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SEISMIC BEHAVIOR OF RC COLUMN-S BEAMMOMENT FRAMES

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SUMMARY

To clarify the overall frame behavior of reinforced concrete column-steel beam moment frames, a two-bay, two-story frame specimen was tested under reversed cyclic loading while keeping a constant axial load in the columns. The span length of the beam and the story height were 2,100 mm and 1,350 mm, respectively. The column section was 300 mm square. The steel beams had flange widths of 100 mm and depths of 250 mm. The specimen was designed to cause the beam yielding and the shear failure of beam-column joints at the 2nd floor at the same time. The ratio M_c / M_b of the flexural strength M_c of the interior column to that M_b of the steel beam at the 2nd floor was 1.24. Based on the test results, a detailed review of the sequence and progression of damages and hysteresis characteristics were discussed in detail. As a conclusion, pinching behavior was observed. However, overall frame behavior was stable although yielding of the steel web panel and concrete crushing caused by bearing above and below the steel beam were observed at early loading cycle.

INTRODUCTION

To develop new composite structural systems composed of reinforced concrete columns and steel beams (hereinafter referred to as RCS composite structure), many kinds of details on steel beam-reinforced concrete joints were proposed in Japan, and many experimental studies using steel beam-reinforced concrete subassemblages have been conducted to make sure of seismic performance of the joint. However, to establish a rational design method of RCS composite structure, it is necessary to clarify the overall frame behavior. Studies on the characteristics of RCS composite frame are a few. According to these previous studies, it has been reported that RCS composite frames designed to fail by beam failure have satisfactory seismic performance. However, little information is available on the effect of joint failure on overall frame behavior. From this point of view, it is necessary to clarify the effect of joint failure on the overall frame behavior experimentally.

EXPERIMENT

A two-bay, two-story frame specimen was tested. The overall dimensions of a specimen, the cross sections and reinforcement details are shown in Fig. 1. The span length of the beam and the story height were 2,100 mm and 1,350 mm, respectively. The Column section was 300 mm square. The main longitudinal reinforcing bars consisted of twelve deformed bars with a nominal diameter of 16 mm (D16 in Japan practice) arranged symmetrically around the perimeter to give a total reinforcement ratio P_g =2.65 %. The transverse reinforcing bars had a nominal diameter of 10 mm (high strength round steel), and transverse reinforcement ratio P_w was 2.10 %. The steel beams were built-up and had flange widths of 100 mm and depths of 250 mm. The flanges of

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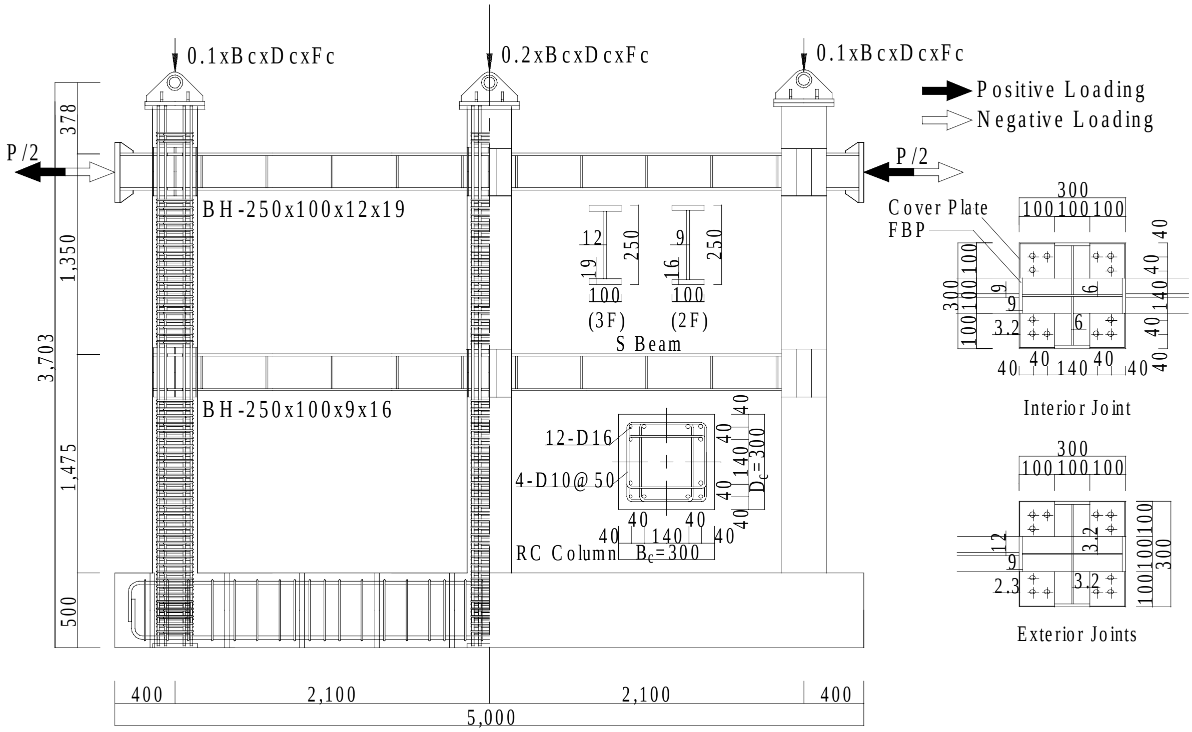


Fig. 1 Details of Test Specimen

Table 1 Summary of Test Specimen

RC Column	Cross Section	300x300 mm	
	Longitudinal Reinforcement	SD685 12-D16	p_g=2.65%
	Transverse Reinforcement	SD785 4-10ϕ @50	p_w=2.10%
	Concrete Strength	27.5 N/mm^2	
	Applied Axial Compression	Interior Column: 0.2xB_cxD_cxF_c Exterior Column: 0.1xB_cxD_cxF_c	
Steel Beam	Cross Section	2F: SM490 BH-250x100x9x16 3F: SM490 BH-250x100x12x19	
Joint	Steel Web Panel	2F Interior Column	: SS400 PL6
		Exterior Column	: SS400 PL3.2
		3F Interior and Exterior Column	: SS400 PL12
	Cover Plate	2F Interior Column	: SS400 PL3.2
		Exterior Column	: SS400 PL2.3
		3F Interior and Exterior Column	: SS400 PL6
	Band Plate	Height: 40 mm	
	Face Bearing Plate	SM490 PL12 Width: 100 mm	

Table 2 Mechanical Properties of Materials

Material	Stress	σ_y	σ_u (N/mm^2)	E_sx10^5
Steel	PL2.3 (SS400)	352	423	2.19
	PL3.2 (SS400)	348	403	1.96
	PL6 (SS400)	253	298	1.88
	PL9 (SM490)	378	548	2.13
	PL12 (SM490)	400	549	2.13
	PL16 (SM490)	387	554	2.12
	PL19 (SM490)	357	540	2.15
Reinforcing Bar	10ϕ (SR785)	884	1116	1.89
	D16 (SD685)	759	978	2.19
Material	**Stress**	**F_c**	**F_t (N/mm^2)**	**E_cx10^4**
Concrete		27.1	2.52	2.21

σ_y: yield stress
σ_u: Maximum stress
E_s: Young's modulus
F_c: Compressive Strength
F_t: Splitting Strength
E_c: Young's Modulus

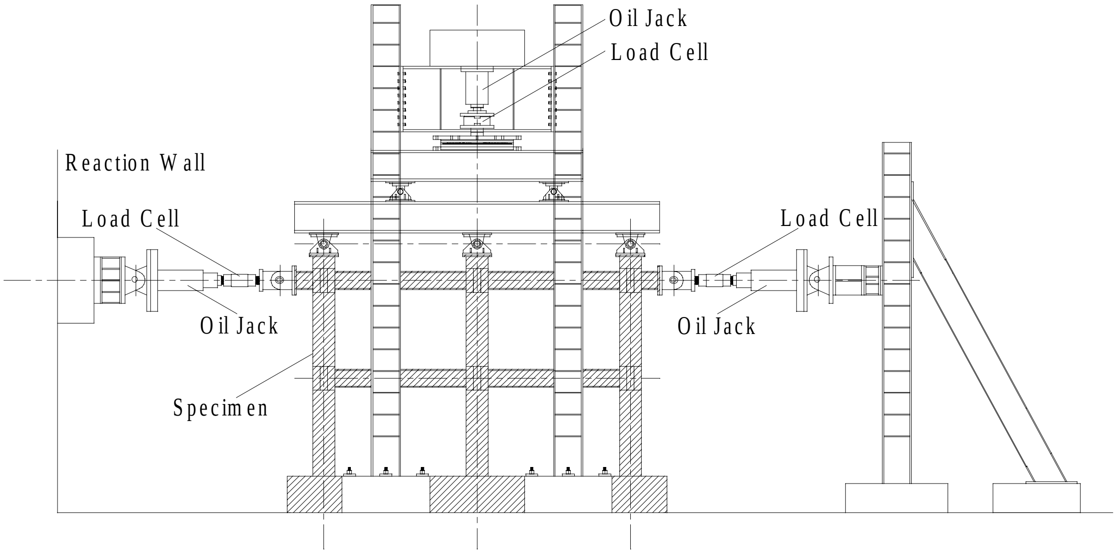


Fig. 2 Test Set-up

the steel beams were continuous through the joint with transverse beams and cover plates (through beam type). The specimen was designed to cause the beam yielding and the shear failure of beam-column joints at the 2nd floor at the same time. Accordingly, the thicknesses of the steel web panel and cover plate at the interior joint are different from those at the exterior joints. The ratio M_c / M_b of the flexural strength M_c of the interior column to that M_b of the steel beam at the 2nd floor was 1.24. The properties of test specimen and the mechanical properties of materials are listed in Table 1 and Table 2, respectively.

Loading apparatus is shown in Fig. 2. Lateral load was applied only at the roof floor while keeping a constant axial load in the columns. Axial loads applied at the interior and exterior columns were 20% and 10% of the compressive strength of the column, respectively. The loading was controlled by the overall drift angle $R=\delta/h$, where δ is the drift at the roof floor and h is the two stories height.

3. TEST RESULTS

A detailed review of the sequence and progression of cracking or damage are shown in Fig. 3. At the cycle of R=0.0025 rad., the initial flexural cracks in the column bases at the first story were appeared. Many flexural cracks were observed around the column bases in comparison with the top of the column at the first story. It is reason that the stiffness of the steel beam is much smaller than that of the column, and accordingly the column behaves like as a cantilever. As the loading amplitude increased to R=0.01 rad., initial flexural shear cracks in the columns and initial concrete crushing caused by bearing above and below the steel beam at the interior joint of 2nd floor were observed. At the cycle of R=0.015 rad., concrete crushing below the steel beam at the interior joint of the roof floor was observed. In subsequent loading, concrete crushing caused by bearing, bond splitting cracks along the longitudinal reinforcing bars at the first story and concrete crushing caused by bending at the column base of the first story occurred. The major cracking loads that were related to the overall failure of the frame are shown in Table 3.

Fig. 4 shows strain distributions of the longitudinal reinforcing bars. From strain distributions, the column inflection point is expected to be at the top of the column at the first story. On the other hand, the inflection point is nearly at midheight in the columns at the 2nd story. These strain distributions correspond to the progression of the flexural cracks in the columns as mentioned above.

Fig. 5 shows shear stress distributions of the steel web panel of the interior and the exterior joints at the 2nd floor. Shear stress was calculated by shear strain obtained from rosette gauges attached at the web panel. At the cycle of R=0.01 rad., shear yielding of the steel web panel at the interior and exterior joints was observed.

The relationship between bearing distortion θ_b and drift angle R of the interior and exterior joint at the 2nd floor is shown in Fig. 6. Bearing distortions were small at the cycle of R=0.00125 rad. and R=0.005 rad.. However, bearing distortions increased with drift angle. Bearing distortions of the interior joint were approximately two times as large as those of the exterior joint. This result corresponds to the progression of the concrete crushing caused by bearing above and bellow the steel beam at the joint as mentioned above.

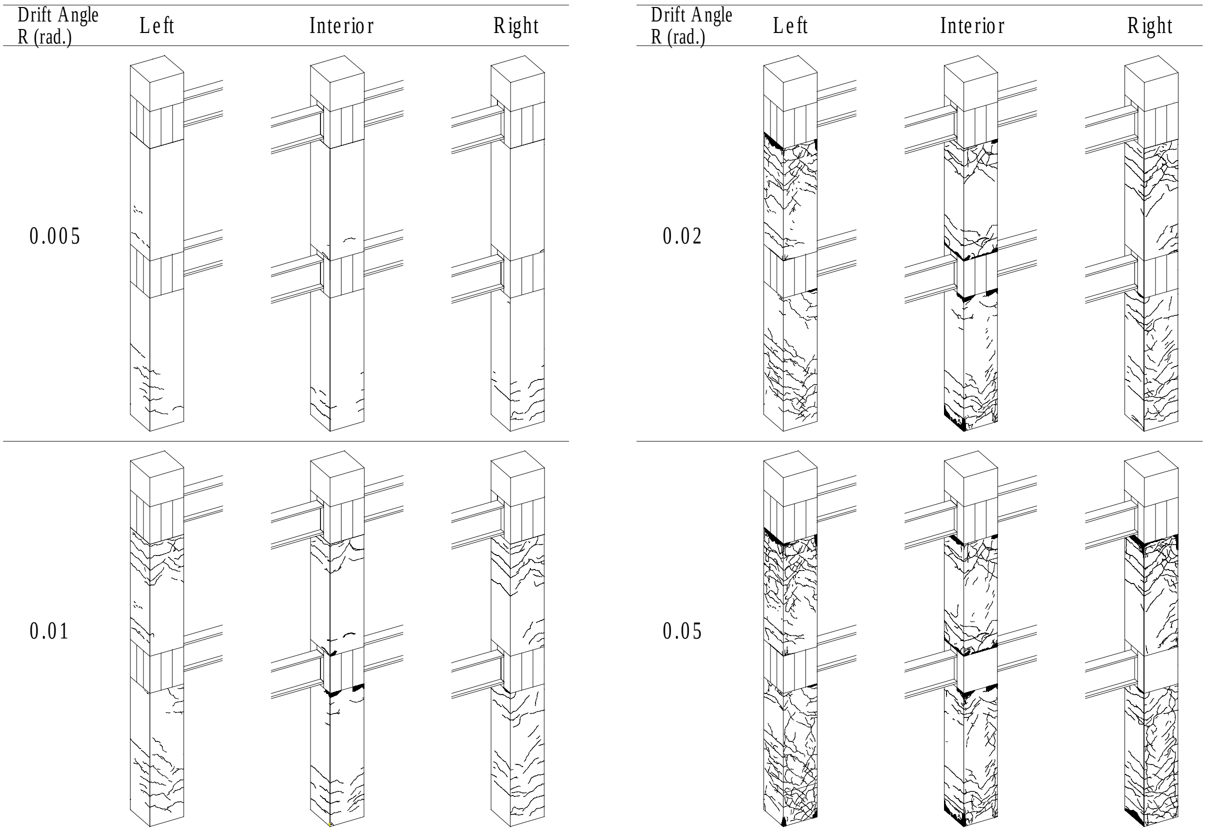


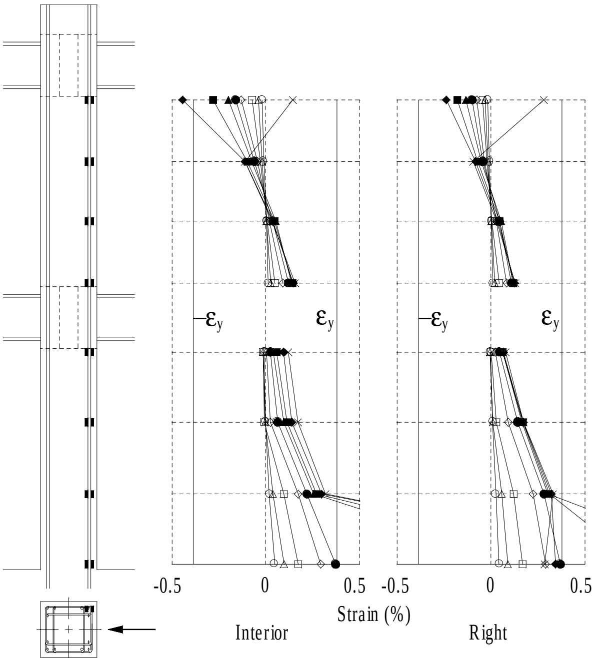
Fig. 3 Crack Patterns

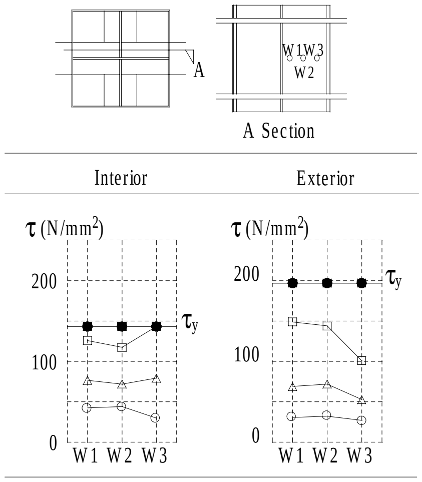
Table 3 Summaries of Test Results

		Flexural Crack P_{fc} (kN)		Flexural Shear Crack P_{fs} (kN)		Bond Splitting Crack P_{bs} (kN)	
		1F	2F	1F	2F	1F	2F
Right Exterior Column	Top Base	399 (387)' 216 (219)	482 (292) 583 (-)	374 (507)	591 (535)	- (723)	824 (-)
Interior Column	Top Base	843 (554) 251 (312)	501 (354) 649 (555)	407 (423)	720 (549)	721 (739)	720 (549)
Left Exterior Column	Top Base	796 (656) 251 (242)	520 (320) (-) 243	262 (518)	540 (387)	775 (-)	843 (-)

		Concrete Crushing caused by bearing above and bellow the beam P_{cc} (kN)		Concrete Crushing at the Column Base of 1F P_{cb} (kN)	Maximum Load P_{max} (kN)
		1F	2F		
Right Exterior Column	Top Base	491 (554)	837 (820) - (-)	778 (-)	
Interior Column	Top Base	412 (-)	697 (820) 748 (-)	638 (-)	843 (832)
Left Exterior Column	Top Base	762 (-)	569 (678) - (535)	- (637)	

( ): Negative Load

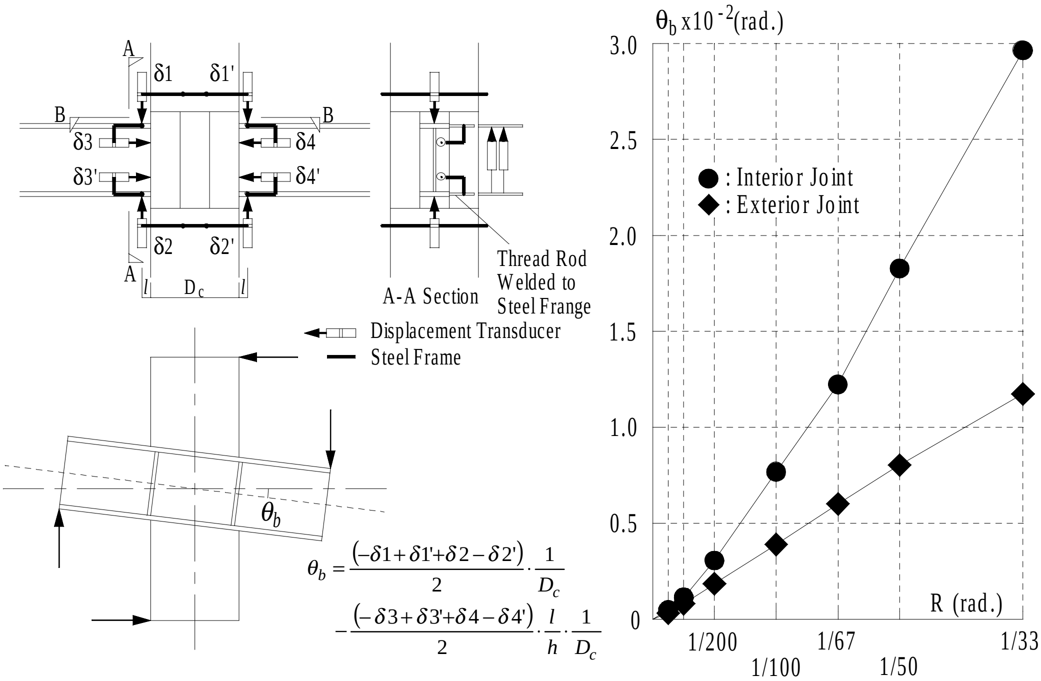




○ : R=1/800, △ : R=1/400, □ : R=1/200, ◇ : R=1/100,
● : R=1/67, ▲ : R=1/50, ■ : R=1/33, ◆ : R=1/25, × : R=1/20
ε_y : Yielding Strain, τ_y : Yielding Stress

Fig. 4 Strain Distributions of Longitudinal Reinforcing Bars

Fig. 5 Shear Stress Distributions of Steel Web Panel



$$\theta_b = \frac{(-\delta 1 + \delta 1' + \delta 2 - \delta 2')}{2} \cdot \frac{1}{D_c} - \frac{(-\delta 3 + \delta 3' + \delta 4 - \delta 4')}{2} \cdot \frac{l}{h} \cdot \frac{1}{D_c}$$

Fig. 6 Bearing Distortion-Drift Angle Relation

The relationships between the applied lateral load at the roof floor and drift angle are shown in Fig. 7. The vertical axis represents the applied lateral load P. Horizontal axis gives the overall drift angle R. P_u is the plastic collapse load of the frame calculated by assuming beam mechanism (total collapse mechanism). Numbers on the figure indicate the order of yielding of the steel flange, steel web panel and longitudinal reinforcing bar obtained by the measured strains. The yielding occurred in the following order: interior steel web panel at the 2nd floor, exterior steel web panel and the steel flange at the 2nd floor, longitudinal reinforcing bar of the column base at the first story, the steel flange at the roof floor and longitudinal reinforcing bar of the top of the column at the 2nd story. Although slight pinching was observed during initial loading cycle of R=0.0025 rad., the overall behavior was almost elastic. In subsequent loading, pinching behavior was remarkably observed.

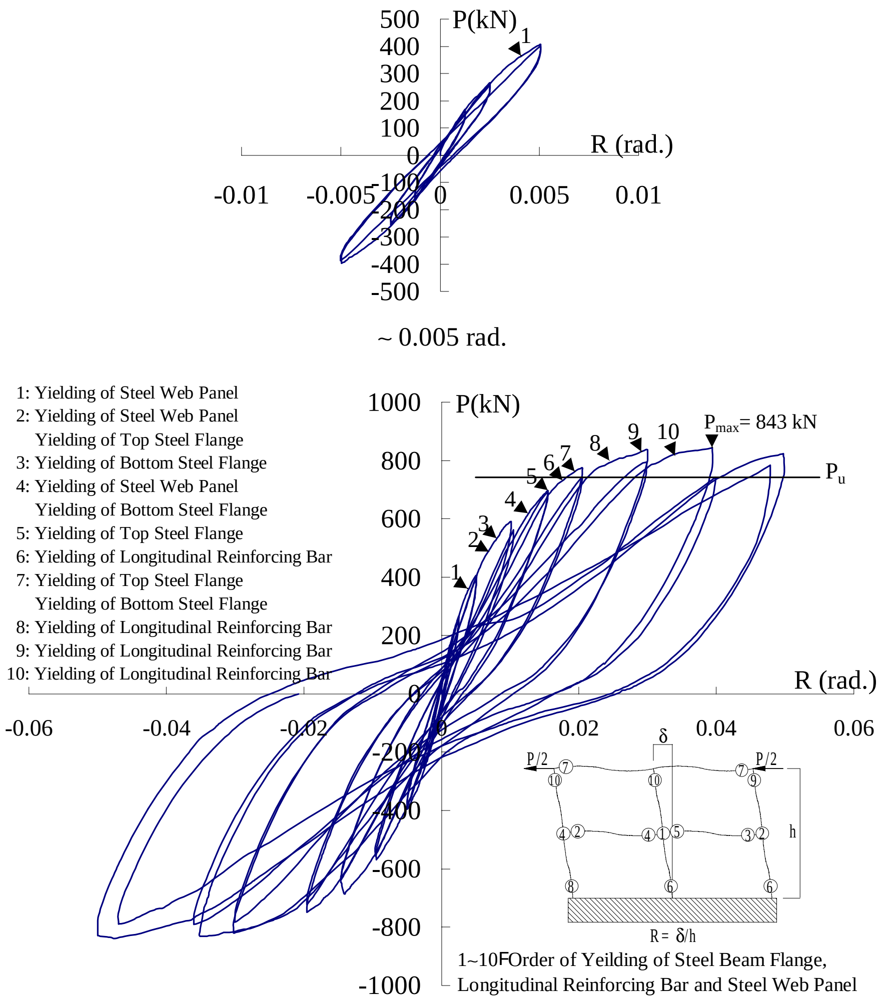


Fig. 7 Hysteresis Characteristics

However, the overall frame behavior was stable although yielding of the steel web panel and concrete crushing caused by bearing above and bellow the steel beam were observed at early loading cycle. The specimen reached its maximum strength at R=0.04 rad.. Beyond this point, large strength degradation was not observed. The maximum strength was about 10% larger than the calculated plastic collapse load.

4. CONCLUSION

A two-bay, two story frame specimen was tested to clarify the effect of joint failure on the seismic performance of the RCS frame. From the test results, pinching behavior was observed. However, the overall frame behavior was stable although yielding of the steel web panel and concrete crushing caused by bearing above and bellow the steel beam were observed at early loading cycle.

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