INVESTIGATION ON SEISMIC BEHAVIOR OF JOINTED PRECAST CONCRETE FRAME STRUCTURES

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ABSTRACT:
The seismic behavior of precast concrete frame structures is greatly influenced by the beam-to-column connections, which can be classified into emulation monolithic connections and jointed precast connections. During the past twenty years, the studies in jointed precast connections were much less than those in emulation monolithic one. The pseudodynamic tests of two jointed precast concrete frame structures including one frame with one floor and double spans, and another frame with one floor, single span, and three bays have been carried out to investigate the seismic behavior of jointed precast concrete frame structures on failure mechanism, strength, ductility, hysteretic characteristics and energy dissipation. The precast beams and columns were assembled together with a prestressed bolt and a rubber cushion was inserted between beam and column in the connections. The test results showed that this kind of structures can provide satisfactory seismic performance. When drift angle arrived at 1/25 the structures still held enough carrying capacity. The beam-to-column connections with rubber cushion and bolt worked well and the failure pattern of the structures was ductile with column bending failure, and the displacement ductility factor was larger than 3.5. Finally, a nonlinear-elastic model was adopted to represent the jointed connection in numerical simulations of the structures. By comparing the results of tests and simulations, the nonlinear-elastic model was proved properly to represent this kind of jointed connection.

KEYWORDS: precast concrete, frame, seismic behavior, pseudodynamic test, jointed precast connection

1. INTRODUCTION

Owing to a variety of advantages including: increasing productivity, improving products' quality, reducing environmental disturbance, and contributing to sustainablility, precast concrete has been widely applied in the world. Precast concrete structures can be classified into emulation monolithic precast concrete structure and jointed precast concrete structure by NEHRP2000 [1] and ACI318-02 [2]. The investigation of seismic behavior of precast concrete structures commenced in 1970s. However, studies in the jointed connections and structures has been much less than those in the emulation monolithic one during the past twenty years [3]. The international cooperation program "Seismic Behavior of Precast Concrete Structures with Respect to Eurocode 8" initiated by European Community aimed to investigate seismic behavior of the jointed precast concrete structures and to offer help for modification of Eurocode8 [4]. As one of the participants, Tongji university has taken in hand a series of testing works including cyclic tests on jointed connections [5], pseudodynamic tests and shaking table tests on jointed precast concrete frame structures. This paper is about pseudodynamic tests.

2. DESCRIPTION OF TEST MODELS

The prototype of test model was industry buildings with jointed precast concrete frame structures, which was widely used in Europe. Two models were designed, one named SJ1 is a one floor, double spans and one bay frame, another named SJ2 is one floor, single span and three bays frame with roof slabs. Components prefabricated before the test were assembled together and fixed in the test table. The beams were fixed to the top of the columns employing jointed connections with bolts and rubber cushions. In SJ2 model the roof slabs were fixed in the beam through welding of the four slab corners. The test's layout was shown in Figure 1.

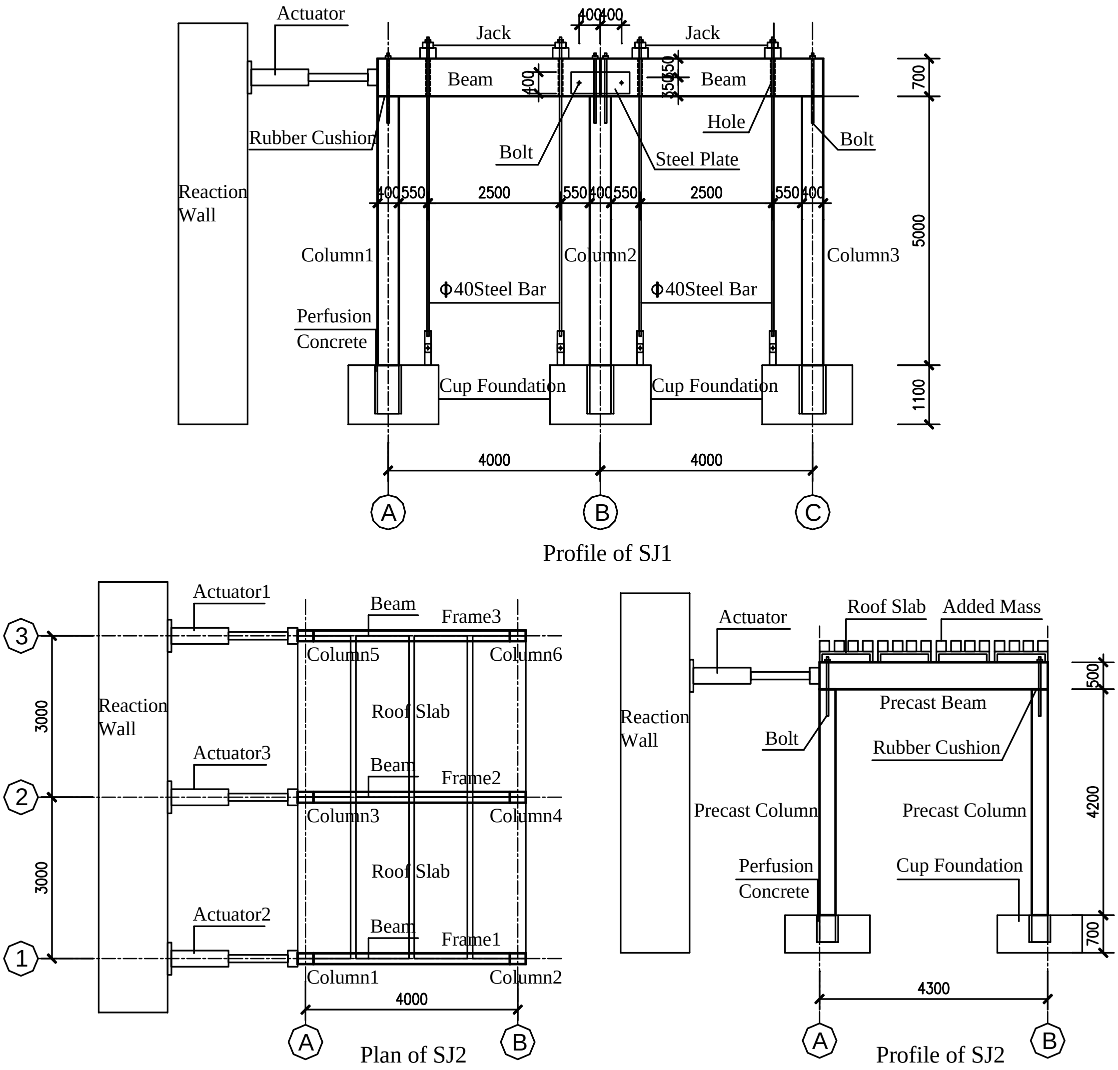


Figure 1 Test layout

The actual material properties were shown in Table1 and Table2. The rubber cushions used in the jointed connections were natural rubber with A65 hardness and 8mm thickness. The SJ2 model was reduced to 1/2 scale of prototype and was designed according to the dynamic similitude relationship [6]. Reinforcements of columns and beams were shown in Figure 2 and beam-to-column and slab-to-beam connections were shown in Figure 3. Four jacks and anchor bars were employed in SJ1 and added mass blocks were employed in SJ2 to ensure that the axial compressive stress of columns in model is equal to that in prototype.

Table 1 Properties of concrete

Concrete	f_{cu} (MPa)	f_c (MPa)	E_c (MPa)
Model SJ1	44.9	38.5	3.57×10^4
Model SJ3	42.8	35.3	3.21×10^4

Note: f_{cu}—Compressive strength of concrete cube, f_c—compressive strength of concrete,
E_c—Modulus of elasticity of concrete.

Table 2 Properties of rebar

Reinforcement	Diameter of reinforcement	$f_y(MPa)$	$f_b(MPa)$	$E_s(MPa)$	$\varepsilon_y(10^{-6})$
Rebar of SJ1	16mm	397.9	586.9	2.00×10^5	1990
Stirrup of SJ1	6mm	401.2	592.4	2.01×10^5	1996
Rebar of SJ2	10mm	359	533	2.00×10^5	1795
Stirrup of SJ2	4mm	382.5	416.5	2.05×10^5	1866

Note: f_y —Yield strength of reinforcement, f_b —Ultimate strength of reinforcement,
E_s —Modulus of elasticity of reinforcement, ε_y —Yield strain of reinforcement.

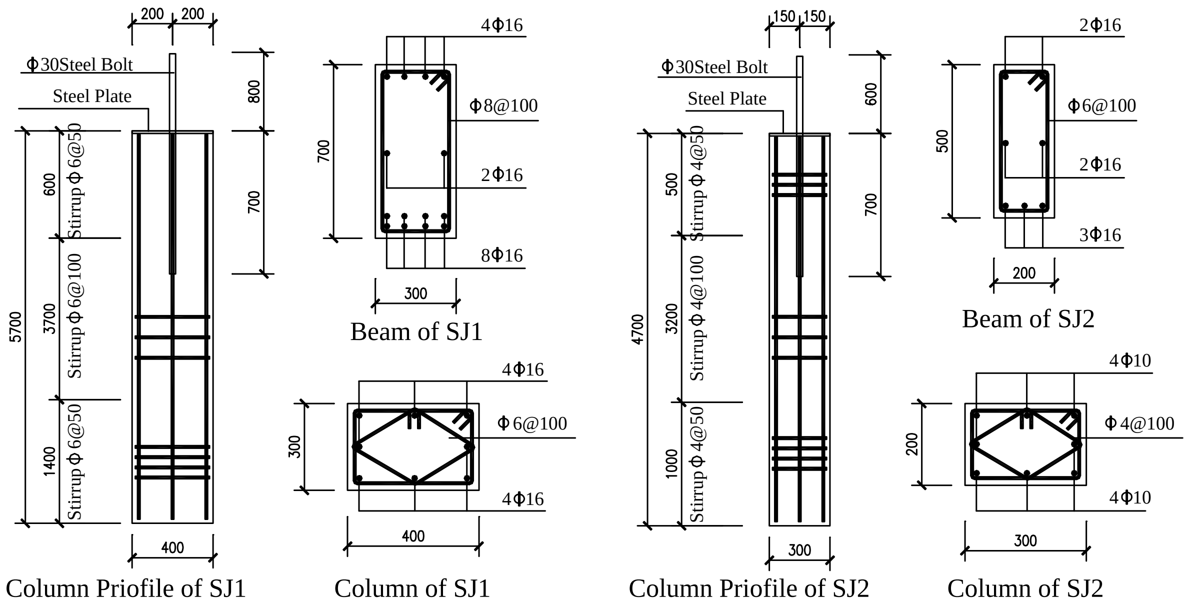


Figure 2 Reinforcements of columns and beams

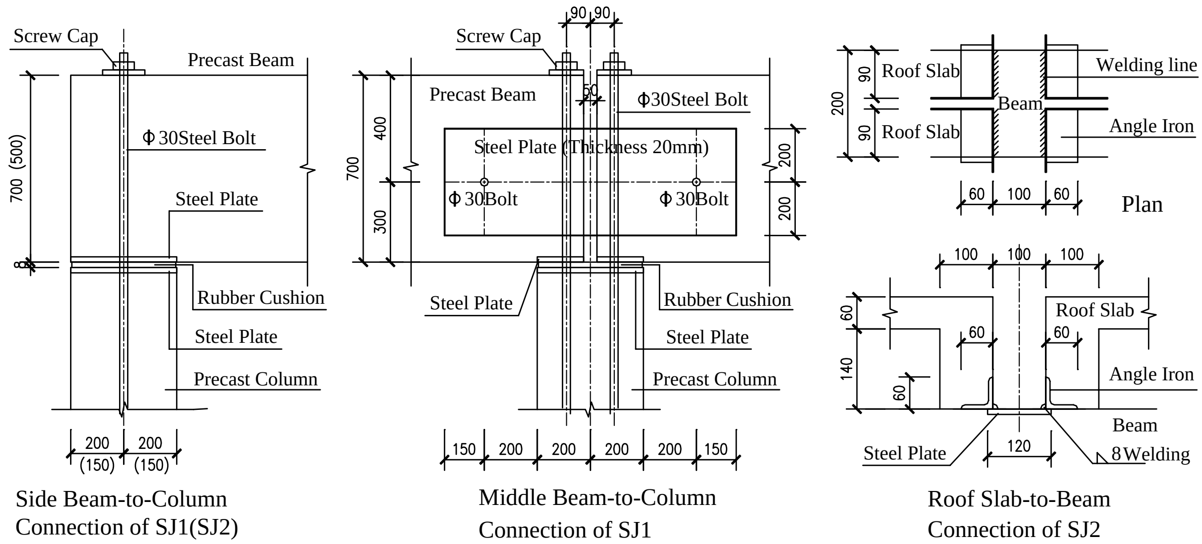


Figure 3 Details of beam-to-column and slab-to-beam connections

3. TEST LOADING

Tolmezzo earthquake wave was chosen as loading wave, which shared the similar response spectrum with Eurocode8 site 1B. Standardized Tolmezzo wave and the response spectrums were shown in Figure 4. Test loadings were determined according to the estimated ultimate loadings. SJ1 included six loadings: 0.05g, 0.11g, 0.23g, 0.39g, 0.47g, 0.59g. SJ2 included six loadings: 0.05g, 0.1g, 0.2g, 0.4g, 0.6g, 0.8g.
SJ2 was regarded as two DOF system through keeping the displacement of actuator1 and actuator2 same and ignoring the global torsion before the roof slab-to-beam connection was destroyed. After the roof slab-to-beam connection destroyed, the SJ2 regarded as SDF system by keeping the three actuators hold the same displacement.

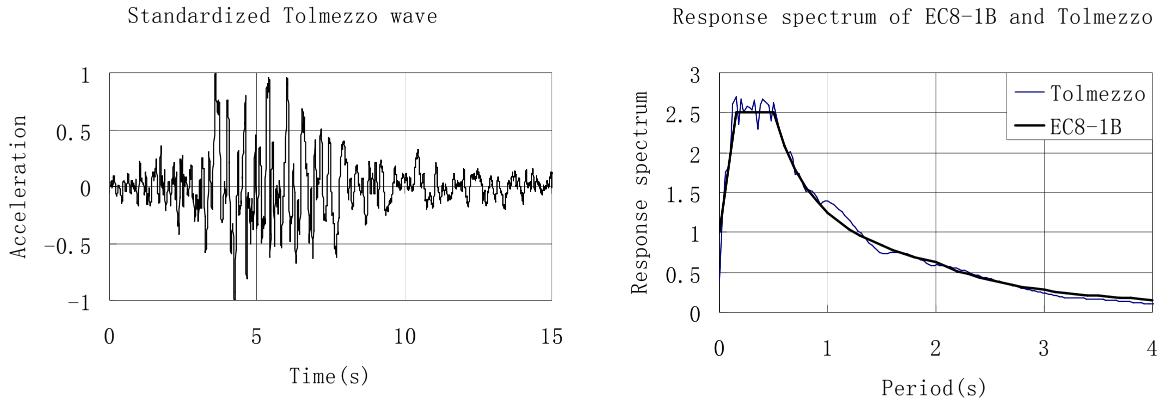


Figure 4 Standardized Tolmezzo wave and the response spectrum

4. TEST RESULTS

4.1 Test Phenomena

SJ1 model: in 0.11g test, visible cracks in the bottom of column were found for the first time and closed when load was taken off. In 0.23g test, residual deformation appeared for the first time. In 0.35g and 0.47g tests, visible cracks were distributed in the range of 1800mm high of column from the base with the 1.8mm maximal width. In 0.59g test, visible cracks were distributed in the range of 2400mm high of column from the base with the 2.2mm maximal width. Model could not revert to the original state when load was taken off. In the test, rubber cushions were extruded from the connection (shown in Figure 5), but the beam-to-column connection maintained well after the test.

SJ2 model: in 0.1g test, visible cracks in the bottom of column were found for the first time and closed when load was taken off. In 0.2g test, visible cracks were distributed in the range of 800mm high of column from base with the 0.7mm maximal width. Breakages in the roof slab-to-beam connections appeared for the first time. In 0.4g test, when test arrived at 2.71s, most roof slab-to-beam connections were destroyed and the maximal slippage between slab and beam arrived at 100mm. In order to avoid roof slab falling off, the test was ceased. Next, the model was regarded as SDF system and was retested. Visible cracks were distributed in the range of 1600mm high of column from base with the 1.2mm maximal width. The model could not revert to the original state after the test. In 0.6g and 0.8g test, visible cracks were distributed in the range of 2000mm high of column from base with the 2mm maximal width and maximal residual deformation of column was 35mm. The breakages of column bottom after test was shown in Figure 5.

Figure 5 Extruded rubber cushion and breakages of column bottom

4.2 Displacements and Hysteretic Curves

The maximal displacements and drifts of SJ1 and SJ2 models in each test were shown in Table 3. Some displacement time history and load-displacement hysteretic curves were shown in Figure 6 and Figure 7.

Table 3 Maximal displacements and drifts of SJ1 and SJ2

SJ1				SJ2			
Test	Displacement (mm)		Drift	Test	Displacement (mm)		Drift
	Push	Pull			Push	Pull	
0.05g	10.2	11.7	1/427	0.05g	6.7	8.3	1/506
0.11g	24.8	24.2	1/202	0.1g	12.9	16.0	1/256
0.23g	53.0	54.8	1/91	0.2g	37.9	31.9	1/109
0.35g	136.6	112.3	1/37	0.4g	91.2	65.1	1/46
0.47g	188.6	138.8	1/27	0.6g	112.9	110.5	1/37
0.59g	188.0	208.1	1/24	0.8g	135.1	173.2	1/24

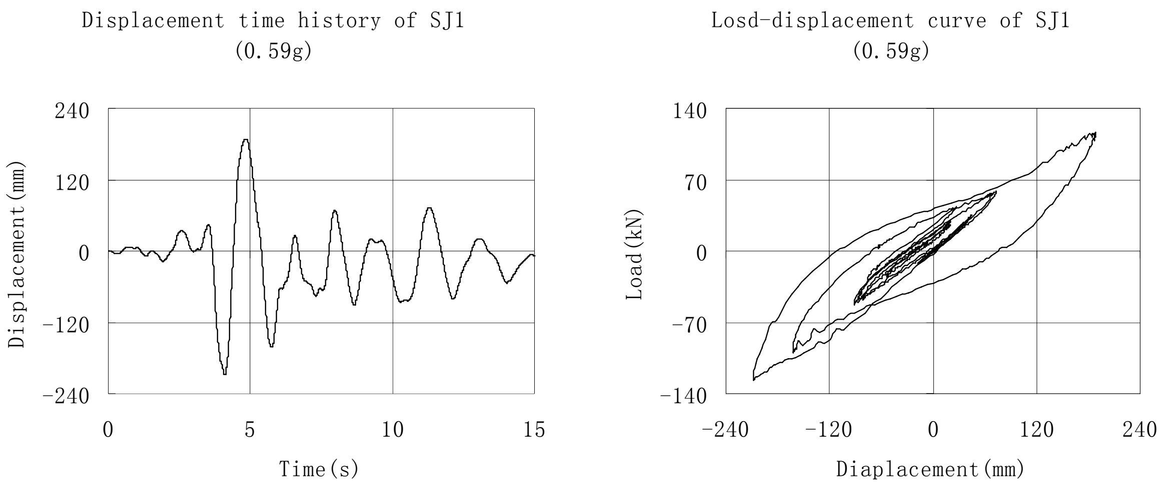


Figure 6 Displacement time history and Load-displacement curve of SJ1 (0.59g)

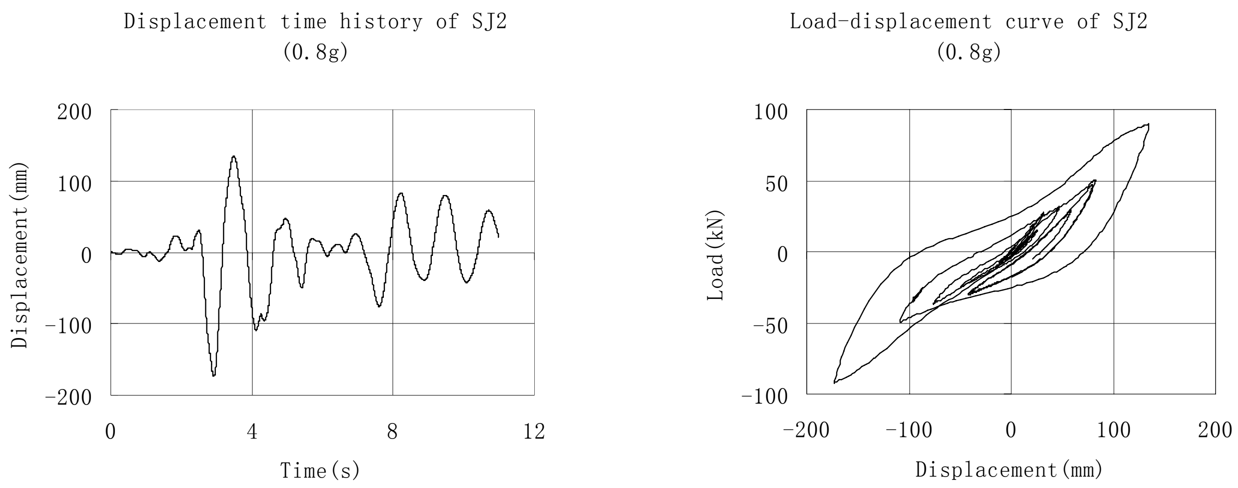


Figure 7 Displacement time history and Load-displacement curve of SJ2 (0.8g)

4.3 Energy Dissipation

The energy dissipations in each test, which were calculated according to the load-displacement hysteretic curves, were shown in Figure 8.

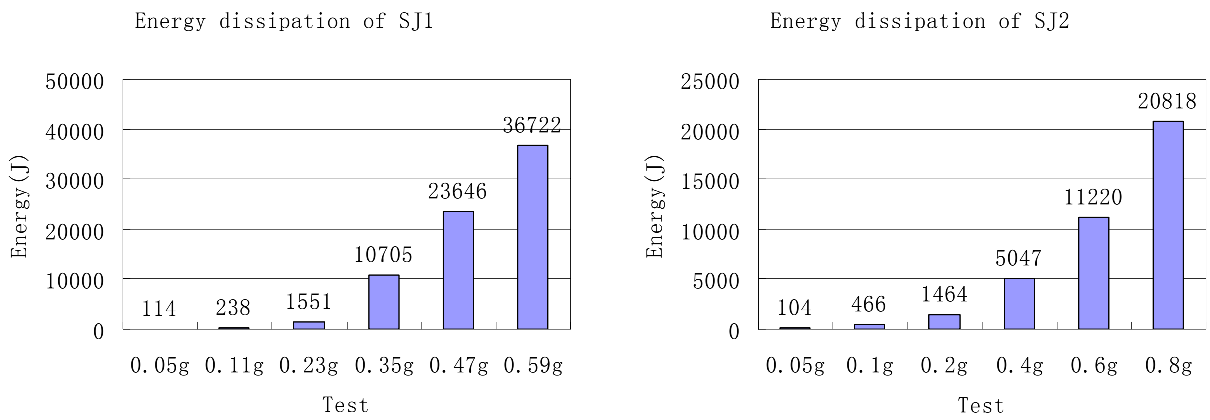


Figure 8 Hysteretic energy dissipations of each test

4.4 Envelop of Load-displacement curves and Ductility

The envelop of load-displacement curves of SJ1 and SJ2 were shown in Figure 9. The displacement ductility factor can be calculated according to Eqn. (4.1) as below.

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{4.1}$$

Where, μ is displacement ductility factor, Δ_u is ultimate load, Δ_y is yield load. Displacement ductility factors were calculated and shown in Table 4.

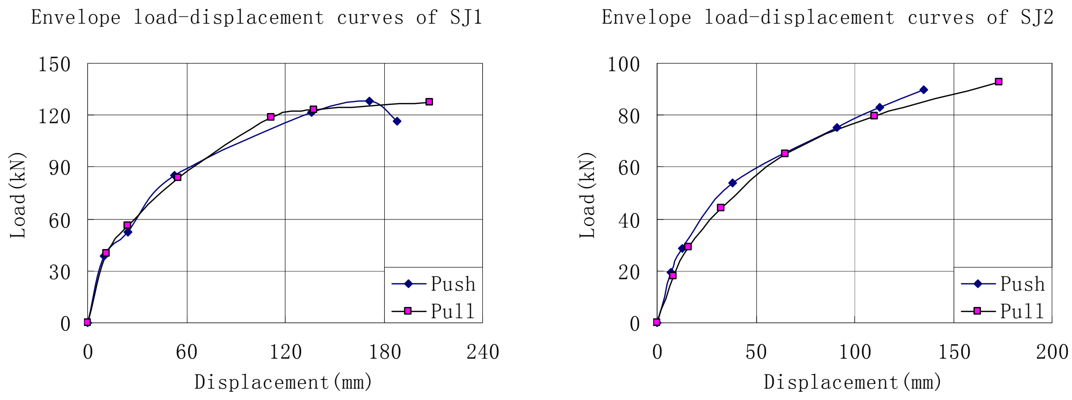


Figure 9 Envelope of load-displacement curves

Table 4 Ductility of the frame

	Model SJ1		Model SJ1	
	Push	Pull	Push	Pull
Yield displacement Δ_y (mm)	53.0	54.7	38.12	32.22
Ultimate displacement Δ_u (mm)	188.0	208.1	135.69	173.22
Ductile factor $\mu = \Delta_y / \Delta_u$	3.55	3.80	3.56	5.38

5. NUMERICAL SIMULATIONS

Resorting to Strand7 software numerical simulations of the tests were carried out. By applying the nonlinear-plastic restoring moment-curvature relationships of beams and columns which were calculated by USC_RC and the nonlinear-elastic restoring moment-rotation relationships of jointed precast connections which were calculated by the connection numerical simulation employing the entity element model, the nonlinear transient dynamic analysis of each test were conducted. Some comparisons of displacement time history and load-displacement hysteretic curves of the tests and numerical simulations were shown in figure 10, 11. It can be seen that the results of the numerical simulations were close to the test results, and the nonlinear-elastic model was proved reliable for this kind of jointed precsat connection.

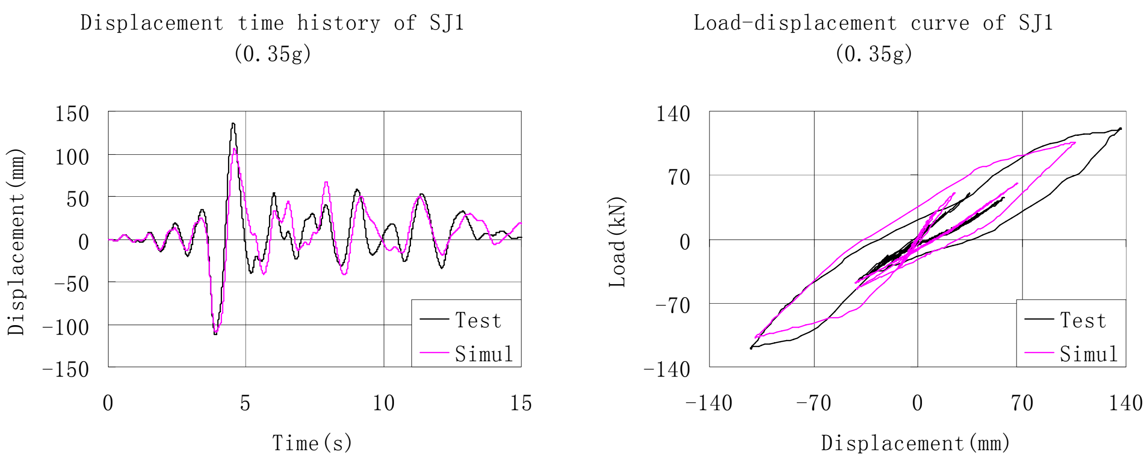
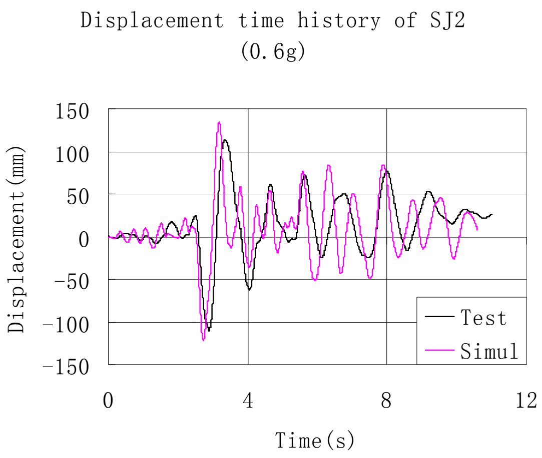


Figure 10 Comparison of test and numerical simulation results (SJ1, 0.35g)



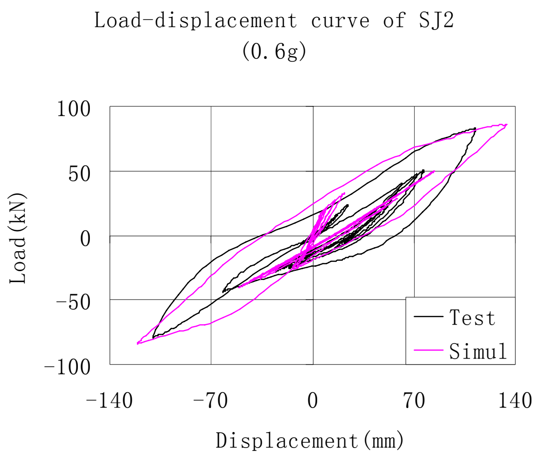


Figure 11 Comparison of test and numerical simulation results (SJ2, 0.6g)

6. CONCLUSIONS

1) Test results indicated that the failure pattern of the jointed precast concrete frame structures was ductile with column bending failure, and the displacement ductility factor was larger than 3.5.
2) Jointed precast concrete frame structures can provide enough energy dissipation and residual loading capacity. When drift angle arrived at 1/25, the structures still held enough carrying capacity without sharp collapse and could restore to the original state without overmuch residual deformation.
3) The beam-to-column connections with rubber cushion and bolt worked well without visible damage. However, the thickness of the rubber cushion should be chosen carefully. Too thin rubber cushion will cause the collision of beam and column and too thick rubber cushion will cause large shear deformation between beam and column.
4) In numerical simulation, nonlinear-elastic model can be adopted to represent this kind of jointed precast connection, the model parameters should be decided by connection analysis or connection test.
5) The roof slab-to-beam welding connections being destroyed firstly during the test may be attributive to the weakness of this sort of structures. It is suggested to strengthen the slab-to-beam connections or to employ flexible joints for the roof slab-to-beam connections.

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