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INFLUENCE OF TRANSVERSE REINFORCEMENT IN BEAM ENDS AND JOINTS ON THE BEHAVIOR OF R/C BEAM-COLUMN SUBASSEMBLAGES

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SUMMARY

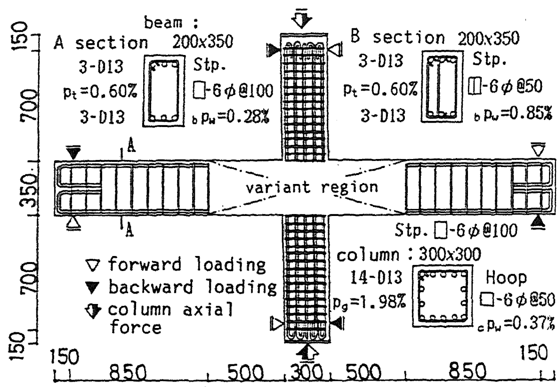
The experimental work was carried out in order to know the influence of transverse reinforcement in joint panels and/or the connecting ends of beams on the behavior of R/C beam-column subassemblages. The test results showed that the heavy transverse reinforcement in joints reduced the slippage of beam bars from the joint panel and enhanced the panel stiffness after cracking, and that the like reinforcement in the beam ends had few effect on relieving the stiffness degradation of frames after yielding. One of specimens that had been treated as to be *bondless within the joint region of the beam bars developed enough ductility but* low energy absorption.

INTRODUCTION

Reinforced concrete interior beam-column joints designed as to develop weak-beam strong-column frame mechanism under lateral loads have generally the flexural yield regions at the ends of connecting beams. These joints have a tendency to show undesirable hysteretic behavior due to the bond deterioration of beam bars within the joint panels during severe cyclic loadings after yielding at the adjacent beam ends. In this paper, the effect of heavy transverse reinforcement in joint panels and/or in beam ends on improving the hysteretic behavior of frames is discussed on the basis of the test data.

EXPERIMENTAL WORK

Test Specimens: The test specimens are interior beam-column subassemblages corresponding to ones extracted from the intermediate stories of reinforced concrete multistory frames. Five specimens shown in Fig. 1 were subjected to constant axial column loads and lateral load reversals. They had the cross shape with no perpendicular-direction beam and no slab, and about a half scale of actual frame members. All specimens were designed so that plastic hinges in the beam ends should form prior to flexural yielding in the columns and shear failure in the beams, columns or joints. The columns were identical in all specimens with cross-section of 30cm x 30cm, a distance of 175cm between top and bottom reaction points, a longitudinal reinforcement of 14-D13 with a gross reinforcement ratio of 1.98% and hoops of 6mm in diameter at every 5cm spacing with a shear reinforcement ratio of 0.37%. The beams were fundamentally composed of a cross-section of 20cm x 35cm, a distance of 300cm between two loading points, longitudinal bars of 3-D13 with a reinforcement ratio of 0.60% at the both top and bottom, and stirrups of



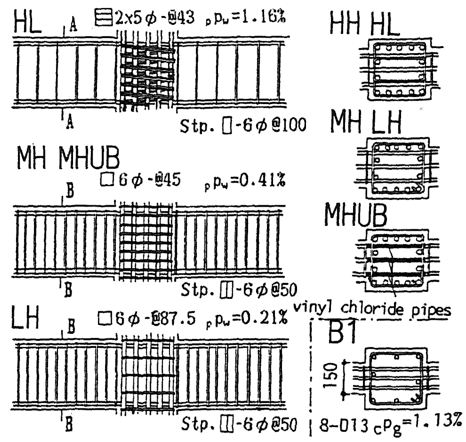


Fig. 1 Details of specimens

Table 1 Variations of specimens and properties of concrete

name	variations			concrete		
	transverse reinforcement		others	comp.	modulus 10^5	split.
	Joint	Beam				
HH	high	high		261	2.12	25.4
HL	high	low		280	2.65	31.0
MH	middle	high		287	2.93	27.1
LH	low	high		274	2.62	27.6
MHUB	middle	high	*1	266	2.41	26.6
B1	low	low	*2	216	1.81	23.5

**1 unbonded beam bars within joint*
**2 beam width of 15cm, column axial bars of 8-D13 (previous test series[1])*

Table 2 Properties of reinforcement

	yielding	fracture	elongation
D13	3850	5970	22.9
6φ	3800	5220	23.0
5φ	10800	12900	8.3

note : unit of Tables 1 and 2 is kgf/cm^2, except for elongation of %

modulus (secant) = value at 33% of compressive strength

6mm in diameter at every 10cm spacing with the shear reinforcement ratio of 0.28% at the middle parts of beam span. The beam bars were passed through the joints and the ratio of the column depth to the bar diameter was 23.

Five specimens had variations in the lateral reinforcement of the joint panels and/or in the transverse reinforcement of the beam ends. As the first variation, the three type of lateral reinforcement in the joint panels were provided as follows:-

'H': higher reinforcement type, where spiral hoops and spiral ties of high strength steel were arranged at the reinforcement ratio of 1.16%, corresponding to the requirements of NZS 3101-1982,

'M': middle reinforcement type, where hoops were arranged at the reinforcement ratio of 0.41%, applying to the requirement of shear reinforcement for columns, other than the special requirement for the flexural hinge regions, in the code of ACI 318-81,

'L': lower reinforcement type, where hoops were arranged at the reinforcement ratio of 0.21%, according to the usual practice in Japan.

As the second variation, two types of transverse reinforcement in the beam ends were provided as follows:-

'L': lower reinforcement type, where stirrups were arranged equally to the middle part of the beam span at the reinforcement ratio of 0.28%,

'H': higher reinforcement type, where stirrups and sub-ties were arranged at the ratio of 0.85%, three times the ratio in the lower type specimens.

The specimens were named for the combinations of the above-mentioned two variations; for example, 'HL' means a specimen which was arranged by higher lateral reinforcement in the joint panel and lower transverse reinforcement in both beam ends. The specimen 'MHUB', one of 'MH' types, was treated to be bondless within the joint region of the beam bars by using vinyl chloride pipes on purpose to clarify the role of the bond capacity beam-bars within the joint in the stress transmission mechanism. Table 1 shows the variations of the specimens and also

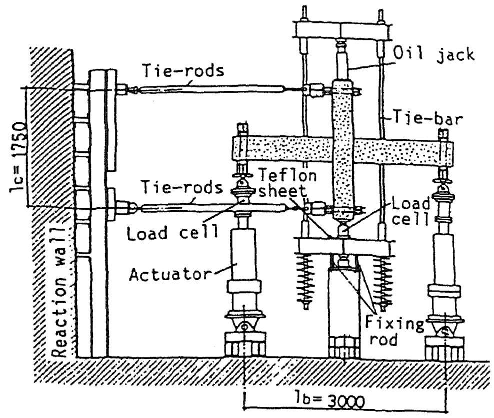


Fig. 2 Loading arrangement

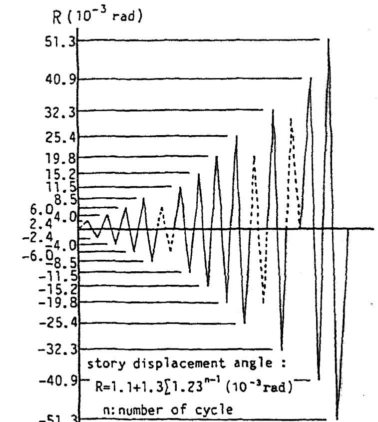


Fig. 3 Forced displacement history

shows the properties of concrete. Compressive strength of the concrete was from 261 to 287 kgf/cm^2. The properties of reinforcement were shown in Table 2. The specimen 'B1' was tested in another experimental series which had been detailed in the previous paper (Ref. 1), and was compared in this discussion. The situations of this specimen was identical to 'LH' except the beam width of 15 cm, the axial reinforcement in the column of 8-D13 and the transverse reinforcement in the beam ends of the lower type specimen.

Loading The loading arrangement is schematically shown in Fig. 2. Two servo-actuators were installed vertically connecting the tips of the beams and the reciprocal loading history given to the specimen is shown in Fig. 3 as represented with the story displacement angle. The axial load on the column was kept constant as $f_c'b_ch_c/6$ during the test.

EXPERIMENTAL RESULTS and DISCUSSIONS

Cracking and Failure Crack patterns in the final stages of the specimens are shown in Fig. 4. The shear cracks in the joint panels of 'HH' and 'HL' with the higher lateral reinforcement in the joints dispersed on the whole joint regions. And contrastively the shear cracks of 'LH' with the lower lateral reinforcement appeared concentratively as a few wider diagonal cracks. No shear crack occurred in the joint of the unbond specimen 'MHUB'. These specimens failed due to flexure of the beam ends while the joint had not developed to the ultimate stages. Only the specimen 'B1' failed due to joint shear under the large deformation forced after the ultimate strength of the beams. The difference of the failing mode of 'B1' from that of 'LH' which had the same lateral reinforcement in the joint might result from the narrower beam width and the less axial reinforcement in the middle depth of column.

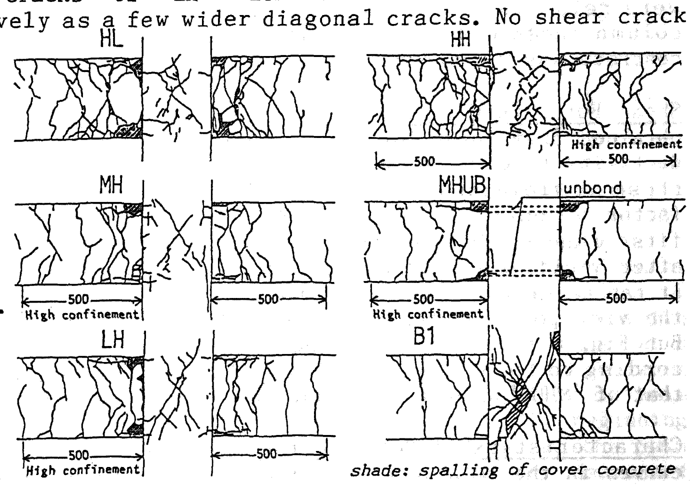


Fig. 4 Crack patterns after test

Table 3 Observed and calculated values

Specimen	loading	shear crack in joint						flexural yield in beam					ultimate strength				
		cycle	(1)	(2)	(3)	(4)	(5)	cycle	(6)	(7)	(8)	(9)	cycle	(10)	(11)	(12)	(13)
			R (10^{-3})	$_{exp}V_{col}$ (tonf)	$_{exp}v_{jh}$ (kgf/cm^2)	$_{cal}v_{jh}^{*}$ (kgf/cm^2)	(3)/(4)		R (10^{-3})	$_{exp}V_{col}$ (tonf)	$_{cal}V_{col}$ (tonf)	(7)/(8)		R (10^{-3})	$_{exp}V_{col}$ (tonf)	$_{cal}V_{col}$ (tonf)	(11)/(12)
HH	+	5	11.4	4.80	32.2	37.6	0.86	3	5.31	5.53	5.14	1.08	9	32.4	6.48	5.03	1.22
	–	4	8.05	5.30	35.6		0.94	4	5.98	5.03		0.99	9	32.3	6.59		1.24
HL	+	3	6.18	5.88	39.4	38.5	1.02	3	4.80	5.28	5.21	1.01	8	25.5	6.61	5.37	1.23
	–	4	8.02	5.86	39.3		1.02	3	4.67	5.40		1.04	8	25.5	6.77		1.26
MH	+	4	7.23	5.73	38.4	38.9	0.99	3	5.46	5.39	5.21	1.03	8	25.7	6.42	5.37	1.20
	–	3	6.07	5.93	39.8		1.02	3	4.74	5.36		1.03	8	25.4	6.72		1.25
LH	+	3	5.42	5.30	35.6	38.3	0.93	3	4.92	5.07	5.21	0.97	9	32.7	6.67	5.37	1.24
	–	5	5.95	5.45	36.6		0.96	3	4.72	5.01		0.96	8	25.4	6.42		1.20
MHUB	+					37.9		4	8.14	5.63	5.21	1.08	6	15.4	5.81	5.37	1.08
	–							4	6.01	4.21		0.81	9	32.4	5.77		1.07

■crack ■yield ■ultimate

$$v_{jh} = \frac{1}{b_j j_c}\left[\frac{M_{b1}+M_{b2}}{j_b} - V_{col}\right] \quad (3) \qquad {}_{cal}V_{col} = \frac{M_{by}}{{}_{e}l_b}\frac{l_b}{l_c} \quad (8) \qquad {}_{cal}V_{col} = \frac{M_{bu}}{{}_{e}l_b}\frac{l_b}{l_c} \quad (12)$$

$$v_{jh}^{*} = f_t'\sqrt{1+\frac{\sigma_0}{f_t'}} \quad (4)$$

${}_{e}l_b$ = claer span of the beam $M_{bu} = 0.9\ a_t f_y d_b$

$M_{by} = 0.8\ a_t f_y d_b$ $j_c = 7d_c/8$ $j_b = 7d_b/8$

$b_j = b_c$; joint width (for this case) $f_t' = 1.4\sqrt{f_c'}$ σ_0 = column axial stress, $P/b_c h_c$

Strength The summary of the test results and the comparison with the calculated values are shown in Table 3. The equations to obtain the calculated values are presented below the table. The calculated values of the shear cracking stresses in joints are close to the test values independently of the transverse reinforcement ratio. In the specimen 'MHUB' with unbonded beam bars in the joint panel, however, the shear crack did not occur in the panel though the test values were above the calculated values. The reason may be that the diagonal tension stresses which might result from the bond stresses of the beam bars and column bars had not developed.

All specimens failed with flexure of the beams and the ultimate strength ratios of the test values to the calculated values lay between 1.2 and 1.3 except 'MHUB', hence it appeared that the beam bars reached the strain hardening range at the maximum loads. The reason why the ratio of 'MHUB' was not enhanced so much may be that the strain of beam bars remained smaller even at large deflection of the frame after the yielding because the deformable length of the beam bars was not less than the column depth and the uniform tension stress occurred along this length uniformly, and that the moment arms in the cross-section of the beam ends were relatively short because the beam bars in the compression regions at the column faces did not work as compression bars and the compressive stress in concrete was duplicated.

Shear Force in Column vs. Story Deflection Angle Relations The envelope curves obtained from shear force in column vs. deflection angle relation curves and the ductility factors which are the ratios of the deflections at ultimate strength to those at yielding are shown in Fig. 5 and Table 4, respectively. The ductility factor of only the specimen 'MHUB' at forward loading looked extremely small at first glance. This is because the peak strength values at reversed loading cycles after yielding slightly fluctuated in the case of 'MHUB'. As observed in Fig. 5 it can be said that all specimens had good deformability in the re-estimation from the view point of the limit deflections to which the yielding strength was held. But Fig. 6 showed that the equivalent viscous damping factor became smaller according to the decrease of lateral reinforcement ratio in the joint panels, and that of 'MHUB',unbond specimen, was maintained the smallest.

Characteristics of Deformation A superimposed illustration of the skeleton curves in the relations of joint shear stress vs. deformation is shown in Fig. 7, except 'HH' and 'LH' in which the shear deformation measuring was defective. The

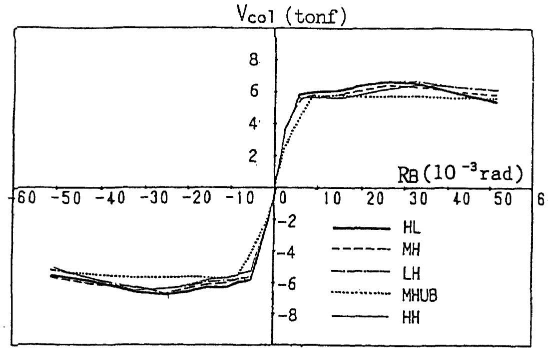


Fig. 5 Skeleton curves in the relations of column shear - story drift angle

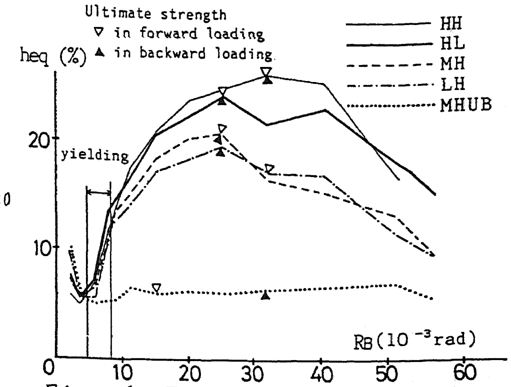


Fig. 6 Equivalent viscous damping factors

Table 4 Deformability

specimen		HH	HL	MH	LH	MHUB
R_y (10^{-3}rad)	+	5.31	4.80	5.46	4.92	8.14
	−	5.98	4.67	4.74	4.72	6.01
R_u (10^{-3}rad)	+	32.4	25.5	25.7	32.7	15.4
	−	32.3	25.5	25.4	25.4	32.4
μ_u	+	6.0	5.3	4.7	6.6	1.9
	−	5.0	5.5	5.4	5.4	5.4
R_y' (10^{-3}rad)	+	>51.8	53.2	>53.2	>53.8	48.3
	−	49.6	>59.2	>54.5	52.8	>59.2

R_y : Story drift angle at yielding of beam bars
R_u : Story drift angle at ultimate strength
R_y': Story drift angle at limit deflection
μ_u : Ductility factor, R_u/R_y
+ : forward loading
− : backward loading

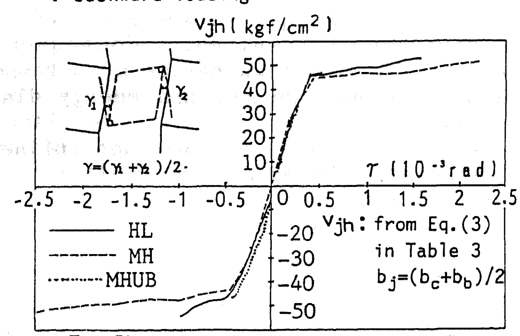


Fig. 7 Skeleton curves in the relations of joint shear stress - shear deformation relations

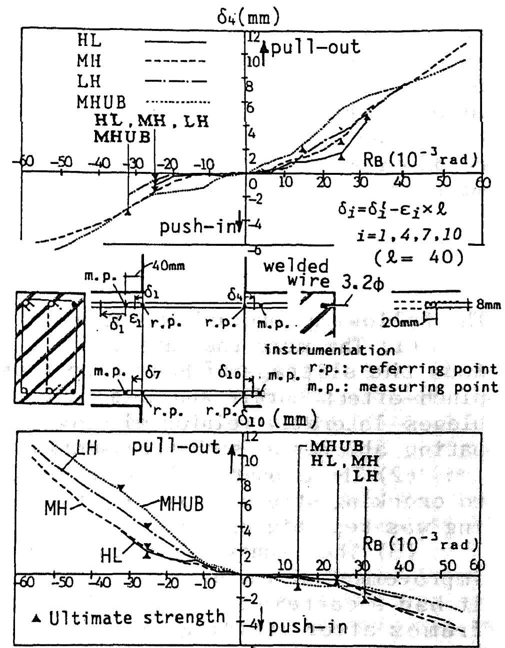
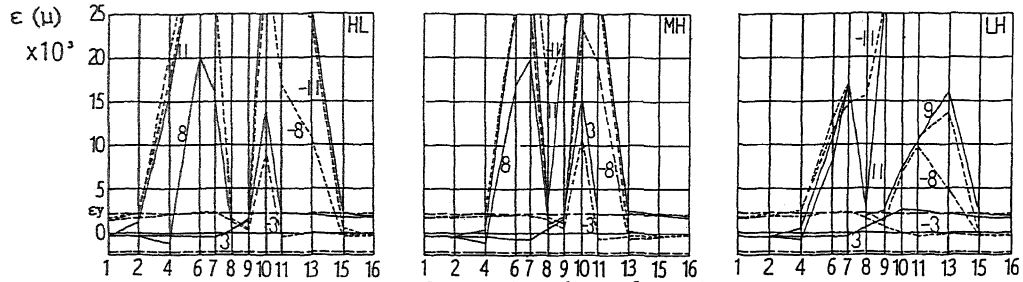


Fig. 8 Beam bar slippage - story drift angle relations

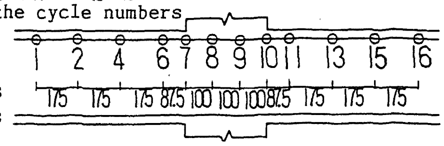
shear stiffnesses of 'HL' and 'MH' degraded just after cracking diagonally in the joint panels. As in Fig. 7, the shear deformation after cracking became larger with the less lateral reinforcement in the joint panel, it was shown that the lateral reinforcement could relieve the stiffness degradations of joint panels.

The Fig. 8 indicates the relation between slippage of beam bars and story deflection. The slippage was the relative displacement between the center line of column depth and the referring point which had been situated at column face on the beam bars before test. As the measuring points on the beam bars were located at a distance of 4 cm from the column faces, the correction for the difference between the measuring points and the referring points was made using the strain of beam bars. Properly the slippage of 'MHUB' appeared simultaneously with the beginning of loading. The slippage of the other specimens began after yielding of the beam bars, and the amount of slippage, especially during pull-out loading, decreased roughly in accordance with the amount of lateral reinforcement in the joints.



numbers in figure show the cycle numbers

Fig. 9 Distributions of beam bar strains



Strains in Reinforcement Fig. 9 shows the distributions of strains along the top bars at their yielding (corresponding to the third or 4th cycle), at the ultimate strength (the 8th or 9th cycle) and at the larger deflection (the 11th cycle). The length of yield region measured from the column face developed larger toward the beam tips as the transverse reinforcement in the beam ends decreased, and penetrated more into the joint as the lateral reinforcement in the joint panels decreased. In the case of 'HL', the lengths toward the beam tips were the largest and extended to the length equivalent to the beam depth of 35 cm at the final stage of loading, and the length penetrated into the joint were the smallest and less than one-third of the column depth.

CONCLUSIONS

The following conclusions may be drawn from the test results:

(1) The more the lateral reinforcement in the joint panels was provided, the less the slippage of beam bars from the joint panels resulted. Consequently pinch-effect hardly appeared on the shear force - deflection curves of subassemblages laterally reinforced heavily in the beam-column joints, and energy dissipating ability of such subassemblages was large.

(2) The amount of lateral reinforcement in the joint panels did not influence to cracking stress. However, the shear stiffness of the joint panels after cracking was kept higher with the heavier lateral reinforcement.

(3) The transverse reinforcement in the beam ends was scarcely affected on improvement of the bond deterioration along the beam bars within the joints. But it had a certain effect, as it was, on relieving the stiffness degradation of frames after yielding because the reinforcement confined the concrete of the beams, and it obstructed the development of yield regions of the beam bars toward the beam tips.

(4) The specimen with 'unbonded beam bars' within the joint panel showed low stiffness even on the elastic region of the frame response. However any shear crack did not occur in the panel nor the shear stiffness of the joint panel degraded, because shear force in the panel was transmitted mainly through the diagonal compression strut. The ratio of ultimate strength to yield strength was not so much as compared with that of the other specimens, but the deformability was same or more in the unbonded bar specimen rather than in the others.

(5) The longitudinal bars arranged in the mid of column depth had an effect on enhancement of the shear strength and shear stiffness of joint.

REFERENCE

1. Shibata,T. and Joh, O., Behavior of R/C interior beam-column joints with various details under cyclic lateral loads. A report prepared for U.S.-N.Z.-Japan Seminar, Monterey, California. (1984)