Shaking Table Test and Numerical Analysis on a Shear Wall High-Rise Structure with Huge Podium

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SUMMARY:
Owing to limited availability of urban land, a type of complex structure system characterized by upper multi-tower buildings combined with a huge podium has been appeared so as to make full use of the above-space of podium structure. Aiming at evaluating the seismic behavior of a shear wall high-rise building with huge podium, a 1/20 scaled model was tested on the shaking table under a series of seismic excitations. Furthermore, a numerical model was set up by using computer program ETABS, and dynamic time-history analysis was carried out. Seismic responses of the structure were analyzed in terms of dynamic property, failure pattern, displacement response. The obtained results indicate that maximum inter-story drift happened at the upper floor, where severe shear failure was observed in the short-leg shear walls. More attention should be paid to the seismic design of the upper shear wall structure for its application in high intensity zones.

Keywords: shaking table test；numerical analysis；shear wall；huge podium；seismic behavior

1. INTRODUCTION

Modern cities have experienced a rapid development of Subway construction, which is considered as a convenient and efficient public traffic system. Traffic facility such as metro depot which serves for metro vehicles has to cover a large area of urban land. Owing to limited availability of urban land and multi-functional requirement for modern buildings, a type of complex structure system characterized by upper multi-tower buildings combined with a huge podium has been appeared so as to make full use of the above-space of the podium structure, resulting in significant economical saving. In most cases, the podium structure can provide a large open space for shopping mall and parking, while the upper structures are built for residential buildings or hotels. However, the combination of the podium and upper structures may lead to a sudden change of the lateral stiffness and mass at the top of the podium structure, thus the high-rise buildings with a podium may suffer from the so-called whipping effect that the seismic response of the upper part of the tower structure may be significantly amplified under earthquake excitation. For this purpose, Fang and Wei (1995) conducted a series of time-history analysis on single-tower and multi-tower structures with a large podium to investigate the effects of stiffness, height and strength of the podium on dynamic responses of the tower. Furthermore, theoretical and experimental studies were carried out to explore the possibility of using magnetorheological (MR) dampers to connect a podium structure to a multi-story building to prevent the whipping effect (Qu et al., 2001 and Xu, et al., 2005). It was concluded that MR dampers could significantly mitigate the seismic whipping effect on the building and reduce the seismic responses of both the building and the podium structure if the control algorithm was selected properly.

In spite of the relatively more application of such type of structure system, its seismic behavior has not yet been investigated extensively, thus, it is significant to understand the seismic response of such type of structure by shaking table model test and corresponding numerical analysis. In the current paper, shaking table test on a 1/20 scaled model of a shear wall high-rise building with huge podium was carried out under a series of selected excitations, and failure patterns as well as dynamic responses of the structure were discussed according to the experimental results. Then, dynamic time-history analysis on the prototype structure was implemented and compared to the test results to obtain a rational evaluation of the seismic behavior of the building. In addition, some suggestions are given for improving structural design of the building.

2. DESCRIPTION OF THE PROTOTYPE STRUCTURE

A comprehensive project involving metro depot and residential buildings which was located in Beijing, China was initiated previously, the podium covering a large area of land provides a huge platform which can be profitably used for developing a residential community, resulting in effective use of urban land. A total of nine residential buildings were planed above the huge podium, and one typical high-rise building presented in the current paper is selected for shaking table test because of its structural complexity which will be explicated later. The main building is a shear wall high-rise structure, while frame structure system is adopted in the podium to achieve a large open space for metro and car parking. The combinational structure including 2-story podium and 23-story residential building has a total structural height of 78.5m. The first story of the podium serving as metro depot is 9.25m high, and the second story is 4.95m high for car parking. The typical story height of the upper residential part ranging from the 3rd floor to 25th floor is 2.8m, except that the equipment story above the podium is 2.7m high. The layout of the first floor and typical floor of the prototype structure are given in Figure 2.1 and 2.2, respectively.

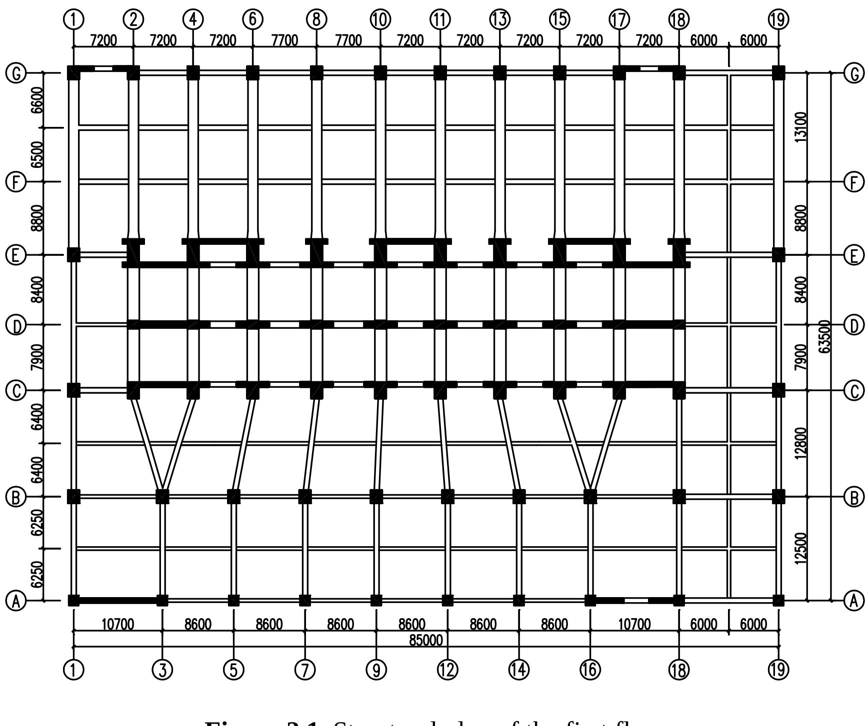


Figure 2.1. Structural plan of the first floor

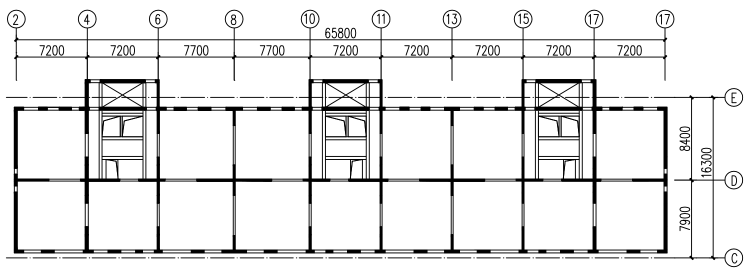


Figure 2.2. Structural plan of the typical floor

The combination of the huge podium and the upper residential structure make the structural design of such multi-functional building more complicated, the main characteristics of the structural system are summarized as follows.

(1) The shear walls of the main building in the transverse direction are continuously arranged along the height of the upper structure and transfer to special members such as T-shaped and cross-shaped short-leg shear walls in the podium structure, allowing the metro lanes running continuously through the main building at the ground level. Nevertheless, the story height and shear wall layout between the 1st floor and 2nd floor are rather different that lead to a sudden change of the lateral stiffness, resulting in vertical irregularity.

(2) Due to the existence of the metro lanes at the ground floor, long-span transfer girders are adopted to support the columns of the second floor at the north side of the podium.

(3) For the upper structure, necessary openings are required for the residential building, leading to a large amount of short-leg shear walls in the longitudinal direction of the structural plan, which are considered as seismic-deficient and likely to damage when experience strong seismic event.

Taking the above irregularities and specialties of the structure into account, some necessary seismic improvements have been applied during the preliminary structural design. Key structural members such as long-span transfer girders, columns of the podium, T-shaped and cross-shaped short-leg shear walls were designed in large size and embedded with H-shaped steel. Moreover, shear walls are arranged between the columns at the edge of the podium so as to enhance the torsional resistance capacity of the entire podium, as well as reducing the displacement of the corner columns.

3. TEST PROGRAM AND ANALYTICAL MODELING

3.1. Model design

To make sure that the model behaves in a similar manner to the prototype structure, the model structure should be designed on the basis of dynamic similitude theory. Three major scaling factors should be determined in the model similitude design. With the size and load-carrying capacity of the shaking table taken into consideration, the dimension scaling factor (S_l) was chosen to be 1/20. The stress scaling factor (S_σ), which was firstly designed and finally determined according to the test results of material properties, was 0.2. The acceleration scaling factor (S_a) was selected to be 2.5. After the three controlling factors were determined, the rest similitude factors were obtained accordingly. The main scaling factors for the test model are presented in Table 3.1.

Table 3.1. Scaling factors for the test model

Parameter	Length (S_l)	Stress (S_σ)	Acceleration (S_a)	Frequency (S_f)	Density (S_ρ)	Force (S_F)
Scale factor	1/20	0.2	2.5	7.07	1.6	5.0×10^{-4}

Reinforced concrete components of the prototype structure were modeled using fine-aggregate concrete together with fine wires. Besides, copper sheets were adopted to simulate the steel structural elements embedded in the steel reinforced concrete members.

Iron blocks and plates were used as additional mass and distributed on each floor of the model. The total mass of the model is estimated to be 21.6 tons including the material itself and additional mass, as well as the base. The total height of the model was approximately 4.6m including the height of the base. The base of the model was securely fixed on the shaking table by tightened bolt connections, ensuring an effective transmission of the earthquake excitation to the base of the test model. An overview of the test model after installation on the shaking table facility is shown in Figure 3.1.

3.2. Instrumentation and testing procedure

A total of 39 accelerometers and 14 displacement transducers were installed on floor levels along the height of the model to monitor acceleration and displacement response of the model during the test, respectively. In addition, 18 strain gauges were also placed on the surface of special structural members such as long span transfer girders, columns of the podium, T-shaped and cross-shaped short-leg shear walls for recording the variation of their strains during the test. All the test data were gathered by data acquisition system for further analysis.

With the spectral density properties of Type Ⅲ site soil taken into consideration, El Centro and Dsst ground motion records were selected as input excitations during the test. In addition to the two records, a synthetic accelerogram designated as 5063 was also chosen for the table excitation. The test was carried out in three stages corresponding to the minor, moderate, and major earthquake levels based on the seismic intensity 8, and peak ground accelerations (PGA) for these three levels adopted during the model test were 0.175g, 0.5g and 1.0g, respectively. White noise excitations were applied before and after each stage of test to determine dynamic characteristics of the model, then the excitation of 5063 synthetic accelerogram, El Centro and Dsst records were input to the model in turn. Structural damage was inspected after each testing stage. For each earthquake record or synthetic accelerogram, the ground motion simulations were conducted twice with the main excitation firstly in direction Y and then in direction X, with the ratio of PGA in three directions as 1:0.85:0.65.

Figure 3.1. Test Model

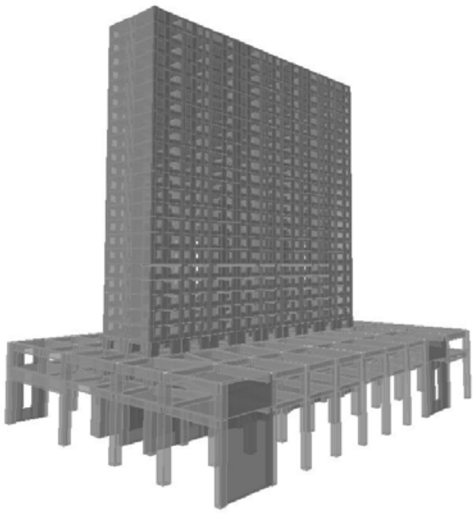


Figure 3.2. Analytical model

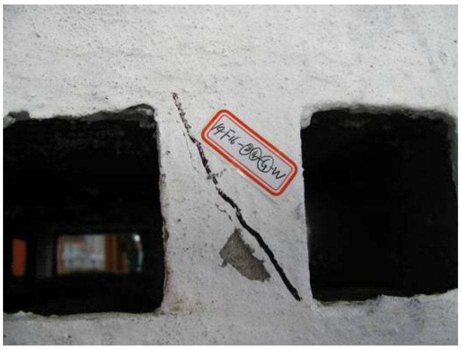
3.3. Analytical modeling

The numerical model of the prototype structure was set up by using computer program ETABS (Computers and Structures Inc., 2000), shell element was selected for shear walls and slabs, and frame element was adopted for columns and beams. The developed analytical model is shown in Figure 3.2. The total mass of the analytical model is 78870 tons. Dynamic time-history analyses including six excitation cases were implemented. It should be mentioned that the longitudinal direction of the structural plan is defined as axis X and the transverse direction as axis Y.

4. EXPERIMENTAL AND ANALYTICAL RESULTS

4.1. Cracking and failure pattern

No visible cracks were observed on the surface of the entire model after the first test stage referring to frequent intensity 8. During the second test stage, majority of diagonal cracks occurred extensively on the longitudinal shear walls at the 16th floor, especially for short-length walls (see Figure 4.1(a)). Moreover, a horizontal bending crack could be seen on the shear wall between the 15th and 16th floors, as shown in Figure 4.1(b). The first excitation case of the third test stage caused relatively serious damage to the model. Severe shear failure was observed mainly on the short-length shear walls at the 16th floor, characterized by a large amount of micro-concrete crushed severely, as well as reinforcing wires exposed and buckled (see Figure 4.1(c)), resulting in local collapse in the end, which is demonstrated in Figure 4.1(d). Special structural members of the podium including columns of the podium, T-shaped and cross-shaped short-leg shear walls remained undamaged and still work well except for only minor damage.



(a) Diagonal crack on wall

(b) Horizontal crack on wall

(c) Concrete crush on wall

(d) Local collapse

Figure 4.1. Typical failure pattern

The main reasons for the observed failure pattern are summarized from both the experimental and structural viewpoints. It is worth noting that most short-length shear walls arranged at the upper floors especially in the longitudinal direction are significantly weakened by architectural openings with certain size and quantity, leading to a relatively low percentage of shear walls for lateral resistance. In addition, it has to be mentioned that the obtained compressive strength values of fine-aggregate concrete used for the 13th floor and above were somewhat lower than that of design requirement, causing a premature shear failure of the shear walls at the upper floors.

4.2. Dynamic property

The variations of the nature frequencies of the model during the test are summarized in Table 4.1. It was found that the natural frequencies of the model decreased slightly after the first test stage. When the model was subjected to stronger excitations, natural frequencies decreased noticeably, indicating the stiffness of model reduced in both X and Y directions.

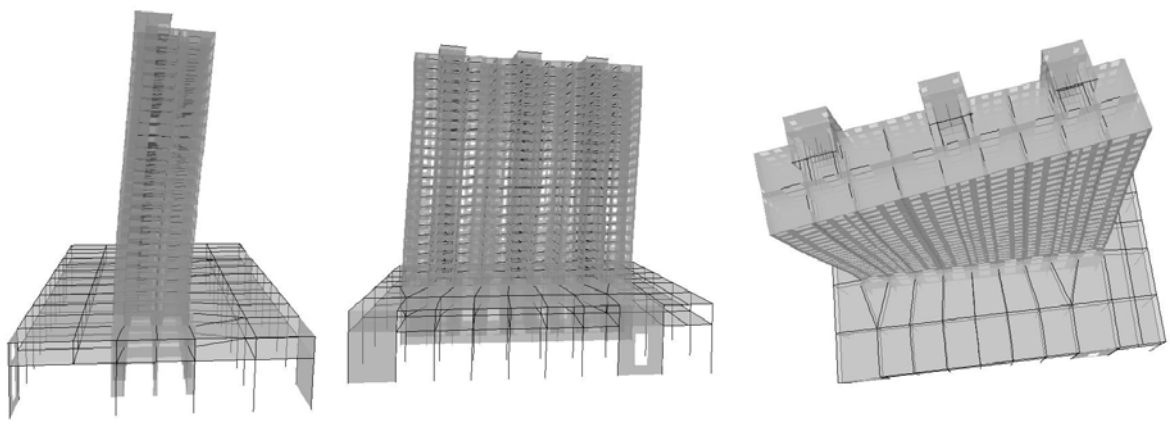
The frequencies of the model before test were extrapolated to that of the prototype structure by the similitude relation in order to comparing with the analytical results, as shown in Table 4.2. The first three vibration shapes of the prototype structure obtained from the numerical analysis are demonstrated in Figure 4.2. The results clearly show that the first vibration mode of the prototype structure is translations in the Y direction, and the second one is mainly translations in the X direction with somewhat torsion.

Table 4.1. Variation of natural frequencies of the test model

Mode	Initial	After inputs of minor level	After inputs of moderate level	Mode of vibration
1	6.877	6.189	4.126	Translation in Y
2	9.456	8.425	5.158	Translation in X
3	11.347	9.800	6.877	Torsion

Table 4.2. Comparison of the experimental and analytical results

Mode	Mode of vibration	Experimental results		Analytical results		Error (%)
		Frequency (Hz)	Period (s)	Frequency (Hz)	period (s)	
1	Translation in Y	0.973	1.028	0.936	1.068	3.9
2	Translation in X	1.337	0.748	1.533	0.652	12.8
3	Torsion	1.605	0.623	1.767	0.566	9.1



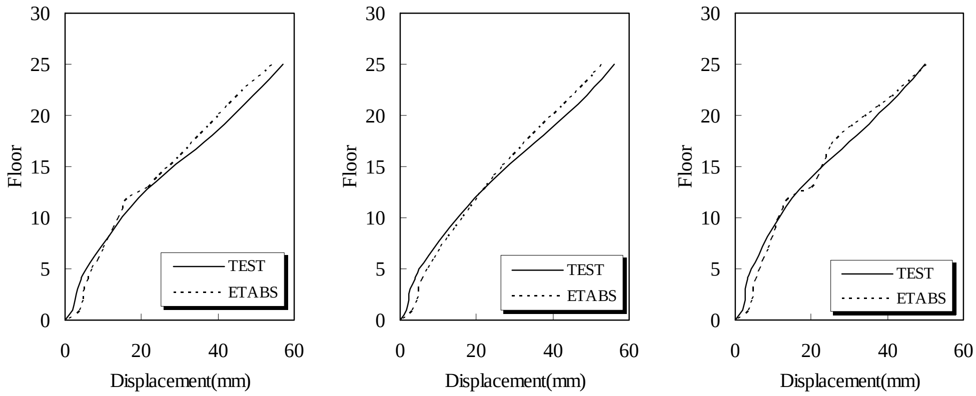
(a) First order (b) Second order (c) Third order

Figure 4.2. First three vibration modes

4.3. Seismic response

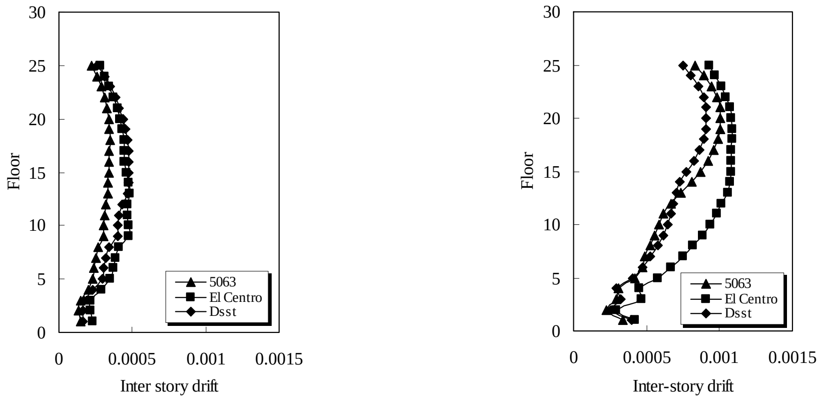
Displacement response of the model structure can be obtained by acceleration integration and differences between the peak values of the target and actually achieved input acceleration were considered in calculating the responses of the prototype structure. Envelops of the story displacement response under minor earthquake level are shown in Figure 4.3. In general, the maximum story displacement curves predicted by numerical analysis agree well with those of the experimental results.

Figure 4.4 shows envelops of the inter-story drift distribution of the prototype structure in both X and Y direction, the maximum values of inter-story drift obtained from both the experimental and analytical results are also summarized in Table 4.3. It can be seen that maximum value of inter-story drift of the upper structure which happened under El Centro excitation slightly exceeds the limit value of 1/1000, as stipulated in Chinese Technical specification for concrete structures of tall building (JGJ3-2002), while the maximum drifts of the podium structure are relatively small compared to those of the upper structure and can achieve the code requirement. Thus, lateral stiffness of the upper shear wall structure should be improved properly in further structural design.



(a) 5063 (b) El Centro (c) Dsst

Figure 4.3. Envelopes of story displacement response in Y direction



(a) Direction X (b) Direction Y

Figure 4.4. Analytical results of inter-story drift

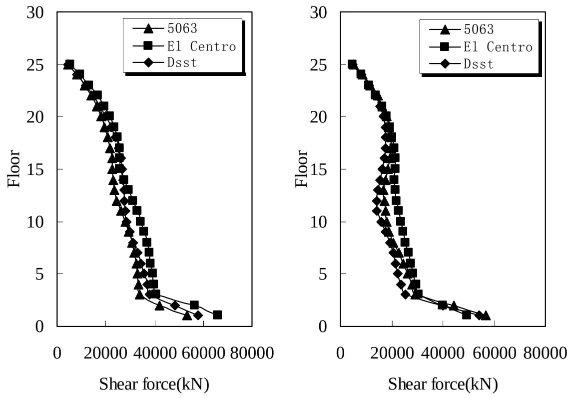
Table 4.3. Maximum inter-story drift

Type	PGA(g)	Experimental results			Analytical results		
		5063	El Centro	Dsst	5063	El Centro	Dsst
Upper structure	0.175	1/993	1/921	1/1099	1/957	1/939	1/974
Podium structure	0.175	1/3223	1/1988	1/2136	1/2985	1/2398	1/2525

The story shear was obtained by accumulating the inertia forces of each floor above, which can be calculated by means of multiplying the mass of each floor by measured accelerations. The distributions of story shear force along the height in both X and Y directions under three selected excitations are shown in Figure 4.5 and 4.6. Both the experimental and analytical results indicate that shear force between the podium and the upper shear wall structure changes sharply, as a result of the sudden change of mass between each other.

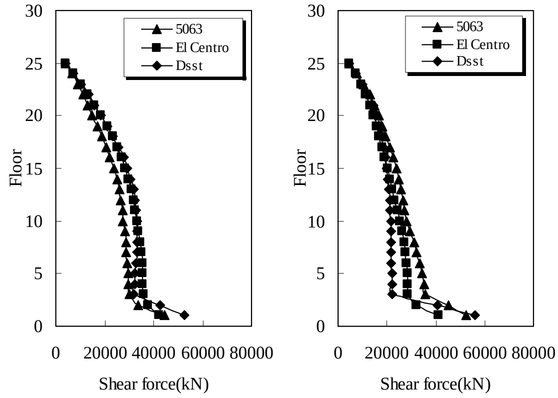
5. CONCLUSIONS

Both Experimental and numerical investigation on seismic behavior of a shear wall high-rise building with huge podium are presented. The following conclusions and suggestions can be drawn from experimental and analytical results:



(a) Direction X (b) Direction Y

Figure 4.5. Analytical results of story shear distribution



(a) Direction X (b) Direction Y

Figure 4.6. Test results of story shear distribution

(1) Severe shear failure of short-length shear walls occurred at the upper floors. No visible failure of members at the podium was observed throughout the shaking table model tests, which demonstrates the reliable seismic performance of the podium structure.

(2) The maximum inter-story drift of the upper shear wall structure slightly exceeds the limit value of code requirement under minor earthquake level, indicating that necessary enhancement of lateral stiffness in the Y direction should be devoted in further structural design.

(3) The shear walls at the upper floors should be designed with sufficient strength and ductility, thus, shear failure of the shear walls characterized by its brittleness may be avoided. In addition, more attention should be paid to the shear wall layouts of the upper structure, especially the short-leg walls.

(4) Finally, it should be mentioned that in order to evaluate the seismic performance of the structure under strong earthquake levels, dynamic inelastic analysis using computer program PERFORM-3D is currently in progress.

ACKNOWLEDGEMENT

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