



IN-SITU PUSHOVER TEST AND NUMERICAL SIMULATION OF A REAL REINFORCED CONCRETE FRAME STRUCTURE

Xilin Lu¹, Jiang Qian¹, Tuo Lei², Liang Lu³ and Guangye Zhao²

ABSTRACT

In order to study the seismic damage and failure patterns of existing buildings, an in-situ pushover test has been performed. A single span, single bay frame cut from a three-story reinforced concrete frame built in 1980s was taken as the test object while the rest was used as the reacting wall. The process of structural damage has been reported, its failure mechanism is analyzed, and the assessment on its seismic performance has been given. It is shown that the torsional responses are obvious because of the irregular distribution of perimeter walls, and no realistic plastic hinges are observed at the end of columns and beams. It is concluded that the final breakdown of the structure should be of brittle type characterized by shear failure of the first floor columns. Also, a detailed finite element model of the structure has been established and analyzed. Emphases are focused on shear failure of beams and columns and additional stiffness effects of slabs and brick walls. The simulation results agree well with the in-situ test and the general damage pattern observed in Sichuan earthquake.

Introduction

In pace with the rapid economy growth and technology progress in recent decades in China, seismic design standard for buildings has been modified several times from its draft version in 1959 to the most currently revised one in 2008, that is the Code for Seismic Design of Buildings (GB50011-2001). It is obvious that structures constructed in different times will reflect the design concepts and standards of that age. Seismic damages observed in Sichuan earthquake in 2008 indicates that most structures built after 1990 stand well against the earthquake while those designed based on older version codes are damaged seriously and even collapse (Wang 2008). In view of a large number of aged buildings are still widely on

¹Professor, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, 1239 Siping Road, Shanghai, 200092, P. R. China

²Graduate Student Researcher, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, 1239 Siping Road, Shanghai, 200092, P. R. China

³Associate Professor, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, 1239 Siping Road, Shanghai, 200092, P. R. China

active service in towns and villages all over China, there is a urgent demand, technically and practically as well, on seismic assessment and strengthening to those existing aged buildings for improvement in seismic safety and further reduction of casualties and financial losses in possible forthcoming earthquakes.

It is commonly agreed that aged existing structures are usually weak in seismic safety and may cause serious damage due to their material property degeneration and outdated design standards as well. Post-earthquake observation and investigation can provide detailed information in damage distribution and failure pattern but can hardly tell us how and why it occurs and proceeds. In general, we are still in lack of deep understanding on the damage mechanism for those aged structures. Laboratory scaled-model test is up to now a dominant means to investigate seismic performance of structures. However, a full-scale test will give a more realistic picture of damage and has the advantage of no influence of scale effects. (Zhu 1981) and (Li 1986) carried out in-situ damage test to some ten masonry and framed light-weight panel buildings. More recently, (Corte 2006) and (Corte 2008) performed a series full-scale test on a real RC building including masonry infill panels and FRP strengthening. In this paper, an in-situ pushover test on a real three-story reinforced concrete frame building is reported. The damage process is described and the failure mechanism is analyzed. Meanwhile, a detailed finite element model has been established and a comprehensive numerical simulation has been carried out. Seismic behaviors of the structure are estimated and some suggestion to seismic evaluation of existing structures is presented also.

Description of the Original Building

The original building was a three-story frame structure built in 1983 with story height 4.84m (the first floor), 4.18m (the second floor), and 4.18m (the third floor) respectively. Outside the internal frame there is an external masonry wall system attached. Floors are made by 120mm thick precast concrete slabs. Fig. 1 shows its south-east elevation and the typical structural layout is given in Fig.2.



Figure 1. South-east elevation view

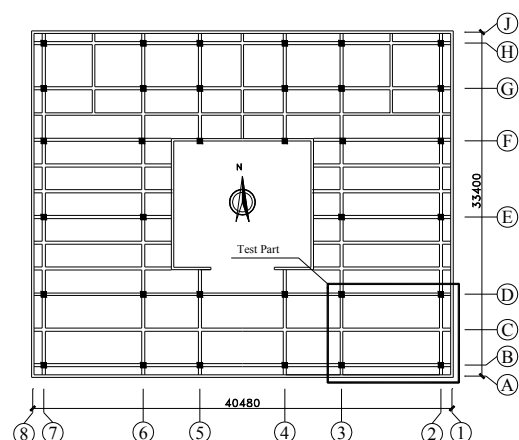


Figure 2. Structural plan of the test building

Details of the Testing Frame

Due to the limitation of in-situ test conditions, a single span, single bay frame cut from the original structure shown in Fig.2 was taken as the test object while the rest part was used as the reacting wall. The testing frame was loaded from north to south. Perimeter walls in axis ① and ④ were preserved, while other internal walls were demolished. Structural layout of the frame and reinforcement of beams and columns are shown in Fig.3. It can be seen in Fig.3 that the test frame presents an irregular configuration due to perimeter walls. It has been checked that sectional reinforcement of beam and column does not meet the requirements of current seismic code. More details of pretreatment for the test can be found in (Lu 2008).

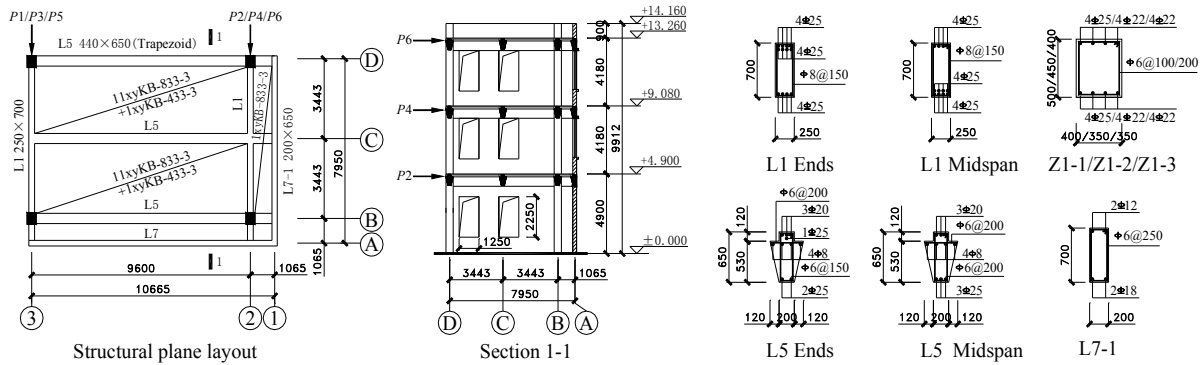


Figure 3. Structural layout and reinforcement of beams and columns

Test Setup and Instrumentation

Decision of Lateral Loading Ratio

In view that the testing frame is low-rise with a regular layout both in its plan and elevation, the conventional inverted triangular acceleration distribution in lateral has been adopted, which can be written as $A_6: A_5: A_4: A_3: A_2: A_1 = 1: 1: 0.69: 0.69: 0.38: 0.38$. Seismic loads are assumed to be proportional to the mass distribution. Thus, the applied load at each hydraulic jack will be the product of the acceleration coefficient $A_1 \sim A_6$ multiplied by the represent concentrated mass $M_1 \sim M_6$ at each loading point. The relative mass distribution factor for the testing frame is calculated as $M_6: M_5: M_4: M_3: M_2: M_1 = 426.37: 308.36: 554.69: 432.06: 600.84: 464.60$. The final lateral loading ratio at each load point $P_1 \sim P_6$ has been determined as $P_6: P_5: P_4: P_3: P_2: P_1 = 1: 0.723: 0.898: 0.699: 0.535: 0.414$.

Layout of Instrumentation

Based on pre-analysis results, hydraulic jacks were placed at six loading points ($P_1 \sim P_6$), and each had a load capacity as: 300KN, 300KN, 500KN, 500KN, 1000KN and 1000KN respectively. At each floor, two displacement transducers were placed to record

displacements of column ③-④ in south-north and east-west directions and one to record displacement of column ②-④ in south-north direction. Flexural and shear deformations at the ends of the beam-column members were monitored by 40 displacement transducers instrumented at the ends of beams and columns in the first and second floor. Strains were obtained on surface of beams and columns by 36 strain gauges. In addition, eight acceleration sensors were used for measuring modal frequencies of the testing frame.

Test Procedure

Pushover test was performed in two phases. In the first phase, the testing frame was under load control with a force incremental of 15kN each step at the hydraulic jack *P6*. When the monitored response displacement became sensitive to the change of load, the corresponding lateral displacement was recorded and test shifted to the second phase. In the second phase, the testing frame was under displacement control with a displacement incremental of 20mm each step at the hydraulic jack *P5* while force ratio for other loading points kept unchanged as that used in the first phase.

Pushover Test Results

Cracking and Failure Pattern

In the first 5 load steps, no visible cracks were observed on the surface of major structural members. At the load step 14, an inclined crack appeared on the East side of upper end of column ③-④ in the first floor and the total base shear reached its maximum value of 861KN. At the load step 15, cracks expanded suddenly with a blare and the shear failure occurred as shown in Fig.4. After that, loading at points *P2*, *P4* and *P6* maintained increasing proportionally while oil pressure of jacks at points *P1*, *P3*, *P5* kept unchanged. As shown in Fig.5, at the load step 23, shear failure again suddenly occurred on upper end of column ②-④ in the first floor. The testing frame became unstable. For safety, the test was terminated. Comparing Fig.4 and Fig.5, it can be seen that the crack pattern in two columns are similar. Fig.6 shows the final crack development in column ③-④ in the first floor. Crash of concrete cover on the northern upper end of the column ②-④ in the first floor is shown in Fig.7. Fig.8 and Fig.9 show the failure characteristic of the wall ① in the first floor, there are inclined shear cracks as well as horizontal cracks between windows.

The overall features of structural failure are summarized as follows: (1) due to asymmetry of perimeter walls, torsional deformation of the structure was distinctive; (2) structural damages in upper floors were not so serious as in the first floor and the most serious damage occurred in bottom columns; (3) damage in columns was more serious than in beams; (4) the existence of ledger beams and casting concrete layers made precast slabs much stronger in its integrity and this prevented them from cracking even in the final stage of the test; (5) the final breakdown of the structure would be of brittle type characterized by shear

failure of the first floor columns.



Figure 4. Shear failure of column ③-⑧ at the load step 15



Figure 5. Shear failure of column ②-⑧ at the load step 23



Figure 6. Shear failure of column ③-⑧ at the load step 23



Figure 7. Crushing of column ②-⑧ at the load step 15



Figure 8. Inclined crack in wall ① at the load step 23



Figure 9. Horizontal crack in wall ① at the load step 23

Dynamic Characteristics

Natural frequencies of the testing frame were obtained using pulsating method. It is summarized as following: (1) before test, frequencies of the first three vibration mode were 2.2 Hz (torsion), 3.3Hz (E-W transition) and 6.1Hz (torsion) respectively; (2) after installation of jacks, they became 2.59 Hz (torsion), 3.52Hz (E-W transition) and 6.69Hz (torsion) respectively. It was because that jacks made the testing frame in connection with the reacting part and thus increased its stiffness; (3) after test, a marked decrement in vibration frequencies were observed. The corresponding frequency values were 1.66 Hz (torsion),

2.29Hz (E-W transition), and 3.96Hz (torsion) with a decrement by 24.54%, 30.61%, 35.08% respectively, which indicated that the test frame suffered severe damage.

Story Displacement and Torsion

Fig.10 shows the measured base shear force versus the Story lateral displacements at each loading point. Due to a successive shear failures of column ③-Ⓑ and column ②-Ⓑ, the response curves in soften stage for the testing frame behave sudden and unstable. The maximum displacement of the roof at point P5 is 171.14mm and, in contrast, that at point P6 is 111.14mm. Comparing the two figures, it might be concluded that, frame ② was stiffer than frame ③ because of the presence of perimeter walls while the latter had more displacement ductility than the former.

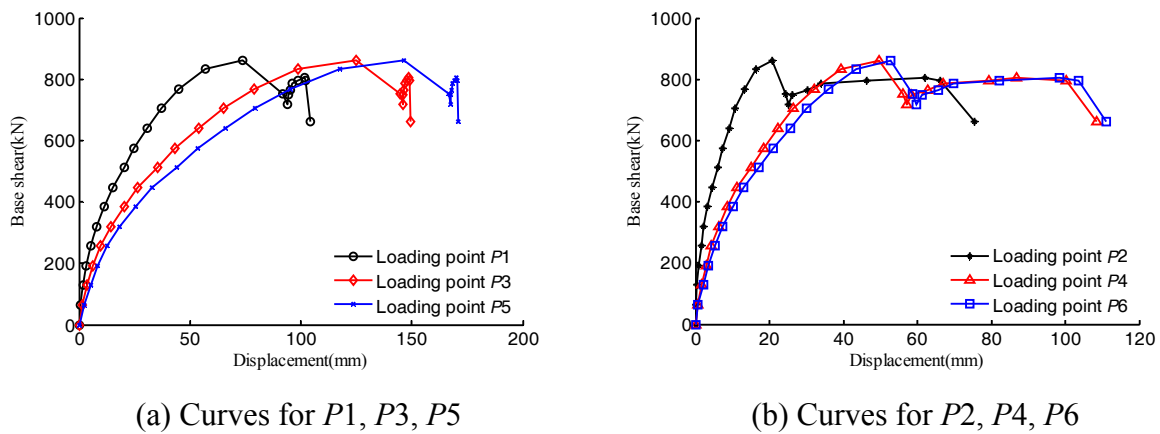


Figure 10. Base shear - displacement curves

Story drift angles at each load step are shown in Table 1. As damage was mainly occurred in the first floor, the first Story drift angle was 1/53 at the 15th load step and 1/47 at the 23rd load step in S-N direction. Due to the irregularity of perimeter walls, an obvious Story drift angle was also stimulated in E-W direction.

Table 1. Story drift angle at each load step

Load step	S-N direction			E-W direction		
	First floor	Second floor	Third floor	First floor	Second floor	Third floor
1~5	1/622	1/657	1/1052	1/1222	1/1292	1/4200
6~9	1/198	1/229	1/412	1/385	1/445	1/1963
10~12	1/109	1/123	1/260	1/209	1/231	1/1728
13~15	1/53	1/79	1/192	1/93	1/176	1/378
16~19	1/51	1/83	1/194	1/101	1/191	1/364
20~23	1/47	1/93	1/194	1/240	1/243	1/379

For quantitative measuring of torsional response for the testing frame, the torsional irregularity coefficients have been calculated as shown in Table 2. It is observed that torsional

irregularity coefficients are all greater than 1.2, that is the permissible value specified by (JGJ 3-2002), and reached a maximum 1.787 in the first floor. It is also found that torsional irregularity coefficient in the first floor is always greater than that in upper floors. It might be concluded that the rigidity of perimeter walls caused an obvious shift of the stiffness center from the mass center especially in the first floor.

Table 2. Torsional irregularity coefficients

Load step	The first floor	The second floor	The third floor
1~5	1.787	1.381	1.498
6~9	1.546	1.405	1.435
10~12	1.550	1.424	1.448
13~15	1.580	1.440	1.478
16~19	1.576	1.437	1.474
20~23	1.362	1.301	1.346

Finite Element Simulation

FE Model

A finite element model for the testing frame has been established for post-test simulation purposes by using ABAQUS package, in which beams and columns are modeled by C3D8R solid elements, floors and walls by S4R shell elements, and steel bar by T3D2 truss elements. Shell-to-solid couple technique has been employed to simulate the interactions of wall-to-beam and floor-to-beam. The damage plasticity model is adopted for concrete with a skeleton curve recommended by (GB 50010-2002). An equivalent ‘concrete’ model has been applied to brick masonry with a skeleton curve proposed by (Liu 2008). There is evidence that floors can be assumed in elasticity. All material properties used in computation take the mean value measured in the test. Fig.11 shows the finite element model in detail.

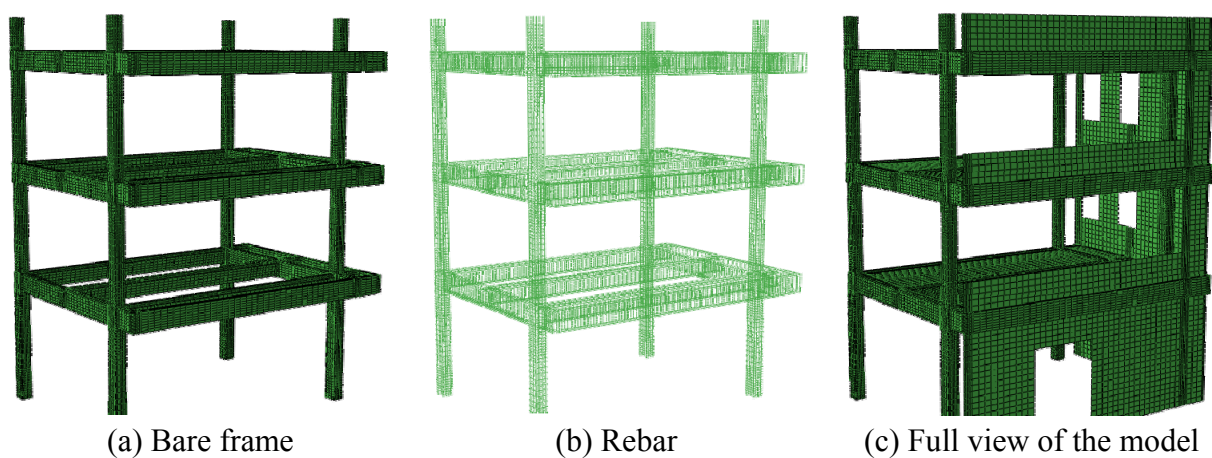


Figure 11. The Computational Model for the test building

Comparisons

The first three modal frequencies are listed in Table 3. They compare well with those measured in the test.

Table 3. Comparison of dynamic characteristics

Mode No.	Test (Hz)	Numerical analysis (Hz)	Error
1	2.2	2.08	5.5%
2	3.3	2.66	19.4%
3	6.1	5.04	17.4%

A preliminary computation has been performed using the established FE model. The loading pattern used in pushover computation follows the similar way as that in test. Fig.12 shows base shear-displacement curves at loading points *P5* and *P6* respectively. There are some difficulties in simulating post-ultimate behavior of the testing frame by using a loading program following the test proceeding. The arc length method might be considered in the further computation. However, the extreme values of base shear compare well with that tested one.

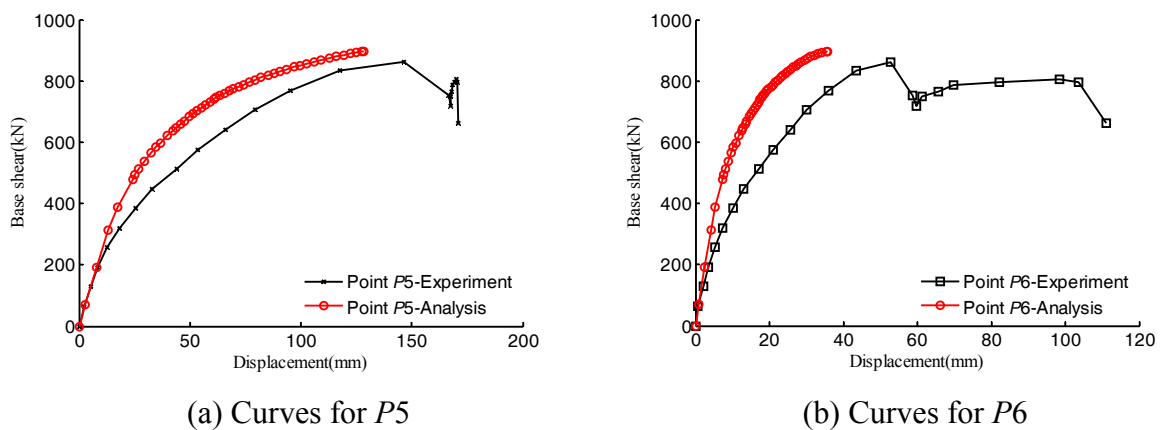


Figure 12. Base shear-displacement curves and Comparison

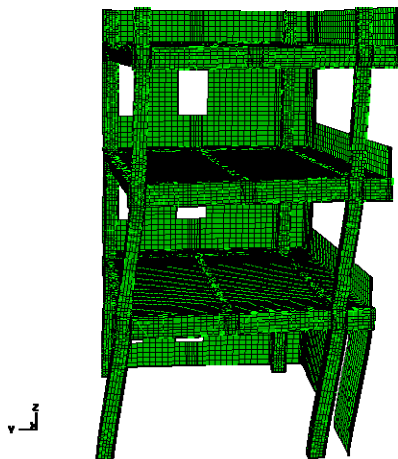


Figure 13. Deformed shape of the structure

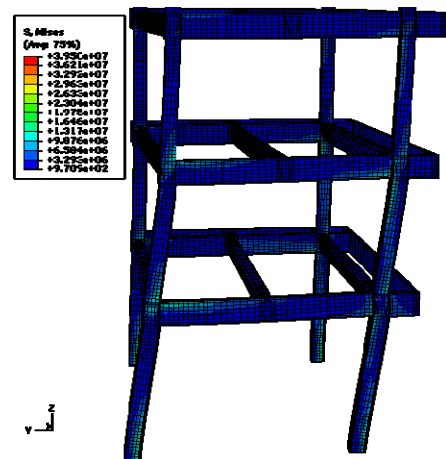


Figure 14. Stress distribution in concrete

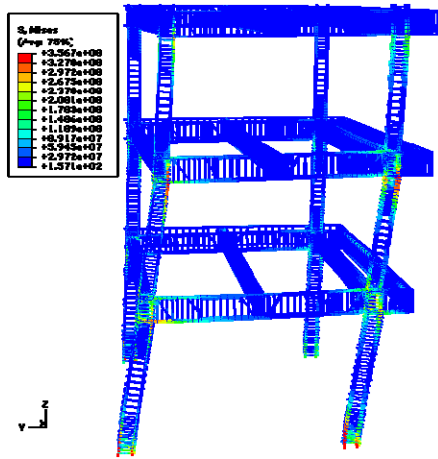


Figure 15. Stress distribution in rebar

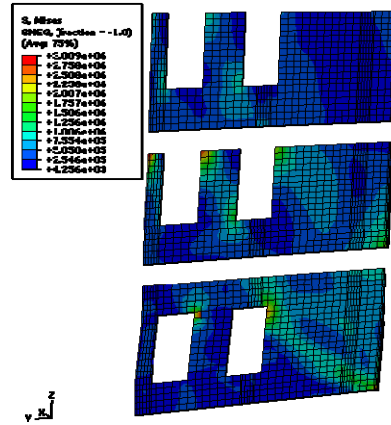


Figure 16. Stress distribution in brick wall

It can be seen in Fig.13 that the structure is undertaken an obvious torsional deformation due to the participation of the perimeter walls. Figs.14 shows the Mises stress distribution in concrete at the last load step. Fig. 15 indicates that some longitudinal reinforcement yield at this load step, they occur at the end of columns on axis ③. Fig.16 shows the stress level in walls with a maximum value about 3.0MPa, which is far beyond the possible strength stress of the equivalent material property.

Conclusions

Based on the field testing result, columns in the axis ② and ③ at bottom floor were in brittle shear damage, while damage in other members were not obvious. There was no significant plasticity hinge appear. Damage to the structure can be concluded as follows: (1) the discipline of strong in shear and weak in bending for columns is not fulfilled; (2) the discipline of strong in column and weak in beam for the frame structures is not fulfilled. (3) Columns at the bottom floor is weak to lateral loads; (4) structural failure appears in a clear pattern of brittle shear damage and this may cause danger of inevitably collapse under serious earthquake.

A number of suggestions for seismic evaluation on this kind of existing buildings can be raised: (1) more attentions should be pay to the construction measure of beams and columns since appropriate construction measure provides guarantee for improving structural ductility; (2) an appropriate evaluation on the shear resistance of columns especially those base columns are necessary for existing structures because there are always shear resistance defects due to small sectional size and insufficiency of shear stirrup.

For existing aged structures, the effect of masonry wall on stiffness and deformation of the frame need to be considered. Due to the unsymmetrical arrangement of the wall there is always stiffness eccentricity in the structure and this may cause torsional response, which is most harmful to the structure. The strengthening effect of floor slab to beams should be taken into account because the combination deformation of beams and slabs may affect the relative stiffness of beams to columns. Thought the effect of slabs to beams at elastic stage

has been considered in the current design code, efforts need to be extended to nonlinear stage in further.

Acknowledgments

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