EFFECTS OF VARIOUS BEAM-BAR ANCHORAGES ON THE STRENGTH AND DEFORMATION OF PRECAST REINFORCED CONCRETE BEAM-COLUMN JOINTS

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SUMMARY

This experimental investigation was carried out to study the influence of the anchorage method of precast concrete beam main bars on the strength and deformation of interior beam-column joints. Bars passing straightly through the joint, bars mechanically connected at the joint panel, bars bent 90 degrees inside the joint and bars lapped at the column face were the main parameters for this experiment. Seven specimens, four of them designed to have beam yielding failure, and the rest to fail in shear at the joint panel, were tested. The following information was obtained.

Among the beam yielding type specimens, the specimen with bottom bars bent 90 degrees has almost same strength that the obtained for the monolithic specimen. On the other hand, the specimens with main bars lapped at the column face or mechanically jointed inside the joint panel, showed a gradual increase of their maximum strength because of a local increase of the strain in the beam bars. For the beam column joint shear failure type specimens, the ratio joint shear stress ($_J\tau_{max}$) to compressive concrete strength ($_J\sigma_B$) was about 0.27, regardless of the main bar anchoring method. The deformation of precast beam due to the pull out of main reinforcing bars at the column face for specimen with mechanical joint was smaller compare to the monolithic specimen. On the other hand the specimen with lapped bars showed most of the deformation concentrated at the column face due to pull out of the beam main bars.

INTRODUCTION

In general, the precast concrete beams have their top main bars passing straightly through the beam column joint, and the lower main bars are bent 90 degrees inside the panel zone. However, in case of beams using thin precast shell forms, the main bars could be connected by lapping splices located at the beam critical section, that is at the column face according to the Concrete Journal [1]. Moreover, top and bottom bars connected at the joint panel using mechanical joints, like grout filled sleeves, are being also increasingly used in this days. In this experimental study, the effects of the connection method of precast concrete beam main bars on the strength and deformation of interior beam-column joints were examined.

OUTLINE OF TEST SPECIMENS

The seven specimens were prepared as shown in Table 1. Based on the failure mode designed, the specimens were divided between four of the beam flexural failure type (called Series B) and three which would develop a shear failure at the joint panel prior to beam yielding (Series J). The details of the specimens are shown in Figure 1. Each specimen represents an interior beam-column subassembly, measuring 2.6m in span and 1.8m in story height. Except for two specimens (M-B and M-T) of monolithic casting, the specimens consisted of

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precast beams and columns, having the beam-column joint panels filled with concrete cast in site. The details of each beam-column joint are shown in Figure 2.

Table 1: Structural Characteristic of Specimens

Specimen		M-B	H-B	PI-B	PII-B	M-J	H-J	PI-J
Column	Main bar: p_g	12-D16(SD345), 1.49%				12-D19(SD685), 2.1%		
	Hoop: p_w	2-D6@50(SD295A), 0.32%				←		
Beam	Main bar: P_t	Both top and bottom: 4-D16(SD345), 0.82%				Both top and bottom: 4-D19(SD685), 1.18%		
	Stirrup: P_w	2-D6@50(SD295A), 0.43%				2-S8@50, 0.66%		
	Main bar anchorage	Both top and bottom: passing through	Both top and bottom: Mechanical joint	Top: Passing through Bottom: 90° bent	Both top and bottom: Lapping joint	Both top and bottom: Passing through	Both top and bottom: Mechanical joint	Top: Passing through Bottom: 90° bent
Common factors		Span: L =2600, Story high: H=1800, Cross section: Column $b_C \times D_C$=400, Beam $b_B \times D_B$=300 × 300 Shear span ratio: Column $M/(Q{\bullet}D_C)$=1.80, Beam $M/(Q{\bullet}D_B)$=3.06 Axial of force of a column: N=0.2$F_c{\bullet}b_c{\bullet}D_c$=848kN, Design standard strength of concrete: F_c=26.5 N/mm^2 Lateral reinforcement: 2-D6@50, p_{jh}=0.32%, Bearing width of a precast beam: 18, S8: High strength shear reinforcement						
Designed failure mode		Beam flexural failure				Joint shear failure		
$_BQ_{su}/_BQ_{mu}$ *		1.96	←	←	←	1.25	←	←
$_jQ_{mu}/V_{ju}$ **		0.37	0.41	←	0.46	0.99	1.05	←
μ ***		8.88	9.37	←	←	18.74	19.29	←

* Ratio of shear strength to bending strength of a beam (calculated value)
** Ratio of joint shear strength at beam yielding to joint shear failure strength (calculated value), $V_{ju} = 0.3\sigma_B{\bullet}b_j{\bullet}D_j$
*** $\mu = \sigma_B{\bullet}\sigma_y/(D_C{\bullet}\sqrt{\sigma_B})$

V_{ju}: shear strength of beam-column joint [kgf]
σ_B: compressive strength of beam-column joint concrete [kgf/cm^2]
b_j: effective width of beam-column joint [cm]
$D_j = D_c$ (cm)
μ: bond index
d_b: diameter of beam main bars [cm]
σ_y: yield strength of beam main bars [kgf/cm^2]
D_c: depth of a column [cm]

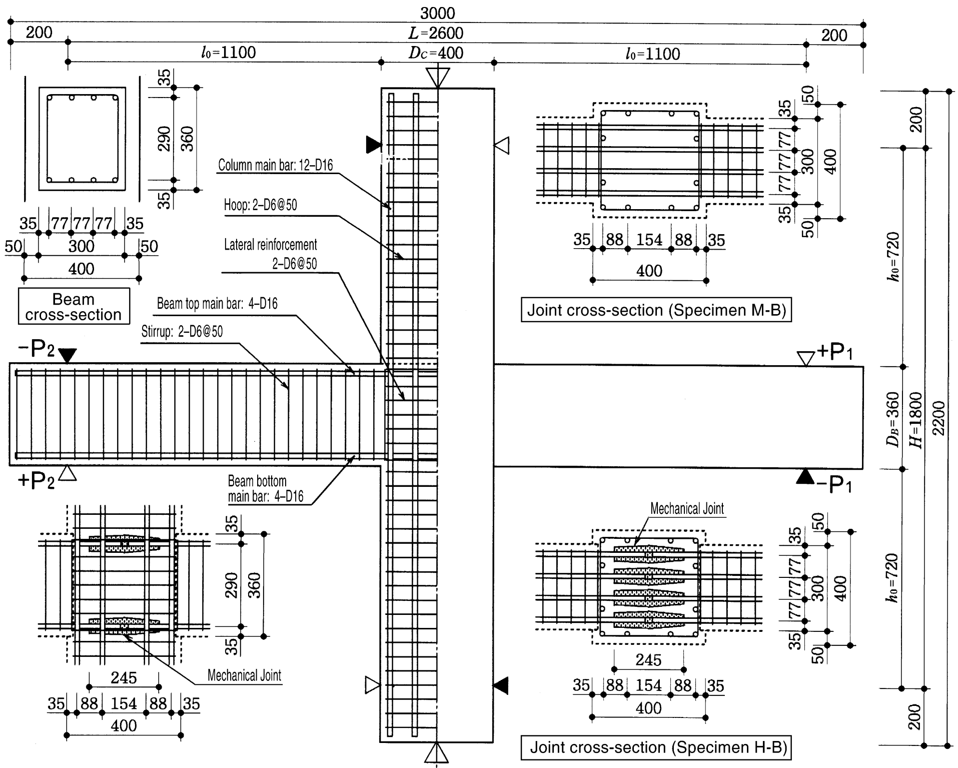

Figure 1: Details of Specimens

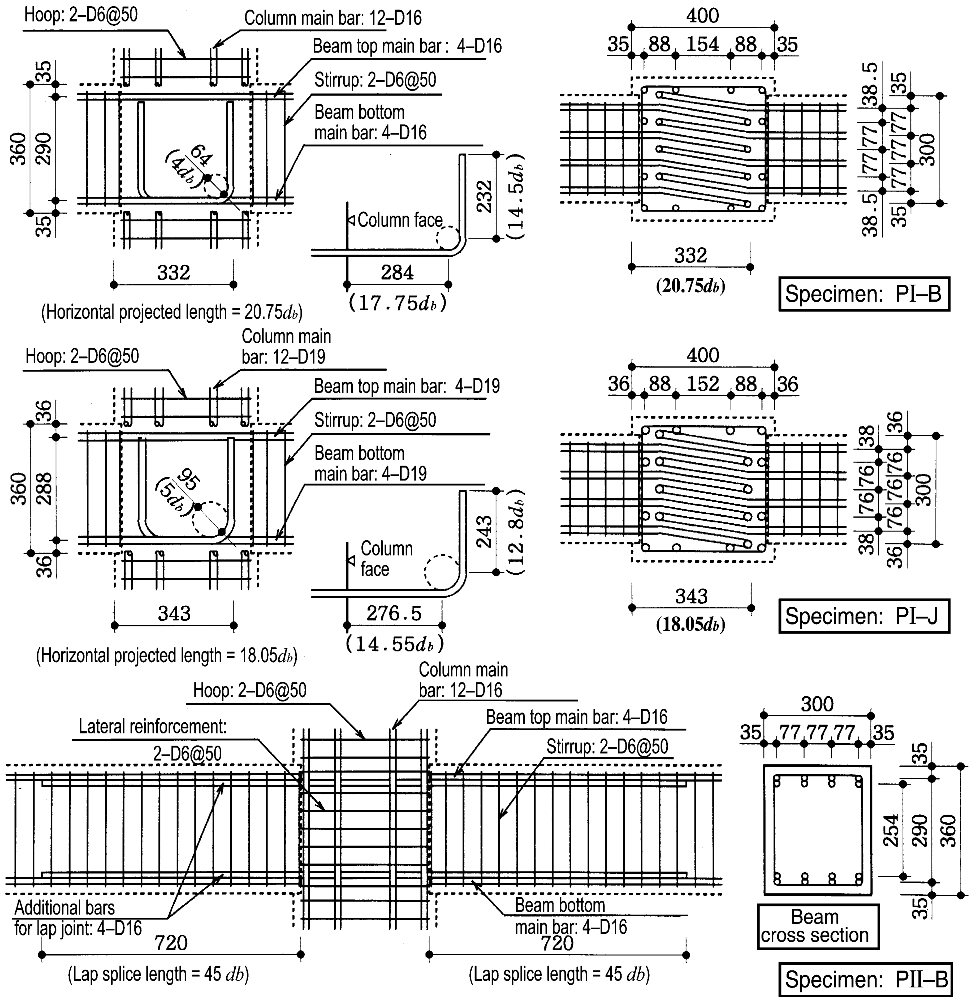

Figure 2: Details of Beam-Column Joints

Table 2: Material Characteristics

a) Reinforcement	σ_y	ε_y	σ_t	E_s	Elongation
Longitudinal reinforcement D19	693	3710	884	198	12 (%)
Longitudinal reinforcement D16	403	2050	549	199	19
Shear reinforcement S8	1048 *	6960 *	1152	211	13
Shear reinforcement D6	410	2320	579	183	22

b) Concrete	Monolithic casting beam and lower story column and their joint, Precast beam, Precast column			Monolithic casting upper story column, Precast specimen joint		
Specimen	σ_B	E_c	${}_c\sigma_t$	σ_B	E_c	${}_c\sigma_t$
Series B	33.6	27.9	3.35	30.2	26.6	2.89
Series J	31.5	27.4	2.89	29.7	25.9	3.03
Grout	93.6	44.5	—	*0.2% offset point		

σ_y: yield stress, ε_y: yield strain, σ_t: tensile stress, E_s, E_c: modulus of elasticity (× 1000)
σ_B: conpressive strength ${}_c\sigma_t$: split tensile strength

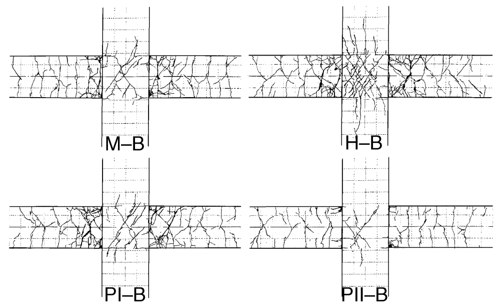

Figure 3: Cracking Pattern (Series B)

In Series B, Specimen M-B is a conventional on-site monolithic cast construction in which has the both top and bottom beam bars passing straightly through the beam-column joint. The beam top and bottom main bars of Specimen H-B are connected at the joint panel using mechanical joints, like grout filled sleeves. The beam top main bars of Specimen PI-B pass straightly through a beam-column joint, and the bottom bars are bent 90 degrees inside the joint (horizontally projected length of 90-degree hook: 20.75 d_b, development length: 36 d_b, d_b: diamter of bar). The beam top and bottom main bars of Specimen PII-B are lapped at the column face (lap splice length: 45 d_b).

In Series J, Specimen M-J is a conventional cast-in-place beam-column interior joint specimen with all beam main bars passing straightly through the joint. The beam top and bottom main bars of Specimen H-J are connected at the joint panel using mechanical joints. The beam main bars of Specimen PI-J are bent 90 degrees inside the joint (horizontally projected length of 90-degree hook: 18.05 d_b, development length: 32 d_b).

The reinforcement and concrete used in the specimens were tested and the results are shown in Table 2. Longitudinal reinforcements of the Series B specimens were all of deformed bars D16 and grade SD345 (345 N/mm^2). The Series J specimens were all reinforced with bars of D19 (SD685 N/mm^2). In addition, lateral reinforcements of beam-column joints, D6□-@50, were common to the seven specimens and the lateral reinforcement ratio of the beam-column joints, p_{jh}, was 0.32%. The longitudinal reinforcements of columns consisted of bars passed through the joints. The specimens were cast horizontally. The bearing width of the precast beam members on the columns was 18 mm and there was no shear-key at the end face of the precast beam.

LOADING AND MEASURING METHOD

Each specimen was firstly supported with pin rollers at the inflection points of the upper and lower story columns. Next the axial load was applied to the top of the upper story column (N=848kN). The displacement was so controlled that the overall deformation of each beam might be anti-symmetrically bending at the points of contraflexure of the left and right beams. Loading was repeated in positive and negative directions alternately. Loading was applied one time each at the lateral drift angle of R=±1/500(rad.) and ±300, two times each at ±1/200, ±1/100, ±1/67, ±1/50 and ±1/33. Loading was concluded by application at R=+1/20.

As for measurement, the overall deformation of beams, the beam-column joint shear distortion, slippage and flexural deformation of beams from the column face were measured. In addition, the strain in main points of longitudinal reinforcements of beams and columns and shear reinforcements were measured.

TEST RESULTS

Cracks and Maximum Strength of Beam-Column Joints:

The final cracking patters in Series B are shown in Figure 3. The shear stress of the beam-column joints at the beam maximum load and at occurrence of the panel first cracking are shown in Table 3.

The maximum strength of Series B was observed at R=1/33 and beyond in almost all specimens. Comparison based on the value (${}_J\tau_{max}/\sigma_B$) found by dividing the shear stress of beam-column joints at the maximum strength by the compressive concrete strength of the beam-column joints shows that precast specimens outperform the monolithic casting Specimen M-B more or less. The failure mode of the specimens is judged to be a bending failure of beam at the column face. Specimen PII-B showed less amount of cracking than any other specimen, but they were remarkable wider at the column face.

On the other hand, the maximum strength of Series J was reached around of R=1/33 in all the specimens. The value of ${}_J\tau_{max}/\sigma_B$ was about 0.27 for the three specimens. The failure mode was judged to be a shear failure in the beam-column joint for all the specimens. An anchorage failure was also observed in Specimen PI-J.

The effect of the difference in precast beam main bar anchoring methods on the maximum strength of the beam-column joints was considered small for both Series B and J specimens.

Hyesteresis Characteristics:

The relation between the story shear force (Q) and the relative story displacement (δ) of several specimens is shown in Figure 4. It should be noted that only the envelope on the positive loading is shown for Series J. Q_{mu} represents the ultimate bending strength of beams by simplified equations, and Q_{mf} is the maximum strength calculated by the inelastic flexural theory.

As shown in Table 3, the ratios of maximum strength of Specimens H-B and PII-B to Q_{mu} were 1.21 and 1.l7 respectively. Loading was accompanied by displacement control to maintain the displacement of same absolute value at points of loading of beams right and left. As a result, the strength increased gradually in H-B and PII-B until the final loading stage because the beam main bars mechanically jointed and the additional bars used for lapping joint entered into the strain hardening region locally near the critical region.

Table 3: Test Results (Loading in positive direction)

Specimen		$_J\tau_{sc}$	Q_{max}	$_J\tau_{max}$
M–B	Observed (*R*)	4.09 (5.06)	136.5 (29.79)	4.15 0.124*
	Calculated	4.88	123.3	——
	Observed/ calculated	0.84	1.11	
H–B	Observed (*R*)	4.16 (4.69)	149.3 (50.47)	4.53 0.150*
	Calculated	4.71	123.3	——
	Observed/ calculated	0.88	1.21	
PI–B	Observed (*R*)	4.01 (5.02)	136.0 (29.78)	4.13 0.137*
	Calculated	4.71	123.3	——
	Observed/ calculated	0.85	1.10	
PII–B	Observed (*R*)	4.97 (10.00)	136.0 (50.50)	4.85 0.161*
	Calculated	4.71	116.4	——
	Observed/ calculated	1.06	1.17	

Specimen		$_J\tau_{sc}$	Q_{max}	$_J\tau_{max}$
M–B	Observed (*R*)	4.78 (4.59)	276.3 (29.81)	8.46 0.269*
	Calculated	4.77	——	7.94
	Observed/ calculated	1.00		1.07
H–B	Observed (*R*)	4.98 (5.08)	266.6 (29.89)	8.17 0.275*
	Calculated	4.68	——	7.94
	Observed/ calculated	1.06		1.03
PI–B	Observed (*R*)	4.83 (5.12)	260.8 (29.86)	7.99 0.269*
	Calculated	4.68	——	7.94
	Observed/ calculated	1.03		1.01

Q: story shear force (kN)
$_J\tau$:   joint shear stress (N/mm²)
R: story drift angle (× 1/1000)

$_J\tau_{sc}$: At joint shear cracking, $\overline{_J\tau_{sc}}$ (calculated)$=1.6\sqrt{\sigma_B}\bullet\{1+\sigma_0/(1.6\sqrt{\sigma_B})\}^{0.5}$ [kgf/cm²]
Joint effctive depth to be distance between tension and compression resultants of column (*jc*) and effective width to be overall width of column (*bc*).

$Q_{max}(_J\tau_{max})$: Under maximum load, (Series B): Q_{max} (calculated)$=0.9a_t\bullet\sigma_y\bullet d/l_0\bullet L/H$, (SeriesJ): $_J\tau_{max}$ (calculated)$=0.9\bullet 7/8\bullet(95.1+0.5p_{jh}\bullet{}_{jh}\sigma_y)$ [kgf/cm²]
$_J\tau_{max}=Q_{max}\bullet H/L\{(L-D_c)/j_b-L/H\}/\{D_c\bullet(b_B+b_c)/2\}$

* Ratio of joint shear stress and joint concrete compressive strength under maximum load.
Joint effective depth to be column overall depth (D_c) and effective width to be obtained by adding beam width to effective width of column ($(b_B+b_c)/2$).

σ_0: axial stress of column [kgf/cm²]
a_t: sectional area of tensile reinforcing beam bars [cm²]
j_b: distance between tension and compression resultants of beam [cm]
d: effective depth [cm]
l_o, L, H, D_c, b_c, b_B: shown in Figure 1 and Table 1
$_{jh}\sigma_y$: lateral reinforcement strength [kgf/cm²]

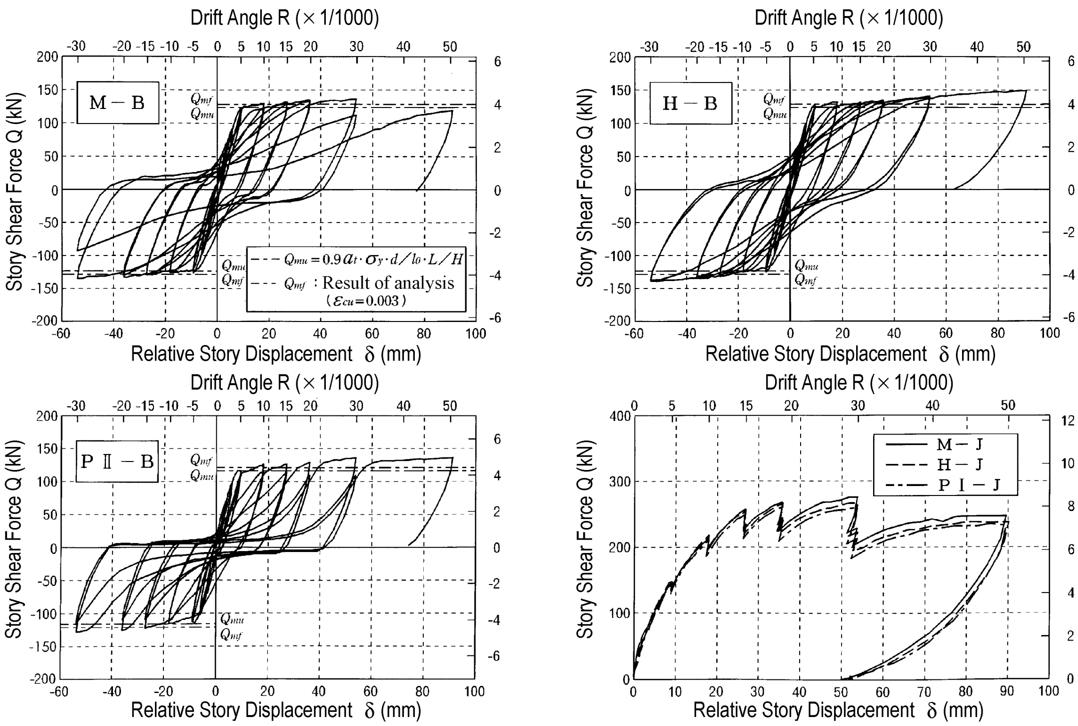

Figure 4: Relation between Story Shear Force and Relative Story Displacement (*Q*–δ)

The relationship equivalent viscous damping coefficient (h_{eq}) to the drift angle is shown in Figure 5. All the specimens were able to meet the requirements of h_{eq}=0.1 at *R*=1/50 given by the Design Guidelines for Reinforced Concrete Buildings [2]. The main bars of specimens H-B and PII-B were supposed to be under severe bond conditions inside the panel joint, compared with specimen M-B. However because h_{eq} showed almost same value until *R*=1/33, it is possible to say that specimen H-B has a performance comparable with the monolithic specimen. On the other hand, for specimen PII-B an increment of the maximum strength induced a rapid bond deterioration of the main bars inside the panel joint. Therefore from R=1/67 the specimen developed a slip type hysteresis loops with a poor energy absorption capacity.

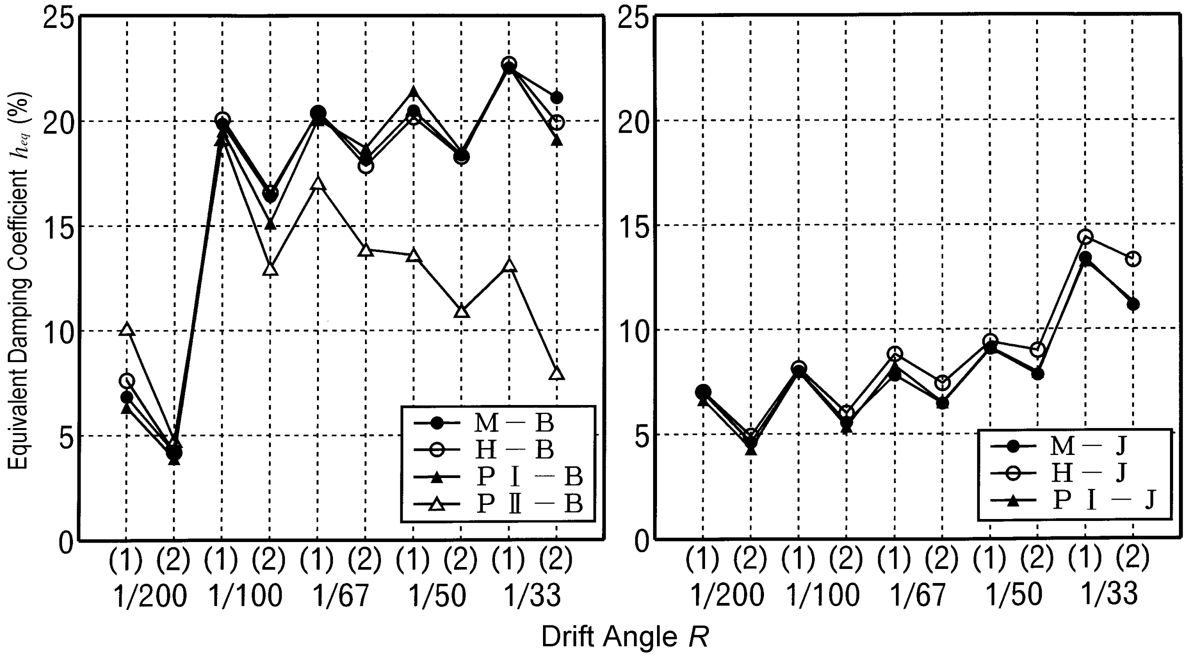

Figure 5: Comparison of Equivalent Damping Coefficient

The value of h_{eq} of the three Series J specimens was less than 10% up to R=1/50. The hysteric energy absorption capacity of Specimen H-J using a mechanical joint was somewhat better than the observed for the other two specimens.

EFFECTS OF PRECAST LONGITUDINAL REINFORCEMENT ANCHORING METHODS ON DEFORMATION COMPONENTS

The deformation components of members for Series J specimens are shown in Figure 6. The deformation components of each member of specimens M-J and H-J are similar. Therefore, no differences are recognized between the specimen with the beam bars mechanically jointed and the one with the bars passing straightly through the beam-column joint. For specimen of Series B (omitted in Figure 6) the beam deformation was about 90% of the total deformation.

Figure 7 shows the deformation components of the specimens with beam yielding type failure. The beam deformation was divided into the deformation due to the slip of main bars from the column face, displacement of the beam respect to the column and the beam flexural deformation itself. Comparison was made with behaviors of Specimen M-B. The pullout of beam main bars from the column face of H-B is evidently smaller than that of M-B at large drift angles. The reason is that deformation is smaller in the beam-column joint of Specimen H-B, while it is large on the loading point side of the beam. In contrast, the beam deformation of PII-B beyond R=1/100 was mostly dependent on the pullout of the additional bars used for lapping joint from column face.

As for Specimens PI-B and PI-J, it was observed that, with an increase of the drift angle, the bent-bar anchorage on the tensile side in the bottom suppressed the pullout from the beam left end zone.

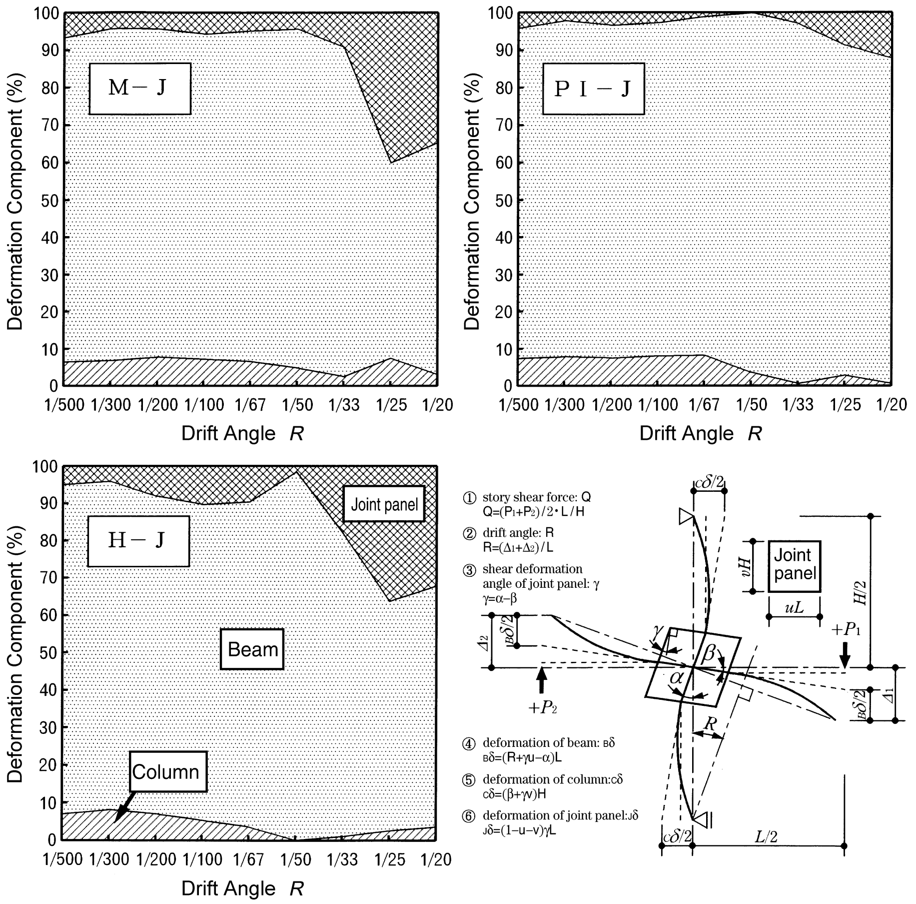

Figure 6: Deformation Component of Member (Series J)

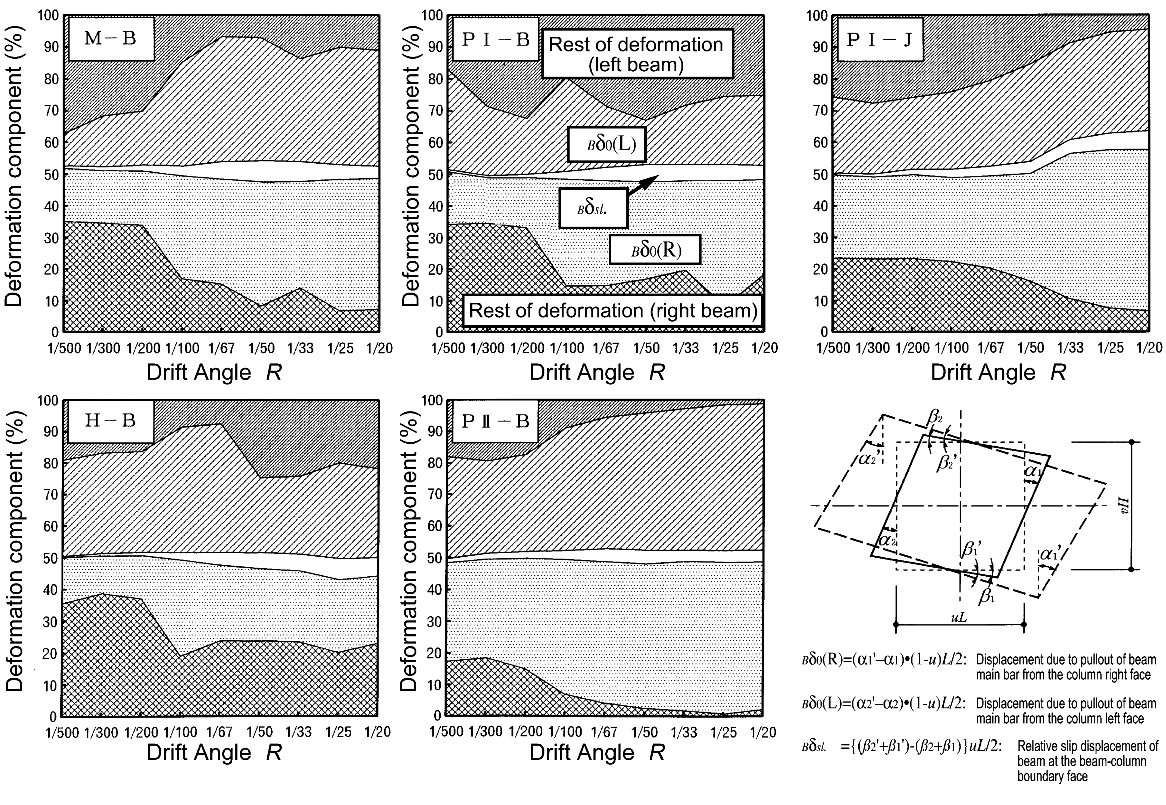

Figure 7: Deformation Component of Beam Member

CONCLUSIONS

1) Among the specimens with beam flexural failure type, the specimen with the bottom bars bent 90 degrees showed almost same maximum strength compared to the monolithic specimen. The specimens with the main bars mechanically jointed or lapped, showed a gradual increase of their maximum strength throughout the loading process because of local strain hardening of the beam main bars.

2) The specimens with beam flexural failure type, were able to keep the equivalent damping coefficient of 0.1 until a drift angle of 1/50. For specimens using mechanical joint and lapping joints for beam main bars, the strength and the bond index increased steadily through the loading process. However, as for specimen with mechanical beam bars joints the hysteresis loops were similar to the monolithic one, the specimen with lapping joints showed a remarkably bond strength decay as indicated by the slip type hysteresis loops.

3) For the specimen with joint shear failure type, the ratio joint shear strength to compressive concrete strength was 0.27, regardless of the beam main bar anchoring method. The specimen with the main bars mechanically jointed showed an equivalent damping coefficient (h_{eq}) slightly greater than the other two specimens.

4) The specimen with mechanical bar joints showed smaller bending deformation of the beam at the column face, compared to the monolithically cast specimen which has the beam main bars passing straightly through the joint. On the other hand, the specimen with lapped main bars developed most of its flexural deformation at the column face.

REFERENCES

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