5th Canadian Conf. Earthquake Engineering / Ottawa / 1987 / 5ème Conf. Canadienne Génie Sismique

Study of seismic behavior of precast concrete large panel buildings

Liangduo Zhang & Xiangqian Na
Tsinghua University, Beijing, People's Republic of China

ABSTRACT: Three models of the precast concrete large panel buildings of 6 stories, 10 stories and 14 stories were constructed according to the Chinese practice (with wet joint connections). The dynamic characteristics and earthquake response tests are carried out on a shaking table. By the tests of the structures, the following results are obtained: the change of frequencies and modes of vibration in different stages of test; the elasto-plastic responses; the process of damage and the energy - consuming mechanism of the cracked joints. The deformation mechanism of precast large panel structures of tall building are studied and revealed that the vertical joints of the panels mainly have shear-slip deformation, and the horizontal joints mainly have tension -uplift deformation. The mathematical models of the frame with rigid zone with two kinds of joint element are established and elasto-plastic responses are computed by inputing earthquake record. The results approximately agree with the tested.

1 INTRODUCTION

The appearance of the precast concrete large panel buildings has about one hundred years history. After the World War II, the use of this industrialized production building system was enlarged in Europe and some countries of Asia because of lacking of appartment and labour.

In China, the first group of this system was built at the sixties in Beijing. At seventies some 10-12 stories precast concrete large panel buildings were constructed and at eighties some 16-18 stories tentative buildings were built. Their use in seismic regions and increasing in height require an understanding of their seismic behavior.

For the typical precast concrete large panel system, the wet joint connections are used in Chinese practice. The cast-in-site R.C. keys in the horizontal joints are used between the panels. The vertical reinforcements in the keys are welded in place before casting concrete. For the vertical joints, besides to weld the steel bars at the upper and lower corners of the panels, a vertical bar inserts into the loops which are stretch out from the vertical sides of both adjacent panels, then concrete is casted into the joints.

In order to investigate the seismic behavior and damage mechanism of the large panel structural system, three models of 6 stories, 10 stories and 14 stories were tested respectively on a small capacity shaking table by inputing the earthquake acceleration.

2 TEST MODELS AND SCALE FACTORS

Three models were built according to the typical precast concrete large panel system in Chinese practice. It was taken one bay and extended half bay at its both sides for the model building. The plan, elevation and connection joints of the model are shown as fig. 1. The scale of the models is 1/15. The material of the panels and for grouting the conections are cement mortar. The tested strengths of the mortars, steel wires and welding joints are shown in table 1 and table 2.

Fig.2 shows the layout of the reinforcements of the panel. The prototype of the model can satisfy the Chinese earthquake design for intensity VIII. The scale factors are shown in table 3.

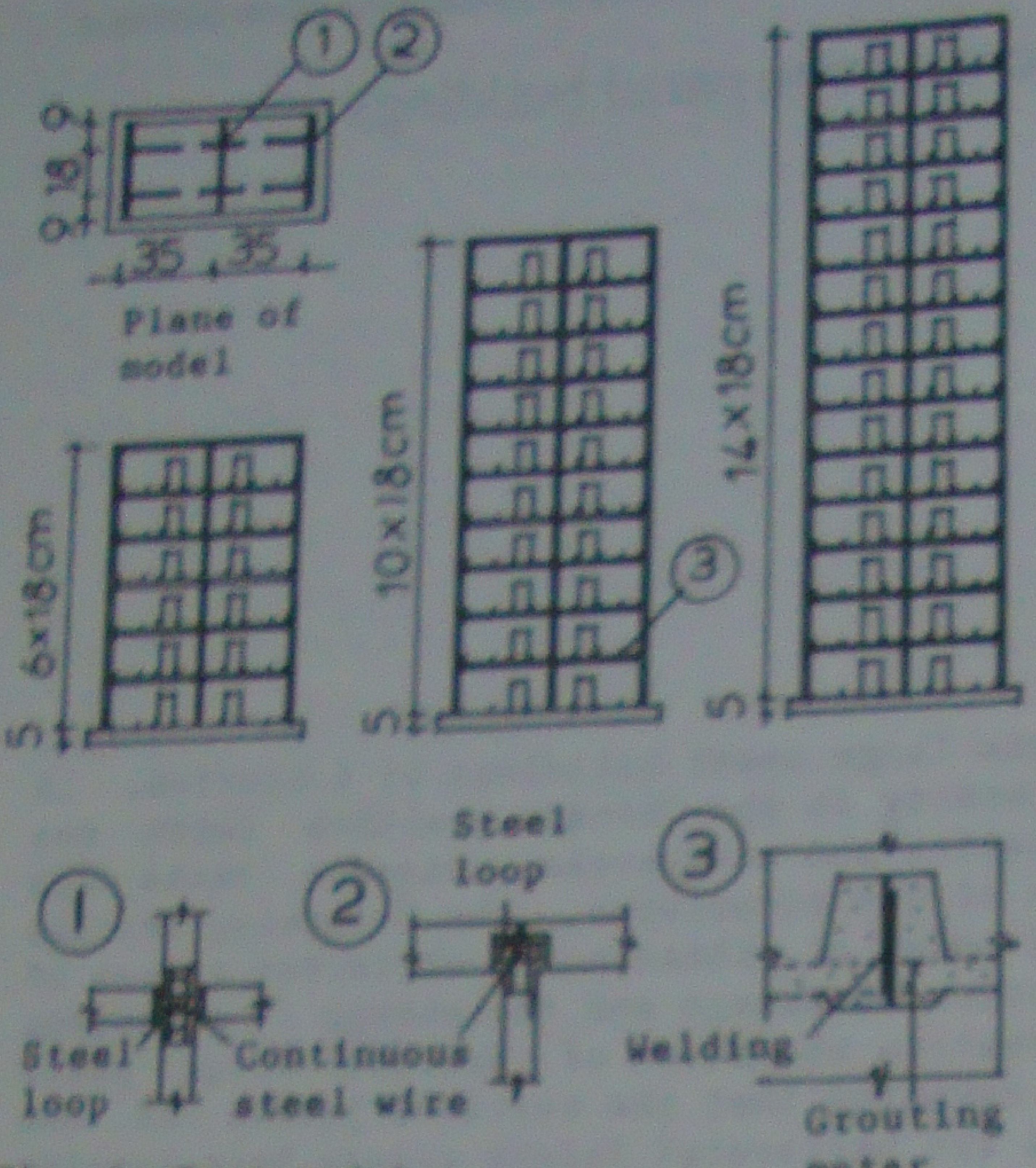

Fig. 1. Test model.

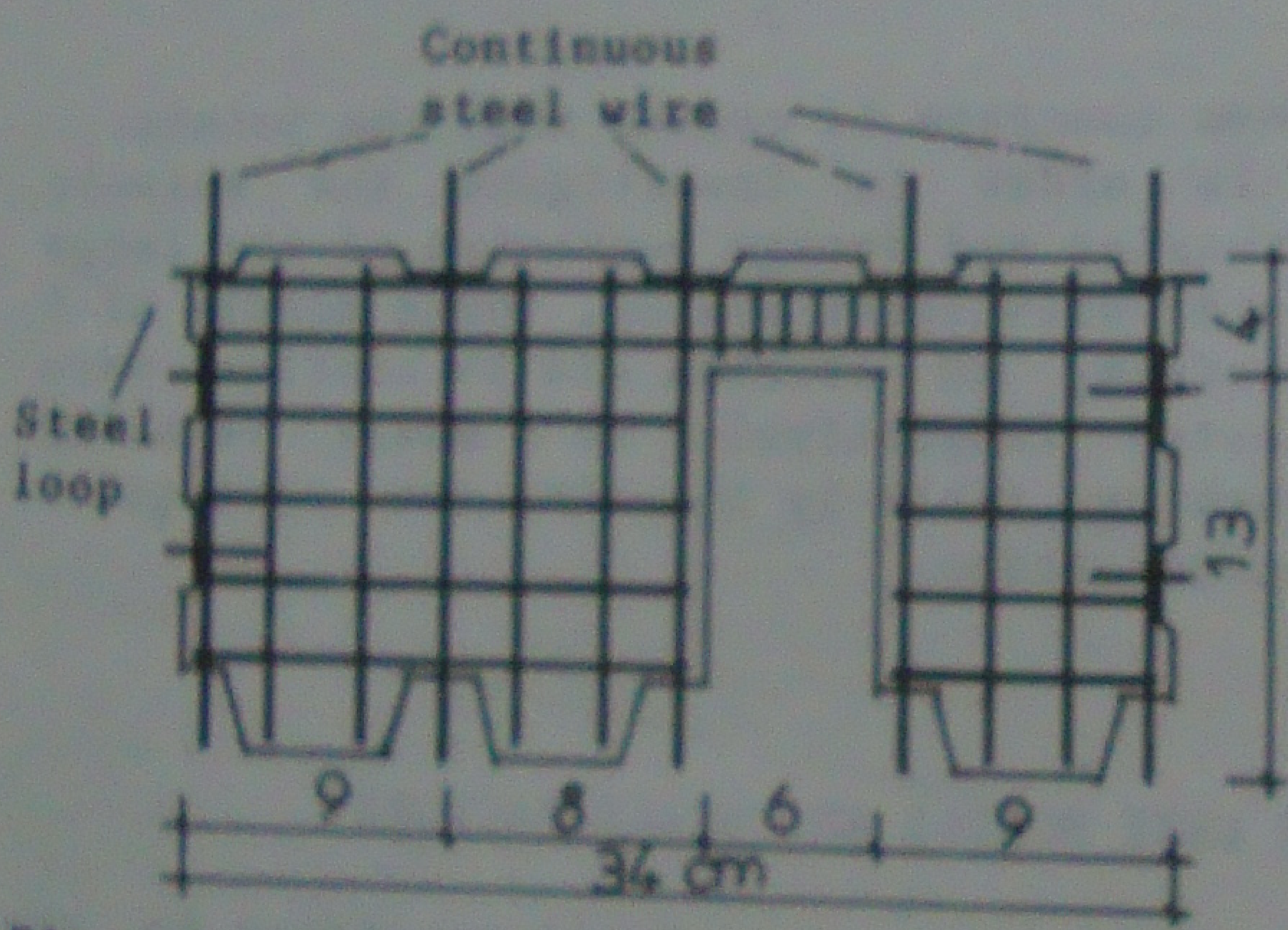

Fig. 2. Layout of reinforcements of the panel.

Table 1. Proportion and tested strengths of mortars.

	sand	Proportion*	Compression Strength (MPa)
For panels	aggregated middle sand	1:3:0.7	33.4
For joints	Silicon Sand	1:2.5:0.7	30.2

*Proportion of Cement: Sand: Water.

Table 2. Tensile strengths of steel wires and welding joints.

Diameters of steel wires(mm)	0.7	0.9	1.2	1.6	2.2
Tensile strengths (N/piece)	100	180	311	720	1140
15mm welding joints (N)				740	1080

Table 3. Scale factors for earthquake response of the models.

	Dimension	Scale factor
Linear dimension L	L	$C_l=1:15=0.067$
Modulus E	FL^{-2}	$C_e=325:300=1.08$
Mass density P	$FL^{-4}T^{2}$	$C_p=2.22$*
Acceleration A	LT^{-2}	$C_a=C_e/C_l \cdot C_p=7.25$
Time T	T	$C_t=\sqrt{C_pC_l^2/C_e}=0.1$

* Small custom made lead plates were attached to every floor to increase the mass indensity.

3 DYNAMIC TEST ARRANGMENT AND DATA MEASUREMENT

The tests were conducted on a small electro-magnetic shaking table which is capable of reaching a maximum force of 15000 N in the random mode. Fig.3 shows the shaking table and the test arrangement. The movement of the shaking table is controlled by a computer, and white noise signal and earthquake loading with variable frequency and amplitude may be input as a model base acceleration. The tests were conducted using the properly scaled N-S component of the 1949 El Centro earthquake and to increase the amplitude step by step to simulate its different intensity.

Prior to input the earthquake loading under different intensity to the models, white noise signal was input to the model base. The response of different floors was recorded and processed by a special computer to find the frequencies and modes of vibration and their change in different test stages.

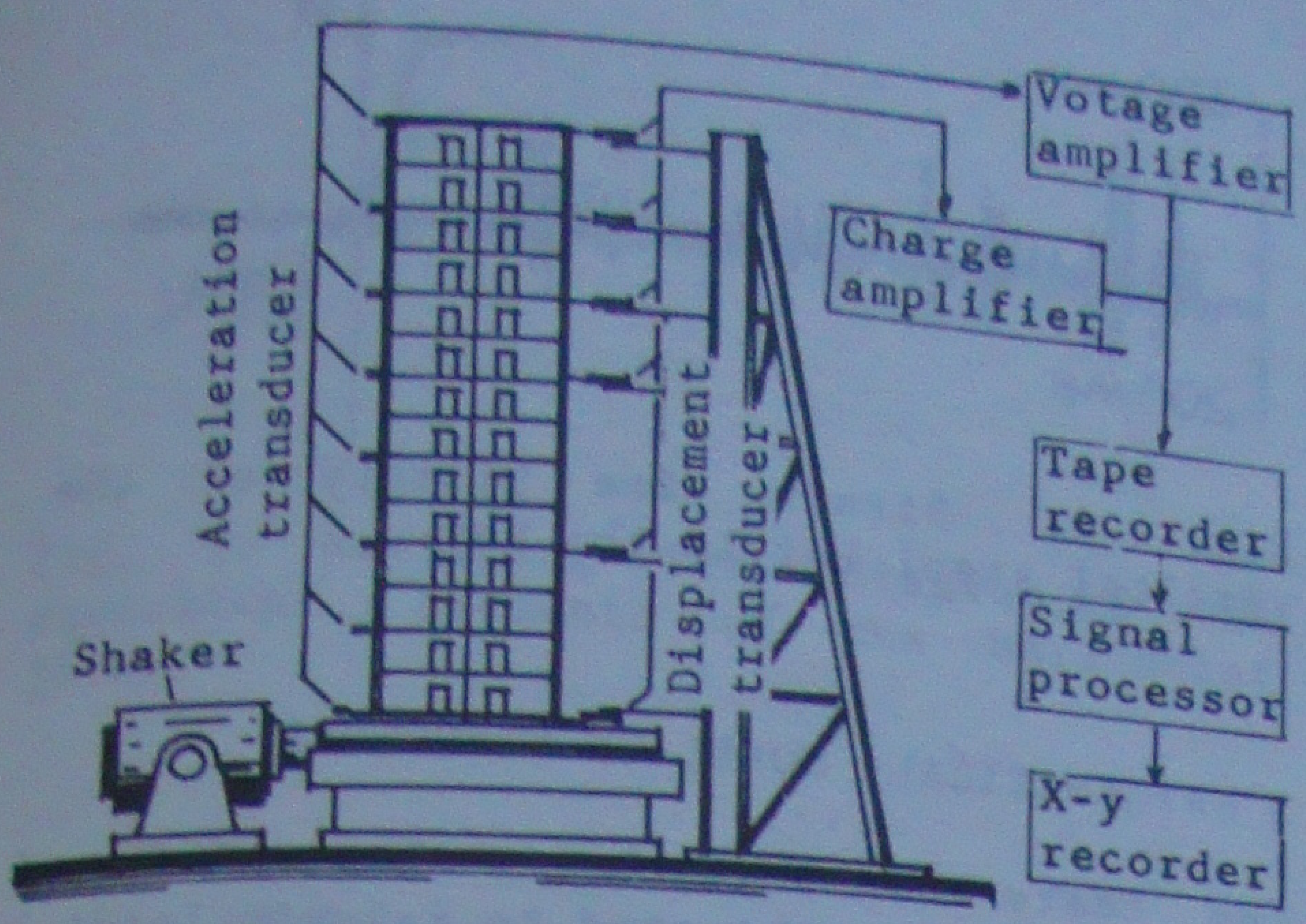

Fig. 3. Test arrangement.

4 DAMAGE MECHANISM OF MULTI-STORY PRECAST CONCRETE LARGE PANEL BUILDINGS

The dynamic response tests of the three models have been conducted according to the above-mentioned method. The test procedure and results are shown in table 4, table 5 and table 6.

Table 4. Dynamic response of the 6-story model.

No.	Peak value of acceleration of El Centro Earthquake			Frequencies stimulated by white noise (Hz)		
	Base input	Roof response	$\beta=a_R/a_B$	f_1	f_2	f_3
1	0	0	0	56.6	195.0	434.8
2	0.80g	1.48g	1.85	54.6	191.1	432.9
3	1.37g	2.49g	1.81	54.6	181.4	409.5
4	1.53g	2.77g	1.81	52.7	174.7	397.2
5*	3.07g	4.95g	1.61	40.0	142.5	347.5

From the tables, it is seen that the frequencies of the three models decrease as the intensity of the earthquake inputs increase. The frequencies decrease slowly during lower earthquake input. The reason for stiffness degradation of the structures is only because of loosing of the joints. No cracks was observed on the structures at the lower earthquake input.

During more strong earthquake input (with signal * in the tables) some cracks were observed at the horizontal joints of the lower stories, and the stiffness of the structures decreased notably. This caused a markable decrease of the frequencies of the structures.

Table 5. Dynamic response of the 10-story model.

No.	Peak value of acceleration of El Centro Earthquake			Frequencies stimulated by white noise(Hz)		
	Base input	Roof response	$\beta=a_R/a_B$	f_1	f_2	f_3
1	0	0	0	30.3	124.9	247.9
2	0.65g	1.35g	2.08	30.3	120.1	242.1
3	1.36g	3.92g	2.89	27.3	110.3	222.5
4*	2.36g	5.44g	2.30	22.5	94.7	204.5
5	3.01g	4.58g	1.50	17.6	72.2	164.5

Table 6. Dynamic response of the 14-story model

No.	Peak value of acceleration of El Centro Earthquake			Frequencies Stimulated by White noise (Hz)		
	Base input	Roof response	$\beta=a_R/a_B$	f_1	f_2	f_3
1	0	0	0	18.5	81.7	176.6
2	0.40g	0.92g	2.2	18.5	77.1	163.0
3	1.23g	2.42g	2.0	17.6	73.2	158.1
4	1.75g	2.50g	1.43	15.6	71.3	150.3
5*	2.30g	3.06g	1.33	12.7	61.5	128.8

The cracks of the three models after the last loadings are represented on fig. 4. The distribution of the cracks may be used to explain the damage mechanism of the structures. For the 6-story model, besides horizontal cracks on the bottom joints, inclined shear cracks were appeared on the upper and lower corners of the door opening. It shows that for low multi-story precast concrete large panel building it subject to bending and shear deformation. For the 10-story model, only horizontal cracks along the bottom joints were appeared. The damage is obviously caused by bending of the structure. For the 14-story model, besides cracks on the horizontal joints of the two lower stories, thin cracks on the vertical joint were appeared between the panels. It shows that for high multi-story large panel building the vertical joints subject shear force caused by greater bending moment of the structure.

It is observed during test that as soon as the horizontal joint cracked through, the whole structure would rock under earthquake loading. The rocking action made the horizontal joints to open and close, especially at the ends of the joints, and finally, the vertical steel bar in the key yielded. Fig. 5 shows the strain history of the steel bar. The rocking action of the whole structure consumed a large amount of the earthquake energy, and enable the upper parts of the structure to remain essentially undamaged. For high large panel building, the rocking action was rather strong, and caused crushing of the panel corners.

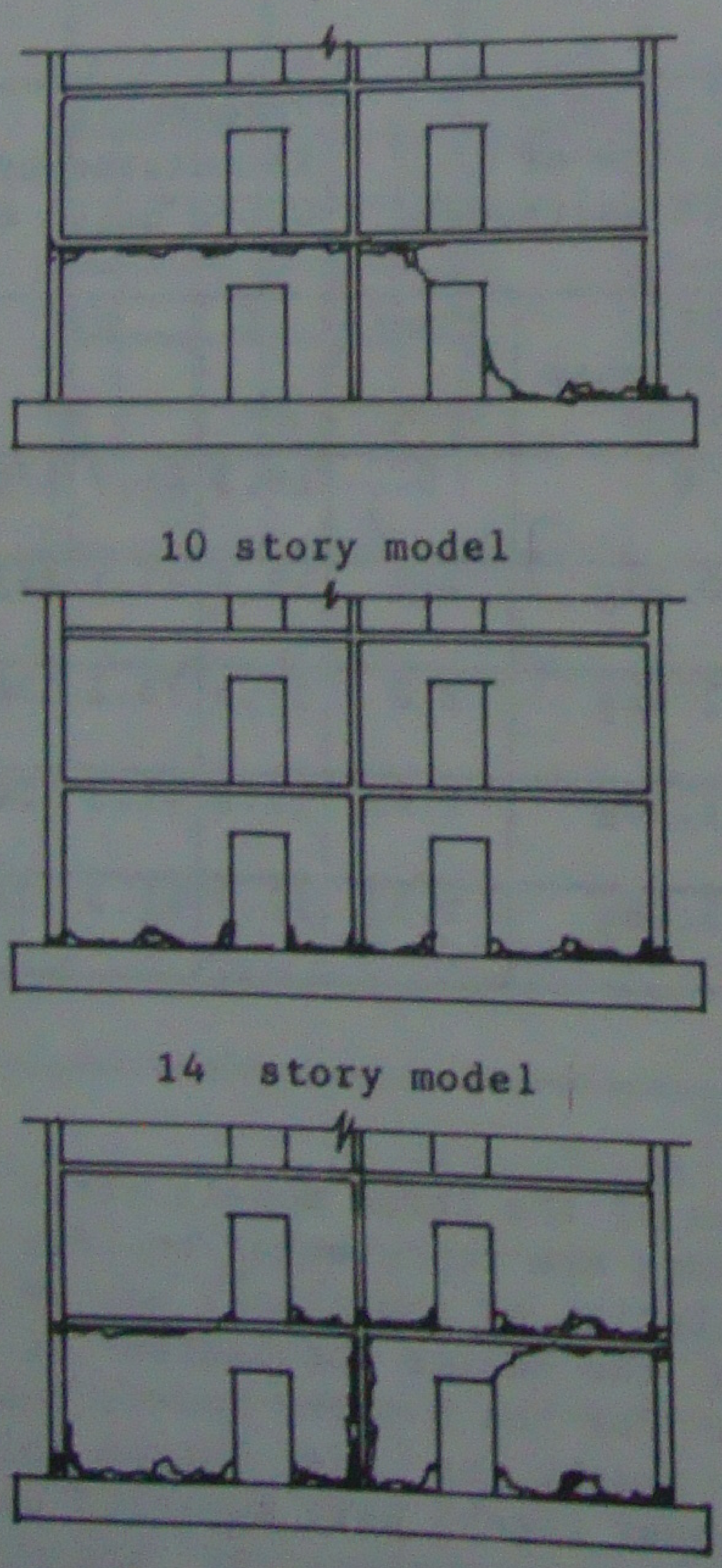

Fig. 4. Obsered Failure of the test model.

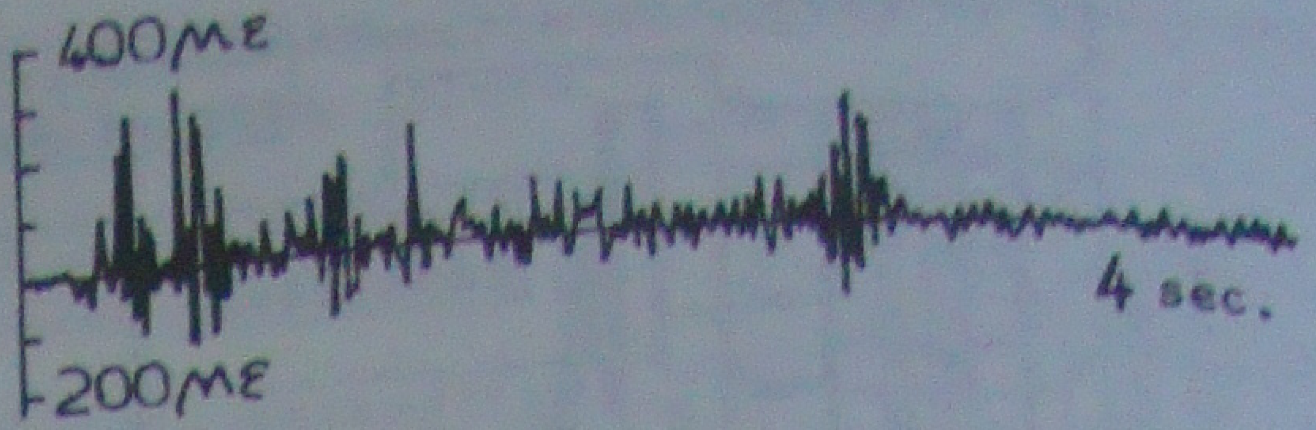

Fig. 5. Strain time history of the vertical steel bar in the key.

5 MATHEMATICAL MODEL OF STRUCTURE

The tests show that the vertical joint of the structure subjected to seismic loading were found to posses mainly shear slip action and for horizontal joint, shear slip and rocking action were found. For high structure, the horizontal joint mainly possess rocking action. Base on the tests, following two mathematical models to consider the joints action are suggested: Firstly, as shown in fig. 6, the panels with openings are considered as a frame with rigid zones, the horizontal joints are considered as some small truss member elements, and the vertical joints are considered as two small beams. The elasto-plastic deformation of the diagonal truss members simulates the shear slip between the upper and lower panels. The elasto-plastic tension and compression deformation of the truss verticals at the ends of the joints simulates their opening and closing action. The restoring force diagrams of the truss member elements are shown in fig. 7. The lengths of the small beams of the vertical joints are equal the width of the joints. The bending deformation at both ends of the beams simulates the shear slip action of the joints. Their restoring force diagram is shown in fig.8.

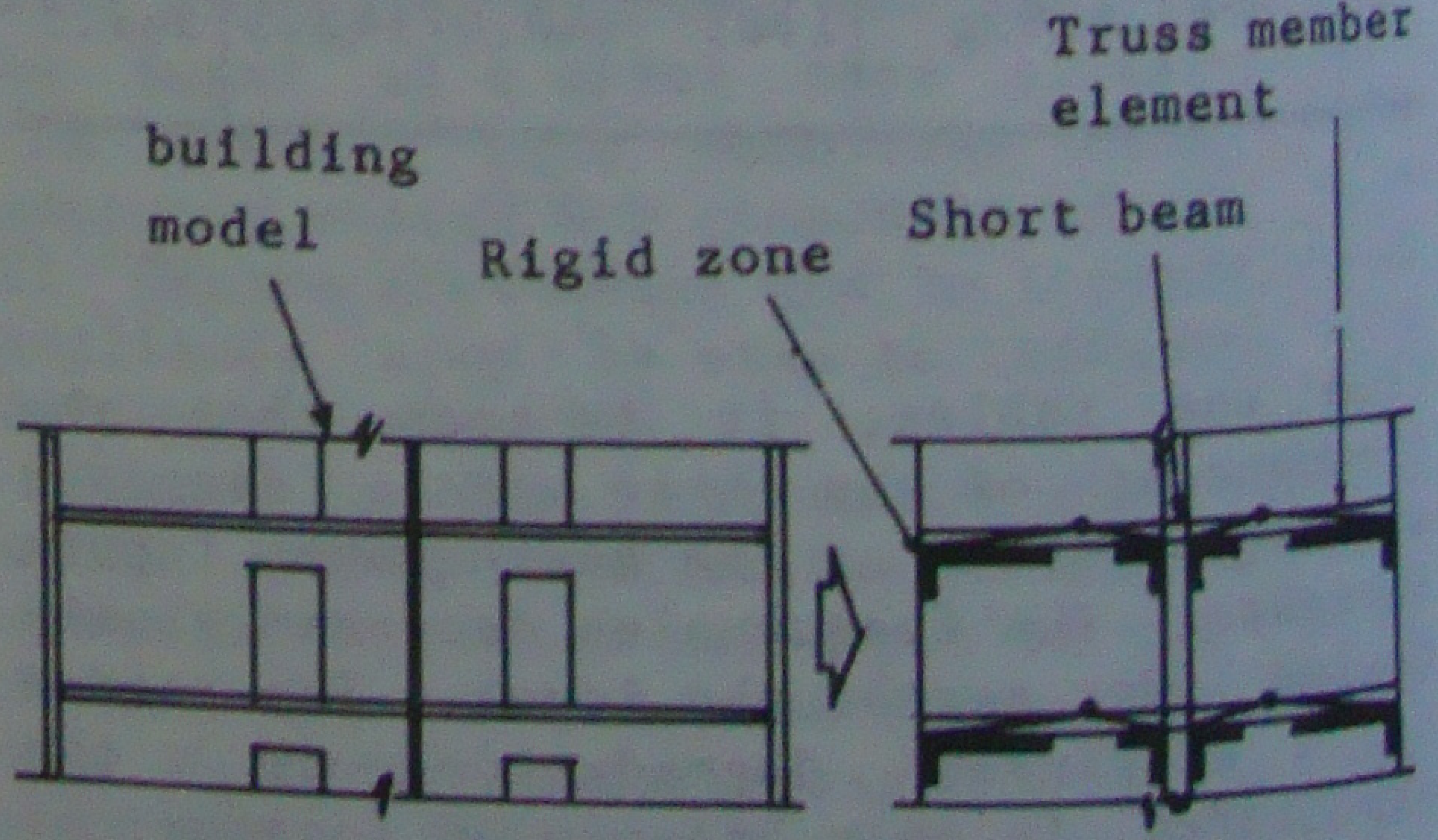

Fig. 6. First mathematical model.

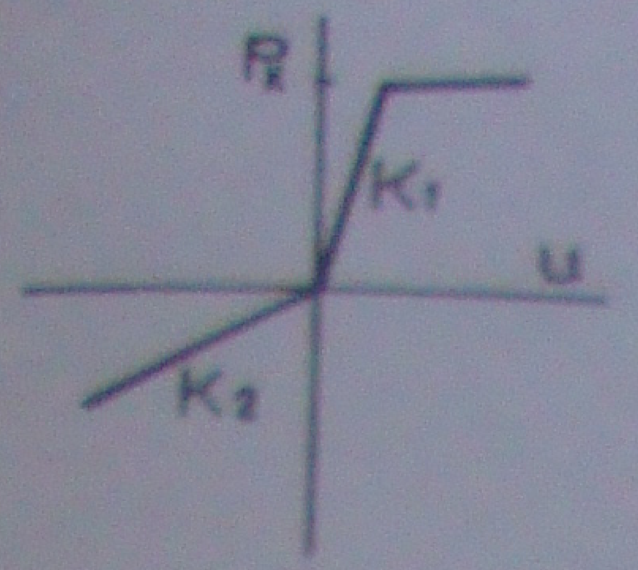

Fig. 7. Restoring force diagram of the equivalent elements of the horizontal joint.

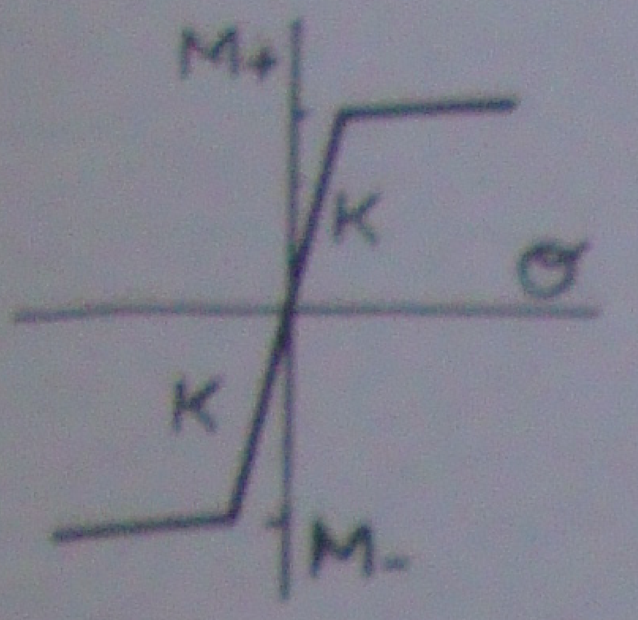

Fig. 8. Restoring force diagram of the equivalent element of the vertical joint.

Secondly, as shown in fig. 9, the structure also be considered as a frame with rigid zones. The horizontal joint would be considered as some short beams, whose length are equal to the width of the joint. The bending deformation of the short beams are same as the panel pier and their shear deformation is same as the joints. In addition to these, the short beams have the property of tension and compression. These short beams are so called the physically equivalent non-linear combined elements (Chen Dan 1981). The bending and shear restoring force diagram is the same as fig. 8, and tension and compression restoring force diagram is the same as fig. 7. The vertical joint still consist of two small beams as in the preceding paragraph. Both of the above-mentioned two mathematical models may be used in the elasto-plastic analysis of large panel structures by a commonly used program. Using the mathematic model shown in fig. 9, the displacement responses of different floors of the 14-story model calculated by inputing recorded base acceleration (peak value 1.23g) and by method of integration step-by-step are shown in fig. 10. These results approximately agree with the tested ones as shown in fig. 11.

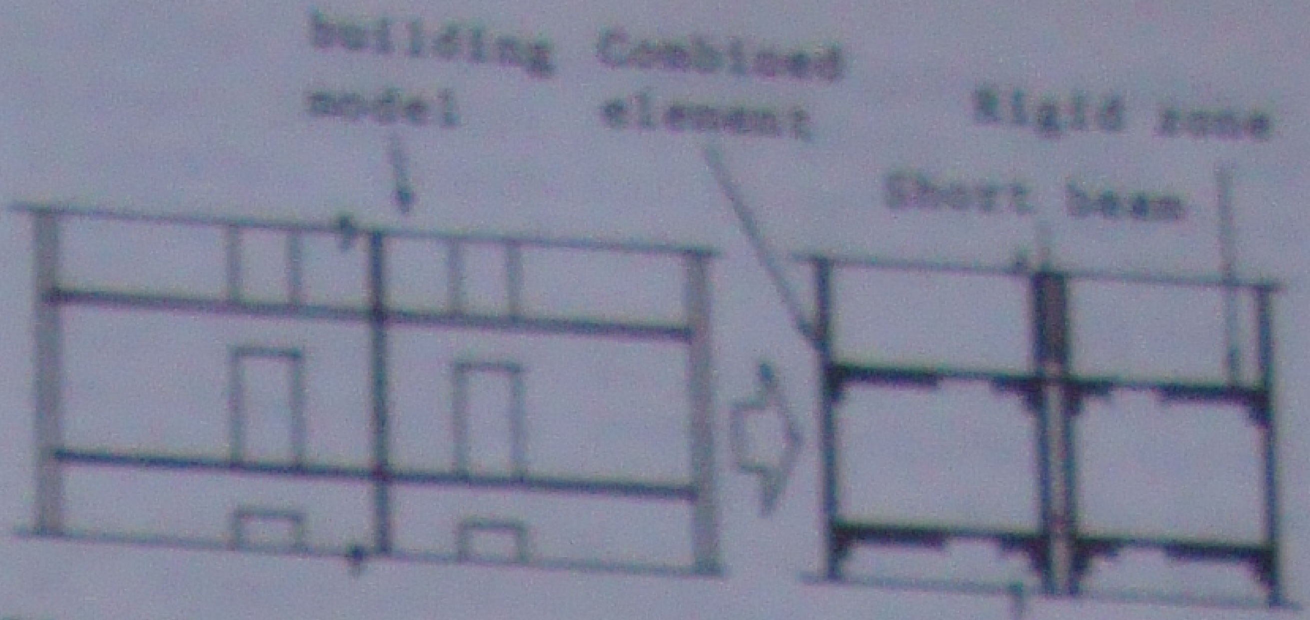

Fig. 9. Second mathematical model.

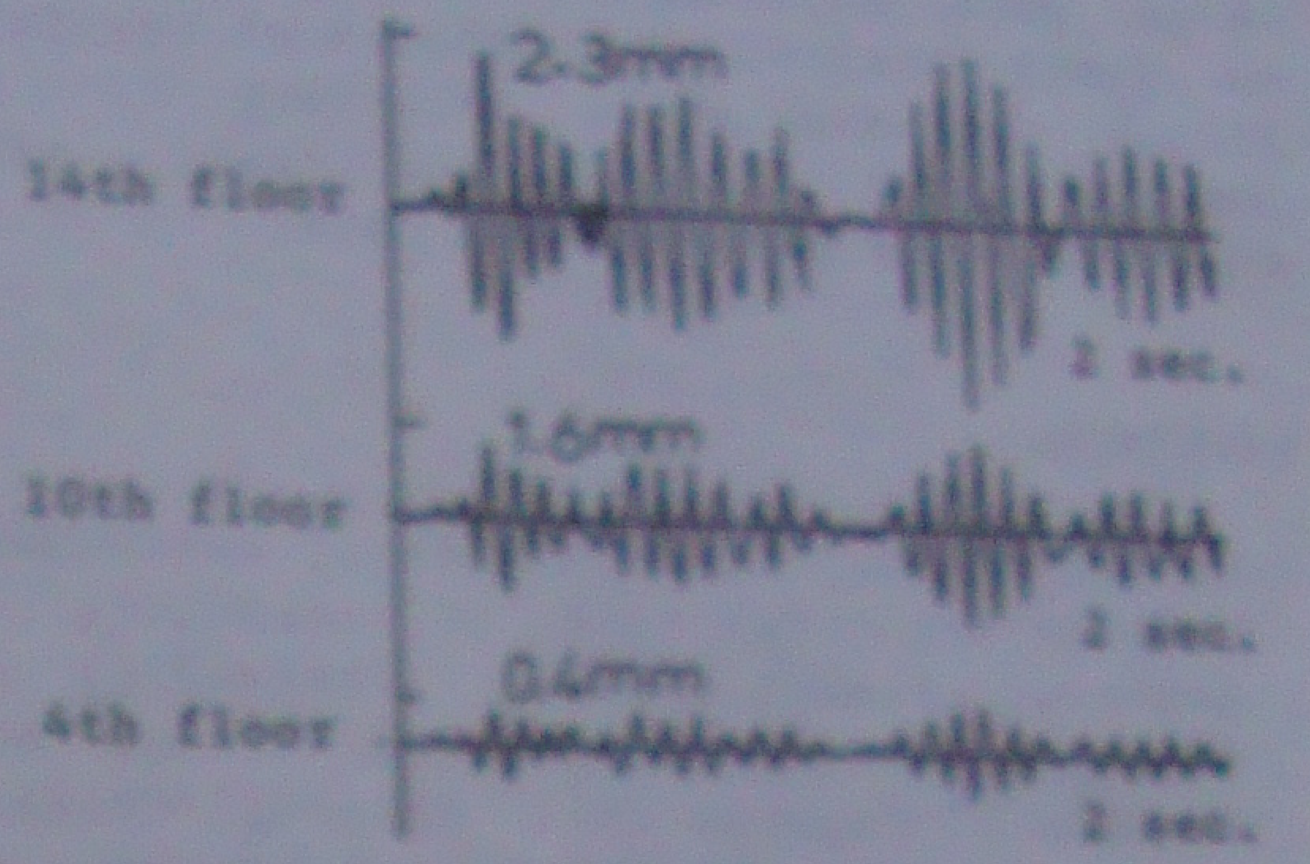

Fig. 10. Calculated displacement time history of the model.

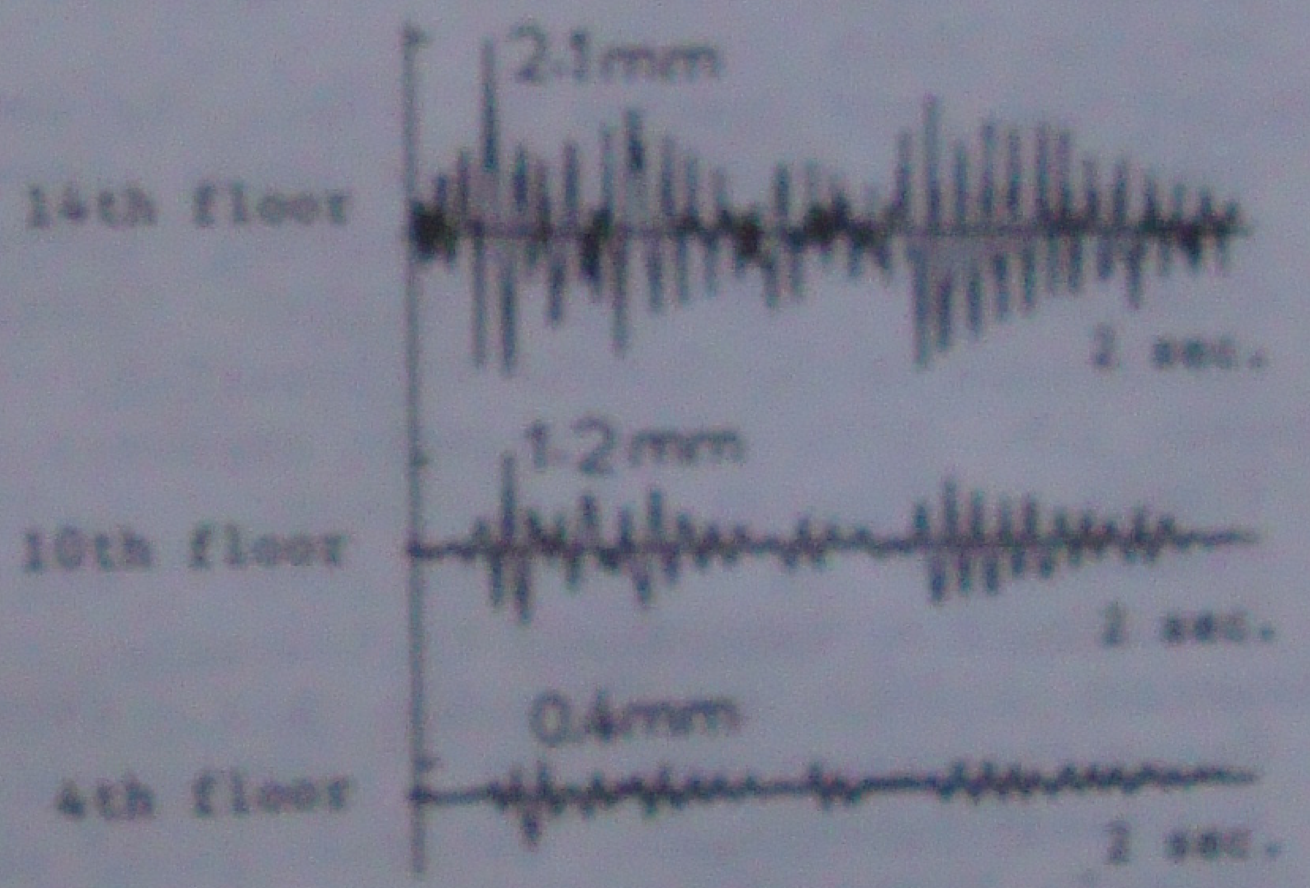

Fig. 11. Measured displacement time history of the model.

6 CONCLUSIONS

The damage mechanism of precast concrete large panel building is as following: The horizontal connections become loose at first and the stiffness of the structure and hence the frequency of vibration decrease. The horizontal bottom joints of the structure crack mainly because of bending action of the whole structure. Then the horizontal joints appear the opening and closing action, and the vertical joints appear shear slip cracks. The upper parts of the structure remain undamaged because of the energy consuming effect of rocking and shear slip of the lower parts.

Using the physically equivalent non-linear combined element to represent the panel joints, the dynamic response of the large panel structure may be analysed by finite element method. The result is approximately agree with the tested one.

Some further measures to improve the earthquake resistance of precast concrete large panel building of 16-20 stories are to exert vertical post-tension at vertical joints of the whole structure or to construct the elevator and stair case monolithically as a stiffen core.

REFRENCES

Chen Dan, 1981. The analysis of earthquake slip-uplift response of structure by combined element models. Proceeding of Sino-American Symposium on Bridge and Structural Engineering. Beijing, China.

Harris H.G. 1981. Earthquake resistance of wet connection and simple shear walls in precast concrete large buildings. Proceeding of Sino-American Symposium on Bridge and Structural Engineering. Beijing, China.

Harris H.G. and Caccese V. 1984. Seismic behavior of precast concrete large panel buildings using a small shaking table. Proceedings of the Eighth World Conference on Earthquake Engineering.

Becker J.M. 1980. The Seismic Response of Simple Precast Concrete Wall. Proceedings of the Seventh World Conference on Earthquake Engineering.

Bouwkamp J.G. and Stephen R.M. 1980. Dynamic properties of prefabricated apartment buildings. Proceedings of the Seventh World Conference on Earthquake Engineering.

Wan Molin and Zeng Bing 1986. Strength and rigidity of joints in large panel buildings. Jounal of Building Structures Vol. 7 No. 4