters long sky corridor bridges the four 230 meters high towers at the top floor. The complexity of the structure makes it very difficult for seismic resistance design. To reduce earthquake response and member forces of the towers and sky-corridor, passive control strategy is adopted. Connection joints between the towers and sky corridor are designed as flexible links. Friction pendulum bearings and viscous dampers are installed in the connection joints. To Study the seismic performance of this complex structure, a 1/25 scaled model structure was tested on the shaking table under minor, moderate, and major earthquake levels. Eight earthquake records with different frequency spectrum property were selected to test the model structure. The maximum responses of acceleration and deformation of the four towers and sky-corridor were measured, and the seismic performance of friction pendulums ,viscous dampers, dynamic characteristics, cracking pattern, failure mechanism of the building were also evaluated. Results show that using friction pendulum bearings and viscous dampers to connect the sky corridor to the four towers can effectively reduce earthquake responses of each tower and member forces of the sky corridor if the properties of friction pendulum bearings and viscous damper are appropriately selected.

*Keywords: four-tower connected structure; passive control; shaking table test; friction pendulum; viscous damper*

# 1. Introduction

Seismic analysis and dynamic control of multi-tower connected structure are a difficult problem. Researchers conducted a series of studies on the seismic performance of multi-tower connecting structure. These researches include rigidly connected multi-tower, multi-tower connected by isolated corridor, and multi-tower installed semi-active or active control bridge. Moshe Safdie (2011) divide the continuous corridor into several parts, and movement joints are used to connect the divided parts to make sure that each tower can move independently under earthquake or wind. So that the coupling effect of each tower can be reduced. Dong-Guen Lee *et al.* (2010) have done the Evaluation of coupling–control effect of a sky-bridge to adjacent tall buildings. Several types of connector configurations were investigated to find an appropriate configuration for the tall buildings considered. Hideki Haramoto *et al.* (2000) proposed an active method for multiple high-rise buildings arranged in parallel. Mitsugu Asano *et al.* (2003) have done the vibration control analysis of triple high-rise buildings connected by active-damping bridges. Verification tests of damping performance were performed at the site, and test results confirmed that the devices demonstrate damping performance capable of meeting the vibration control target. R. E. Christenson *et al.* (1999) study on the dynamic control of three tall buildings connected by semi-active dampers. These semi-active dampers can produce a variety of control forces by changing their dynamic characteristics in real-time, and no significant energy is required. A clipped optimal control strategy is employed for the smart dampers that can provide increased performance, over a comparable passive control strategy, during moderately severe seismic events.

Some researchers study the seismic performance of multi-tower connected structures using experimental method. Lu *et al.* (2007a) used a shake table to test a scaled model of a tall building with a large podium structure, to study the torsional responses and check the effect of linking viscous fluid dampers (VFDs) between the main tower and the podium structure. Zhou *et al.* (2009) conducted a shake table testing of a multi-tower connected hybrid structure. Zhou *et al.* (2011) have a study on the seismic performance of a multi-tower connected structure. A 1/25 scaled model structure was tested on the shake table under minor, moderate, and major earthquake levels.

In this paper, a high-rise, four-tower connected hybrid structure is used as a representative irregular structure not currently included in Chinese codes. Detailed shaking table model test was performed by a working group of the State Key Laboratory for Disaster Reduction in Civil Engineering at Tongji University, China. Dynamic similitude design approaches for small-sized model of multi-tower high-rises connected with an isolated sky-corridor on the top are introduced in this paper. Experimental responses such as dynamic property, acceleration and displacement of the towers and sky corridor, the earthquake response of friction pendulum bearings and viscous dampers were analysed. Finally, conclusions for evaluating the seismic performance of this type of structure are drawn.

# 2. Description of the building structure

## 2.1 Building structures

The project is designed in accordance with the Code for seismic design of buildings (GB50011-2010) in China. The building is designed as a four-tower building connected by an isolated long-span corridor at its top.

The structure consists of four 235 meters towers and a sky corridor, the sky corridor is isolated at the top of the four towers by 28 friction pendulum bearings and 16 viscous dampers. The T2 and T5 tower is 58 stories, the T3S tower is 51 stories, the T4S tower is 54 stories. All the towers are Frame-Reinforced Concrete core Tube systems, there are 3 strength layers along the structure height, each strength layer consists of outrigger truss and belt truss. The overall structure of the prototype is shown in figure 1.

The sky corridor bridges the four towers at the top floor is a continuous space steel truss system, which is curve layout along east-west direction. The steel truss contains 3 main truss, and perpendicular to the direction of the main truss, the secondary steel truss is installed to connect the three main trusses per 4.5 meters. The sky corridor was supported by 28 friction pendulums on the top of four towers, each tower of T2, T3S and T4S have

6 friction pendulums, T4S tower has 8 friction pendulums. There were 16 viscous dampers installed between the four towers and corridor.

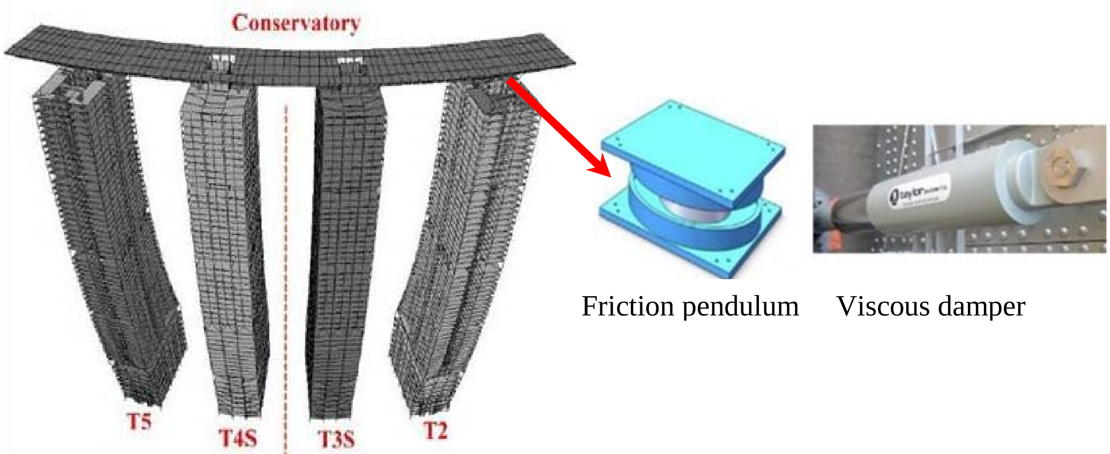



Fig. 1 –Overall structure of building

## 2.2 Structural irregularities

According to the Chinese Code for Seismic Design of Buildings (GB50011-2010) (CMC, 2010) and the Technical Specification for Concrete Structures of Tall Building (JGJ3-2002) (CMC, 2011), the main characteristics of the four-tower connected structure are summarized as follows.

(1) In the elevation, the columns of the frame are curved, the arch shape of the tower makes the force transmission path becomes complex. On the roof of suspending floor, straight columns become inclined columns, and the existence of the bifurcation of the inclined column makes the path of force transfer complex. On the roof of the four towers, a 300 meters long steel truss sky-corridor bridges the four towers, and there are 3 strength layers in every tower. All these structural characteristics cause mutation of the lateral stiffness of the tower along the structure height.

(2) In the plan layouts, the four towers do not arrange in a line, curved layouts of four towers are not conducive to resist earthquake. There are large openings in each storey of the T2 and T5 tower. Those large openings divide the floor into 2 parts.

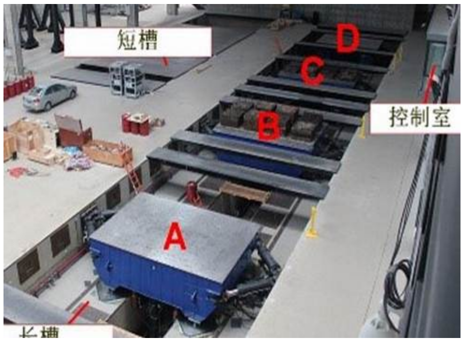
(3) Connections between the four towers and sky-corridor are flexible links. The sky-corridor is isolated by friction pendulums, and viscous dampers are installed to dissipate the earthquake energy by adding supplemental damping.

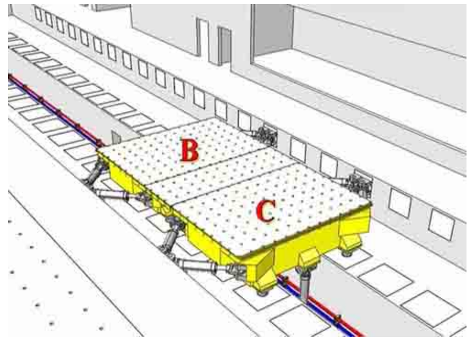
Given the above irregularities and complexity of the structure, it is necessary to study the seismic behavior of the multi-tower connected structure in detail and evaluate its ability to resist strong earthquakes.

# 3. Shaking table testing of the model structure

## 3.1 Shaking table facility

There are four shaking tables in Tongji Multi Function Shaking table test lab. The dimension of the shaking table is 6 m×4 m. The maximum payload of table B and C is 700kN. The four tables can be arranged into a line(in figure 2 (a)) to test large span bridges and also can be merged into a large shaking table to test large span structure. In this test, table B and table C are connected to be a large shaking table by a connecting table(in figure 2 (b)), the dimension of the large table is 10m×6 m, the total load capacity is 1400 kN. Synchronous control technology for the two connected table is used to test the experimental model.






(a) (b)

Fig. 2– Shaking table for the test

## 3.2 Model designs

Copper plates were used to simulate the steel structural members and fine-aggregate concrete with fine wires was chosen to construct the RC components in the model. The dynamic behaviour of a structure is fully identified by means of three basic quantities:mass, stiffness and restoring force, and these three quantities are in turn related to mass density, elastic modulus, time and length.

### 3.2.1 Similarity design of towers

First, based on the capacity and the size of the shake table, the scaling factor of dimension $S_l$ was chosen to be 1/25. According to the material test results, the overall scaling factor of the elastic modulus was determined to be 0.2. $S_a$ was set to be 2.0 and additional iron blocks were evenly distributed on the model to compensate for the difference in vertical load. The total weight of the model was estimated to be 1350 kN, including iron blocks with a weight of 700 kN. All the other scaling factors could be derived and the typical factors are listed in Table 1.

Table 1 Typical similitude factors of model structure

| Physical parameter | Length | Elastic modulus | Frequency | Acc | Mass density | Concentrated force |
|---|---|---|---|---|---|---|
| Similitude factors | 1/25 | 0.2 | 7.07 | 2.0 | 2.5 | $3.2\times10^{-4}$ |

### 3.2.2 Similarity design of friction pendulums and viscous dampers

The scaled friction pendulum bearing is designed base on the similitude of friction pendulum equivalent stiffness. The scaled damper is designed base on the similitude of energy dissipation capacity. The scaled friction pendulum bearings and viscous dampers of model structure is shown in figure 3. There are 18 friction pendulum bearings and 16 viscous dampers installed between the four towers and sky corridor. There are four bearings on the roof of T2, T3S and T5 tower. The remaining 6 bearings are installed on the roof of T4S tower.


Fig. 3– scaled friction pendulum and viscous damper in the experimental model

### 3.2.3 Similarity design of sky-corridor

The two principles of the similar design of steel truss sky-corridor are: (1) dynamic property similarity, (2) loading capacity similarity. The model sky-corridor should reflect the dynamic property and load capacity of the prototype structure. Installation of the model towers and corridor is shown in figure 4.

Fig. 4– Installation of four towers and shy-corridor

## 3.4 Instruments and transducers

The plan layout of the experimental model is shown in figure 5. There are a total of 303 sensors installed on the experimental model structure, which included 81 accelerometers on the ground, L1, L12, L24, L36, top floor of four towers and sky corridor respectively; 44 displacement transducers on the ground, L1, L24, top floor of four towers and sky corridor respectively; 18 triaxial forces sensors on the friction pendulum bearings, and 160 strain gauges on the surface of some important elements of towers such as the outrigger truss, belt truss, joint of transfer column of towers and important members of sky-corridor.

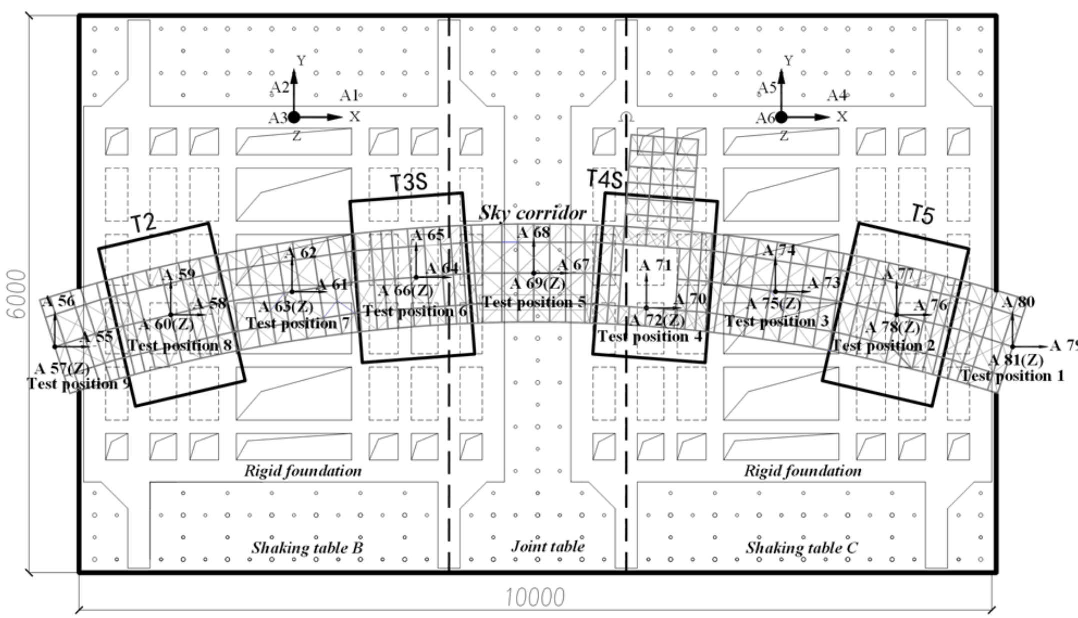
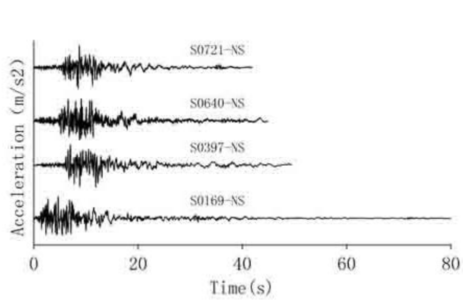



Fig. 5–Plan layout of model structure

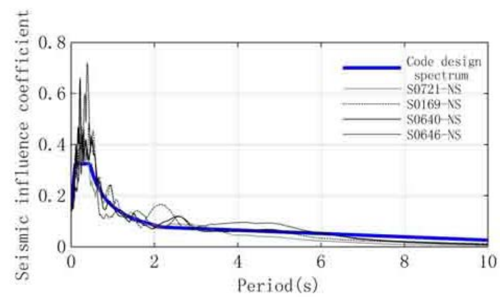
### 3.5 Input seismic waves

The location of the multi-tower connected structure is assigned to an earthquake zone of intensity 6.5. The peak ground accelerations corresponding to the minor, moderate and major levels of seismic intensity 6.5 are specified as 0.025g, 0.07g, 0.175g, respectively. The acceleration scaling factor (where *Sa* = 2.0) was used to obtain the target input peak value in the tests.

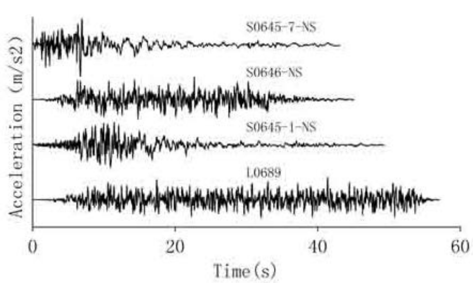
According to the soil condition and design intensity of the location of the structure, 7 ground motions were selected as the input motions during the minor and moderate earthquake test: (a) S0721; (b) S0169; (c) S0641; (d) S0397; (e) S0647; (f) S645-1; (g) S645-7. And 1 ground motion(L0689) was selected as the input motions during the major earthquake test. Input seismic waves time history and response spectrum were shown in figure 6. These earthquake acceleration time histories were scaled to have the same target input peak value for each intensity level.



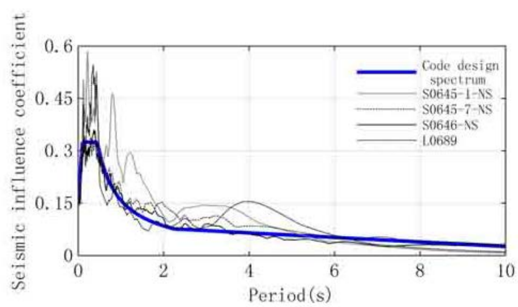

(a) Time history




(b) Response spectrum




(c) Time history




(d) Response spectrum

Fig. 6 – Input seismic waves and response spectrum.

### 3.6 Test programmes

The test was carried out in four stages. The first three stages represented minor, moderate and major levels of intensity 6.5, respectively. The last one represented a major earthquake of intensity 7, which was applied for further investigation of the multi-tower connected structure subjected to extremely strong earthquakes. In the two-direction excitations in the test, the peak acceleration ratio of the principal direction to the other direction is designated to be 1 to 0.85, as specified in Chinese design code.

## 4. Experimental observation of model structure

For minor earthquakes of intensity 6.5, no visible damage was observed. After the white noise 2 was used to scan the model, it was found that the frequencies were slightly reduced. This reveals that there was no damage in the model. The structure remains the elastic state.

For moderate earthquakes of intensity 6.5, there was no obvious damage appeared on the model structure. There was only a few cracks at the place of the beams, columns and coupling beams above the middle part of the model structure, and the performance of crack closing is well. The natural frequency of the model was reduced to less than 5%, it reveals that although the whole structure has been damaged to a certain extent, the structure still retains considerable stiffness after damage.

For major earthquakes of intensity 7, existing cracks were extended. Severe damage does not appear on most components of the model. Cracks of tower T4 were more than the other three towers. The natural frequency of the model was reduced about 10%. The whole structure has been damaged to a certain extent, but still has certain stiffness and integrity.

**(a)** **(b)** **(c)** **(d)** **(e)**

Fig. 7–Failure patterns under major earthquakes of intensity 7: (a) beam at the top floor of T4S tower; (b) beam at the 38 th floor of T4S tower; (c) beam at the 34th floor of T5 tower; (d) beam at the 36th floor of T5 tower; (e) column at the 12th floor of T5 tower

## 5. Experimental results of structure

### 5.1 Experimental dynamic characteristics

Frequencies of the model at different phases were measured by inputting a white noise signal before further seismic input simulations. Variations of frequencies and stiffness at the end of every earthquake level were listed in Table 2.

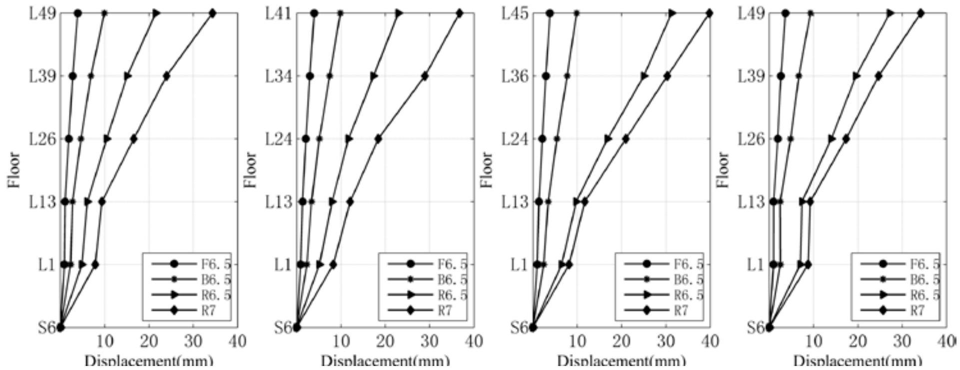
Table 2 Vibration frequencies and vibration modes of prototype structure

| Mode of vibration | $f_t$ / Hz | After inputs of 6.5 minor levels | | After inputs of 6.5 moderate levels | | After inputs of 6.5 major levels | | After inputs of 7 major levels | | Vibration modes |
|---|---|---|---|---|---|---|---|---|---|---|
| | | $f_t$ / Hz | $\Delta f$ | $f_t$ / Hz | $\Delta f$ | $f_t$ / Hz | $\Delta f$ | $f_t$ / Hz | $\Delta f$ | |
| 1 mode | 0.229 | 0.229 | -0.1% | 0.228 | -0.4% | 0.219 | -4.5% | 0.211 | -8.0% | translation |
| 2 mode | 0.349 | 0.346 | -0.7% | 0.338 | -3.1% | 0.327 | -6.2% | 0.314 | -9.9% | translation |
| 3 mode | 0.450 | 0.449 | -0.2% | 0.448 | -0.4% | 0.441 | -2.0% | 0.425 | -5.4% | torsion |

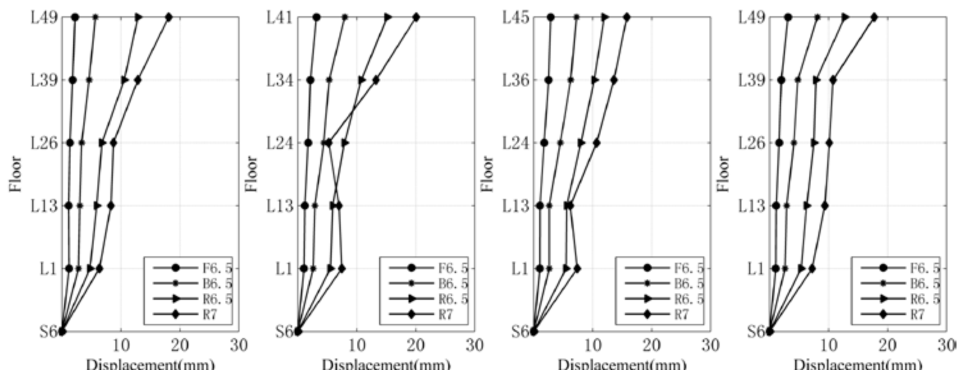
### 5.2 Displacement responses of the model structure

As showed in figure 8, the peak displacements of the four towers increased with the peak acceleration of the inputed excitation increased. These curves show that the slope of displacements above the bottom strengthen layer become larger than that under the bottom strengthen layer. It indicates that displacement response of the upper portion of the towers is violent, which is consistent with the phenomenon of more damage in the upper part of the structure.

In order to observe the torsion deformation at the top of the structure, time history of lateral displacements at sky-corridor roof level under the rare earthquake of intensity 7 were shown in figure 9. The results show that the maximum difference value of roof is 28.57mm (t = 6.58s) in direction Y.

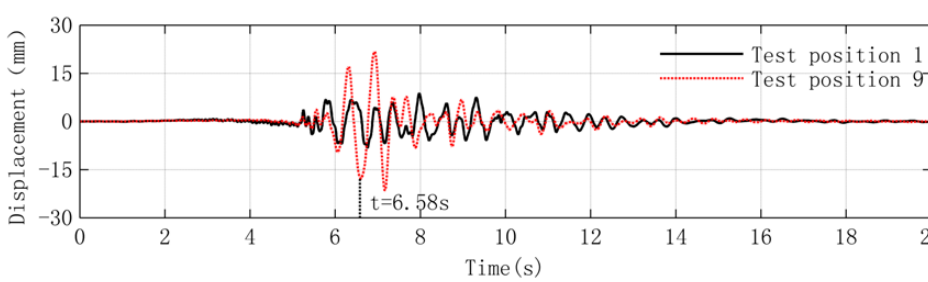



(a) T2 tower (b) T3S tower (c) T4S tower (d) T5 tower X-direction

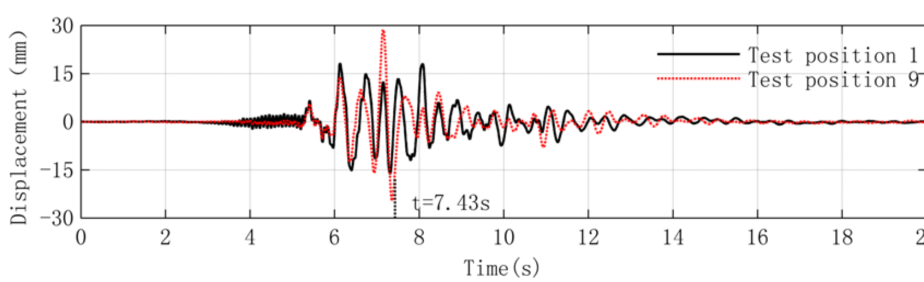



(a) T2 tower (b) T3S tower (c) T4S tower (d) T5 tower Y-direction

Fig. 8–Distribution of max displacements of towers




(a) Case of X principle direction




(b) Case of Y principle direction

Fig. 9–Displacement time history of sky-corridor under rare earthquake of intensity 7

### 5.3 Acceleration responses of the model structure

Earthquake input is in horizontal direction X and Y, not include the vertical component. While the earthquake response on the top floor of the four towers includes three translational motions and three rotation motions. The earthquake response can be seen as the input in the corridor. It contains six degrees of freedom component. Vertical acceleration of sky corridor under different earthquake level is shown in figure10. The vertical acceleration of the cantilever end is larger than other places of the corridor, and the vertical acceleration of supports (test point at 3, 5, 7) is larger than vertical acceleration of mid-span. The vertical acceleration response is symmetrical distribution along the plane layout of the corridor. The maximum vertical acceleration of the sky corridor is 0.33g, 0.75g, 1.37g, 1.80g for the four test phases, the vertical response of the sky corridor should not be neglected even if there is not vertical earthquake input on the base of the overall structure.

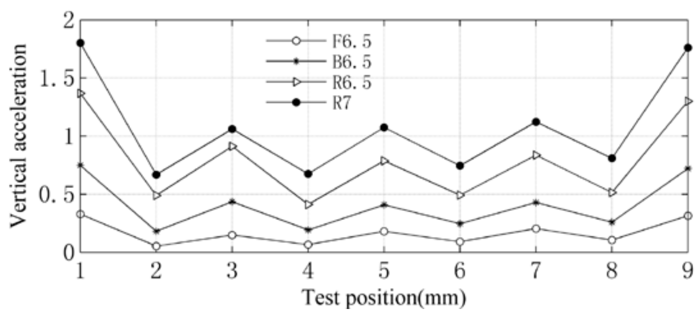
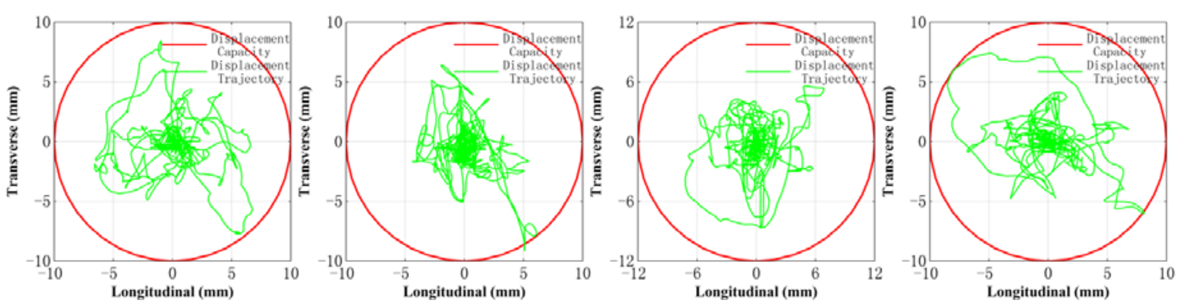



Fig. 10–Vertical acceleration of sky corridor under different earthquake level

### 5.4 Experimental responses of friction pendulum and viscous damper

When the model was tested under the minor earthquakes of intensity 6.5, the friction pendulums did not slide. The connections between the four towers and sky corridor have the rigid connection properties. This phenomenon verifies the design principle of the friction pendulums were locked under minor earthquakes. Friction pendulum bearings begin to slide, while the displacement is small. The sliding displacement is increased with the increase of the earthquake intensity. The displacement orbit of the pendulums under the test of 7 major levels is presented in figure 11. In the rare earthquake test, the displacement demand does not exceed the capacity of the isolation system and there was slight uplift.




(a) T2 tower (b) T3S tower (c) T4S tower (d) T5 tower

Fig. 11–Movement orbit of each Tower’s bearing under 7 major levels

## 6. Conclusions

The following conclusions are obtained from the shaking table test of the structure.

(1) According to the results of shaking table model test, basic knowledge of the dynamic property can be got. Such as the vibration frequency and mode. The first three modes are translation mode in X-direction, Y-direction and torsion mode respectively. Because of the irregularity and complexity of the structure, the

torsion mode contains the overall structure torsion and the single tower torsion.

(2) The four-tower connected structure will not be damaged by a minor earthquake. It would have some visible structural cracking under a moderate earthquake, and cracks would develop under a major earthquake, but would not suffer serious damage. There are no cracks on key members of the tower. Those members keep elastic state under major earthquake.

(3) The friction pendulum does not slide until under the moderate earthquake test. The displacement demand does not exceed the capacity of the isolation system. There are slight uplift of the pendulums under major earthquake test.

(4) The sky corridor still keeps elastic under the major earthquake test. For the isolation function of friction pendulum bearings and energy dissipation function of viscous dampers, the horizontal acceleration-amplification coefficient of corridor degrade with the increase of earthquake input intensity. The maximum acceleration of the cantilever end of the corridor is seriously larger than other places, especially the vertical acceleration.

## 7. Acknowledgements

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