Shaking Table Test on a Reinforced Concrete Core Wall-Steel Frame Hybrid Structure

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ABSTRACT :

An efficient hybrid structural system is obtained when reinforced concrete (RC) core walls are used in conjunction with steel perimeter frames. Core walls can be effectively formed by coupling individual wall piers with the use of coupling beams. Outrigger beams are framed between the core walls and columns in the perimeter frame. The successful performance of such one-tower hybrid structural systems depends on the adequacy of the primary individual components which are the core walls, steel frames, and frame-core connections. For the multi-tower hybrid structures, the problem becomes more complex. A multi-tower RC core wall-steel frame hybrid structure, rigidly connected at the middle height of the building, was thus introduced in this paper. A 1/15 scaled model structure was tested on the shaking table at Tongji University. Earthquake records of El Centro and Pasadena, local accelerogram of Shanghai Wave II, and the white noise were sequentially input to the shaking table under frequent, basic, and rare earthquake intensities. In the experiment, the structural damage was observed and the stress, acceleration, displacement responses were recorded. Then, the dynamic responses of the model structure were interpreted to the seismic design of the prototype structure. It was concluded that the torsional effect of the multi-tower structure was so obvious that those walls at the corner of the floor plan cracked first. The largest inter-story drift happened at the connected truss story, where buckling of the steel members was observed. Some suggestions to the structural design of the multi-tower hybrid structure were also given.

KEYWORDS: RC core wall-steel frame, hybrid structure, multi-tower, shaking table test

1. INTRODUCTION

In China, the irregular high-rise buildings are becoming more extensive. Most of these buildings have irregular structures against traditional structural concept. Their irregularities may exist in configuration of the building, in differences between the storey heights, in distribution of masses and rigidities, in creating short columns, and also in non-orthogonal placement of columns and shear walls (Tezcan and Alhan, 2001). According to the past experience, it is the irregularities in the structural design that directly or indirectly cause the collapse or severe damage of the buildings under strong earthquakes. Detailed investigation is thus necessary to verify the safety and rationality of the seismic performance of the irregular buildings.

In the past several decades, substantial progress has been made in computer-based procedure for analysis of structures. Using one analytical method, however, it is still difficult to predict the seismic performance of irregular structures. Shaking table test is another useful procedure to examine the structural seismic performance. Several shaking table facilities have lately been constructed, including E-defence shaking table (Katayama, 2005), EU Centre shaking table (Pavese, et al., 2005), shaking table at Montreal Structural Engineering Laboratory (Tremblay, et al., 2005), shaking table at China Academy of Building Research, etc. Researchers have more and more investigated the earthquake-resistant behaviors of irregular buildings by shaking tables (Ko and Lee, 2006; Lu, et al., 2007a, 2007b; Lu, et al., 2008; Tong, et al., 2007).

To study the seismic performance of irregular structures, shaking tables tests are used in conjunction with the numerical analysis. First, preliminary analysis is carried out for the peer review on the irregular structure. Then, shaking table model test is carried out to find the weak positions and to obtain the structural parameters of the building, which are the basis for the further experiment on the weak joint and refined analysis of the overall structure.

The target building of this paper is a multi-tower high-rise building, whose hybrid structure is out of Chinese code. A detailed shaking table model test was performed by the working group at the State Key Laboratory for Disaster Reduction in Civil Engineering (SlabDRCE) of Tongji University, China. The calculation from the model responses to the prototype responses was made according to the similitude law, based on which, measures for improving the seismic performance of the structure were given at the last part of the paper.

2. DESCRIPTION OF THE BUILDING STRUCTURE

2.1 Building Structure

Shanghai International Design Centre (SHIDC) is an office building which is built for the centennial anniversary of Tongji University in 2007. For its significant background, prestigious Japanese Architect Tadao Ando was invited to design the architecture of SHIDC. He finally decided to apply the overturned Arabic number "4" as the main elevation of SHIDC, as shown in Figure 1.

It is structural engineers' responsibility to realize the design of the architects. In SHIDC, reinforced concrete core wall-steel frame (RCC-SF) hybrid structure was employed (Figure 1 and Figure 2).

The structure consists of a major tower, a minor tower and a 4-storey podium adjacent to the minor tower. The major tower is 25-storey with a height of 99m, while the minor tower is 12-storey with a height of 48m. Both towers are reinforced concrete core wall-steel frame (RCC-SF) system. There are 7.5m-span cantilever floors at the middle height of the major tower from story 11 to story 13. Five inclined columns (the inclined angle is 15°) in the minor tower support cantilever beams at each storey. The connecting corridor consisting of steel trusses spans 17.5m to join the two towers rigidly between storey 11 and storey 13.

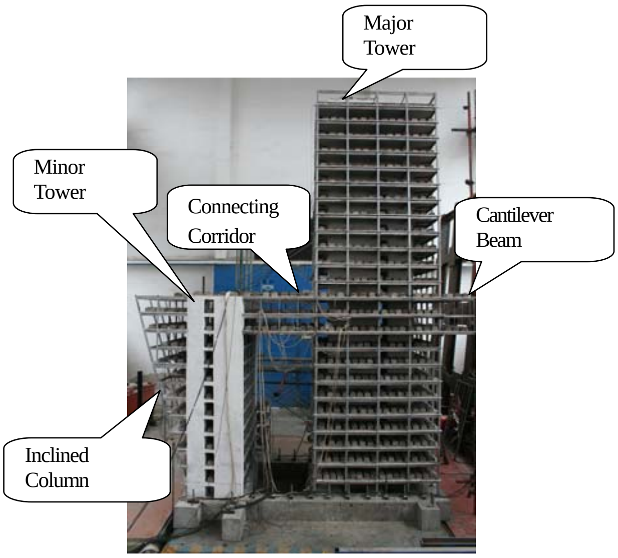


Figure 1 Structural elevation of SHIDC model

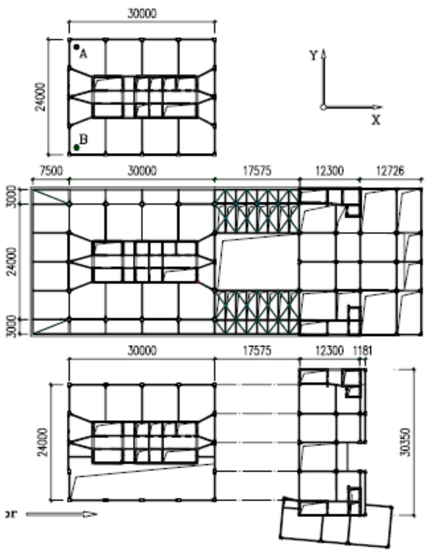


Figure 2 Typical plan layouts of SHIDC

2.2 Structural Irregularities

According to Chinese Code for Seismic Design of Buildings (GB50011-2001, 2001) and Technical Specification for Concrete Structures of Tall Building (JGJ3-2002, 2002), the main characteristics of SHIDC structure can be summarized as follows.

1). In the plan layouts, there are large openings exist in continuous seven stories of the Major Tower, whose area is over 30% floor area and beyond the limitation of the Chinese Code. Second, the length of the cantilever floor adds up to 7.5m and it locates at 50m above the ground, which may potentially vibrate during the strong earthquake.

2). In the elevation, SHIDC is a two-tower-connected hybrid structure and structural height of the Major Tower is two times of the Minor Tower. China has built many single-tower RCC-SF buildings, but it has no experience in designing a different-height multi-tower hybrid systems. And, there are five inclined steel columns in the Minor Tower, which are easier to buckling under non-uniformly distributed loads.

Given the above irregularities and complexity of the structure, it is necessary to precisely study SHIDC seismic behaviour and evaluate its performance to resist designed earthquakes.

3. SHAKING TABLE TEST OF SHIDC

3.1 Shaking Table Facility

The MTS shaking table used for the testing can input three-dimensional and six degree-of-freedom motions. It has a dimension of 4m × 4m with a maximum payload of 250kN. With a 150kN payload, the maximum accelerations are 1.2g, 0.8g and 0.7g for the horizontal, transverse and vertical directions, respectively. Its working frequency ranges from 0.1Hz to 50Hz and 96 channels are available for data acquisition (SlabDRCE, 2008).

3.2 Building Model Material

Material properties are very important in the dynamic model tests. According to the purpose of experiment, shaking table models can be classified into two categories: elastic model and strength model. The material of the former does not need to be exactly the same as that of the prototype provided that it remains elastic during testing and has the same distribution of mass and rigidity as the prototype structure. However, the similitude of elastoplastic material between the model and the prototype is essential in the strength model (Lu, et al., 2007a). Thus, based on past experiences, copper plates were applied to simulate the steel structural members and fine-aggregate concrete with fine wires were chosen to construct the RC components in SHIDC model.

3.3 Similitude Relationship

Corresponding to the fact that the dynamic behaviour of a structure is fully described by means of three basic quantities, only three model quantities can be arbitrarily selected in dynamic problems (Sabnis, et al., 1983). Practically, the similitude equilibrium in the dynamic test is given in Eqn. 3.1, where S_l, S_E, S_a, S_p is scaling factor of dimension, elastic modulus, acceleration, density, respectively.

$$\frac{S_E}{S_\rho \cdot S_a \cdot S_l} = 1 \qquad (3.1)$$

First, based on the capacity and the size of the shaking table, the scaling factor of dimension S_l was chosen to be 1/15. SHIDC model was thus built with a height of 6.6m. Second, since the prototype structure was made of concrete and steel, the overall scaling factor of elastic modulus S_E should be determined by two kinds of materials. It also should be noted that there is a distinct decrease in elastic modulus when copper is welded. According to the material test results, the overall scaling factor of elastic modulus was determined to be 0.35. Third, considering the capacity of the shaking table and avoiding the disturbance of the noise, the scaling factor of acceleration was set to be 2.5. The total height of the model, including the additional artificial mass, was estimated to be 195kN. All the other scaling factors could be derived and the typical factors are listed in Table 3.1.

Table 3.1 Typical Scaling Factors of SHIDC Model

Parameter	Relationship	Model/prototype
Length	S_l	1/15
Elastic modulus	S_E	0.35
Stress	$S_\sigma=S_E$	0.35
Strain	S_σ/S_E	1.00
Density	$S_\sigma/(S_a \cdot S_l)$	2.10
Force	$S_\sigma \cdot S_l^2$	1.56E-03
Frequency	$S_l^{-0.5} \cdot S_a^{0.5}$	6.12
Acceleration	S_a	2.50

It is still difficult to have the same stress scaling factor for both aggregate and steel bars. Therefore, the strength alternation in structural members should be considered in the model design, as shown in the work (Lu, et al., 2007a).

The complete elevation of SHIDC test model is shown in Figure 1.

3.4 Test Program

There were 70 sensors in total installed on the SHIDC model structure, which include 29 accelerometers on the ground, 6th, 9th, 11th, 13th, 15th, 20th and 25th story, respectively; 16 displacement transducers on the ground, 6th, 11th, 13th, 20th and 25th story, respectively; and, 25 strain gauges on surfaces of a few structural members such as lower shear walls and connecting trusses.

Base on the soft site condition of Shanghai, three ground motions were input during the test, include two strong earthquake records (El Centro record from California Imperial Valley earthquake and Pasadena record from California Kern County earthquake) and one Shanghai artificial accelerogram.

In Chinese Codes, Shanghai is assigned to earthquake zone of intensity 7 with peak ground acceleration (PGA) of 0.10g. The test was carried out in four stages. The first three stages represented frequent, basic, rare occurrence of intensity 7, respectively. The last one represented the rare occurrence of intensity 8, which was utilized for further investigation of the SHIDC structure subjected extremely strong earthquakes. (Zhou and Lu, 2008)

4. EXPERIMENTAL RESULTS OF SHIDC MODEL STRUCTURE

4.1 Cracking and Failure Patterns

At the test stage of frequent earthquakes of intensity 7, no visible damage was observed. After the white noise scanned the model, it was found that the frequencies in both Y and X directions reduced slightly. That is to say, micro-cracks of the model had already developed inside.

At the test stage of basic earthquakes of intensity 7, shear wall concrete of the Minor Tower first appeared cracks at storey 3 while the Major Tower remained undamaged. Steel beams at the connecting corridor began to buckling. The results of the white noise showed the stiffness of the structure decreased noticeably.

At the test stage of rare earthquakes of intensity 7, existing crack developed. For the Major Tower, cracks appeared at the RC coupling beam ends from storey 10 to storey 13. An obvious crack was observed at the connected end of the cantilever slabs at story 13 due to negative moment (Figure 3(a)). For the Minor Tower, new cracks appeared at the coupling beam ends from storey 4 to storey 12 and at the bottom corner of the shear walls. More beams of the connecting corridor buckled.

At the test stage of rare earthquakes of intensity 8, existing cracks remarkably propagated. For the Major Tower, more cracks spread at the coupling beam ends and new cracks were also found on the shear walls at storey 5 and storey 15. The Minor Tower damages badly in the middle and bottom part of the shear walls and at most of the coupling beam ends (Figure 3(b)). For the connecting corridor, welded beams of the steel truss ruptured at joints (Figure 3(c)), and steel beams buckled obviously out-of-plane (Figure 3(d)).

(a) cantilever slabs (b) Minor Tower (c) joint at the corridor (d) steel beams at the corridor

Figure 3 Failure pattern of SHIDC model

From the observation, it can be concluded as follows. (1) The RC core wall of the Major Tower stayed undamaged until the rare earthquake of intensity 7, while that of the Minor Tower cracked at the former stage, i.e., basic earthquake stage. (2) There is no obvious buckling observed in the perimeter steel frame members, even in the inclined steel columns. (3) Rigid joints between the steel truss and the core walls worked well but steel members buckled under the earthquakes. (4) Those cantilever slabs located so high that they damaged more severely than those in the expected design.

4.2 Experimental Dynamic Characteristics

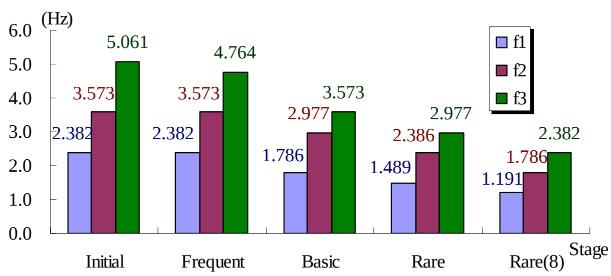


Figure 4 Frequencies at different stages

The first three vibration modes were translation in Y, translation in X, Torsion, respectively. Figure 4 gives the values of the first three frequencies at different earthquake levels. It can be seen that all three frequencies decreased with the increase of the strong earthquake input. The first frequency decreased from 2.382Hz to 1.191Hz and the second frequency from 3.573Hz to 1.786Hz. Thus, the equivalent stiffness decreased about 75% when the model underwent the rare earthquake of intensity 8.

5. EXPERIMENTAL RESULTS OF SHIDC PROTOTYPE STRUCTURE

5.1 Dynamic Characteristics

Model periods can be extrapolated to the prototype structure by the similitude relation. The first six experimental periods are summed up in Table 5.1. The ratio of the torsional period to the first translational period (1.21/2.57) is less than 0.9, which meets the requirement of the Chinese code about structural torsion.

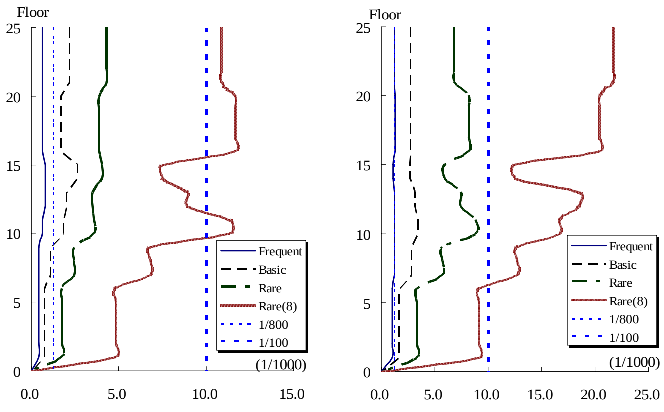
Table 5.1 Dynamic Characteristics (Vibration Periods)

Period	Test (s)	Analysis (s)	Mode shapes
T1	2.57	2.03	Translation in Y
T2	1.71	1.15	Translation in X
T3	1.21	0.89	Torsion
T4	0.73	0.56	Translation in Y
T5	0.59	0.44	Torsion
T6	0.47	0.35	Translation in X

5.2 Inter-story Drift

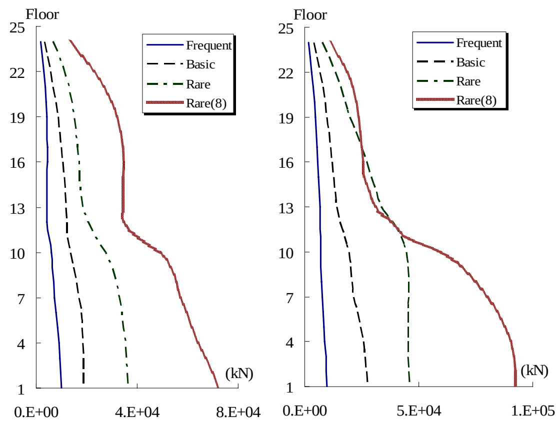
The method of acceleration integration is used here to achieve the final results of displacement. The inter-story drift is calculated by subtracting the lateral displacements of two adjacent floor levels. Figure 5 shows envelops of the inter-story drift of the prototype structure.

It can be shown that the inter-story drifts under frequent earthquake occurrence are 1/778 in direction Y and 1/1234 in direction X, which in direction Y is slightly beyond the code requirement of 1/800. At the rare earthquake stage of the designed intensity 7, inter-story drift increase to 1/114 in direction Y and 1/235 in direction X, both of which are less than the elastoplastic inter-story drift limitation of 1/100 for hybrid systems.



(a) x direction (b) y direction

Figure 5 Envelops of the Inter-story Drift



(a) x direction (b) y direction

Figure 6 Envelops of the Shear Force

5.3 Shear Force

The story shear is calculated by summing the individual floor inertia forces at each floor above that story; these inertia forces are calculated by multiplying the measured absolute acceleration by the floor weight. Distributions of storey shear force are figured in Figure 6. Under frequent earthquake level, the base shear force is 10024kN in direction X and 9730kN in direction Y. That is to say, the ratio of the shear force to total weight is 3.26% in direction X, 3.16% in direction Y, respectively.

6. CONCLUSIONS AND SUGGESTIONS

Shanghai International Design Center (SHIDC) is a hybrid tall building with two different-height towers connecting together, which is extremely irregular in both plan and elevation. As suggested by the review panel, shaking table model test was carried out at the State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, China. The seismic performance of SHIDC structure was evaluated based on the dynamic test as follows.

1. The irregular structure can resist designed frequent earthquakes without damage, resist basic earthquakes with some structural cracking and deformation, and resist rare earthquakes with some major damage, but without catastrophic collapse.

2. When the structure is subjected to earthquake waves of frequent stage, the maximum inter-storey drift in direction Y is 1/778, which is slightly larger than the allowable value of 1/800 according to Chinese code.

3. The RC core wall of the Major Tower stayed undamaged until the rare earthquake of intensity 7, while that of the Minor Tower cracked at the basic earthquake stage. Steel columns are suggested to be encased as the boundary columns of the Minor Tower, which will contribute to the seismic behaviour of the Minor Tower and to the displacement of the overall building.

4. There is no obvious buckling observed in the perimeter steel frame members and the force distribution between the core walls and steel frames is reasonable.

5. Rigid joints between the steel truss and the core walls worked well but steel members buckled under the rare earthquake of intensity 8, which successfully denote the structural strong-columns-weak-beam concept.

6. The strength and the stiffness of the cantilever floors need to be improved considering the dynamic effect.

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