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SHAING TABLE EXPERIMENTAL STUDY TO RC SHORT LIMB SHEAR WALL STRUCTURE

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ABSTRACT :

Following the prototype of a 12-storey residential building, a 1/10 scaled model of RC short limb shear wall structure was designed and constructed. And the seismic resistant behavior of the model was experimentally evaluated on the shaking table with three different stages. According to the site condition and seismic background, two natural and an artificial earthquake waves with varied peak values were employed as the input excitations. The natural frequency and dynamic responses including acceleration, displacement, shear force and torsion displacement were obtained through the measured data. The experimental results indicate that the building has good seismic performance, and is suitable for being applied in regions with the seismic fortification intensity of 7. Based on the experiment, some suggestions are given for the design of the prototype building.

KEYWORDS: Short limb shear wall, High rise building, Shaking table test, Seismic performance, Dynamic analysis

1. INTRODUCTION

As a result of the rapidly incremental demand of residence and the under-supply of land for building, high-rise building has become the first choice to developers. There are two traditional kinds of high-rise residential building, which are frame-tube system and wall-tube system. These two types of systems both have a lot of merits and demerits.

Because of the irregular compartmentation of rooms, columns are very difficult to be arranged in the frame-tube residential structures. Moreover, no matter how the plane arrangement is, the sections of columns are commonly very large in high-rise buildings, which results that the breadth of columns is larger than the thickness of wall, and the protrusion of columns at the corner of room will effect the arrangement of furniture. Furthermore, the lateral load-assistant performance of frame structure is not very good, especially when the placement of columns is irregular.

In the wall-tube structures, walls can be used as partition walls as well as bearing members, which can match with the plane arrangement of structure, and there is no protrusion. However, it is difficult to arrange shear walls rationally because of the irregular partition of rooms. Furthermore, the requirements of plane layout, such as symmetry, uniformity and irregularity, can not be easily satisfied too. On the other hand, since shear walls are very heavy, the amount of building materials is huge which will result in the increment of cost.

For the sake of comfort and individuality, the modern residential buildings have more requirements, for example, it requires large bay, the plane should be convenient for the partition of rooms, the column should not be protuberant as well as the beam under floorslab. However, the two traditional systems above can not satisfy these requirements well. With the continually engineering practice and amelioration, basing on the shear wall, and adopting the merits of frame, engineers proposed a new system of high rise building which was named as the short-limb shear wall (SLSW) structure [1].

The definition of short-limb shear wall is that, the ratio of the wall limb's breadth to its thickness is intervenient of 5 to 8, [2] which is larger than irregular column (the ratio is 3~5) and smaller than normal shear wall (the ratio is larger than 8). Because of the similarity and difference to column and normal shear wall, SLSW processes the merits but eliminates the demerits of them [3].

1) Compared to the normal shear wall structure, the spaces between short-limb shear walls can be filled with lightweight brickworks. The weight of structure and the use of building materials can both be reduced, and the constructing of building can be more convenience, all of these will lead to the reduction of cost.
2) According to the lateral load level, the numbers of SLSW and the lengths of limbs can be adjusted to meet the requirements of design code, and through the modification, the stiffness and the location of stiffness centre can also be adjusted to optimize the performance of structure.
3) The arrangement of plane is very flexible and the architectural restriction to structure is minimum, which is very propitious to the choice of the best design proposal.

Because of its impressive merits, SLSW structures have been adopted widely in the construction of residential buildings in many cities of China, such as Nanjing, Guangzhou and Lanzhou [4]. However, there are no detailed specifications about the computational model, the height scope of application and the constructional measures to SLSW structures in the design codes of China. In the Technical Specification for Concrete Structures of Tall Building (JGJ 3-2002) [5] and Code for Design of Concrete Structures (GB 50010-2002) [6] of China, there are only some more restricted limitations to SLSW than to normal shear walls, such as the arrangement of SLSW, the seismic fortification intensity and the ratio of axial compressive force to axial compressive ultimate capacity of section. Except these limitations, nearly all the design codes of SLSW are the same to that of normal shear wall structures. However, the rationality of these specifications to SLSW structure is suspectable. Therefore, systemic studying to SLSW structures (including the mechanical properties, seismic performance, failure mode, and analytical methods) and establishing the design theory of SLSW structure are an emergency to the Chinese engineering community. According to these backgrounds, a shaking table experiment on a RC short-limb shear wall structure was carried out, the results of which may help to the promotion of this kind of structure and the improvement of code design procedures.

2. DESIGN OF THE TESTED MODEL AND EXPERIMANTAL SETUP

2.1. Design of the tested model

The prototype structure of the tested model is a high-rise residential building in Fuzhou, China. The building has a one-storey basement and 12-storey up-structure. The elevation of the all storeys is 2.9 m except the first floor, whose height is 3.2 m, and the total height of the building is 35.1 m. The base plane of the prototype structure is “凸” shaped, which is composed by two different rectangular parts as shown in Fig.1. The length and width of the larger part are 30.2 m and 15.7 m, and the plane of the smaller part is 18.2 m by 12.8 m, the aspect ratio of height to width is 1.25. The area of structure is 9705 m^2 and the seismic fortification intensity of site where the building will be constructed is 7.

The building has no columns, and the only vertical load bearing members are short-limb shear walls. The thicknesses of some walls in lower storeys (1~5 storeys) vary from 250 mm to 350 mm, and other walls in such storeys as well as all walls in 6~12 storeys are 200 mm. The maximum ratio of breadth to thickness of short-limb shear wall is 8, while the minimum is 3.5

The leading aim of this experiment is to study the seismic performance of the structure under earthquake, so more attention is played on the similitude ratio of the lateral load-resistant members, such as shear walls, tubes and main beams, and the dimension, reinforcement ratio and the strength of concrete of these members are determined strictly according to the similitude ratio [7, 8]. To simply the construction of the tested model, all balcony and some nonsignificant beams of the prototype building are eliminated as shown in Fig.2, which has little effect to the performance of the model. The effect of load in prototype, including the dead load and the live load, is taken into account in the tested model, say, use the additional weight (i.e. plumbum and/or iron blocks) to simulate such loads [9, 10], therefore, the model is the fully artificial mass model, which can furthest meet the similarity of geometry and physics to the prototype structure.

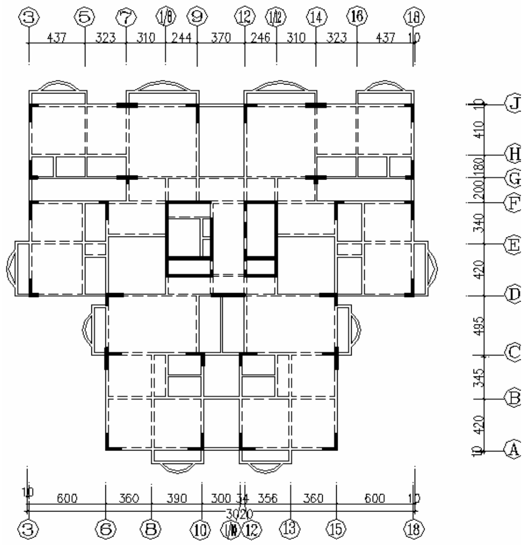


Fig. 1. The plane view of prototype (cm)

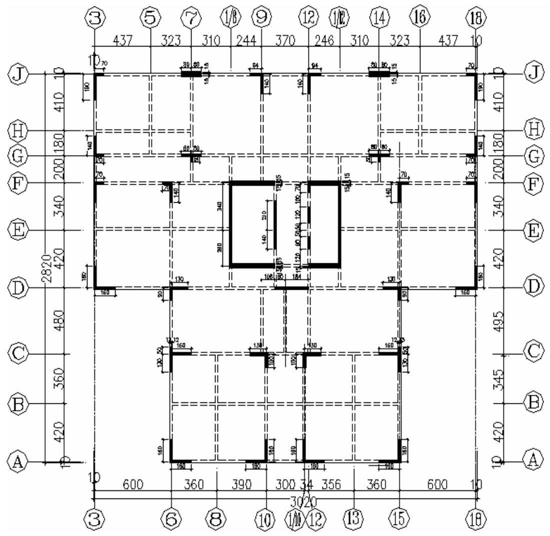


Fig. 2. The plane view of model (mm)

As is known that in shaking table tests, the similitude laws should be satisfied in principle. On the other word, the similarity ratio of geometry, acceleration, elasticity modulus and density, which is denoted by S_l, S_a, S_E, S_ρ respectively, should meet the following expression [11,12]:

$$\frac{S_E}{S_\rho \cdot S_a \cdot S_l} = 1 \tag{2.1}$$

The shaking table is 4.0 m×4.0 m with a capacity of 25t. Considering geometric size of the prototype and the shaking table as well as its bearing capacity, the geometry similitude ratio of model to prototype is adopted as 1/10, and the dominating ratios of similitude are shown in Table 1.

2.2 Design of rigid ground beam

A rigid ground beam, as shown in Fig.4, was designed to simulate the effect of fixation of foundation, through which the tested model was fixed on the shaking table with 26 provided anchors shown in Fig.3. The ground beam not only can make sure there is no relative displacement between the model and shaking table, but also can be applied for hoisting the tested model from the casting site to the shaking table also needs the ground beam. Therefore, the ground beam should provide enough bending and shearing resistance as well as the integral rigidity, and it should also be very convenient for anchoring [13].

Table 1 Dominating ratios of similitude

Physical parameter	Length S_l	Modulus S_E	Stress S_σ	Density S_ρ	Time S_t	Frequency S_f	Acceleration S_a
Ratio	0.1	0.504	0.504	2.016	0.2	5	2.5

Fig. 3. The panoramic view of the tested model

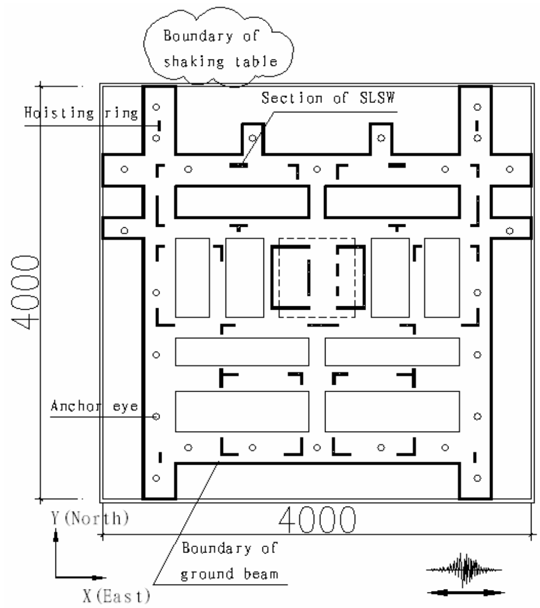


Fig. 4. The rigid ground beam

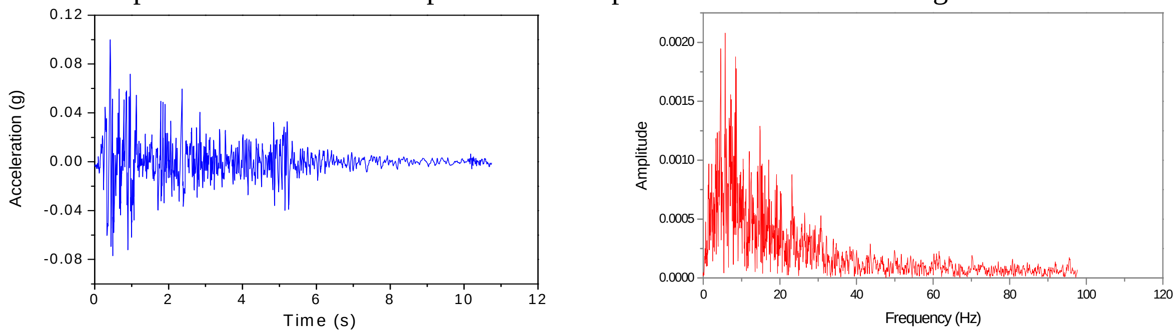
During the construction of the model, the form boards at periphery are made of wood, which can be lift together with the process of construction. While, the inner form boards are made of foam, some of which will not be removed for the sake of convenience, and the residual forms have little effect to the performance of tested model. The construction procedure and maintenance process are strictly according to that of prototype. The finished model is shown in Fig. 3, whose total height is 3.81 m including the ground beam which is 0.5 m high, and the overall weight of model is 22 t, in which, the weight of model itself is 5 t, of additional weight is 12 t, of ground beam is 5 t.

2.3 Instrumentation and experimental setup

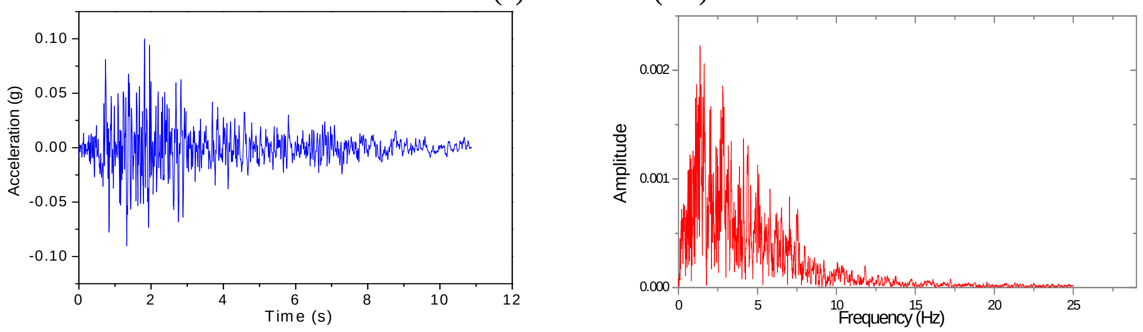
The testing data were acquired by an automatic data acquisition system at 50 Hz in 69 channels. An accelerometer was fixed firmly on the shaking table to measure the table's motion, which is the actual input of the excitation to the base of SLSW structure. 36 accelerometers were installed to measure the accelerations of different floors in the X direction and Y direction. 10 displacement transducers were fixed at one identical wall of different storeys to measure the storey displacement as well as the inter-storey drift, while another 6 displacement transducers were applied to measure the torsion displacement. 16 strain gauges were attached to four critical positions in the first and second storeys to obtain the time histories of strain.

2.4 Experimental program

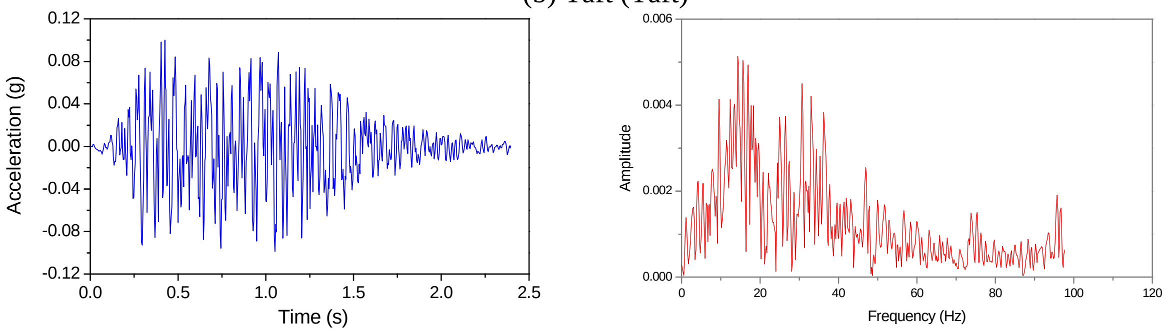
The site classification of prototype building is III with the design characteristic period of ground motion being 0.45 s. The actual recorded earthquake wave at the prototype engineering site is the best choice for the input excitation. However, there is no such recorder in Fuzhou. Therefore, two ground accelerations with similar properties of engineering site are used as the input shaking table accelerations. One is the scaled EI Centro recorded acceleration (N-S component, 1942, briefly donated as EL) [14], which is widely used as the typical input of site classification III. Another one is the scaled Taft recorded acceleration (1952, donated as Taft) [14]. According to Chinese Code for seismic design of buildings (GB 50011-2001) [15], in the time-history analysis, it needs at least two natural and one artificial earthquake waves. Thus, in this experiment, an artificial ground motion used in the seismic design of structures in Guangzhou, China (briefly donated as GZ) is adopted [16]. Since the properties of site's soil in Guangzhou and Fuzhou are similar, it is reasonable to adopt this artificial earthquake wave. As mentioned before, the duration times of the three earthquakes are shortened by the factor of 5 according to the similitude law. Typical time histories and amplitudes of the Fourier spectra of the input waves are shown in Fig. 5.



(a) El Centro (EL)



(b) Taft (Taft)



(c) Guangzhou (GZ)

Fig. 5. Input acceleration time histories and the amplitudes of Fourier spectra of them

Since the seismic fortification intensity of the prototype structure is 7, the experimental process, designed, includes three stages, which are frequent-earthquake stage of seismic fortification intensity 7 (briefly donated as FE7), seldom-earthquake stage of seismic fortification intensity 7 (briefly donated as SE7) and seldom-earthquake stage of seismic fortification intensity 8 (briefly donated as SE8). In each stage, the acceleration time histories of El Centro (EL), Taft (Taft) and Guangzhou (GZ) are input sequentially in the direction of east-west. In addition, before and after each stage, white noise wave scanning is inserted to measure the natural frequency/period of the tested model, and all the 13 experimental cases are shown in Table 2.

Table 2 Program of the experiment

FE7			SE7			SE8		
Case	Earthquake wave (X direction)	Amplitude (g)	Case	Earthquake wave (X direction)	Amplitude (g)	Case	Earthquake wave (X direction)	Amplitude (g)
1	White noise	0.07	5	White noise	0.07	9	White noise	0.07
2	El-Centro	0.12	6	El-Centro	0.62	10	El-Centro	1.02
3	Taft	0.136	7	Taft	0.626	11	Taft	1.05
4	Guangzhou	0.16	8	Guangzhou	0.636	12	Guangzhou	1.05
						13	White noise	0.07

3. RESULTS OF THE SHAKING TABLE TEST

3.1 Experimental phenomena observation

In the first stage (FE7), since the peak ground acceleration (PGA) was relatively small, slight vibration was observed. Because the duration of Guangzhou wave was very short (2.4 s), the corresponding response of the tested model liked an impulse, while under the two natural earthquake waves, the vibrations were more obvious. After the carefully observation to the tested model when the first stage completed, no visual cracks could be found, and the natural frequency (period) decreased by 4.7% from 5.127 Hz at the beginning to 4.883 Hz in the end. It meant that there was only slight damage occurring during the first stage.

In the second stage (SE7), the PGA increased to 0.62 g, and notable vibrations could be observed. After the input of EL, there still had no visual cracks. While a few tiny cracks were found in some beams which located in the north and south elevations of 2th and 3th storeys after the input of Taft. Under GZ, these cracks were kept in developing, but none cross section of beam was penetrated by these cracks. Simultaneously, some other cracks appeared in the fourth storey. By the way, all cracking occurred at the end of beam and none of visual cracks were observed in SLSW. When such stage completed, the white noise wave scanning showed that, the natural frequency decreased by 40% reaching to 2.93 Hz, which indicated that obvious damage occurred in the tested model. Since the cracks only appeared at the end of beams, the integrality of model was kept well.

In the seldom-earthquake stage of seismic fortification intensity 8, the PGA of input waves was set as 1.0 g. Under the acceleration time history of EL, the vibrating response of model was very drastic, and debris was seen dropping slightly. After this case completed, many beams in 2-4 storeys were nearly penetrated by cracks, and more cracks appeared in up storeys, especially in the 5th and 6th storeys. During the input of Taft, the model vibrated with the sound of concrete cracking, and more debris dropped from the model. Cracks kept on appearing, and most of cracked beams in 2-4 storeys had been completely penetrated. Remarkably, there was a trend that the cracks expanded into the limbs of walls. Besides, the damage in north elevation was more severe than that in south elevation. Under the succeeding GZ wave, more cracks appeared in beams even including beams in 11th storey, and several tiny cracks appeared in the walls of west elevation, and some debris flaked off from these walls. During this stage, the natural frequency decreased by 20% compared to the second stage, while compared to the original model, the reduction of natural frequency was 52.4% and reached to 2.44 Hz. Consequently, it could be judged that serious damage occurred in the tested model.

Typical cracks in different cases are shown in Fig.6. According to the observation above, it was noticed that, even under the seldom earthquake waves with the fortification intensity of 8, no collapse occurred and the model even maintained its integrity, which meant that such short-limb shear wall structure had good seismic performance and met the seismic requirements of Chinese code.

Fig. 6. Typical cracks

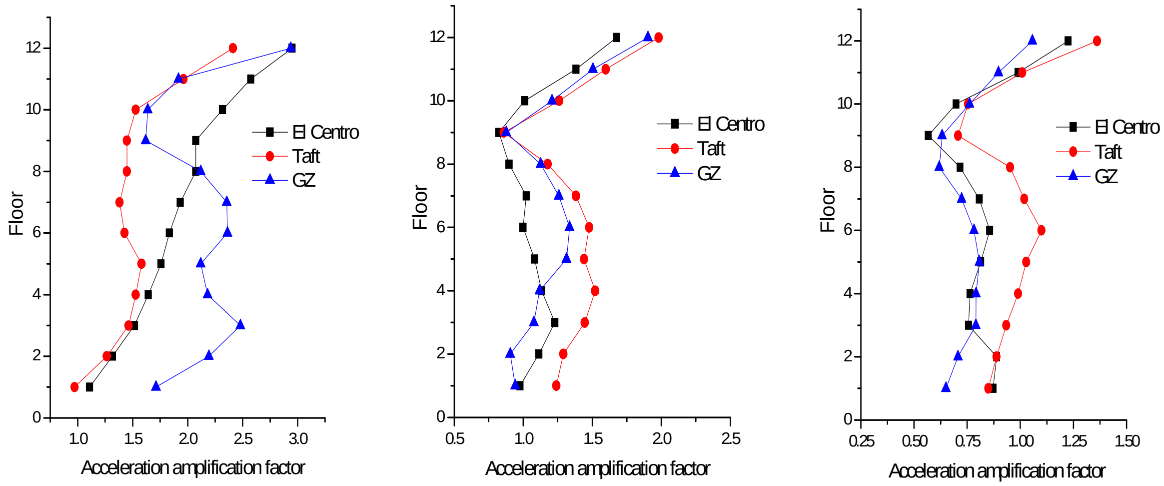
3.2 Dynamic response of acceleration

Different earthquake waves and different PGAs would result different responses. And the properties of the structural dynamic responses can be captured by a comprehensive index effectively, which is named as amplification factor of acceleration (AFA). The amplification factor of acceleration at storey i is defined as

$$\rho_i = \frac{\max\left(\left|\ddot{x}_i\left(t\right)\right|\right)}{\max\left(\left|\ddot{x}_g\left(t\right)\right|\right)} \tag{3.1}$$

$\ddot{x}_g\left(t\right)$ is the time history of base acceleration, namely the input acceleration time history, $\ddot{x}_i\left(t\right)$ is the time history of acceleration response at i th storey, and $\max\left(\left|\bullet\right|\right)$ takes the maximum absolute value of time histories. According to the measured data, the amplification factor of acceleration in different stages can be evaluated, as shown in Fig. 7.

It can be found that, no matter which earthquake wave is and no matter what PGA is, all curves of amplification factor have the same “S” shape with two obvious inflection points. The first point is near the 6th floor, and another one is at the 9th floor, and from that point, AFA increases drastically, namely, the phenomenon of tip effect is very obvious.



(a) Stage of FE7 (b) Stage of SE7 (c) Stage of SE8

Fig. 7. The amplification factor of acceleration in different stages

It is also noticed that, the amplification factor of acceleration decreases with the increment of PGA. For example, under the wave of EL, the AFA at the 12th storey is 2.95 in FE7 stage, and in SE7 stage is 1.667, while in stage of SE8 it is 1.225, which is only 41.5% of that in the first stage. However, the amplitude of acceleration response increases from 0.354 g to 1.25 g. In physical sense, the reason is that, with the increment of PGA, the response of structure is becoming more obvious, which leads to the development of damage, and the stiffness of structure decreases gradually.
Besides, the curves of amplification factor of natural waves are smoother than that of the artificial wave, whose curve has more reflection points, especially in stage FE7. The reason is that, from observing the Fourier spectra of the three waves, it can be noticed that, the energy of Guangzhou wave is more dispersive than that of natural waves, and under such wave, the effect of high order mode shapes to the dynamic response to structure is greater.

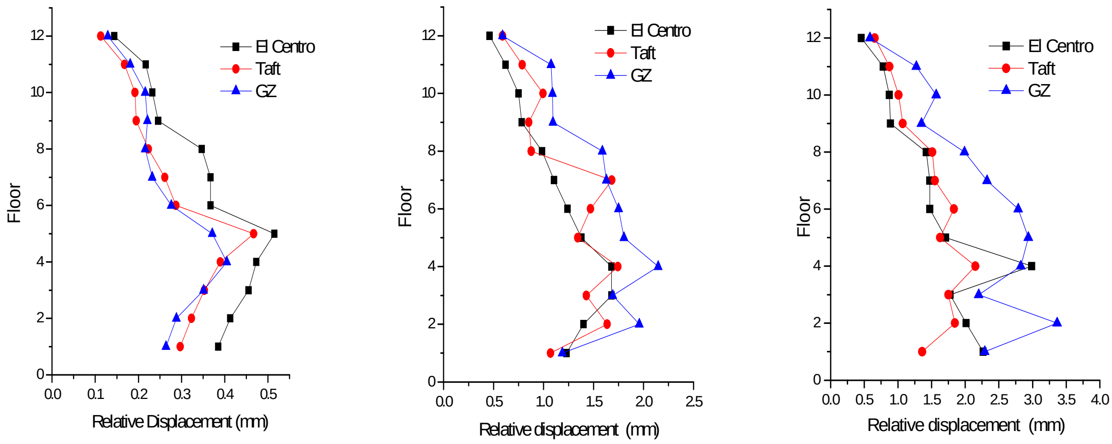
3.3 Dynamic responses of displacement

Except the displacement transducer, whose number is limited in the experiment, the data of displacement can also be obtained through the double integral to the data of acceleration. Of course, such calculational results should be calibrated by the measured data of displacement.
Comparing to the storey displacement, the inter-storey drift and/or storey drift angle are the more important indexes to structural performance, which are defined as:

$$s_i = \max\left\{\left|x_i(t) - x_{i-1}(t)\right|\right\} \tag{3.2}$$

$$\phi_i = \frac{\max\left\{\left|x_i(t) - x_{i-1}(t)\right|\right\}}{h_i} \tag{3.3}$$

where, s_i and ϕ_i are donated as inter-storey drift and storey drift angle respectively, $x_i(t)$ and $x_{i-1}(t)$ are time histories of displacement of i th and $i-1$th storeys.



(a) Stage of FE7 (b) Stage of SE7 (c) Stage of SE8

Fig. 8. The inter-storey drift in different stages

According to the measured data, the maximum storey drift angle (briefly donated as MSDA) is 1/772 in FE7 stage. Though such value is less than the limitation (1/550) to frame structure in Chinese Technical specification for concrete structures of tall building (JGJ3—2002), it is larger than the limitation (1/1000) to shear wall structure as well as SLSW structure. However, observation and measured results have indicated that the there is no obvious damage to the tested model in such stage, which means the limitation in Code is too strict to SLSW structure. In the stage of SE7, the MSDA is 1/156, which is less than the limitation (1/120) to shear wall structure.
In the stage of SE7 and SE8, the storey drift in down structure varies drastically, as shown in Fig.8, while the variation in up structure is little. This phenomenon indicates that the displacement in down structure is

more sensitive to the increment of PGA than that of up structure.

Totally speaking, the storey drift increases with the height rising, and near the 5th floor, it reaches the peak value and then decrease, which means there is an obvious concentration of displacement. The reason is that, as said before, the performance of SLSW structure is intervenient between that of normal shear wall structure and frame structure, and its displacement under horizontal load is composite of flexure and shear, which leads to the maximum storey drift appear in the middle height of structure, which is consistent to experimental result.

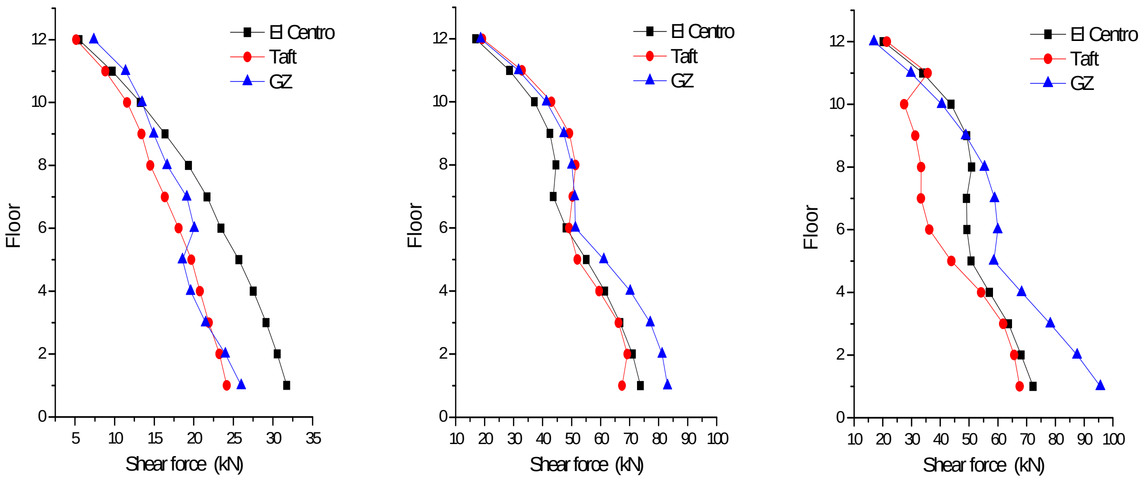
It also can be found that, in stage FE7, the curves of inter-storey drift are relatively smooth, but in stages of SE7 and SE8, these curves become more oscillating. That is because, in the original state, the tested model is continuous and the stiffness is nearly equally distributed along the height. However, with the processing of experimental cases, varied degrees of damage occur in each storey, and the stiffness of structure is no longer equally distributed, namely, the properties of the tested model has been completely changed and it has been in non-linear state.

3.4 Dynamic response of shear force

It is known that, the amount of shear force that each storey bearing is directly relate to the stiffness of storey, therefore, the variation of shear force can reflect the variation of stiffness. The maximum shear force that each storey bears can be defined as:

$$F_k = \left| \sum_{j=k}^{n} m_j \ddot{x}_j(t) \right|_{\max} , \quad k = 1, 2, \cdots n \tag{3.4}$$

where m_j is the weight of j th storey. The diagrams of shear force distribution along the height of structure in different stages are shown in Fig. 9.



(a) Stage of FE7 (b) Stage of SE7 (c) Stage of SE8

Fig. 9. The distribution of shear force in different stages

It is found in Fig. 9(a) that, the shear force that each storey bears increases gradually and nearly equally from the top to bottom of the structure, which means that, in the stage of FE7, the stiffness of the structure is very uniform along the height. The slight disturbance appears in the curve under GZ wave is caused by the effect of high order mode shape as said above.

In the stage of SE7, the shear force distributions under different waves are very similar compared to that of stage FE7. The phenomenon indicates that, in such stage, the distribution of shear force is less sensitive to the kind of wave than PGA. Furthermore, there is one obvious concave near the 6th storey. The reason is that the damage in this part is much larger than other storeys, and this difference affects the uniformity of the stiffness of the structure. However, in the stage of SE8, the curves become even more oscillating, which indicates that the uniformity of stiffness keeps on being destroyed and more other storeys, not only the sixth storey, have been in varied degrees damaged. All of these phenomena are consistent with the results of acceleration and displacement.

3.5 Dynamic response of torsion

Since the plane view of prototype building is "凸" shaped as shown in Fig. 1, the tested model is asymmetric in the east-west axis (or X axis). There are more walls in the north part of the structure, which results in the eccentricity of stiffness and mass of the model and the torsion response under earthquake wave will be very obvious. The torsion response of model can be expressed by torsion angle, which is defined as:

$$\alpha = \max\left\{\frac{\left|x_k\left(t\right)-x_l\left(t\right)\right|}{L}\right\} \tag{3.6}$$

where $x_k\left(t\right)$ and $x_l\left(t\right)$ are the displacement time histories at two different points k and l in identical storey with the vertical distance of L. In this experiment, the torsion displacement of 12th, 8th and 4th storeys are measured, and the most of north and south angular points are selected as the points of k and l, whose L is 2080 mm. Table 3 shows the maximum relative displacements and torsion angles of points k and l in different stages as well as the relative ratios to maximum storey displacement and storey drift in corresponding stages respectively. It is noticed that, relative ratios of torsion displacement to inter-storey displacement in different stages are larger than 25%, and the maximum value nearly reach 35%, which indicates that the torsion is so great that can not be ignored.

Table 3 Comparison between torsion displacement and storey displacement

Experimental stage	Maximum relative displacement (mm)	Ratio to storey displacement	Maximum torsion angle	Ratio to storey drift angle
FE7	1.35	34.4%	1/2089	27%
SE7	4.743	31%	1/595	25.5%
SE8	5.504	33%	1/512	25.3%

On the other hand, even though the excitation of shaking table is unidirectional, the acceleration time histories in perpendicular direction (Y direction) still can be obtained, and Table 4 shows the amplitudes of acceleration time histories in Y direction as well as the relative ratio to that in X direction of corresponding cases. It is found that, the maximum amplitude of acceleration in Y direction is larger than 0.3 g, and the horizontal inertial force caused by such acceleration can deeply affect the seismic performance of the structure.

Table 4 Amplitudes of acceleration response in Y direction and comparison to X direction

Experimental stage	EL		Taft		GZ	
	Amplitude (g)	Ratio to X direction	Amplitude (g)	Ratio to X direction	Amplitude (g)	Ratio to X direction
FE7	0.088	20%	0.0752	23.5%	0.0909	19.9%
SE7	0.194	18.7%	0.193	16.2%	0.277	24.1%
SE8	0.33	26.4%	0.305	22.9%	0.311	29.6%

Both the results of displacement and acceleration indicate that, though the input excitation is unidirectional, the torsion response is very obvious and severe, which is very adverse to the performance of structure, and can aggravate the damage of the structure, especially to some critical parts, such as the edges between north and south rectangular parts, in which, the cracks are much more than other place.

3. CONCLUDING REMARKS

Based on the shaking table experiment, the seismic performance of the short-limb shear wall structure is studied and analyzed, and some conclusions and suggestion can be obtained:

1) The experiment is designed strictly according to the similitude law, and the results indicate that the performance of the structure satisfies the seismic requirements in Chinese codes, that is "no damage under the frequent earthquake and no collapse under the seldom earthquake".
2) According to Chinese Technical specification for concrete structures of tall building, the ratio of longitudinal reinforcement in short-limb shear wall is no less than 1.2% in the strength storeys (i.e. the first and the second storeys), and no less than 1.0% in other storeys. However, there is no visible failure of SLSW even in the stage of SE8, which indicates that the specifications about ratio of reinforcement above is too conservative, and the amount of reinforcement in the prototype building can be reduced correspondingly.
3) The eccentricity of stiffness and mass of the structure caused by the irregular structural plane view results in severe torsion response. Besides, since there is an obvious shape-variation at the edge of the two rectangular parts, remarkable stress concentration occurs. It advises that, the layout of prototype building should be modified to be more regular. Alternatively, if it is very difficult to modify the plane view, it should increase the amount of SLSW or lengthen the breadths of limbs of SLSW in the weakly part (i.e. the south part), which can balance the stiffness of these two parts.
4) In the prototype building as well as the tested model, there are two "一" shaped short-limb shear walls. Since such "一" shaped SLSWs are arranged in the most north elevation where the dynamic response is greatest and the torsion response of the structure is very obvious, stress and strain in these two walls are much larger than other walls nearby. It advises that, the "一" shaped SLSW should be modified to be "T" shaped, in which the flanges can provide the out-plane stiffness effectively and the torsion response in such walls can be reduced.
5) As mentioned above, in Chinese Technical specification for concrete structures of tall building, the limitations of inter-storey drift to normal shear wall structure is too strict to short-limb shear wall structure. In other words, the limitation should be loosened to SLSW, and the design of SLSW structures should not completely according to the code of normal shear wall.

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