Seismic performance of a composite frame structure

Seismic Performance of a Composite Frame Structure

Toshihiko YAMAMOTO
Professor of Civil Eng.
Daido Inst. of Technology
Nagoya, Aichi, Japan

Toshihiko Yamamoto, born 1947, received his doctor degree from Nagoya University 1989. He is a member of AIJ Committee, Composite Structure.

Takeshi OHTAKI
Researcher of Civil Eng.
Inst. of Tokyo Construction
Sagamihara, Kanagawa, Japan

Takeshi Ohtaki, born 1961, received his MEng from Kyoto University 1986. He has worked on structural dynamics.

Summary

In order to investigate the elasto-plastic behavior of composite frame structure consisting of steel beams and reinforced concrete columns, a full-scale three story and two bay (2.8m story height and 5.5m beam span) frame was tested under cyclic lateral loading. The response of the structure was quite ductile and reached its mechanism with plastic hinges at the beam ends and the first story column bases. No serious damage in beam-column joints except some outer beam-column joints were observed throughout the test.

1. Introduction

The composite frame structure consisting of steel beams and reinforced concrete columns is a structural system which utilizes steel and concrete materials effectively and expected to bear the needs for saving cost and labor for construction. In spite of the advantage, it is important to know the seismic performance of this type of structure when built in seismic zones. In order to demonstrate the elasto-plastic behavior of the composite frame structure, a full-scale three story two bay (2.8m story height and 5.5m beam span) specimen was tested.

2. Experimental Program

2.1 Specimen

Fig.1 shows the full-scale three story two bay specimen which has 2.8m story height and 5.5m beam span. The specimen was designed to provide a mechanism with plastic hinges at steel beams and reinforced concrete first story columns based on the AIJ Structural Design Guidelines[1]. The column and beam sections of the specimen are shown in Fig.2. The column had a cross section of 550mm x 550mm and a 2300mm clear height. The longitudinal and lateral reinforcement ratios of the column were 2.54% and 0.92%, respectively. The section of the steel beam was BH400x150x9x12. In order to prevent lateral buckling of the beam, three lateral small beams were implemented based on the Ultimate Structural Design for Steel Buildings[2]. The thickness of the slabs was 100mm and connected to the beams with stud bolts. The beam-column joints were designed based on the previous test results[3] as shown in Fig.3. Cover plates with shear cotter were used to provide enough bond between concrete and steel in the joints[3].

The mechanical properties of the concrete and steel are shown in Table 1 and 2. The specified concrete strength was 21MPa.

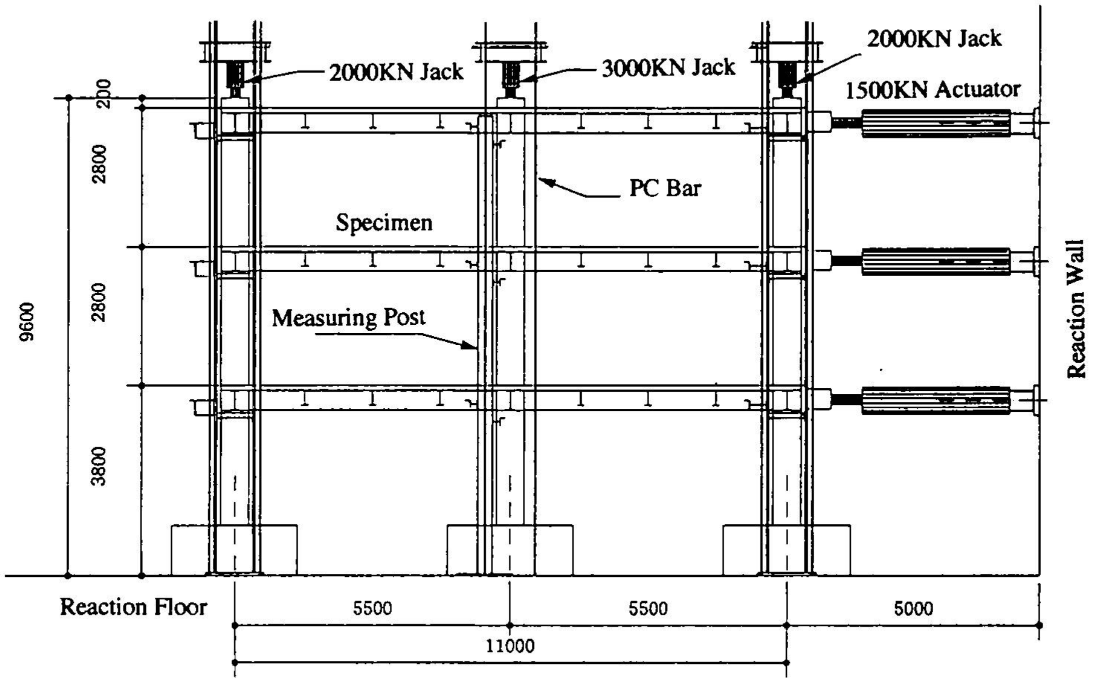


Fig.1 Specimen

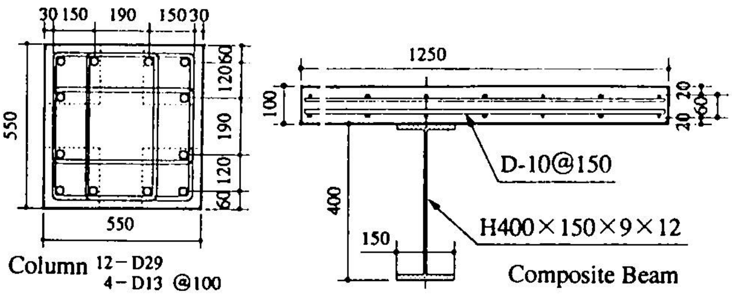


Fig.2 Column and Beam Section

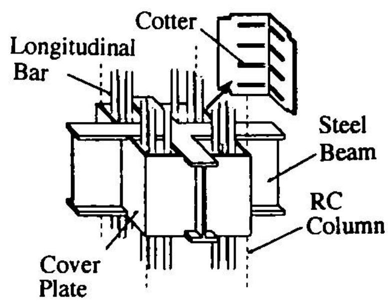


Fig.3 Joint Details

2.2 Test Procedures and Measurements

Prior to the cyclic loading test the natural frequencies, stiffness matrix and flexibility matrix of the frame were measured to examine the elastic behaviors of the structure. The cyclic loading test under inverted triangle load distribution was conducted subsequently. The axial load applied to the top of the inner column was 1600KN and the axial force ratio was 0.25. The load applied to the outer columns was 1270KN and the ratio was 0.20. The lateral shear load was applied to

Position	Fc [MPa]	Ft [MPa]	Ec [GPa]
1FColumn,2FSlab	32.5	2.75	28.1
2FColumn,3FSlab	34.9	2.75	28.3
3FColumn,RFSlab	34.4	2.92	28.9

Fc,Ft:Compressive, Tensile Strength
Ec:Young's Modulus

Table 1 Properties of Concrete

Position	Fy [MPa]	Ft [MPa]	Es [GPa]
Steel Beam Flange	285	441	210
Column Long. Bar	400	577	197
Column Hoop	319	493	177
Slab bar	376	520	191

Fy,Ft:Compressive, Tensile Strength Es:Young's Modulus

Table 2 Properties of Steel

the each floor level with three actuators. The load distribution through three floors was maintained inverted triangular shape from maximum load at RF to zero at base floor. The cyclic loading sequence applied to the specimen was a displacement control defined as the relative deformation angles(R) between base floor and RF, single cycle at 1/1000 and two cycles at 1/400, 1/200, 1/100, 1/50 before reaching ultimate strength.

3. EXPERIMENTAL RESULTS

3.1 Natural Frequencies and Stiffness and Flexibility Matrices

Measured natural frequency of the first mode under non axial force condition was 5.7Hz. The calculated first mode natural frequency obtained from the stiffness and flexibility matrices was about 5.9Hz which gave a good agreement with the directly measured natural frequency.

3.2 Elasto-Plastic behavior

The test results are summarized in Table 3 and the crack pattern at the final loading stage is shown in Fig.4. First flexural cracks were observed in slabs during loading stages of R=1/1000 and 1/800 started from 2F, 3F to RF and propagated rapidly in the following loading stages. Flexural cracking occurred at R=1/600 and R=1/300 in the first story columns and 2nd-3rd story columns, respectively.

Sequence of plastic hinges are shown in Fig.5. The plastic hinge formation was defined as the yielding of column reinforcing bars or beams or slab reinforcing bars. The cracking load agreed well with the prediction. The second floor steel beams yielded at compression side at R=1/240 and the other floor beams except RF beams yielded at compression or tension side between R=1/200 and 1/100. Finally the RF beams and the first floor columns yielded between R=1/100 and 1/75 and formed a total yield mechanism categorized by beam-yield type. After the formation of the mechanism, local buckling was observed in the 2nd and 3rd floor beams at R=-1/60. However beam-column joints did not collapsed.

3.3 Force-Displacement Relationship

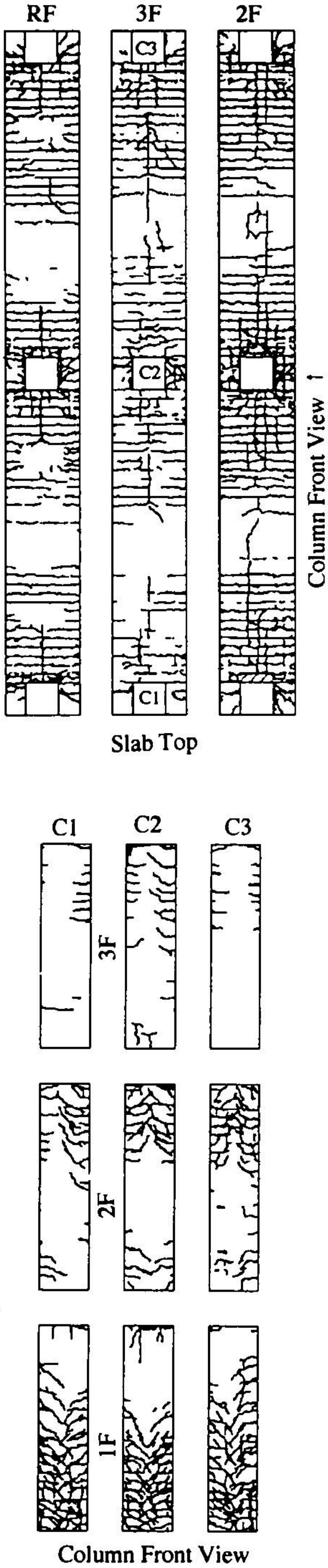
Force-displecement relationships are shown in Fig.6. The frame response showed no strength degradation due to cyclic loading at the same displacement angles. Although the hysteresis loops exhibited slightly pinched shape until R=1/100, they showed quite hysterical energy absorption at R=1/50.

The lateral force at the mechanism agreed well with the calculated strength using virtual work method. The relative story drifts at 3rd, 2nd and 1st story were 35.5mm(R=1/79), 45.0mm(R=1/62) and 31.5mm(R=1/89), respectively. The responses of each story of the specimen showed ductile behavior even after the mechanism formed and the frame capacity did not decrease until R=1/25.

The skeleton curves obtained by the elasto-plastic analysis is also shown in the figures with dotted line. In the analysis, the end spring member model considering rigid zone was used. The column and beam were modeled to have tri-linear characteristics as presented in Table 4. The

Displacement Angle R	Remarks	Story Shear[KN]						Story Drift[mm]					
		RF		3F		2F		RF		3F		2F	
		Exp.	Cal.	Exp.	Cal.	Exp.	Cal.	Exp.	Cal.	Exp.	Cal.	Exp.	Cal.
1/1000	Slab Cracking	104	-	166	-	195	-	2.8	-	3.5	-	2.1	-
1/600	Column Cracking	158	148	254	246	300	292	4.9	3.3	5.7	3.8	3.4	2.1
1/240	Beam Yielding	314	355	519	587	616	698	11.4	8.6	13.6	10.9	8.9	7.9
1/75	Mechanism	638	718	1059	1186	1255	1412	35.5	43.8	45.0	59.4	31.5	50.1

Table 3 Results of Test



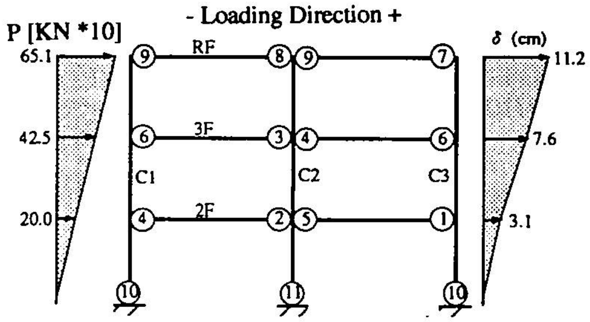


Fig.5 Sequence of Plastic Hinge Formation

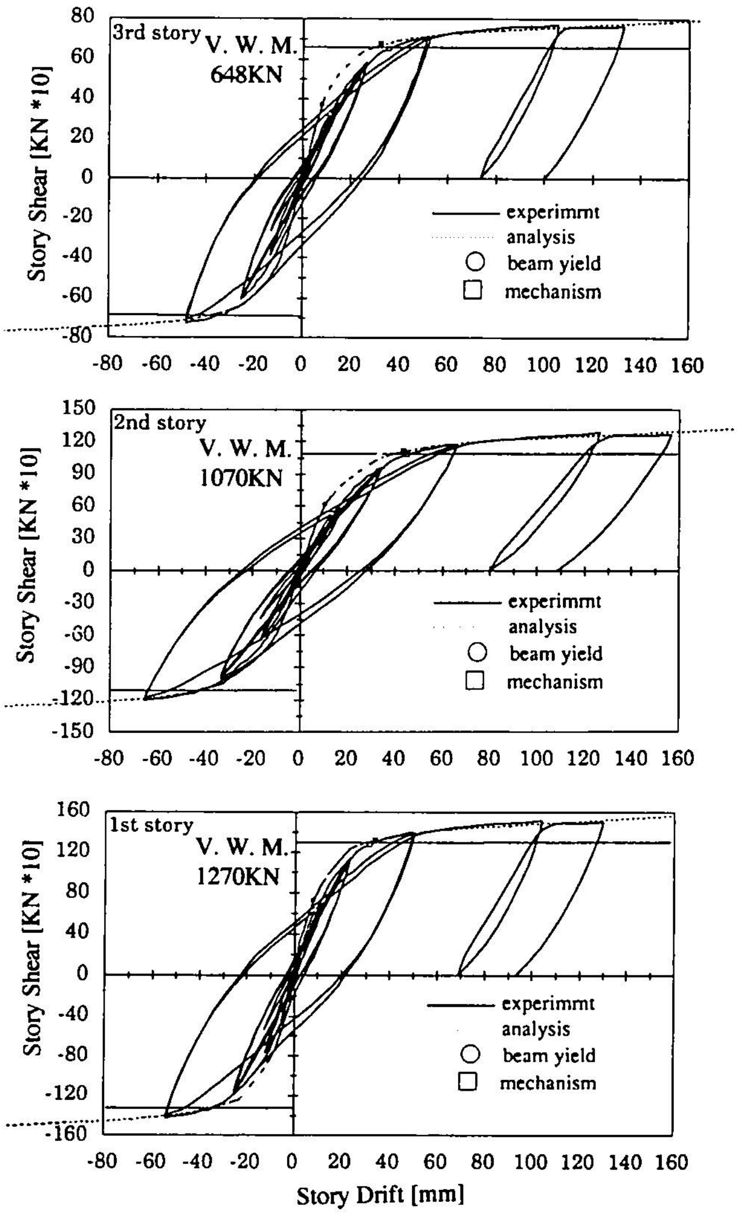


Fig.4 Observed Crack Patterns *Fig.6 Relationships between Story Shear and Story Drift*

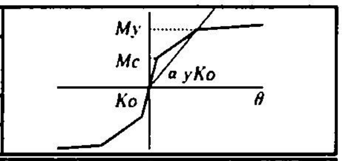
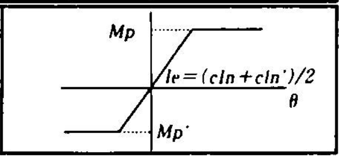
Column	Inner	Outer	
Cracking Moment [KN m]	225	253	
Yielding Moment [KN m]	841	903	
Effective Moment of Inertia	Ie		
[cm4]	925000		
Composite Steel Beam	Positive Bending	Negative Bending	
Full Plastic Moment [KN M]	Mp 543	Mp' 400	
Effective Moment of Inertia [cm4]	cln 50100	cln' 24300	

Table 4 Model for Column and Beam

of the analysis gave good agreements with the measured lateral force and story drift at the mechanism, with the measured initial and post cracking stiffness.

The equivalent damping factors at each loading stages are shown in Fig.7. They were 2-6% during R=1/1000 and 1/200 and increased to 4-8% at R=1/100. After the mechanism formation, the loop area increased significantly and the equivalent damping factor increased up to 16-19%.

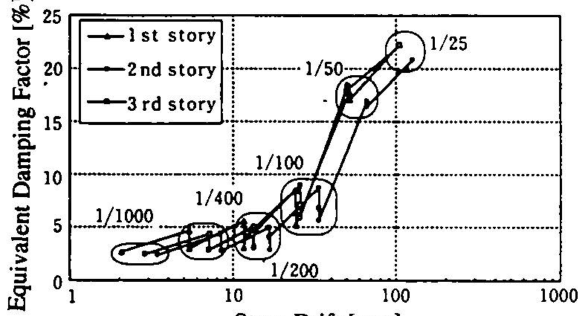


Fig. 7 Equivalent Damping Factor

3.4 Strain Distribution

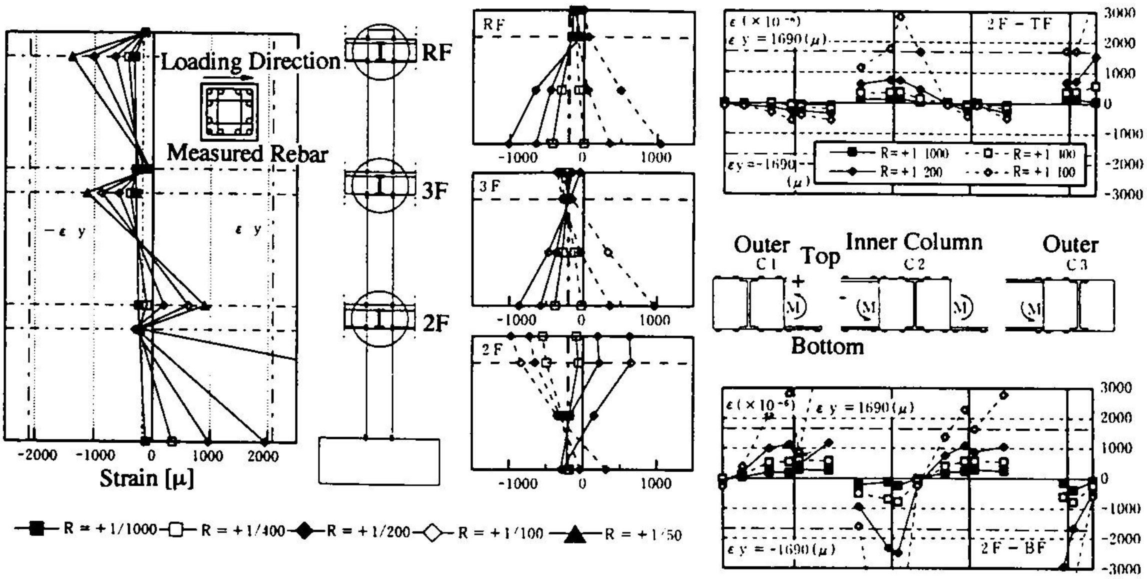


Fig.8 Distribution of Strain

The strain distribution of the longitudinal rebars of the columns and the steel beams are shown in Fig.8. Large strains were recorded at the first story column base and the strain profiles at the second story column was that of double bending. Although the strains of the third story column rebars were large at the column top, reinforcing bars were well anchored in the joint at the end of the test. The strain distributions of outer column reinforcing bars were almost the same as that of inner column. The longitudinal bars in the first story columns yielded at the column base at about R=1/75 and the third story column rebars yielded at the column top at about R=1/50.

The observed beam strains at the interior column faces changed from tension-to-compression in the beam-column joint, indicating the stresses were transferred effectively through the joints.

3.5 Joint Behavior

Final crack patterns inside the joint were investigated after the test by cutting off the cover plates. The joints were not heavily damaged and their crack patterns were corresponding to the direction of the shear force transferred through the joints. It is considered that the cover plates with shear cotter were great effective for enhancing the beam column connection.

4. Conclusions

The following conclusions are derived from the test results.
1) The frequencies obtained from the three different measuring technique; direct measurement, stiffness and flexibility matrices, agreed well each other. It is recommended to use flexibility matrix measuring technique for estimating the frequencies of structures because of their relative simplicity.
2) The specimen formed an estimated total yield mechanism of beam-yield type at R=1/75. The response of the specimen showed stable hysteresis loops and excellent ductility. Although the shear capacity was almost the same as that calculated from virtual work method, the deformation was larger than that expected.
3) The elastio-plastic analysis based on measured material strength made an accurate estimates of the test results especially in terms of the occurrence of the cracking.
4) From the strain distribution adjacent the joints, shear force was considered to be transferred well through the joints. Serious damage was not found in the joints.
5) Although it is shown that this type of structural system has an excellent seismic capacity, it is important to control relatively large story drift of the structure, could be a matter for further research.

Acknowledgement

This research was carried out as a project on composite structures at Technical Institute of Tokyu Construction Co. The authors deeply acknowledge to the members of this project, Mr. Ozawa, Mr. Toyoshima, Mr. Iso, Mr. Aoki and Mr. Kajiwara.

References

[1] Architectural Institute of Japan, "AIJ Structural Design Guidelines for Reinforced Concrete Buildings,", 1994.
[2] Architectural Institute of Japan, "AIJ Ultimate Structural Design for Steel Buildings,", 1975.
[3] Ozawa, J., et al., "Composite Frame Structures Composed of Steel Beams and Reinforced Concrete Columns," Annual Meeting of AIJ, Structural Part C, No.21862, pp.1897-1899, 1992.8.