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EARTHQUAKE SIMULATOR TESTS OF ONE-SIXTH SCALE NINE-STORY RC MODEL

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SUMMARY

An earthquake simulator test of one-sixth scale nine-story reinforced concrete frame-wall model was conducted at Institute of Engineering Mechanics, CEA. The response results are summarized and discussed in this paper. Some suggestions associated with seismic resistant design and evaluating damage of RC structures are presented based on this test.

INTRODUCTION

The performance of RC structures under earthquake excitation is extremely complex, the theoretical and experimental studies have been conducted for a long time and still we have much work to do for better understanding their seismic behavior. The earthquake simulator test is an especially effective method for this purpose and the researchers in this area have presented lots of valuable test studies during last decades, especial those presented by U.S.-Japan cooperation program [1, 2, 3].

An earthquake simulator test of one-sixth scale nine-story reinforced concrete frame-wall model was conducted on the shaking table of 5m×5m. The main objectives of this experiment are:

1. To study the behavior of the RC building under the earthquake excitations of different intensities.
2. To investigate the changes of drift values and natural frequencies etc. with various damage states of structures.
3. To examine the analytical results in order to further improve the analytical methods.
4. To evaluate the realistic performance of building designed according to Chinese Seismic Code.
5. To examine whether there is the effect of short columns when the height/width ratios of columns are less than 4.0, which are designed in order to carry more vertical loads.

The model, programs and results of the test are introduced in the paper. The frequencies of the model under different damage states and the responses of the accelerations and displacements are presented, the damage details are described and the drift values corresponding to different damage stages are discussed.

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DESIGN AND CONSTRUCTION OF THE TEST MODEL

Outline of the model

The test model is shown in Fig.1. Limited by the size of the shaking table, which is 5m×5m and has a maximum loading capacity of 35t, the model was designed to be 1/6th scale nine story reinforced concrete frame-shear wall structure. The plan and section of the model is shown in Fig.2 and Fig. 3 respectively. In China, the cross sections of columns in lower stories of some high-rise buildings are larger than ordinary ones in order to carry more vertical loadings, therefore, the height/width ratio of columns of this model in 2-4 stories is designed to be 3.73, less than 4.0 to examine short column effect.

The model is symmetry to C axis and the direction of earthquake excitation. The steel braces were placed in all stories of 1 axis frame and 3 axis frame to prevent torsion as shown in Fig.1. Whole model was fixed on the RC foundation plate with plan size of 2.5m×2.5m and thickness of 200mm.

Fig. 1 Test model

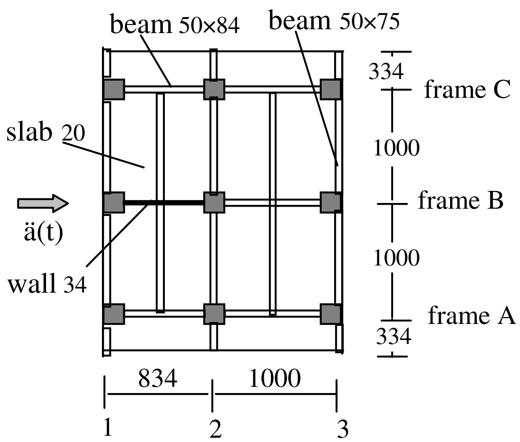

Fig. 2 Plan of the model

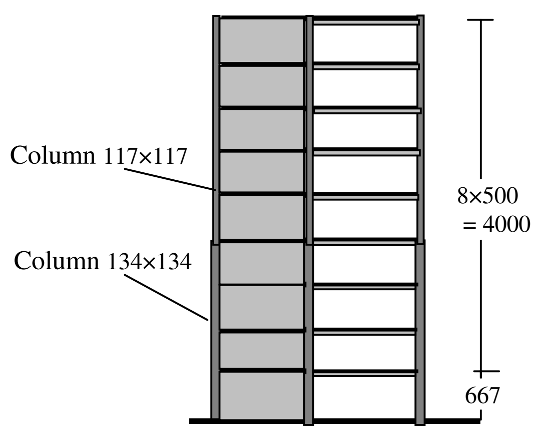

Fig. 3 Section of the model

Model materials

The purpose of this test is to investigate the performance of prototype structure under earthquake excitation, therefore the same materials, namely reinforced concrete, as prototype were used for the test model and at the same time it was tried, as possible as we can, to make the model meet the same time history of the strain with the prototype. For this reason, the model was constructed by micro concrete which consists of C45 Portland cement, fine sand and small sizes of stone of maximum size<15mm with a proportion of 1:2:2.

The micro concrete material properties were tested with 27 specimens of size 7×7×14cm, aged time from 204days to 271days. The average compression strength is 8.07Mpa and variation coefficient is 0.21. The standard cubic compression strength is 12.04Mpa obtained by dividing a conversion coefficient namely 8.07/0.67. The average elastic modulus is 2.09×10^4Mpa measured when concrete reaches 45% of its compression strength.

The main reinforcements of columns and beams are φ6 and φ4 respectively. Limited by reduced scale size the steel wire (φ1.6) mesh is used as reinforcements of two orthogonal directions in the wall and floor and the size of one mesh is 27mm×27mm. The stirrups in all columns and beams are $12^{\#}$ iron wire ofφ2. Total 12 reinforcement specimens were tested for measuring their yield strength, ultimate strength and ductile ratio and the results are listed in Table1.

Table 1 Mechanical properties of main reinforcements

Diameter	Yield strength (Mpa)		Ultimate strength (Mpa)		Ductile ratio (%)	
	Average	Variation	Average	Variation	Average	Variation
φ6	247	0.100	427	0.046	29.2	0.72
φ4	250	0.026	379	0.000		
φ1.6	324		405	0.031	13.7	0.088

Similitude relations

It is most ideal to meet the gravity similitude requirement between prototype and model structures in dynamic test in order to more reliably predict response of prototype. Therefore, the method of adding additional weight is generally employed and this is so called artificial mass model. However, if we do so in this test the total weight (model + foundation plate + artificial mass) would be 50t and greatly exceeds the load capacity of the shaking table and thus only insufficient artificial mass model can be used. The cast iron blocks are added on each floor as artificial mass.

The equivalent mass ratio is calculated as following formula based on the relationship of insufficient artificial mass model [4].

$$\zeta_r = (m_m + m_a + m_{om}) / [l_r^3 (m_p + m_{op})] \qquad (1)$$

in which, m_m, m_a and m_{om} denote the member mass, artificial mass and non-structural member and live load mass of model respectively, m_p and m_{op} represent the member mass and non-structural member and live load mass of prototype building and l_r is the length ratio of model to prototype. Last we can get ζ_r = 4.235 and the other similitude relations are listed in Table 2.

Table 2 Similitude relations of model

Length ratio	Density ratio	Elastic modulus ratio	Time Ratio
1/6	4.235	1	0.343
Frequency ratio	Acceleration ratio	Stress ratio	Strain ratio
2.916	1.411	1	1

Measurement meters and locations

The thirteen acceleration meters and four displacement meters were mounted in the test model. Eight acceleration meters were fixed on the floors of 1^{st} - 8^{th} story respectively and four were fixed on the top of the model, among which two were measuring horizontal response and the other two were charged for measuring vertical accelerations.

Table 3 Test program

Test stage	Test No.	Peak acceleration (g)
Dynamic property	D1	f=5.24Hz ξ=0.57%
Low intensity	TEC1	0.013
	TEC2	0.013
	TEC3	0.015
	TEC4	0.04
	TEC5	0.14
Dynamic property	D2	f=5.00Hz ξ=0.75%
Middle intensity	TNH1	0.030
	TEC6	0.129
	TEC7	0.09
	TEC8	0.07
	TEC9	0.15
	TEC10	0.20
	TEC11	0.22
	TEC12	0.25
	TEC13	0.30
Dynamic property	D3	f=4.60Hz ξ=1.76%
High intensity	TEC14	0.34
	TEC15	0.52
	TEC16	0.57
	TEC17	0.68
	TEC18	0.82
	TNH2	0.51
	TNH3	0.79
Dynamic property	D4	f=2.64Hz ξ=2.06%

Note: first three letters TEC and TNH in test No. column denote El Centro and Ninghe records respectively.

EARTHQUAKE SIMILITUDE TEST

Earthquake input

The two typical ground motions were selected as earthquake input, namely El Centro (N-S) and Ninghe records, their dominant periods are 0.55s and 1.04s respectively. The time interval of the records were compressed according to time ratio.

Similitude test program

The test program is listed in Table 3. The dynamic properties were measured four times: 1) initial stage i.e. before earthquake similitude test, 2) elastic stage 3) visible crack stage i.e. after TEC13 test, 4) last stage namely after all the similitude tests. The earthquake similitude tests were conducted twenty-one times and divided three stages, namely the stages of low, middle and high intensity of earthquake input.

Test results and discussion

Dynamic properties

The first three dynamic tests (D1, D2 and D3) were conducted by sine excitation method and the last one (D4) was carried out by free vibration method i.e. by pushing the model and then suddenly relieving.

The main results are presented in Table 3. The frequency decreases and damping increases with the model damage progress. The change ratios of frequency and damping are, respectively, 4.6% and 31.6% from initial stage to elastic stage, 12.2% and 209% to visible crack stage and 49.6% and 261% to near collapse stage.

Response results of the model

The total response values of ten tests and effective frequencies obtained from the Fourier spectra of roof acceleration time history were given in Table 4. First it can be seen that effective frequencies are obviously lower than those obtained by free vibration and they are 4.1Hz and 1.78Hz in elastic stage and near collapse stage respectively and the change ratio is 56.6%. The inter-story drifts in Table 4 are estimated from the relative values of 1-2 story, 3-6 story and 7-9 story, because only four displacement meters were available in the test.

The relationship between maximum base shears or overturning moments and roof drifts from ten test is shown in Fig. 4 and it can be seen that the significant stiffness changes take place during TEC13 and

TEC17 tests i.e. the model basically keeps elastic before TEC13 (A_{max}<0.30g) and hereafter the stiffness of the model obviously decreases, when maximum acceleration reaches 0.82g (TE18) the model shows a little stiffness and this leads to large displacement observed in following test even though the maximum acceleration of this test is only 0.51g. Another feature can be seen that the strength of the structure is much larger than the one required by China seismic code, according to which the maximum base shear will be 27.2%W supposing that the prototype building is located in seismic zone of nine.

Force and displacement distribution along height of model

The distribution of the seismic forces along the height, corresponding to shear and overturning moment and displacement at maximum response of each test were examined and here only main results are presented. The resultant seismic force is observed at to act between 0.58H to 0.75H from the base of the model. Mostly, the distributions of the seismic force along height at maximum response are inverted triangle and inverted triangle plus rectangular patterns.

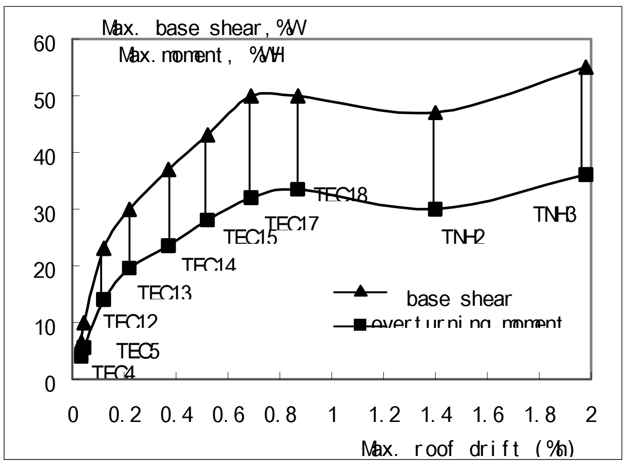

Fig. 4 Max. base shear and overturning moment versus max. roof drift

Table 4 Maximum responses

Test No.	Peak acceleration (g)	Effective frequency (Hz)	Maximum base shear (%W)	Maximum overturning Moment (%WH)	Maximum top acceleration (g)	Maximum roof drift (%H)	Maximum Inter-story drift (%h)
TEC4	0.04		6.6	4.6	0.174	0.04	0.04 (1st story)
TEC5	0.14	4.1	10.1	5.9	0.260	0.05	0.06 (1st story)
TEC12	0.25	4.1	22.7	15.3	0.589	0.14	0.14 (1st story)
TEC13	0.30		29.5	19.2	0.760	0.21	0.23 (7-9 story)
TEC14	0.34		36.1	23.3	0.917	0.36	0.39 (3-6 story)
TEC15	0.52	2.84	42.3	28.4	0.933	0.52	0.55 (7-9 story)
TEC17	0.68	2.4	50.2	31.5	1.300	0.70	0.73 (7-9 story)
TEC18	0.82	2.4	50.5	33.2	1.514	0.86	0.87 (7-9 story)
TNH3	0.51	1.87	45.2	30.3	0.979	1.41	1.47 (7-9 story)
TNH4	0.79	1.78	54.6	34.9	2.598	1.98	2.10 (7-9 story)

Damage observed during tests

Before maximum acceleration reaches 0.3g no visible crack or inelastic behavior was found in the model.

The two inclined cracks were observed at the wall of the same places of two sides of the wall in 1st story (Fig. 5) shows one crack on the C axial side) and very slight crakes were found on a few beams after TEC13 test (A_{max}=0.3g).

The more inclined cracks could be seen on the wall of 1st story, three and two cracks were found on the wall of 2nd and 3rd story respectively and tiny cracks appeared on 1 axial columns of 1st story after TEC14 test (A_{max}=0.34g).

Fig. 5 Inclined crack at wall of 1st story (A_{max}=0.3g)

The old cracks spread and new inclined cracks were observed till the wall of sixth floor and columns of 2nd story respectively, the slight crack occurred at the connection of wall and foundation plate and also were found at the ends of majority of beams and at the middle of some beams after TEC15 (A_{max}=0.57), TEC16 (A_{max}=0.52) and TEC17 (A_{max}=0.68).

Fig. 6 Bucking of reinforcement and Crushing of concrete at the bottom of edge column of wall

Finally, The connection between wall and foundation plate cracked thoroughly, the crushing of concrete and bucking of reinforcing bars occurred at the bottom of its two edge columns (Fig. 6) after the last one test.

The model is obviously the type of strong beam and weak beam, however although the cracks were found at most of beams, no severe damage was observed. This is a questionable problem and possibly the slabs provide enough influence to the beams to lighten their damage.

Based on the results of Table 4, Fig. 4 and observed response behavior the relations between maximum inter-story drift and damage degree of the structure can be estimated as following: θ_{max} <0.2%, structure stays in full operation state and no or only a few tiny cracks occur at structural members; 0.2%>θ_{max} <0.4%, structure is slightly damaged and a few small cracks can be observed; 0.4%>θ_{max} <0.7%, structure is in middle damage state and a few obvious cracks appear; 0.7%>θ_{max} <2.1%, structure is severely damaged and more cracks, crushing of concrete and buckling of reinforcement can be found.

CONCLUSION

1. Natural frequency of structure decreases as damage progress. It decreases by about 12% from original state to initial crack state and by about 50% to approaching collapse state.
2. The limitation values of maximum inter-story drifts in Chinese Code for Seismic Design of Buildings are less than 0.125% and less than 1.0% under the earthquake excitations of 63% and 2% exceeding

probability respectively for this type of structure and these values are too conservative compared with the test results.

3. The prototype structure is supposed to be located at the earthquake zone of intensity of nine and the maximum base shear should be 27.2% of total weight of structure, required by Chinese Code for Seismic Design of Buildings. However, a maximum value of 54.6% is reached in this test and it shows that the designed structure has a significantly higher lateral strength than the demanded one.
4. Although the ratio of length/width of some columns is less than 4.0, no short column damage phenomena is observed and the reason is that the stiffness of slab and beam does not provide enough restraint to make these columns become real short columns.

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