INELASTIC EARTHQUAKE RESPONSE ANALYSIS OF REINFORCED CONCRETE FRAME-SHEAR WALL STRUCTURES

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ABSTRACT :

The fiber model and the equivalent beam model are all used in this thesis. Calculations were conducted to simulate a shaking table test of a nine story 1:6 scale frame-shear wall building model. After comparing the test results of top floor displacement, acceleration, base shear response and maximum story drift ratio, the simulation effects were evaluated. Based on the tests and simulations, the paper gives a conclusion as follows: The simulation results which based on three-dimensional analysis of fiber model and two-dimension analysis simulated by IDARC-2D4.0 are very similar. The reliability and stability of them are very well. And the fact that the results of numerical simulation are similar to the test proves that numerical simulation can simulate the elastic-plastic earthquake response of R.C. structures very well. This is helpful to the research of the elastic-plastic earthquake response of R.C. structures.

KEYWORDS: frame-shear wall structures, Fiber model, inelastic dynamic analysis

1. INTRODUCTION

There is not a proper way to process that analysis of elastic-plastic deformation of complex high-rise building and extra high-rise building to meet the safe needs. Both shaking table tests and numerical simulations are primary approaches in research of response behaviors in earthquake. Shaking table tests need cost bigger and research period longer, numerical simulations are still important approaches in research of elastic-plastic response in earthquake.

As reinforced concrete frame-shear wall structures are widely used in high-rise buildings now. But most of the software can only do the analysis with two-dimensional or pseudo-three-dimensional. As we know, the reinforced concrete frame-shear wall structures can not be simplified to two-dimensional problem, so the best way to solve the problem is setting a new three-dimensional model. It's very important to adopt proper shear wall model and hysteretic model when proceeding elastic-plastic earthquake response analysis of R.C. frame-shear wall structures. However the studies on the hysteretic relations and performances of shear-wall are much less than those of beams and columns, the traditional macroscopic models such as the models used in DRAIN-2D and IDARC-2D are difficult applied in three-dimensional. Fiber model and multi-spring model(Li Kangning, 1998) are considered to be promising models which represents the interactions among axial load and bi-directional bending moments.

In this paper, the three-dimensional spatial rod system analysis model for nine-story reinforced concrete structure which considered the deteriorative character of reinforced concrete member in inelastic hysteretic model is established. By comparing the results of calculating and test, the validity of model of reinforced concrete shear walls based fiber model are evaluated. The difference of the simulations effect between the fiber model and the traditional model is analyzed.

2. OVERVIEW OF SHAKING TABLE TESTS

This paper presents the results from shaking table tests of one-sixth scale nine-story reinforced concrete frame-wall model which was conducted at Institute of Engineering Mechanics, CEA(Sun Jingjiang, 2001). The experimental results and associated study are summarized and discussed in this paper.

2.1. Outline of The Model

The test model is shown in Fig.1, Fig2 and Fig3. 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. The model is symmetry to Ⓑ axis, which is also 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.

The model was constructed by micro concrete .The concrete cubic strength f_{cu} of specimens was 13.5MPa and modulus of elasticity was 2.09×10^4MPa . 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 of φ2. The yield strength f_y of reinforcement with diameters of 6mm, 2mm and 1.6mm was 247MPa, 250MPa and 324MPa, respectively.

Due to the limited capacity of the shaking table facility, the test model was designed according to the general similitude law that considers the effect of the short of artificial mass. Therefore, the method of adding additional weight is generally employed and this is so called artificial mass model. The Similitude ratios are listed in Table 1.

2.2 Test Program

The dynamic properties were measured four times: 1) initial stage i.e. before earthquake simulator test, 2) elastic stage, 3) visible crack stage i.e. after TE13 test, 4) near collapse stage after all the simulator tests. The total tests are twenty-one times and divided three stages, namely the stages of low, middle and high intensity of earthquake. Meanwhile, four dynamic property tests were carried out. The first three dynamic property 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 excitation direction was in X direction to study the seismic performance of frame-shear wall structures. 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 was compressed according to time ratio. The test program and more information can be found in reference (Sun Jingjiang, 2001).

3. COMPARISON BETWEEN SIMULATION AND TEST RESULTS

In this paper, to verify the reliability of fiber element model, ten times typical test were simulated by the seismic time history analyses (Wang Tao, 2006). The two computer programs Canny99(Li Kangning, 1998) and Idarc-2D were used to perform the seismic time history analyses, in which the test model was respectively discredited by equivalent beam elements and fiber elements.

3.1 Analysis Model

In this paper, the three-dimensional spatial rod system analysis model for nine-story reinforced concrete structure which considered the deteriorative character of reinforced concrete member in inelastic hysteretic model is established. The structural members (beam, column and shear wall) were idealized as line elements, and the slabs were treated as a rigid diaphragm having three degrees of freedom (Ye Xianguo, et al. 2004).

Fixed support was assumed at the first-story column base. Beams and columns were considered rigidly connected. The mass was concentrated at nodes. The effect of the gravity load on the lateral displacement(P-△ effect) was taken into account in the analysis. The beam element had uniaxial bending in the frame plane and was idealized in a one-component rotational spring model. The nonlinear hysteretic model of moment-rotation relationships had the aspects of strength deterioration, stiffness degradation and pinching effect by a series of control parameters. Moment-rotation relation for beam was shown in Fig.4.The shear walls and the columns were idealized by the Fiber model, as shown in Fig.5. The fiber element model accounts for the interactions between the bidirectional bending and the axial load fluctuation in exterior columns and shear walls. Every steel bar in columns and shear walls was treated as a steel fiber, and the concrete of section was represented by a fairly large number of concrete fibers. The fiber properties were based on the material stress-strain relationships. Hysteretic modeling of concrete and steel were shown in Fig.6 and Fig.7. The nonlinear shear behavior of all elements was considered to be proportional to the flexural stiffness degradation. By this simplification, the shear deformation was assumed to be elastic.

3.2 Comparison of Natural Frequency

The predicted responses were compared with the shaking table test results. To ensure comparability, the input for the dynamic analysis of test model was the acceleration time history recorded on the shaking table surface, as shown in table2. The acceleration values of the records were divided by the similarity ratio 1.411 and the time interval ratio was 0.343. The input direction was taken as the same as the shaking table test in X horizontal direction. Natural frequency comparison between analysis by CANNY99 and shaking table tests are presented in Table3. The frequency decreases and damping increases with the model damage progress. The variation of frequency error rate are, -1.72% from initial stage to elastic stage, -0.22% to visible crack stage, -27.6% to near collapse stage, 6.1% respectively. From the results of comparison, it can be seen that fiber element model can better predict RC frame-shear wall structural damage stage from elastic and crack to near collapse under strong earthquake excitations.

3.3 Comparison of Peak Value Responses

Peak value of acceleration and base shear comparison between analysis and shaking table tests are presented in Table4. Peak value of roof drift and Inter-story drift comparison between analysis and shaking table tests are presented in Table5. From the results of comparison, it can be seen that both fiber element model and equivalent beam element model can basic model the peak value response of structures under strong earthquake excitations, but they may overestimate or underestimate the peak displacement response. The error may come from accidental error in test, especially peak value response of displacement. Additionally, the analysis model was not able to consider concrete crush and spall off and the nonlinear shear deformation of shear wall could not be taken into account. Nevertheless the modeling results are acceptable in practice engineering.

3.4 Comparison of Time Responses Curve

After TE18 test, the model damages severely and has come to completely yielding stage. Some comparisons of analytical and experimental results from TE4 to TN4 tests are presented in Fig.8 to Fig.10. Top acceleration responses, compared with experimental data, are respectively shown in Fig.8. The correlation between analytical and experimental results is quilt satisfactory and that means either equivalent beam element model or fiber element model can better model the acceleration response of structures under strong earthquake excitations. Fig.9 and Fig.10 show experimental base shear and top displacement responses, meanwhile, give out corresponding simulate results by two programs respectively. It can be seen that both equivalent beam and fiber element models can capture the structural dynamic response properties and better simulate the nonlinear displacement response process as a whole, but they may overestimate or underestimate the peak displacement response. Nevertheless the modeling results are acceptable in practice engineering.

4. CONCLUSIONS

From the shaking table tests and response analyses, the following conclusions can be drawn.
1) The fiber element model is capable of simulating many important features of the experimental measured behavior. The fiber element model accounts for the interactions between the bidirectional bending and the axial load fluctuation in exterior columns and shear walls.

2) The fiber element model is capable of simulating the natural frequency of structure decreases with damage progress very well. It can be used to estimate structural damage level for mid-high rise RC frame-shear wall buildings.

3) Both commonly used equivalent beam element model and fiber element model can better simulate nonlinear behavior of RC frame-wall structure under strong ground motion excitation. Especially acceleration responses quite coincide with experimental results. Although the two element models may overestimate or underestimate the peak displacement response, the whole displacement response results are acceptable in practice engineering.

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ACKNOWLEDGEMENT

The seismic damage field survey and the shaking table tests were supported by Scientific Research Fund of Heilongjiang Provincial Education Department (NO:11531329) and Supported by Basic research fund of Institute of Engineering Mechanics, China Earthquake Administration for special project 2007A02.

Table 1 Similitude relations of model

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

Table2 Peak value of acceleration inputted on the shaking table tests (%g)

Test No.	TE4	TE5	TE12	TE13	TE14	TE15	TE17	TE18	TN3	TN4
Objective value	0.04	0.14	0.25	0.30	0.34	0.52	0.68	0.82	0.51	0.79
Real value	0.04	0.13	0.26	0.31	0.33	0.51	0.70	0.79	0.51	0.79

Table3 Comparison of natural frequency between simulation and shaking table tests

Test No.	D1（initial stage）	D2（elastic stage）	D3（visible crack stage）	D4（near collapse stage）
Test(Hz)	5. 24	5. 00	4. 60	2. 64
Simulation (Hz)	5. 15	4. 89	3. 33	2. 80
Error rate (%)	1. 72	0. 22	27. 6	6. 1

Table4 Peak value of acceleration and base shear comparison between analysis and shaking table tests

Test No	Maximum top acceleration (%g)					Maximum base shear (%W)				
	Test	IDARC	Error rate %	CANNY	Error rate %	Test	IDARC	Error rate %	CANNY	Error rate %
TE4	17.4	13.5	-22.4	17.9	2.9	6.6	7.3	10.6	9.7	47
TE5	26.0	35.3	35.8	22.9	-11.9	10.1	17.3	71.0	12.4	22.8
TE12	58.9	56.3	-4.4	41.5	-29.5	22.7	24.2	6.6	22.3	1.8
TE13	76.0	60.1	-20.9	54.2	-28.6	29.5	27.7	-6.1	25.6	-13.2
TE14	91.7	49.1	-46.5	66.4	-27.6	36.1	19.1	-47.1	28.4	-21.3
TE15	93.3	88.0	-5.7	70.8	-24.1	42.3	33.0	-22.1	31.1	-26.5
TE17	130.0	83.9	-35.5	72.8	-44.0	50.2	30.3	-39.7	29.7	-40.8
TE18	151.4	92.0	39.2	80.1	-47.1	50.5	33.5	-33.6	34.8	-31.1
TN3	97.9	85.0	13.2	77.3	-21.0	45.2	37.5	-16.2	30.6	-32.3
TN4	259.8	116.3	-55.2	111.4	-57.1	54.6	43.3	-20.6	32.5	-40.5

Table5 Peak value of roof drift and Inter-story drift comparison between analysis and shaking table tests

Test No	Maximum roof drift (%H)					Maximum Inter-story drift (%h)				
	Test	IDARC	Error rate %	CANNY	Error rate %	Test	IDARC	Error rate %	CANNY	Error rate %
TE4	0.04	0.04	0.0	0.04	0.0	0.04	0.04	0.0	0.05	25.0
TE5	0.05	0.10	100.0	0.08	60.0	0.06	0.13	116.0	0.09	50.0
TE12	0.14	0.18	27.8	0.17	21.4	0.14	0.22	57.1	0.21	50.0
TE13	0.21	0.22	5.2	0.40	94.4	0.23	0.27	17.4	0.26	13.0
TE14	0.36	0.18	-50.0	0.51	42.7	0.39	0.22	-43.6	0.32	-17.9
TE15	0.52	0.36	-30.2	0.55	-5.8	0.55	0.45	-18.2	0.44	-20.0
TE17	0.70	0.29	-58.1	0.91	29.6	0.73	0.37	-49.3	0.43	-41.1
TE18	0.86	0.58	-32.6	1.13	31.2	0.87	0.6	-31.0	0.69	-20.7
TN3	1.41	1.15	-18.3	1.89	34.0	1.47	1.3	-11.6	2.1	42.8
TN4	1.98	2.45	-23.7	2.87	44.9	2.10	2.65	26.2	3.1	47.6

Fig. 1 Test model

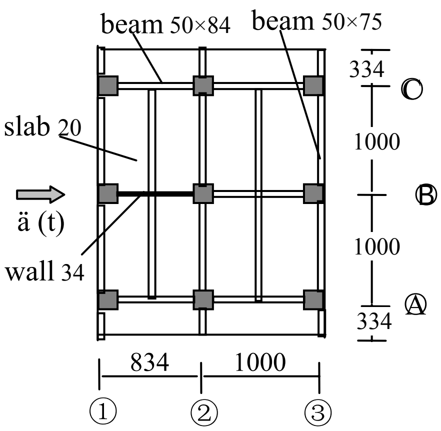

Fig. 2 Plan of the model

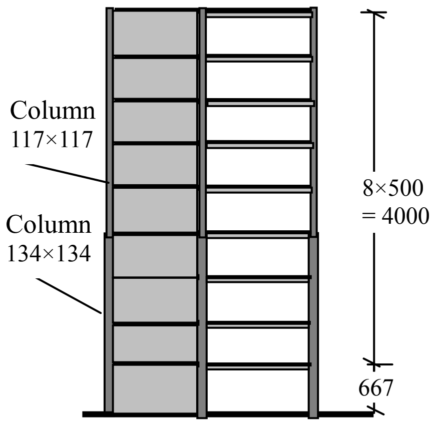

Fig. 3 Ⓑ axis section

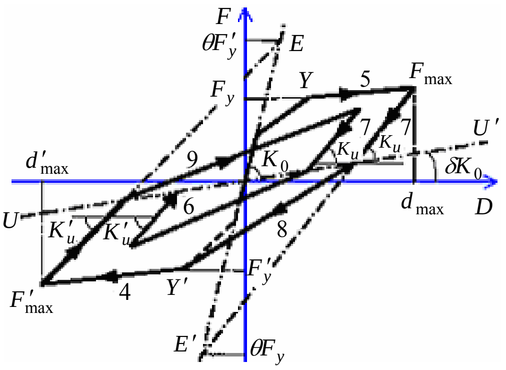

Fig. 4 Moment-rotation relation for beam element

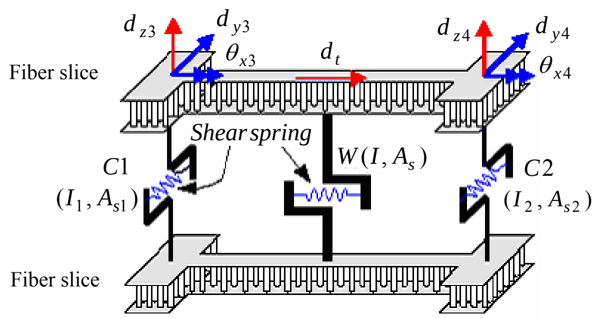

Fig. 5 Fiber model for 3D shear wall element

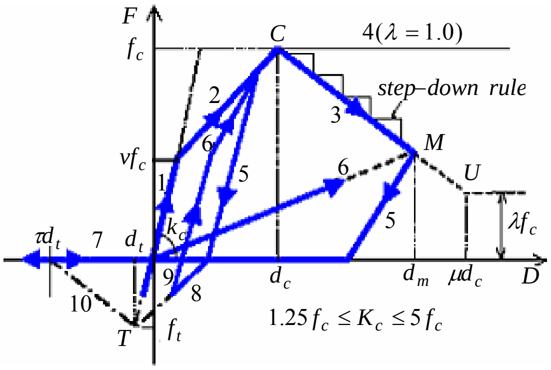

Fig. 6 Hysteretic modeling of concrete

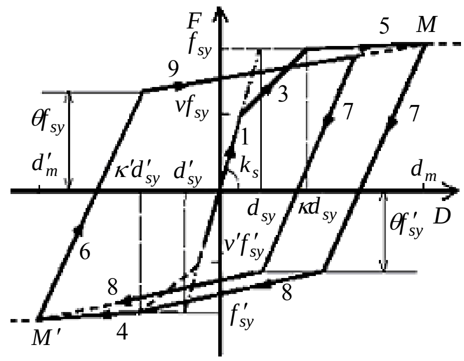

Fig. 7 Hysteretic modeling of reinforced

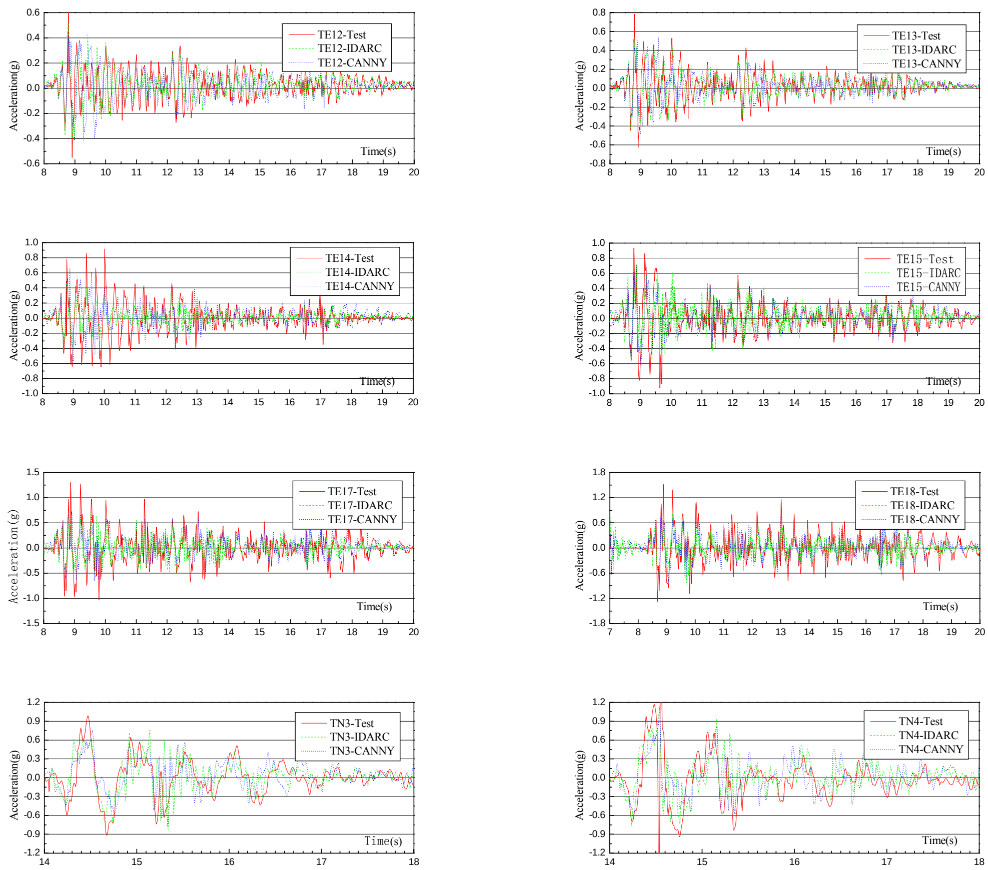

Fig. 8 Time history of accelerations at the top floor between tests and simulation

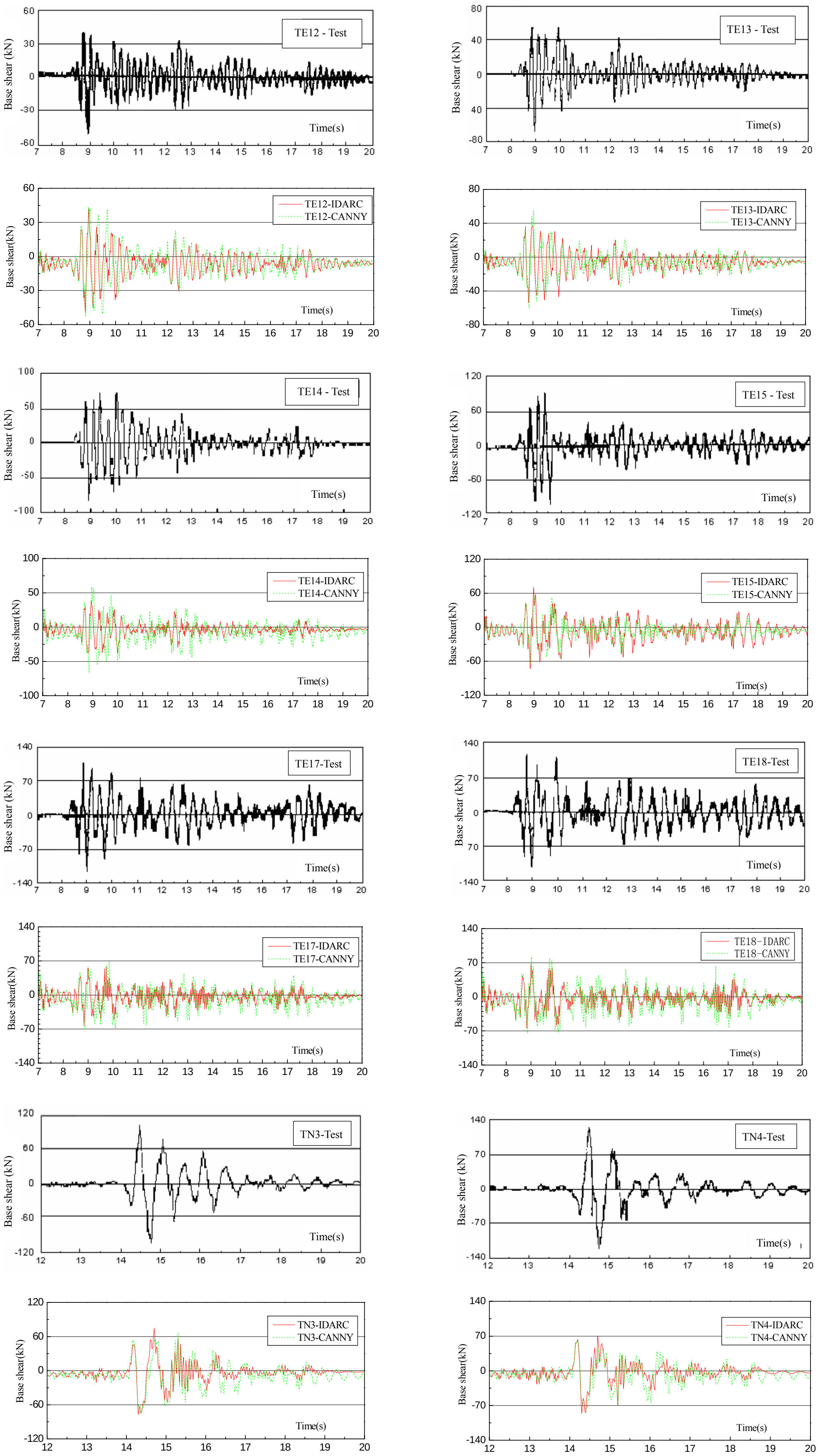

Fig. 9 Time history of base shear between tests and simulation

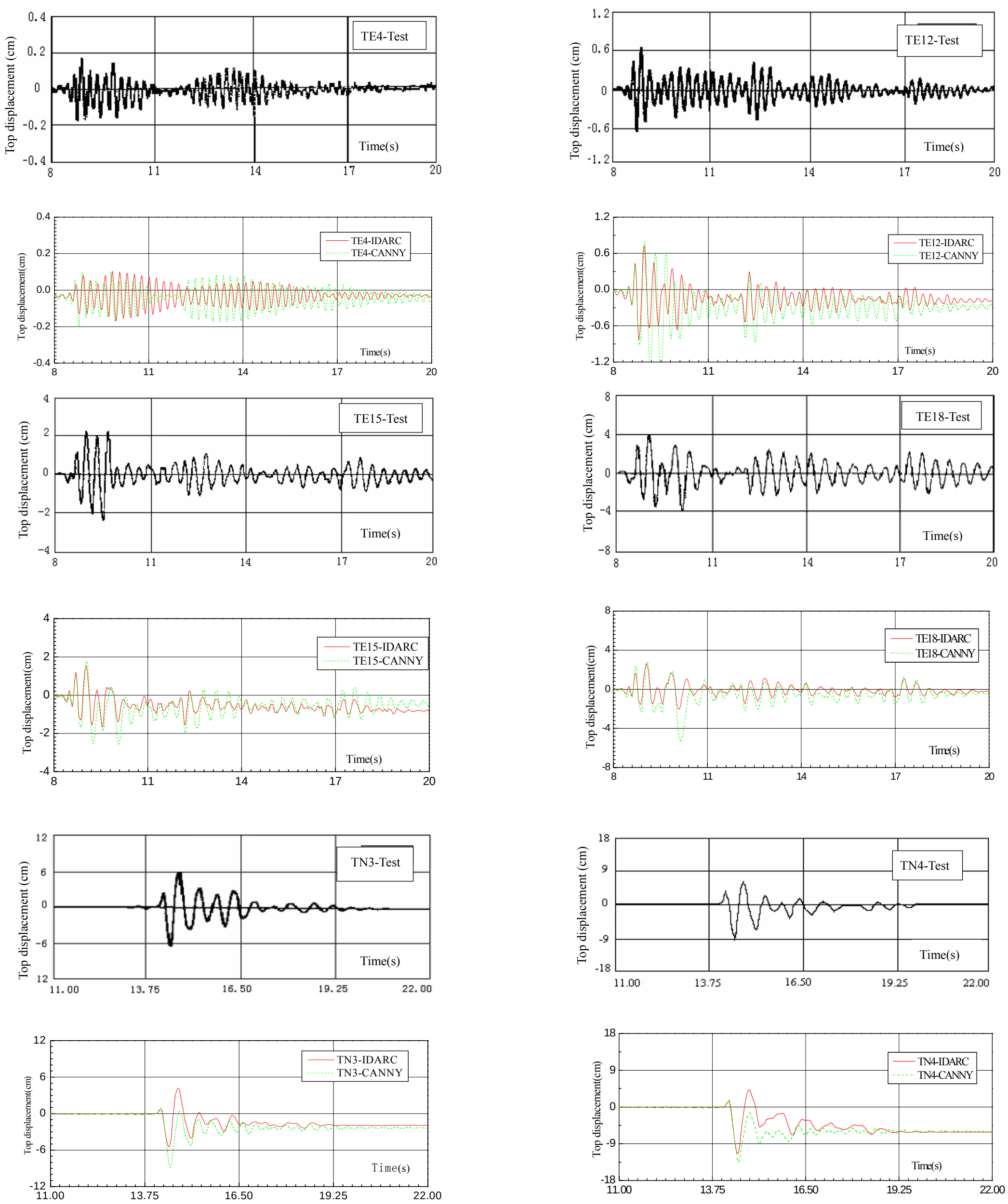

Fig. 10 Time history of displacements at the top floor between tests and simulation