SHAKING TABLE TEST AND NUMERICAL ANALYSIS ON STRUCTURAL MODEL OF BEIJING NEW POLY PLAZA

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ABSTRACT :

In this paper, shaking table test of a 1/20 scaled structural model for Beijing New Poly Plaza, an irregular high-rise steel frame - steel reinforced concrete tube hybrid building, was carried out. The dynamic characteristics and response of the structural model, such as accelerations, displacements, under different base excitations have been measured and analyzed. The response of the structure's special parts is discussed. In addition, a 3-D finite element model was built using the structural analysis software SAP2000. The elastic responses of the model predicted by finite element analysis agree well with the experimental results, even though some local responses can not be captured due to simplifications in the model setup. Both experimental and numerical study show that the structural system meets the requirements of Chinese Seismic Design Code (GB50011-2001). The seismic performance of the whole structure has been evaluated.

KEYWORDS: Shaking table test, Finite element, Seismic behavior, Hybrid structure, High-rise building

1. INTRODUCTION

Steel frame-concrete shear wall hybrid structure system has been widely adopted in building structures in China because it has the advantages of steel structure and concrete structure. Some of them are irregular buildings. For cities located in seismic design zones, it is a key issue for seismic design of those irregular buildings. The National Seismic Design Code (GB50011-2001, 2001) prescribes that elastic-plastic analysis is required for the irregular buildings with evident weak parts, either static elastic-plastic analysis or dynamic elastic-plastic time history analysis should be performed depending on the structural characteristics. However, there are no specified provisions for hybrid structures. Some studies have been carried out during the last several decades. Gong et al (1995) tested a 1/20 scaled 23-storey regular and symmetric structural model and found that hybrid building has a good seismic performance and could be used in regions rated as seismic intensity VIII. Gong et al (2004) carried out shaking table test of a 1/35 scaled structural model for the Shanghai International World Trading Plaza and found the torsion effect is existed due to its irregularity in vertical direction and the eccentricity between the center of mass and the center of stiffness. Jiang et al (2005) tested a 1/25 scaled model of a high-rise building with serious irregularity on shaking table and concluded that very extreme irregular structure was not recommended in high intensity seismic zones.

The objective of this paper is to assess the seismic performance of Beijing New Poly Plaza, an irregular hybrid building in Beijing, through a 1/20 scaled shaking table test and finite element analysis under different levels of earthquake excitations. Detailed results can be found in another paper (Zhao et al, 2007).

2. SHAKING TABLE TEST

2.1. Prototype Structure

Beijing New Poly Plaza is a steel frame-steel reinforced concrete core hybrid structure, with 4 stories underground and 24 stories above the ground. The total height of the building is 105.2 m. The building plane is a triangle and there is a 12-storey, localized 22-storey high atrium, the plan view of the 1st level is shown in Figure 1. The lateral resisting system of the structure is composed of three steel reinforced concrete cores, located at the corner of northwest (Core1), southwest (Core2) and southeast (Core3) respectively, and two steel frames, one connecting Core1 with Core2 and starting from the ground, the other connecting Core2 and Core3 and starting from the 12th floor. On the 22nd floor, a 3-storey, 16-m high steel truss connects Core1 with Core3 together.

A 7-storey, 50 m high steel "special lantern" sticks out from northwest face of Core3 and is hung by four steel cables, two of which are anchored at 22nd floor of Core3, the other two are anchored at 24th floor of Core2 and 22nd floor of Core1, respectively. To connect the steel frame with three cores, steel columns were embedded at the corners of the three cores and connected together by the embedded steel beams. The composite floor system is used for the steel frames while the floors of three cores are made from lightweight aggregate concrete to reduce the self weight of the structure. According to the China national seismic design codes (GB50011-2001, JGJ3-2002), the New Poly Plaza is an irregular structure in both horizontal and vertical directions.

2.2. Model Design and Casting

The test is carried out on a 3-D, 6-DOF shaking table. The dimension of the table is 5m×5m. The maximum payload is 20 tons and the maximum overturning moment is 350KN•m. The maximum acceleration is 1.0g in horizontal direction, and 0.7g in vertical direction. Considering the technical parameters of the shaking table, the reliability of the scaled structural model and the model constructing technique, the similarity ratio of the length S_l is determined as 1/20, the similarity ratio of the Young's modulus S_E is 0.65 and the equivalent mass density ratio S_p is 1.55. According to the consistent similitude law with insufficient additional weight (Zhang, 1997), the scale factors of the main parameters are determined and summarized in Table 1.

The structure model has 24 stories. Its height is 5.46 m, including a 0.2m high soleplate. Its plane dimensions are 3.803m×3.825m. The scaled model was fabricated using fine-aggregate model concrete and the steel frame elements were made of steel tube based on the similitude law. The slab of the model is composite of cast-in-place model concrete and 1 mm thick steel plate. To enhance the shear connecting capacity of the steel beam and concrete slab, rivets were welded onto top of the steel beam at a proper spacing. In addition, the special lantern and the swinging device between the lantern and the cables were fabricated as design requirement to simulate their real working condition. The scaled model before testing is shown in Figure 2.

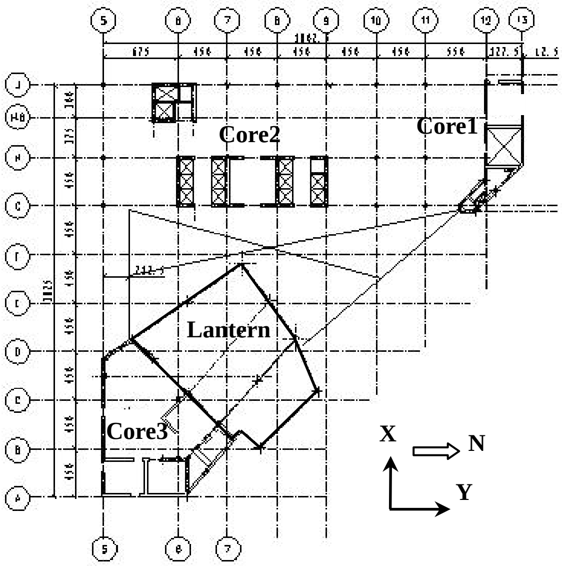


Figure 1 Plan view of the first level


Figure 2 Model on the shaking table

Table 1 Scale factor of the main parameters

Parameter	Scale factor	Parameter	Scale factor	Parameter	Scale factor
Length S_l	1/20	Displacement SB_u	1/20	Time S_t	1/12.95
Strain S_ε	1.00	Mass density S_p	1.55	Frequency S_f	12.95
Y's modulus S_E	0.65	Stress S_σ	0.65	Acceleration S_a	8.39

2.3. Measurements and Testing Procedures

Accelerations and vertical strains at different levels of the model were measured. A total of 44 accelerometers and 55 strain gauges were installed to measure the seismic response of the model. Three stages I, II, and III were carried out in this test, corresponding to the minor, moderate, and major earthquake levels based on the seismic intensity VIII. Earthquake records used in this test were listed in Table 2.

The earthquake excitation for each earthquake record was first input in each direction only. Then the excitations were applied from three directions simultaneously, with the ratio of the peak accelerations in 3 directions as 1.00:0.85:0.65, to investigate the model behavior under 3-D excitations. Each excitation case is denoted as A8BC, in which "A" represents the earthquake record name in short form (as listed in Table 2), 8 refer to the seismic intensity VIII, "B" represents the level of excitations ("S"- minor, "M"- moderate, "L" - major), and C represents the excitation direction ("X", "Y", "Z", represents X, Y, Z direction respectively). Before and after each stage, the dynamic characteristics of the model were tested and recorded by applying white noise excitation. Structural damages were inspected after each testing stage.

Table 2 Earthquake records used

Earthquake Record	Short Form	Duration(s)	Test Stage	Earthquake Record	Short Form	Duration(s)	Test Stage
Artificial S1	s	30.0	Stages I, II	Artificial S2	S	30.0	Stage III
Olympia	O	89.2		MT.Diablo	M	31.6	
Court h	C	13.2		Court h	C	13.2	
Los Angeles	L	63.6					

2.4. Experimental Results

2.4.1. Fundamental periods

The measured frequencies of the first six modes of the model are summarized in Table 3. The corresponding periods for prototype structure estimated through similitude law were also included. It can be found that the nature periods of the model increase from stage I to stage III, which means damage of the model occurred and became more serious when the larger excitations were applied to the model.

Table 3 Measured model periods and estimated prototype periods （s）

Case		Initial		After stage I		After stage II	After stage III
		model	prototype	model	prototype	Model	Model
X-direction	1st mode	0.114	1.476	0.116	1.502	0.126	0.136
	2nd mode	0.028	0.363	0.029	0.375	0.033	0.036
Y-direction	1st mode	0.157	2.033	0.161	2.085	0.186	0.213
	2nd mode	0.039	0.505	0.040	0.518	0.048	0.052
Torsion	1st mode	0.076	0.984	0.082	1.062	0.091	0.114
Z-direction	1st mode	0.022	0.285	0.023	0.298	0.024	0.025
Lantern-local	1st mode	0.018	0.233	0.018	0.233	0.019	0.021

2.4.2. Observations

Minor horizontal cracking between concrete wall and composite slab was observed after the stage I testing. The cracking was observed in the bottom stories and the crack distribution on three cores is different: up to level 8 for Core1, up to level 6 for Core2, and only to level 4 for Core3. The reason for this phenomenon might be that the insufficient gravity load leads to small compression strength in the concrete cores and the horizontal construction joints are located. The natural frequencies before and after the stage I almost did not change, indicating that the structural model remained elastic.

After stage II, the horizontal cracking was increased both in extent and quantity. In addition, bending cracking at the ends of the coupling beams was observed (Figure 3). The natural frequencies of the model reduced, which means the model started to behave plastically. After stage III, the horizontal cracking distributed at both the top and bottom of the concrete cores, the bending cracking on the coupling beams was propagate through the entire height of the beams. The tensile cracking occurred at the connection between big truss and concrete core1 (Figure 4). The connections between steel beams and concrete cores also cracked on the top floor. No visible damages between the connection of special lantern and Core3 were observed. More frequency reductions were observed after stage III.

Figure 3 Cracks in coupling beam

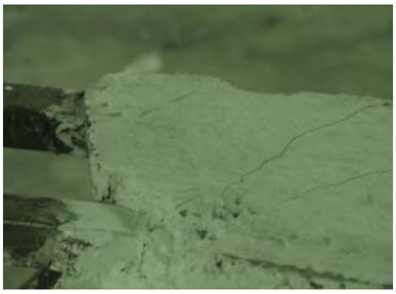


Figure 4 Cracks between the steel truss and Core1

2.4.3. Acceleration and displacement response

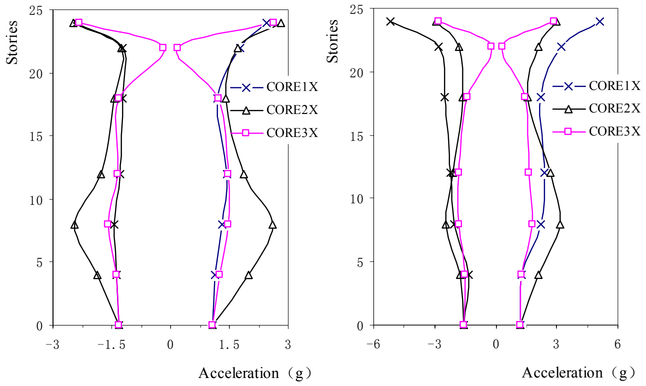
During testing, acceleration responses of the model were recorded by the accelerometers and the corresponding displacement responses can be obtained by integrating the accelerations twice. It was found that the structural responses under the artificial earthquake record excitation were the strongest among all excitations. Figure 5 through 6 show the envelopes of the acceleration responses under the artificial earthquake record excitation both in unidirection and 3-direction in stage III. By comparing the test results, the following conclusions can be obtained: 1) The acceleration response under X-direction excitation is smaller than that under Y-direction excitation, the structural is relatively weaker in X direction. 2) The acceleration response of Core1 is lager than other two, especially for the acceleration above level 22. Core1 is relatively weak among the three cores. 3) The acceleration responses under 3-direction excitation are larger than those under unidirection excitation.

Similarly, the displacement responses under the artificial excitation are larger than those under other inputs. To investigate the actual deflection of the model, the displacement patterns of the each core along its height at the time when the top level of Core1 reached its peak value under excitation "s" during stage II were plotted in Figure 7 and Figure 8. It is obvious that the displacements of the three cores are not consistent, the response of Core1 is greater than the other two, especially in Y-direction, which indicates that Core1 is relatively weak. Torsion effect was observed because the displacement in Y and X directions were coupled even the excitation is inputted in X (or Y) direction only. The displacement responses under 3-direction excitation are larger than those under unidirectional excitation. Whipping effect was observed in stories above level 22.

2.4.4. Response of the special lantern and big truss

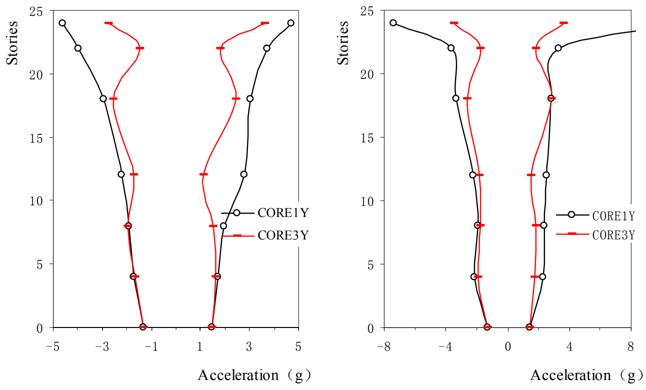
As mentioned in section 2.3, accelerometers and strain gauges were installed in special lantern, hanging cables and the truss of the model to investigate their structural responses. A total of 7 accelerometers and 4 strain gauges were installed in the lantern, one strain gauge was attached to each hanging cable and one accelerometer was located at the mid-span of the big truss. From the test results, there is no significant difference in the peak accelerations among the different locations of the lantern. The maximum dynamic strain measured from the lantern structure is less than 600 με and the equivalent maximum dynamic strain in the hanging cable is less than 450με according to the geometric similitude ratio. Obviously, the special lantern and hanging cables has a very good overall capacity.

The 80mm high (16m in prototype) big truss is another critical part of the structure, the maximum vertical acceleration at the truss mid-span and the corresponding input excitation of three stages were summarized in Table 5. The dynamic amplification factor is somewhere between 2 and 5.5.



a) under S8LX b) under S8LXYZ

Figure 5 Acceleration envelope in X direction



a) under S8LY b) under S8LXYZ

Figure 6 Acceleration envelope in direction Y

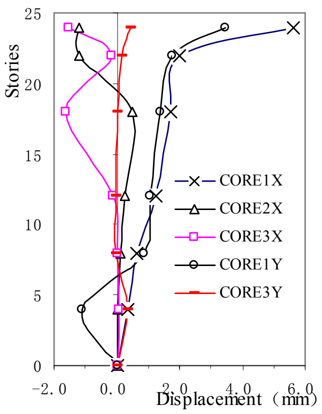


Figure 7 Displacement pattern under s8MX

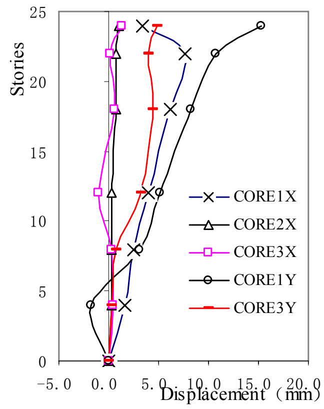


Figure 8 Displacement pattern under s8MY

Table 5 Peak acceleration at truss mid-span and input excitation at soleplate

Case	Location	Positive accl. (g)	Time (s)	Negative accl. (g)	Time (s)
Stage I	Soleplate	0.511	1.248	-0.433	1.265
	truss mid-span	1.000	1.248	-1.671	1.265
Stage II	Soleplate	-0.732	1.290	0.364	1.883
	truss mid-span	-1.923	1.290	1.676	1.883
Stage III	Soleplate	-0.344	1.418	0.085	1.430
	truss mid-span	-1.855	1.418	0.313	1.430

3. FINITE ELEMENT ANALYSIS

To further investigate the seismic performance of the structural model, a finite element model was developed using structural analysis program SAP2000 (CSI, 2002). The elastic time-history analysis of the structural model was carried out.

3.1. Modeling

Walls and all slabs of the model were modeled by “shell elements”. Steel beams and columns were modeled by “frame elements”. Four hanging cables were modeled by “prestressed cable”. The prestressing force was calculated based on the initial strain during the test. A 5% damping ratio was used in the analysis. The finite element model was shown in Figure 9, and the plan view of level 23 of the model was shown in Figure 10. Modal superposition method was used to perform the time-history analysis to reduce the calculation load using direct integral method, and the first 50 modes were included. To compare with the experimental results, only the excitation cases with the model under elastic status were considered. For the 3-direction excitation cases, only two horizontal excitations were inputted and the peak acceleration scale was 1:0.85.

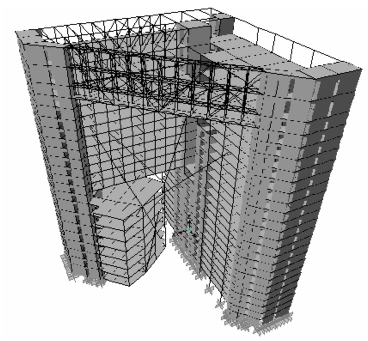


Figure 9 Sap2000 Model

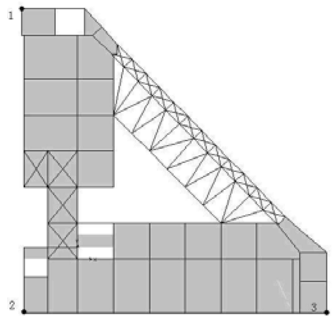


Figure 10 Plan view-level 23

3.2. Finite Element Analysis Results

3.2.1. Natural Frequencies

The first 50 modes of the model were calculated. From the analysis results, torsion component occurred in each mode and the absolute lateral vibration modes did not existed. Table 6 shows the frequencies and periods of the first 10 modes, the vibration description also included. The comparison between the first 10 natural frequencies of the finite element analysis and the shaking table test is shown in Figure 11. The numerical results matched the experimental result.

Table 6 Dynamic characteristics of FE analysis model

Mode	Period (s)	Frequency (Hz)	Vibration	Mode	Period (s)	Frequency (Hz)	Vibration
1	0.1259	7.94	Y+Torsion	6	0.0273	36.58	Vertical (Lantern)
2	0.0952	10.50	X+Torsion	7	0.0247	40.49	Torsion
3	0.0801	12.49	Torsion	8	0.0233	42.88	Breath Type
4	0.0412	24.30	Y+Torsion	9	0.0216	46.24	Vertical
5	0.0299	33.47	X+Torsion	10	0.0206	48.66	Vertical (Frame)

3.2.2. Displacement response

Three corners (as shown in Figure 10) of each level, represent the three concrete cores, are selected to investigate the elastic responses of the model. The model responses under four excitations – “C”, “L”, “O”, and “s” (denoted in Table 2) were analyzed. The displacement envelopes of these three locations under earthquake excitations in X and Y are shown in Figure 12 and Figure 13, respectively. The following phenomena can be seen from the two figures: (1) The structural model behaved like a shear wall structure, and deformed in a typical bending style. (2) The displacements of Core2 and Core3 are close to each other in X direction, Core1 and Core2 are close to each other in Y direction. This is consistent with the structure configuration, Core1 and Core2 are connected in Y direction, while Core2 and Core3 are connected in X direction; (3) Similar as experimental result, torsion effect was significant. Compared with Core2 and Core3, Core1 is the weakest, and the displacement of joint 1 at top of the model in Y (X) direction is about 75% (70%) of that in X (Y) direction when the excitation “s” is applied in X (Y) direction.

The responses of structure in elastic range under bidirectional excitations were also investigated. Structural responses under bidirectional excitations are larger than those cases under unidirectional excitation. The structural response in Y (X) direction might exceed the response in X (Y) direction when the X (Y) direction was the primary direction of the excitation. Obviously, it is necessary to investigate the structural performance under excitations both in unidirectional and in bidirectional (or 3-directional).

It should be noticed that, the displacement envelope of FE analysis shows a relative smooth bending style deformation pattern, while there were abrupt changes of the displacement envelope from the experimental results. One reason of this might be the simplification of finite element model could not represent the special parts of the complicated structure. Therefore, both the shaking table test and finite element method are needed to analyze the seismic capacity of an irregular structure like New Poly Plaza.

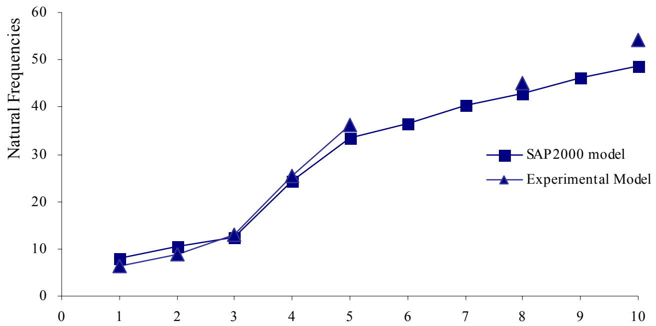
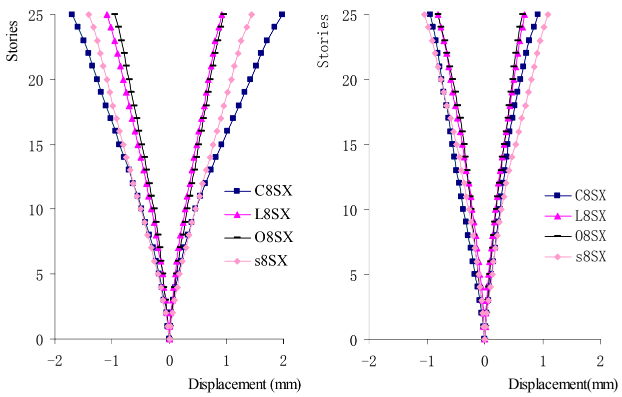


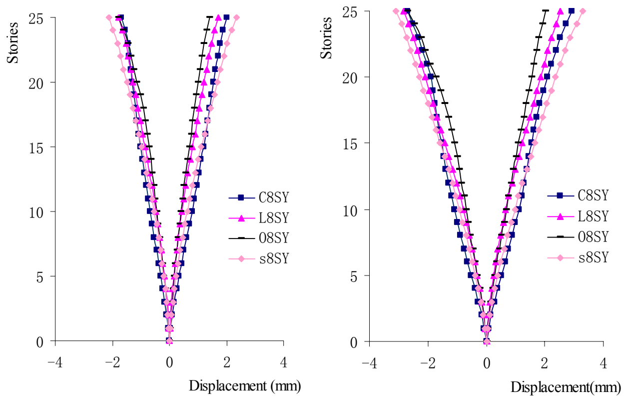
Figure 11 Natural Frequencies of the analysis and the experimental results

3.2.3. Storey drift

The storey drift, the relative inter-storey lateral deflection divided by the storey height, is another important index to evaluate the seismic performance of a structure. The storey drift of the experimental test was not studied due to the limited data. Storey drifts in elastic state of the three corners as shown in Figure 12 were calculated. Most results were within the storey drift limit 1/800 required for concrete structure in the design code (JGJ-2002). Overall, the structure of elastic state was adequate to catch the current design codes.



a) Core1 X direction b) Core1 Y direction
Figure 12 Displacement envelope under X excitations



a) Core1 X direction b) Core1 Y direction
Figure 13 Displacement envelope under Y excitations

4. CONCLUSIONS

The following conclusions can be drawn from this study:

(1) The fundamental frequencies were decreased from stage to stage, which indicated that the structure will work nonlinearly after experiencing higher level earthquakes. In addition, after three stages, there were no significant damage of the special lantern and the hanging cable system. The three concrete cores of the structure behaved differently in both shaking table test and finite element analysis, Core1 is weaker than the other two, especially in Y-direction. Except unidirectional excitation, bidirectional or 3-directional excitations are needed for extreme irregular structure like Beijing New Poly Plaza, because the structure responses under multi-directional excitations are stronger than those under unidirectional excitations.

(2) The experimental results and numerical results agreed well when the structure was in the elastic state. The whipping effect above level 22 was observed from both approaches. To some extent, the finite element analysis reproduced the shaking table test. However, shaking table test shows more local responses such as the abrupt changes of acceleration and displacement responses of levels 12 and 22 due to the architectural changes at these two levels. Therefore, it is necessary to evaluate the design of complicated, irregular structures combining these two approaches.

(3) Based on the experimental and analysis results, it can be concluded that the Beijing New Poly Plaza has a good seismic performance and is satisfactory with the requirements for seismic intensity VIII.

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