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Shaking table tests on China Pavilion for Expo 2010 Shanghai China

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SUMMARY

Modern structural engineering tends to progress towards more novel high-rise structures through gradually improved methods of design and wide use of new materials. The National Hall of China Pavilion for Expo 2010 Shanghai is such a building specially designed with a shape of an inverted trapezoid in elevation and local discontinuity of floor slabs in plan for aesthetic and functional considerations. The preliminary analysis shows that the fundamental vibration mode of this structure is a torsional mode due to the special shape, thus resulting in the period ratio between the first torsional mode and the first translational mode exceeding the limit value stipulated in Chinese code. Shaking table tests of a 1/27-scale structural model were performed to observe the seismic response characteristics, and verify the seismic safety of the structure. This paper presents the results of the shaking table tests in terms of dynamic properties, global responses, as well as local responses. It was found that the torsional effects were not significant by the analysis of the ratio between the maximum displacement and the average displacement of floor and torsional angle. Finally, weak points were identified, and some corresponding suggestions were put forward to improve the overall seismic performance of this structure.

1. INTRODUCTION

The past decades have witnessed a flourish of novel high-rise structures through gradually improved methods of design, and wide use of new structural systems and materials. Many of these structures have irregular plan layout and elevation for aesthetic and functional considerations, which bring new challenge to the structural engineer because of the difficulties in analyzing and predicting the responses of structures.

The seismic behaviour of irregular buildings has been theoretically investigated by several researchers (De la Llera and Chopra, 1995; Das and Nau, 2003; Tremblay and Poncet, 2005). Most of these studies focus on seismic performances of multi-storey reinforced concrete (RC) or steel buildings. For the experimental studies on irregular high-rise buildings, Lu *et al*. (1999) studied the dynamic response of a complex structure with U-shaped floors and specially shaped slant columns. Ko and Lee (2006) preformed a shaking table test on a 1/12-scale model to investigate the seismic performance of a 17-storey high-rise RC structure with a high degree of torsional eccentricity and soft-storey irregularity in the bottom two storeys. Lu *et al*. (2009) carried out shaking table model tests on a complex high-rise building with two towers of different heights connected by trusses. Since modern structural engineering tends to progress towards more novel high-rise structures with irregularities both in plan and elevation, it is necessary to systematically investigate and evaluate the performances of these complex structures through numbers of experimental studies.

The current experiment is conducted on such a high-rise building called China Pavilion for Expo 2010 Shanghai, which is specially designed with a shape of an inverted trapezoid in elevation and local discontinuity of floor slabs in plan. The preliminary analysis shows that the fundamental vibration mode of this structure is a torsional mode due to the special shape, thus resulting in the period

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ratio between the first torsional mode and the first translational mode exceeding the limit value stipulated in Chinese code. It is of great significance to obtain comprehensive understanding of the overall structural behaviour, especially the torsional effect of this structure. With the development in technology and theory of structural test, the shaking table model test has become one of the most effective ways to predict and evaluate the overall performance of complex buildings.

This paper presents shaking table tests on a 1/27-scale model of a high-rise RC core structure with irregularities both in plan and elevation. A series of simulated earthquake ground motions, including the El Centro 1940 earthquake record, Pasadena 1952 earthquake ground motion and Shanghai artificial accelerogram with gradually increasing magnitudes, were input to the table. The results of the shaking table tests in terms of the global and local responses, as well as the dynamic properties, are presented. Based on the test results, the overall behaviour of the prototype structure is evaluated. Accordingly, some suggestions for improving the design of this structure are also put forward.

2. DESCRIPTION OF THE STRUCTURE

The National Hall of China Pavilion for Expo 2010 Shanghai is a public building for exhibition with a total structural height of 68 m. Located in the Pudong part of the Expo Park, it will become a permanent landmark in the Pudong New Area, Shanghai, China. The floors below the height of 33.3 m are hollow, while the upper floors are unfolding outwards level by level, forming a shape of an inverted trapezoid which is called 'oriental crown' (see Figure 1). Accordingly, the plan dimensions are enlarged greatly from 69.9×69.9 m at the height of 33.3 m (see Figure 2) to 136.8×136.8 m at the roof (see Figure 3), resulting in a cantilever span of about 33 m at the roof level. Furthermore, the floors between the height of 38.7 and 46.8 m are staggered and uprise spirally along the outsides of the cores (see Figure 4). In elevation, four RC cores stretch from the bottom to the top, and protrude 7.7 m out of the roof (see Figure 5). The structural system is composed of four RC cores with steel–concrete composite floors. The four cores with the same plan dimensions 18.6×18.6 m are designed to be the primary lateral resisting system, besides to sustain the vertical loads of the structure. In addition, at the height of 33.3 m, 20 inclined columns consisting of concrete-filled rectangular steel tube are placed on the perimeter of the cores as the vertical support of the big-span steel beams on the floors. Since there is no column in the interior, it satisfies the requirements of the architectural function for open and large exhibition areas.

The main characteristics of this structure, which are beyond the limitation of Chinese code, can be summarized as follows:

Figure 1. Perspective view of the structure.

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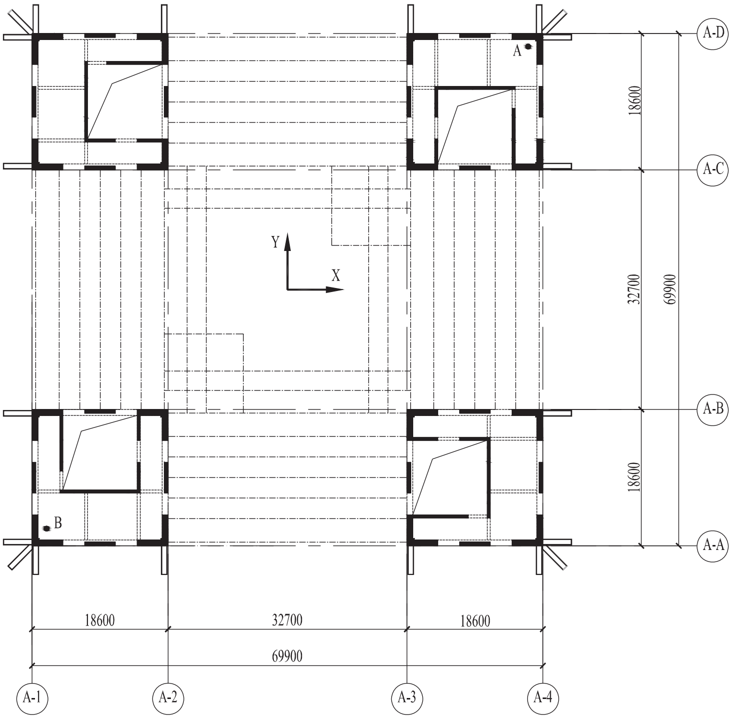

Figure 2. Plan layout at 33.3 m floor (unit: mm).

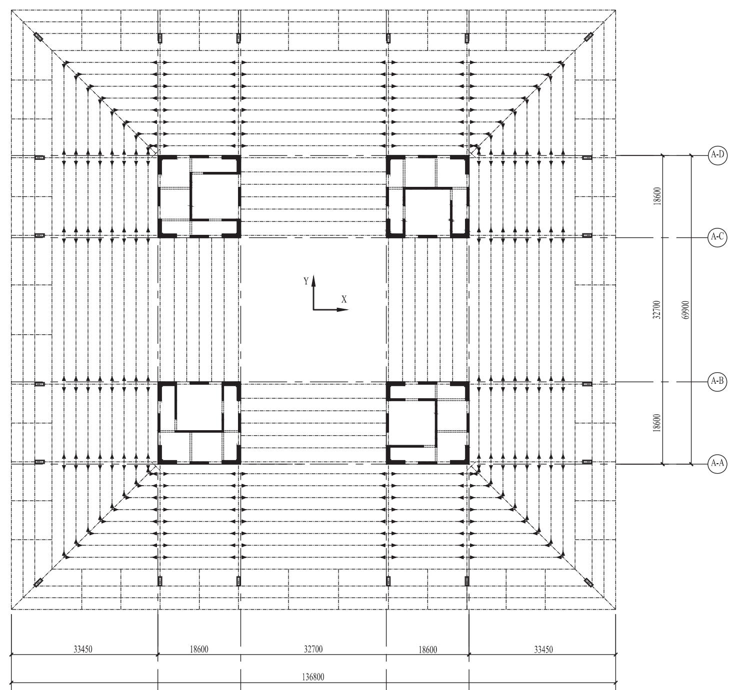

Figure 3. Plan layout at roof floor (unit: mm).

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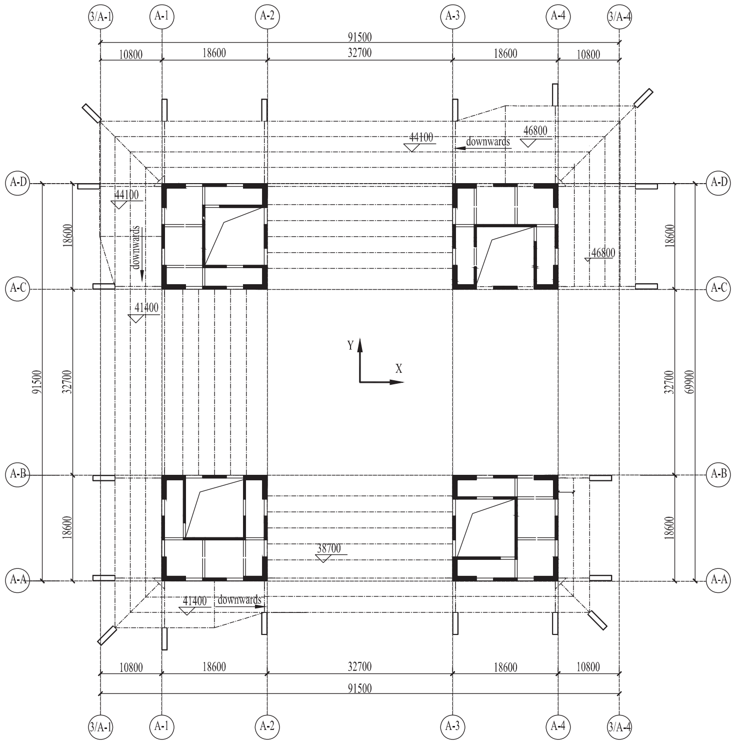

Figure 4. Plan layout between 38.4 and 46.8 m floors (unit: mm).

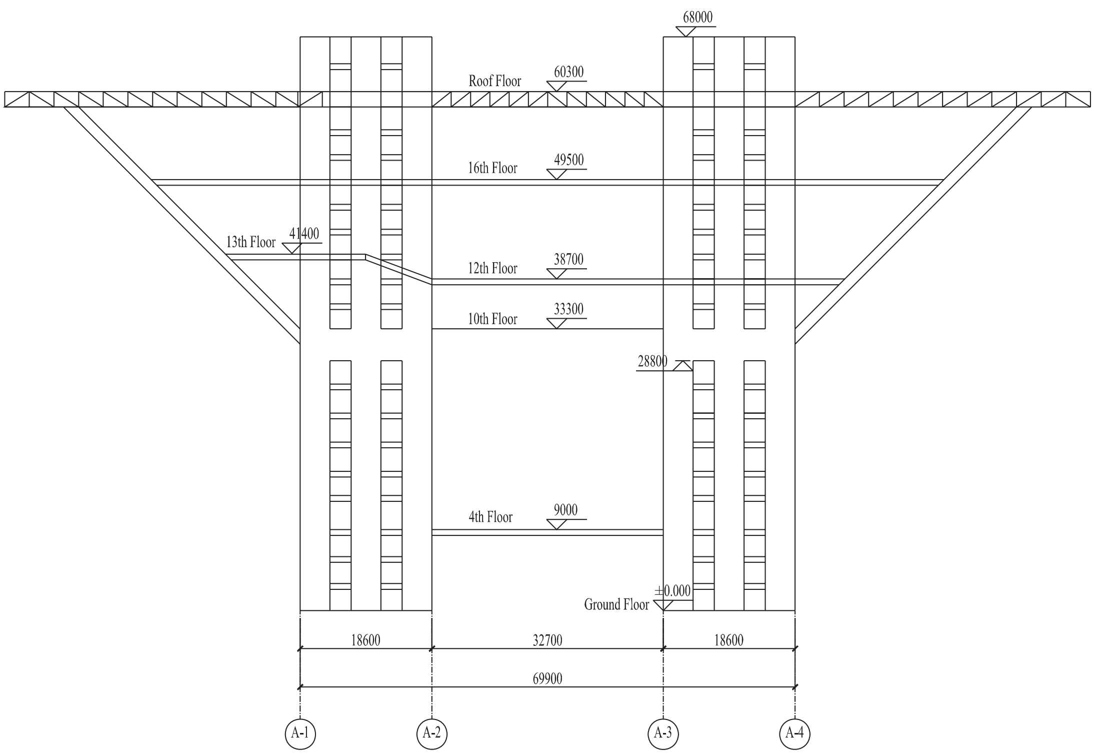

Figure 5. South elevation (unit: mm).

(1) Local discontinuity of floor slabs: There is an atrium with dimensions 32.7 × 32.7 m, and the floors between the height of 38.7 and 46.8 m are staggered, which results in the plan irregularity defined by the Chinese *Code for Seismic Design of Buildings* (CSDB, GB50011-2001) (Ministry of Construction of China, 2001). Obviously, it is not appropriate to apply rigid diaphragm hypothesis in these floors for structural analysis.
(2) Beyond code limitation of torsional period ratio: According to the Chinese *Technical Specification for Concrete Structure of Tall Building* (TSCSTB, JGJ3-2002) (Ministry of Construction of China, 2002), the torsional period ratio is required to be less than 0.9 to prevent excessive torsional response. However, the fundamental vibration mode is a torsional mode due to the special shape of the building, thereby resulting in the torsional period ratio exceeding the limit value.

Taking these irregularities into account, the shaking table model test was recommended by the peer review committee. A 1/27-scale model was constructed according to the similitude laws in State Key Laboratory of Disaster Reduction in Civil Engineering at Tongji University, Shanghai, China.

3. MODEL DESIGN AND CONSTRUCTION

3.1. Description of the shaking table

The MTS shaking table used for this test was able to input three-dimensional and six degree-of-freedom motions. The table is 4 × 4 m in size. The maximum payload is 250 kN, and the maximum accelerations are 1.2, 0.8 and 0.7 *g* in the longitudinal, transverse and vertical directions, respectively. Its working frequency ranges from 0.1 to 50 Hz, and 96 channels were available for data acquisition during testing.

3.2. Model materials and similitude relationship

Micro-aggregate concrete and fine wires were used to construct the RC elements, and steel structural members were simulated by copper plates due to its low elastic modulus and similar yielding properties to steel.

The model was designed by scaling down the geometric properties from the prototype according to the scaling theory (Sabnis *et al*., 1983), which was systematically introduced and discussed especially for civil engineering application. Since the commercially available materials, components and loading equipments used in a test usually cannot meet all the scaling requirements simultaneously, it is hardly possible to satisfy all the similitude laws strictly.

According to the dynamic similitude theory, there are three controlling scaling factors, which are independent with each other, and other scaling factors are derived accordingly. Given to the bearing capacity and the size of the shaking table, the dimension scaling factor (S_l) was chosen to be 1/27. The stress scaling factor (S_σ), which was first designed and finally determined according to the test results of material properties, was 0.143. The third controlling factor concerning acceleration (S_a) was determined to be 1.0 for the consideration of eliminating gravity distortion effect. Hence, the density scaling factor (S_ρ) needs to be modified using artificial mass to make up the discrepancy between the similitude-required mass and the model mass.

Since the three controlling factors were chosen, the other scaling factors were obtained and listed in Table 1.

3.3. Model construction

The model structure was fabricated in the casting site adjacent to the shaking table for the convenience of hoist before testing. A rigid base beam was constructed as the basement of the model, assuming that there was no interaction between the soil and the prototype structure. Fine wire meshes were assembled to simulate the reinforcement, and copper plates were fabricated to model the steel elements. The concrete was placed using internal and external forms; the internal forms made use of

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Table 1. Main similitude relationships.

Variable	Equation	Scaling factor	Remark
Length	S_l	1/27	Controlling factor
Elastic modulus	$S_E = S_\sigma$	0.143	
Stress	S_σ	0.143	Controlling factor
Strain	S_σ/S_E	1.000	
Density	$S_\sigma/(S_a S_l)$	3.861	
Force	$S_\sigma S_l^2$	1.960E-04	
Frequency	$S_l^{-0.5} S_a^{0.5}$	5.196	
Velocity	$(S_a S_l)^{0.5}$	0.193	
Acceleration	S_a	1.000	Controlling factor
Time	$(S_l S_\varepsilon/S_a)^{0.5}$	0.193	

Figure 6. Base beam construction.

plastic foam for easy removal, and the wood plates were applied as external forms in slipform construction. Figure 6 shows the construction of base beam, and Figure 7 shows the assembling of the steel beams and trusses at the roof level.

The model was 2.819 m high in total, including the superstructure height of 2.519 m and base beam height of 0.300 m. After cured for about 2 weeks, the wood forms were removed from outside, and the internal foam plastic forms were taken out by breaking them into pieces.

4. TEST PROGRAMME

4.1. Model installation

The model was hoisted by a crane and placed at the centre of the shaking table. To ensure an effective transmission of the table motion to the model, the model base beam was firmly mounted on the shaking table through bolt connections.

Iron blocks and disks were used as artificial mass to make up the discrepancy between the required and the model mass according to mass similitude relationship. These weights were distributed on each floor of the model conforming as closely as possible to the prototype. The total artificial mass attached to the model was calculated to be 16.49 tons, and the total mass on the table, including the artificial

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Figure 7. Steel beams and trusses assembling at roof level.

Figure 8. Overview of the model on shaking table.

mass and the mass of the model, was estimated to be 24.78 tons. Shown in Figure 8 is an overview of the model with artificial mass after installation on the shaking table.

4.2. Instrumentation

Three types of instrumentations, including accelerometers, displacement transducers and strain gauges, were installed on the model so that both the global and local responses of interest could be measured.

The distribution of the accelerometers and the displacement transducers is shown in Figure 9, where letters A and D refer to accelerometer and displacement transducer, respectively. In total, there were 45 accelerometers used to monitor the acceleration responses of the model. It should be pointed out that the displacement can also be obtained by integrating the recorded acceleration twice. Apart from three accelerometers on the base beam to check the table input, four at the perimeter of the roof level to measure the maximum accelerations, other accelerometers were distributed on floors along the height of the RC cores as uniformly as possible. Taking the 10th floor (corresponding to the height of 33.3 m) as an example, three accelerometers were placed at a corner of one RC core denoted by point A (see Figure 2) in the directions of *X*, *Y* and *Z* of the shaking table, respectively; additional two along the two horizontal directions were set at the corner of the opposite core denoted by point B (see Figure 2) for the measurement of torsional effect on this floor. There were 12 displacement transducers placed at the base beam, as well as the perimeter of the cantilever floors, for the purpose of monitoring the maximum floor displacements. In addition, 15 strain gauges were concentrated on members whose stresses are of interest, such as the inclined columns, the bottom of the RC core, the steel beams and the trusses both at top and bottom chords, where the local behaviour was of particular interest.

4.3. Test programme

The condition of the site soil is one of the most important factors to select earthquake inputs for dynamic tests. According to the CSDB, the site soil in Shanghai belongs to type IV, which is defined that the overlaying thickness of the site is more than 80 m, and the average velocity of shear wave in the soil layer is not more than 140 m/s. Considering the power spectral density properties of type IV site soil, three different ground motions were applied as input accelerations to the shaking table: (a) the EI Centro record from the California Imperial Valley earthquake on 18 May 1940; (b) the Pasadena record from the California Kern County earthquake on 21 July 1952; and (c) the Shanghai artificial accelerogram (SHW2), which is specified in the *Shanghai Code for Seismic Design of Buildings* (SHCSDB, DGJ08-9-2003) (Shanghai Government Construction and Management Commission, 2003). Figure 10 shows the time history of acceleration of SHW2, and the comparison of response spectrum between SHW2 and design spectrum in Shanghai Code.

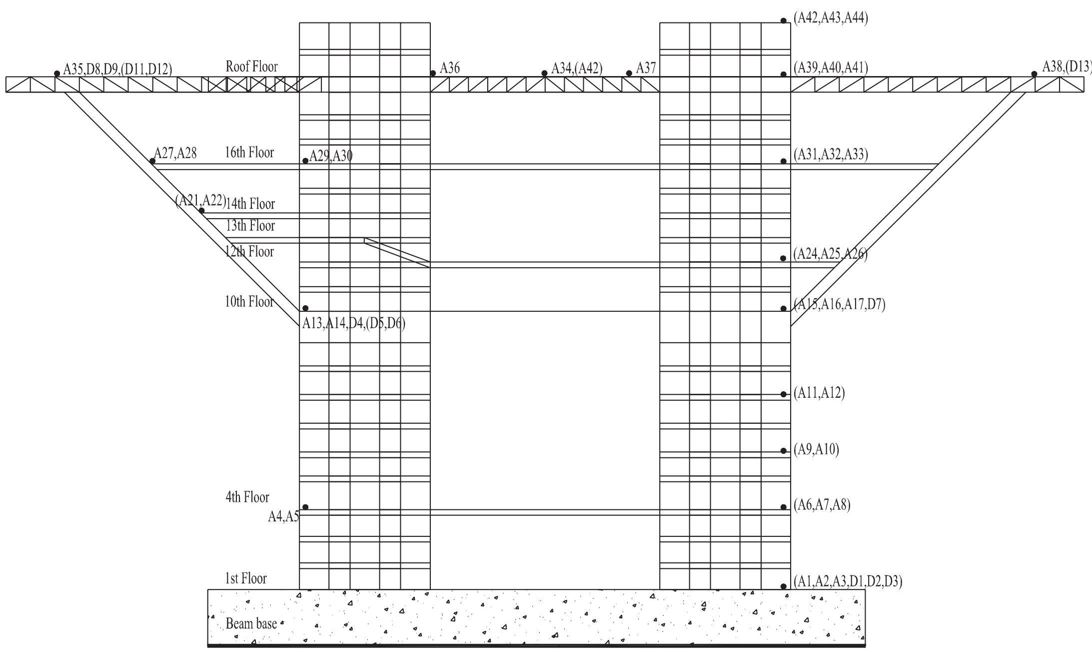

Figure 9. Arrangement of accelerometers and displacement transducers.

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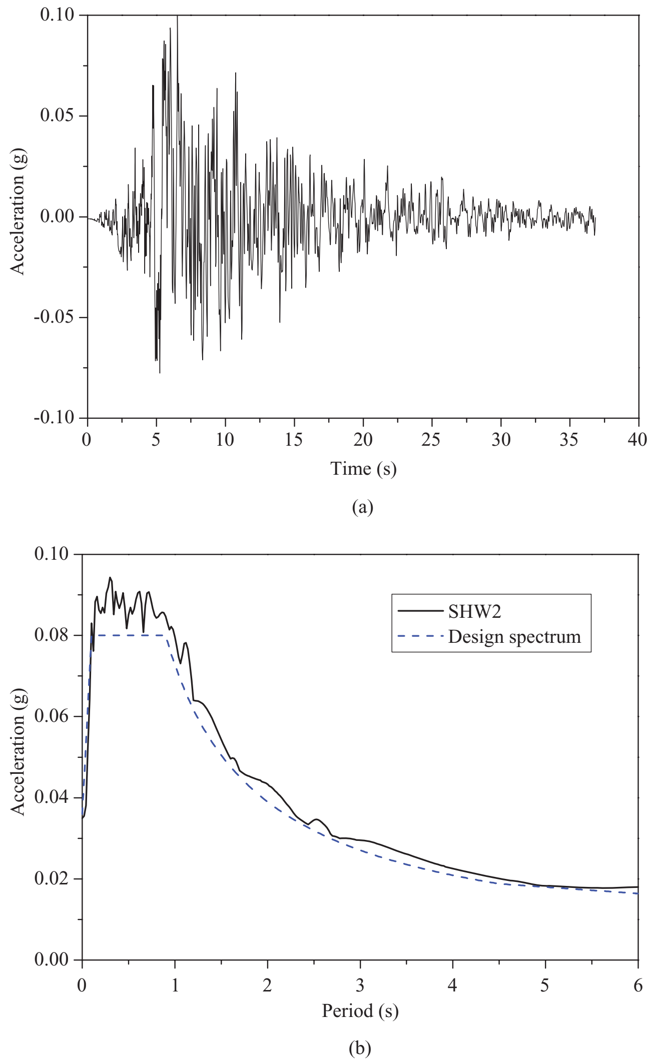

Figure 10. SHW2 accelerogram: (a) time history of acceleration, and (b) response spectrum comparing to the design spectrum in Shanghai Code.

According to CSDB, buildings in seismic regions should be designed to sustain earthquakes of minor, moderate and major levels, which correspond to 63.2, 10 and 2% probability of being exceeded in 50 years, and return period of 50, 475 and about 2475 years, respectively. That is to say: when buildings are designed to be subjected to the influence of frequently occurring earthquakes with an intensity of less than the design intensity, the buildings will not be damaged, or will be only slightly damaged and will continue to be serviceable without repair; when they are subjected to the influence of earthquakes equal to the design intensity, they may be damaged but will still be serviceable after ordinary repair or without repair; when they are subjected to the influence of expected rare earthquakes with an intensity higher than the design intensity, they will neither collapse nor suffer damage that would endanger human lives. As Shanghai belongs to the seismic zone of intensity 7 (roughly equivalent to a modified Mercalli intensity of 7), the peak ground accelerations (PGAs) corresponding to earthquakes of minor, moderate and major levels are specified to be 0.035, 0.100 and 0.200 *g*, respectively.

A total of 43 test cases were conducted, and a summary of the inputs used for each case is presented in Table 2. The model structure was subjected to the following sequence of test. First, a white noise with PGA of 0.030 *g* was input to measure the dynamic properties of the intact model structure. Then,

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Table 2. Test programme.

Test case	Case name	Phase	Input wave	Principal direction	PGA (g) Direction X	Direction Y	Direction Z	Note
1	W1	White noise		–	0.030	0.030	0.030	3D
2	F7EX	Frequent 7	EI Centro	X	0.035	–	–	1D
3	F7EY		EI Centro	Y	–	0.035	–	1D
4	F7EZ		EI Centro	Z	–	–	0.023	1D
5	F7EXYZ		EI Centro	X	0.035	0.030	0.023	3D
6	F7PX		Pasadena	X	0.035	–	–	1D
7	F7PY		Pasadena	Y	–	0.035	–	1D
8	F7PZ		Pasadena	Z	–	–	0.023	1D
9	F7PXYZ		Pasadena	X	0.035	0.030	0.023	3D
10	F7SHX		SHW2	X	0.035	–	–	1D
11	F7SHY		SHW2	Y	–	0.035	–	1D
12	W2	White noise		–	0.030	0.030	0.030	3D
13	B7EX	Basic 7	EI Centro	X	0.100	–	–	1D
14	B7EY		EI Centro	Y	–	0.100	–	1D
15	B7EZ		EI Centro	Z	–	–	0.065	1D
16	B7EXYZ		EI Centro	X	0.100	0.085	0.065	3D
17	B7PX		Pasadena	X	0.100	–	–	1D
18	B7PY		Pasadena	Y	–	0.100	–	1D
19	B7PZ		Pasadena	Z	–	–	0.065	1D
20	B7PXYZ		Pasadena	X	0.100	0.085	0.065	3D
21	B7SHX		SHW2	X	0.100	–	–	1D
22	B7SHY		SHW2	Y	–	0.100	–	1D
23	W3	White noise		–	0.030	0.030	0.030	3D
24	R7EX	Rare 7	EI Centro	X	0.200	–	–	1D
25	R7EY		EI Centro	Y	–	0.200	–	1D
26	R7EZ		EI Centro	Z	–	–	0.130	1D
27	R7EXYZ		EI Centro	X	0.200	0.170	0.130	3D
28	R7PX		Pasadena	X	0.200	–	–	1D
29	R7PY		Pasadena	Y	–	0.200	–	1D
30	R7PZ		Pasadena	Z	–	–	0.130	1D
31	R7PXYZ		Pasadena	X	0.200	0.170	0.130	3D
32	R7SHX		SHW2	X	0.200	–	–	1D
33	R7SHY		SHW2	Y	–	0.200	–	1D
34	W4	White noise		–	0.030	0.030	0.030	3D
35	R8EXY	Rare 8	EI Centro	X	0.360	0.306	–	2D
36	R8EYX		EI Centro	Y	0.306	0.360	–	2D
37	R8EXYZ		EI Centro	X	0.360	0.306	0.230	3D
38	R8PXY		Pasadena	X	0.360	0.306	–	2D
39	R8PYX		Pasadena	Y	0.306	0.360	–	2D
40	R8PXYZ		Pasadena	X	0.360	0.306	0.230	3D
41	R8SHX		SHW2	X	0.360	–	–	1D
42	R8SHY		SHW2	Y	–	0.360	–	1D
43	W5	White noise		–	0.030	0.030	0.030	3D

a series of base accelerations were input to the shaking table with increasing intensities. These inputs were divided into four phases, i.e. frequently occurring (frequent 7), basic (basic 7), rarely occurring earthquakes (rare 7) of intensity 7 and rarely occurring earthquake of intensity 8 (rare 8). It should be pointed out that the rare 8 phase was only intended to obtain further investigation of the failure mechanism of the targeted structure subjected to extremely strong earthquakes. After each phase of test, the white noise was input to study the dynamic characteristics of the model.

There were 1D, 2D and 3D earthquake inputs used in this test. As stipulated for 2D horizontal inputs by CSDB, the PGA ratio of the principal direction to the other direction should be 1:0.85; as to 3D inputs, the ratio is 1:0.85:0.65, corresponding to the horizontal principal direction, the other horizontal direction and the vertical direction, respectively. The earthquake inputs were compressed in time by applying the scale factor of 0.193 shown in Table 1.

Due to the noise of the shaking table system and the feedback effect of model to the table under different excitations, there were some discrepancies in the peak acceleration values between the input and recorded table motions. The effect was considered to modify the results accordingly in evaluating the seismic responses of the prototype structure.

5. TEST RESULTS

The test structure was subjected to the ground motions as described in Table 2. The global responses of the structure in terms of the dynamic properties and torsion, as well as the failure patterns of the test structure, were discussed as follows.

5.1. Dynamic responses

The frequencies of the model were obtained by inputting a white noise signal, whose power spectral density distributes evenly over a frequency range. Table 3 presents variations of model dynamic properties, which were measured after each test phase. The fundamental vibration mode was found to be a torsional mode with initial natural frequency of 4.590 Hz, and the next two modes were of translation in directions *Y* and *X* with frequencies of 5.340 and 5.343 Hz, respectively, which infers that the difference in initial stiffness between directions *Y* and *X* is very little. From this table, it can be seen that the vibration mode of translation in direction *X* became the first one, while the torsional mode changed to the second one after the rarely occurring earthquake of intensity 7, which suggested that severer damage had taken place in direction *X* than in *Y*.

From Table 3, it can also be calculated that the period ratio between the first torsional mode and the first translational mode was 1.163, which exceeded the limit value of 0.9, stipulated to control torsional effect in the code TSCSTB. It was attributed to the irregular shape of an inverted trapezoid, and the distribution of mass was enlarged accordingly as gradually extending of the exposition areas from mid-height to top, thereby increasing the moment of inertia, which results in the torsinal vibration to be the first mode. However, monitoring results of torsional responses show that the torsional effect is not significant. Figure 11 shows the time history of torsional angle at roof level under the input of F7PXYZ (case 9), a 3D ground motion that was designed to excite large torsional response. It can be seen that the maximum value of torsional angle at roof level is 0.0005 rad (t = 6.19 s) in direction *X*, and 0.0006 rad (t = 5.33 s) in direction *Y*, which show that the torsional responses are not significant.

Table 3. Dynamic properties of model structure.

Test phase	Dynamic property	Vibration order		
		1	2	3
Initial	Frequency (Hz)	4.590	5.340	5.343
	Damping ratio	0.0732	0.0571	0.0622
	Vibration modes	Torsion	Translation in *Y*	Translation in *X*
After Frequent 7	Frequency (Hz)	4.570	5.340	5.342
	Damping ratio	0.0758	0.0380	0.0560
	Vibration modes	Torsion	Translation in *Y*	Translation in *X*
After Basic 7	Frequency (Hz)	4.570	5.088	5.332
	Damping ratio	0.0802	0.0543	0.0398
	Vibration modes	Torsion	Translation in *X*	Translation in *Y*
After Rare 7	Frequency (Hz)	4.069	4.322	4.832
	Damping ratio	0.0847	0.0730	0.0429
	Vibration modes	Translation in *X*	Torsion	Translation in *Y*
After Rare 8	Frequency (Hz)	3.305	3.305	3.812
	Damping ratio	0.0993	0.0463	0.0487
	Vibration modes	Translation in *X*	Torsion	Translation in *Y*

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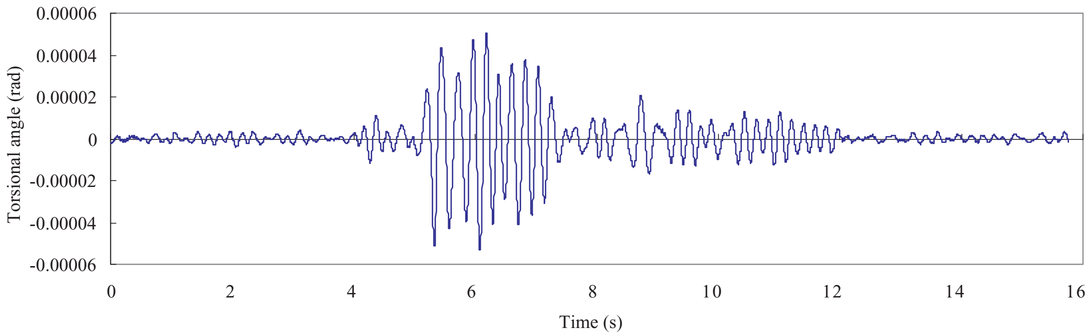
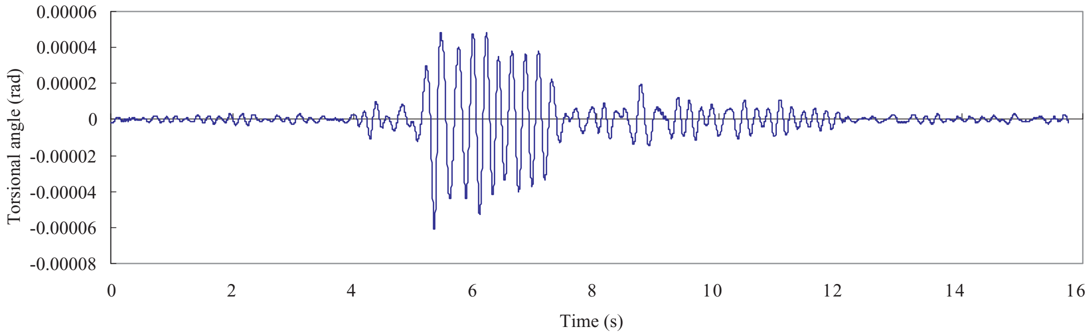


(a)



(b)

Figure 11. Time history of torsional angle at roof level under F7PXYZ in: (a) direction *X* and (b) direction *Y*.

5.2. Failure patterns

Under the earthquake simulations of the first phase (cases 2–11), corresponding to the minor earthquake of intensity 7, no visible cracks were observed. After the second white noise test, it was found that the frequencies of the model decreased very slightly, which indicated that the model structure still behaved in a linear state during this phase.

For simulations of moderate earthquake of intensity 7 (cases 13–22), four fine diagonal cracks occurred on the exterior faces of the RC core wall at the 10th floor, corresponding to the height of 33.3 m in prototype where the floor starts to extend outwards. Several fine vertical cracks were also detected at the ends of the coupling beams located between the 4th and 10th floors (see Figure 12(a)). Two of the inclined columns had a slight twist (see Figure 12(b)), which may be attributed to the instability of inclined columns.

Under the simulation tests with rare intensity of 7 (cases 24–33), new diagonal cracks occurred and previously observed cracks developed on the core wall between the 4th and 10th floors. Both ends of most coupling beams below the 10th floor showed vertical cracks (see Figure 13(a)), and for those with sectional dimensions 800 × 4500 mm at the 10th floor, diagonal cracks were observed at this phase (see Figure 13(b)). However, previously observed twist of inclined columns did not develop further.

The last test of rare intensity of 8 (cases 35–42) caused relatively significant damage of the model. Almost all coupling beams within the core wall showed vertical cracks at both ends, and developed throughout the whole height of beams. Horizontal cracks were first observed at the base of the core wall (see Figure 14(a)), and two vertical cracks also occurred first at the bottom of the core wall (see Figure 14(b)). No new twist of other inclined columns was observed. In addition, no damages of the joints, which were thought to have concentrated stress because of connecting three inclined columns

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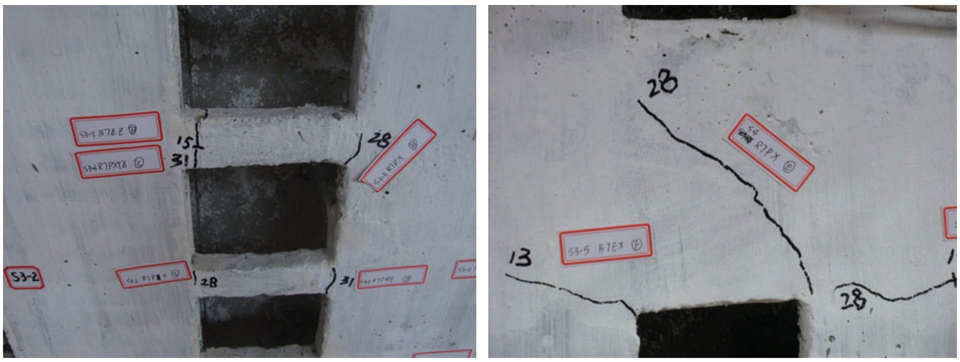
(a) (b)

Figure 12. (a) Crack at the end of coupling beam. (b) Slight twist of inclined column.



(a) (b)

Figure 13. (a) Cracks throughout the height of coupling beams. (b) Diagonal cracks at the coupling beam at 10th floor.

(a) (b)

Figure 14. (a) Horizontal crack at the base of core wall. (b) Vertical cracks at the bottom of core wall.

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with steel reinforced concrete columns embedded in the corner of the core wall, were detected throughout the test.

6. SEISMIC PERFORMANCE EVALUATION OF THE PROTOTYPE STRUCTURE

Based on the test results of the model, the dynamic response of the prototype structure can be calculated according to the similitude relationship. The seismic performances of the prototype structure in terms of dynamic properties, displacement responses, as well as distribution of shear force, are evaluated.

6.1. Dynamic properties

The frequencies of the prototype structure can be derived from model using the equation:

$$f_p = f_m / S_f \tag{1}$$

where f_p and f_m refer to the frequencies of the prototype and model structure, respectively; and S_f is the scaling factor of frequency. The first three frequencies of the prototype structure are presented in Table 4. Comparison between W1 and W2 shows that there is almost no variation of the frequencies after frequently occurring earthquake of intensity 7, which infers that the structure is still in an elastic state. After the major earthquake of intensity 7, the reductions of the initial three frequencies are 5.8, 9.5 and 23.8% corresponding to the torsional mode and translational modes in directions *Y* and *X*, respectively. It can be concluded that severer damage has taken place in direction *X*, thereby producing greater loss of stiffness in direction *X* than in *Y*.

6.2. Displacement responses

The displacement of the prototype structure discussed herein consisted of torsional and translational displacements, calculated by the following formula:

$$D_p = \left(\frac{a_t}{a_a}\right) \cdot \frac{D_m}{S_d} \tag{2}$$

where D_p and D_m refer to the displacements of the prototype and model structure, respectively; a_t and a_a refer to the target and actual PGA of the test; and S_d is the scaling factor of displacement. Figure 15 illustrates the maximum floor displacements of the prototype structure along the height of RC core wall under different levels of earthquake. Generally, the floor displacement responses in direction *X* are larger than that in direction *Y*, which may be attributed to the test programme shown in Table 2. As can be seen in this table, there are two runs of 3D input motions at each test phase with the principal direction only in direction *X*, thus resulting in larger dynamic response in direction *X* than in *Y*. Accordingly, the structural damage in direction *X* is severer under a large earthquake, thereby producing greater loss of stiffness in direction *X* than in *Y* after the test phase of rare 8.

Table 5 lists the maximum drift ratios of the prototype structure in different phases. The maximum inter-storey drifts are 1/2003 in direction *X*, and 1/2893 in direction *Y* at frequent earthquake level, taking place at floors between the height of 9 and 33.3 m. Both of them satisfy the limit value of 1/1000, which is stipulated in the code TSCSTB for structural wall system. These values are increased to 1/352 and 1/634, respectively, under a rarely occurring earthquake of intensity 7, which also meet the limit value of 1/100 specified in the code.

The torsional responses of the prototype structure are evaluated based on two indexes, i.e. torsional displacement ratio and torsional angle. In order to prevent excessive structural torsion, the torsional displacement ratio is suggested to be no more than 1.2 and should be no more than 1.5, as stipulated in the code TSCSTB. Table 6 provides the maximum torsional displacement ratios under different earthquake levels. From this table, it can be observed that only two values exceed the suggested limit

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Table 4. Dynamic properties of the prototype structure.

Test phase	Dynamic property	Vibration order		
		1	2	3
W1	Frequency (Hz)	0.883	1.027	1.028
	Vibration modes	Torsion	Translation in *Y*	Translation in *X*
W2	Frequency (Hz)	0.880	1.027	1.028
	Vibration modes	Torsion	Translation in *Y*	Translation in *X*
W3	Frequency (Hz)	0.880	1.026	0.964
	Vibration modes	Torsion	Translation in *X*	Translation in *Y*
W4	Frequency (Hz)	0.783	0.832	0.930
	Vibration modes	Translation in *X*	Torsion	Translation in *Y*
W5	Frequency (Hz)	0.636	0.636	0.734
	Vibration modes	Translation in *X*	Torsion	Translation in *Y*

Note: W1–W5 refer to the test cases of white noise, as shown in Table 2.

Table 5. Maximum drift ratios of the prototype structure.

Test phase	Roof displacement/total height		Maximum inter-storey drift ratio	
	Direction *X*	Direction *Y*	Direction *X*	Direction *Y*
Frequent 7	1/2459	1/3499	1/2003	1/2893
Basic 7	1/820	1/2062	1/690	1/1667
Rare 7	1/444	1/749	1/352	1/634
Rare 8	1/314	1/463	1/243	1/370

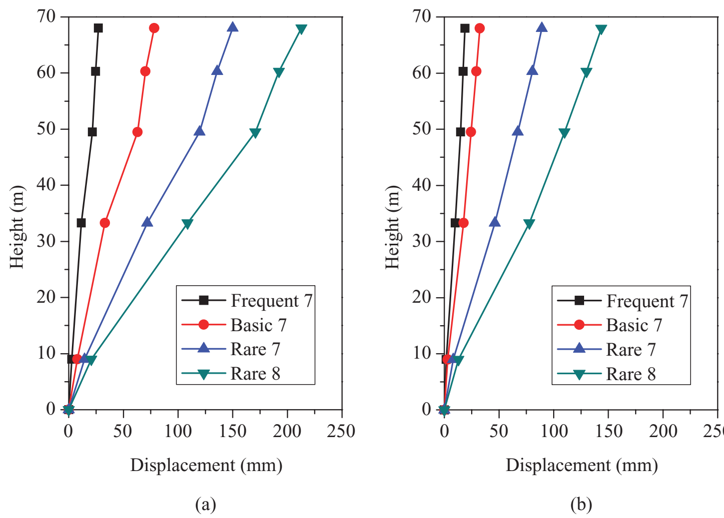

Figure 15. Maximum floor displacements in: (a) direction *X* and (b) direction *Y*.

of 1.2, but far less than 1.5. This indicates that the torsional effect is not very obvious, even though the torsional period ratio is larger than 0.9. The maximum torsional angles under different earthquake levels are also presented in Table 6. It can be seen that the maximum torsional angle is 1/12 646 rad under frequent 7, and 1/4108 rad under rare 7, which also prove that the torsional effect is not significant.

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Table 6. Maximum torsional responses of the prototype structure at roof level.

	Torsional displacement ratio			Torsional angle (rad)		
Test phase	Direction *X*	Direction *Y*	Direction *XYZ*	Direction *X*	Direction *Y*	Direction *XYZ*
Frequent 7	1.21	1.18	1.22	1/13 009	1/12 646	1/13 871
Basic 7	1.13	1.05	1.17	1/5 483	1/28 776	1/7 949
Rare 7	1.07	1.03	1.05	1/4 108	1/14 737	1/5 522
Rare 8	1.02	1.01	1.03	1/5 629	1/11 902	1/2 440

Note: Directions *X*, *Y* and *XYZ* refer to the 1D inputs in direction *X* and *Y*, and 3D input with principal direction in *X*, respectively.

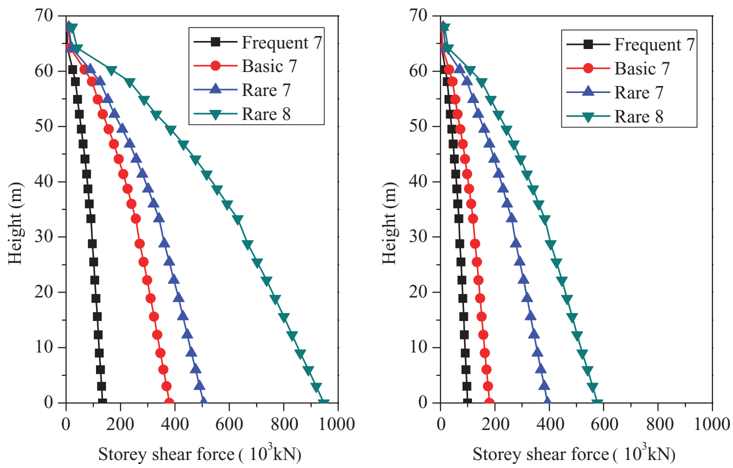

Figure 16. Distribution of storey shear force in: (a) direction *X* and (b) direction *Y*.

6.3. *Distribution of shear force*

Shown in Figure 16 is the distribution of storey shear force under different earthquake levels. In general, the distribution of shear force along the structural height is triangular, and the storey shear forces become larger as the PGAs of input excitations increase. The storey shear forces in direction *X* are larger than in *Y*, which is consistent with the displacement response.

7. CONCLUSIONS AND SUGGESTIONS

Based on the shaking table model tests and analysis of test data, the following conclusions can be drawn:

(1) The first vibration mode of the prototype structure is a torsional mode with initial natural frequency of 0.883 Hz, and the next two modes are of translation in directions *Y* and *X* with initial frequencies of 1.027 and 1.028 Hz, respectively, demonstrating that the difference in initial stiffness between the two directions is very little.
(2) The frequencies of the first three modes decrease very slightly after the frequent earthquake of intensity 7, indicating the prototype structure is still in an elastic stage; under the major earthquake of intensity 7, the frequencies decrease apparently. Quantitatively, the frequencies of the first three modes are reduced by 5.8, 9.5 and 23.8%, corresponding to the torsional mode and translational modes in directions *Y* and *X*, respectively. It infers that severer damage has taken place in direction *X*, thereby producing greater loss of stiffness in direction *X* than in *Y*.
(3) In spite that the torsional period ratio exceed the limit value of 0.9, stipulated in Chinese code, the torsional responses are not significant. The maximum torsional displacement ratios are within rational ranges of Chinese code.

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(4) The coupling beams except for those at the height of 33.3 m are able to dissipate energy, thus provide additional global structural ductility.
(5) Under a major earthquake of intensity 8, the weak points of the prototype structure are detected as follows:
 (a) The coupling beams (800×4500 mm in sectional dimensions) at the height of 33.3 m are damaged severely, forming lots of crossing inclined cracks which run almost throughout the whole height of the coupling beams.
 (b) The bottom of the core in the north side experiences severe damage, forming long vertical cracks with concrete cover spalling.
(6) Considering the existence of the weak points, the following suggestions are put forward accordingly so as to improve the seismic behaviour of the prototype structure.
 (a) The coupling beams at the height of 33.3 m may have too large stiffness, thus resulting in not only the severe damage of themselves, but also the core walls adjacent to these beams. It is suggested to reduce the sectional dimensions of these coupling beams appropriately.
 (b) The transverse connection of inclined columns, located at the perimeter of the structure, should be strengthened so as to increase the lateral stability and resist the earthquake effect out of plane.

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APPENDIX: NOTATION

In this paper, the subscript 'm' refers to the corresponding quantities of the model structure, while subscript 'p' corresponds to the prototype structure. The following symbols are used in this paper:

a_{a} actual peak ground acceleration of model test
a_a actual PGA

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a_{t} target peak ground acceleration of model test
a_t target PGA
D displacement response
D_m displacement of the model
D_p displacement of the prototype
f frequency
f_m frequency of the model
f_p frequency of the prototype
S_{a} scaling factor of acceleration
S_d scaling factor of displacement
S_{E} scaling factor of elastic modulus
S_{f} scaling factor of frequency
S_{l} scaling factor of dimension
S_σ scaling factor of stress
S_ε scaling factor of strain
S_ρ scaling factor of material density

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