19861120 JCI Transaction
本採用原稿
K. Kitayama.

1

BEHAVIOR OF REINFORCED CONCRETE THREE-DIMENSIONAL BEAM-COLUMN CONNECTIONS WITH SLABS

Kazuhiro KITAYAMA*, Shin'ichiro ASAMI**, Shunsuke OTANI* and Hiroyuki AOYAMA*

ABSTRACT

The tests of the three-dimensional beam-column connections with slabs were executed as a part of the tri-lateral cooperative research project among the United States, New Zealand and Japan. Two interior and one exterior connections were tested under bi-directional lateral load reversals. The influence of the beam bar bond situation within a connection on the hysteretic behavior was investigated for interior connection specimens. The existence of the slab was found to cause a pinching behavior regardless of a bond situation. The behavior of transverse beams and slab was studied for an exterior connection specimen.

1. INTRODUCTION

The hysteretic behaviour of a beam-column connection is influenced by the bond condition of beam bars within the connection. An improvement in bond of beam bars makes it possible to develop a good spindle-shape hysteresis with flexural yielding at the critical region at beam ends (Ref.1). On the other hand, the bond deterioration of beam bars yields a pinching hysteresis loop attributable to the pull-out of the beam bars from the connection, followed by the shear failure of the connection at a large deformation (Ref.2).

In the past, most of beam-column sub-assemblage tests were carried out on plane beam-column connections, loaded in one horizontal direction. The beam-column connection in an actual structure has both slabs and transverse beams and is subjected to bi-directional loading by earthquake motions. Therefore, it was decided that three-dimensional beam-column connections with slabs be tested in the trilateral program under bi-directional loading. The main variable in the test was chosen to be the bond situation of beam bars within the connection.

----

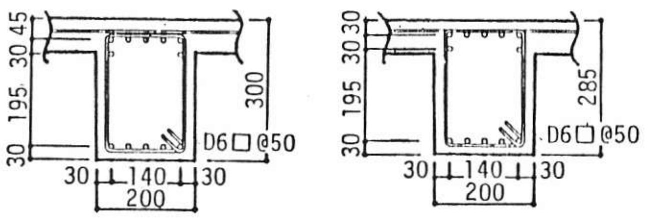
* Department of Architecture, Faculty of Engineering, Univ. of Tokyo
** Mitsubishi Heavy Industry Co.

2. EXPERIMENTAL PROGRAM

2.1 SPECIMENS

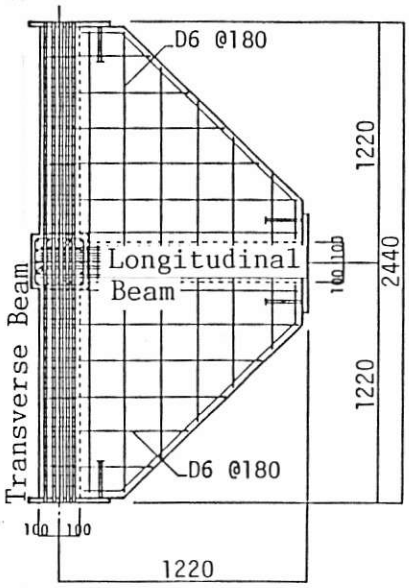
Three half-scale reinforced concrete three-dimensional beam-column connections with slabs (called K-series) were tested; two interior connections (Specimens K1 and K2) and one exterior connection (Specimen K3). The dimensions of the column were varied in the two interior connections; i.e., the column dimensions were 275x275 mm in Specimen K1 and 375x375 mm in Specimen K2. The column dimensions in Specimen K3 were the same as those in Specimen K1. The dimensions of beams were common in the three specimens; 200x300 mm for longitudinal beams (in the primary loading direction) and 200x285 mm for transverse beams. The thickness of slabs was 70 mm. The four corners of the square slab were trimmed to fit into the testing apparatus.

Reinforcement details of the specimens are shown in Fig.1. Beam bars passed through an interior connection, whereas the top and bottom beam bars were anchored within an exterior connection. D13 bars were used as the column reinforcement in the three specimens. The size of the beam bars was varied in the two interior specimens; D13 bars in Specimen K1 and D10 bars in Specimen K2. D10 bars were used as the beam reinforcement in Specimen K3. The amount of lateral reinforcement (D6 bars) within a connection was decided to be the same as the amount of shear reinforcement of a column in accordance with the AIJ Standard. The slab was reinforced with D6 bars at 180 mm on centers in a single layer, with a 180^{o} hook at each end, but the slab bars in Specimen K3 parallel to the longitudinal beam were anchored in the transverse beams with 90^{o} hooks.

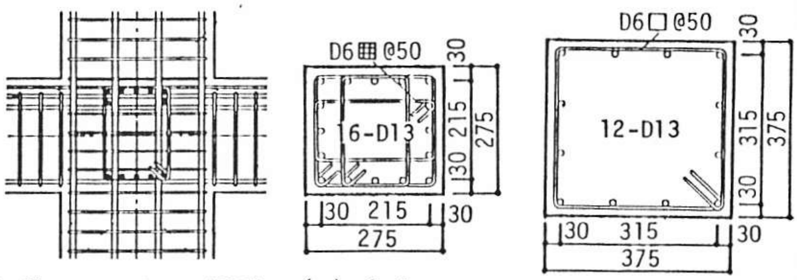


(b) Longitudinal Beam (K2)

(c) Transverse Beam (K2)



Specimen	Beam Top Bar	Beam Bottom Bar	Connection p_w(%)
K1	4-D13	3-D13	2-D6 4 sets (0.42%)
K2	7-D10	5-D10	2-D6 4 sets (0.33%)
K3	7-D10	5-D10	2-D6 3 sets (0.34%)



(a) Slab (K3) (d) Connection (K2) (e) Column (K1 and K3) (f) Column (K2)

Fig.1 : Member Sections and Reinforcement Details

The bond situation of beam bars was made significantly different in the two interior connection specimens by varying the column width and beam bar size. The bond index, defined as an average bond stress of a beam bar within the connection under tensile and compressive yielding assumed at the column faces (Ref.1), was 102 kgf/cm^2 for Specimen K1 and 57 kgf/cm^2 for Specimen K2 using the actual yield strength of the beam bar. From these index values, the bond of beam bars in Specimen K1 was expected to be quite severe compared to Specimen K2.

The concrete was cast in the upright position in two stages; i.e., the concrete was first placed to the top of the slab, and then cast in the upper column after a day.

2.2 MATERIAL PROPERTIES

The compressive strength of the first batch of the concrete was 244 kgf/cm^2 for Specimens K1 and K2, and 199 kgf/cm^2 for specimen K3. The compressive strength of the second batch was 266 kgf/cm^2 for Specimens K1 and K2, and 196 kgf/cm^2 for Specimen K3.

The yield strength was 4,420 kgf/cm^2 for the D13 bars (of which nominal area is 1.27 cm^2), 4,460 kgf/cm^2 for the D10 bars (0.71 cm^2), and 4,010 kgf/cm^2 (0.2 % offset) for the D6 bars (0.32 cm^2).

2.3 TESTING METHOD and INSTRUMENTATION

The loading apparatus is shown in Fig.2. The specimens were tested in the upright position. The base of the specimen was supported by a universal joint. The free ends of the beams were supported by vertical rigid members equipped with universal joints at their ends, creating roller support conditions in the horizontal plane. The distance from the column center to the beam-end support was 1,350 mm, and the distance from the beam center to the bottom support or to the top horizontal loading point was 735 mm. The constant vertical load (an average axial stress of 20 kgf/cm^2) and reversing bi-directional horizontal loads were applied at the top of the column through the tri-directional joint by three actuators. Counter-weights were used to balance the weight of the horizontal actuators. A set of pantograph was attached parallel to the longitudinal beam to prevent a specimen from rotating around the vertical axis.

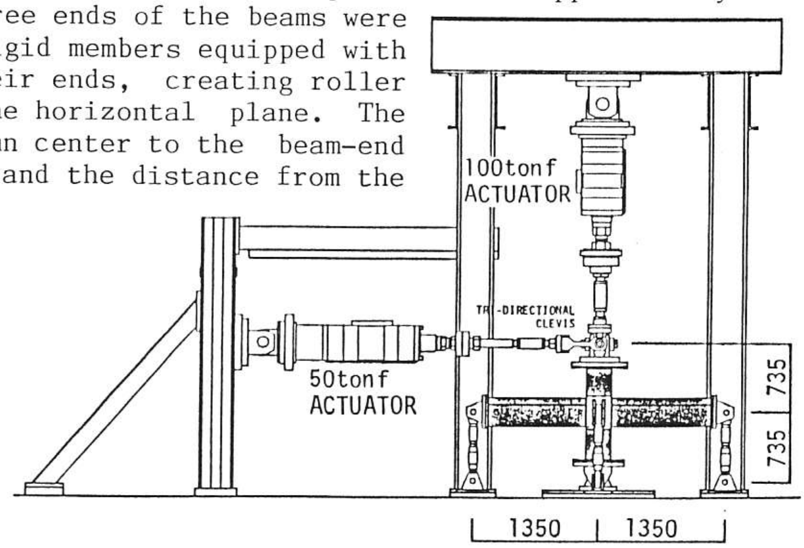
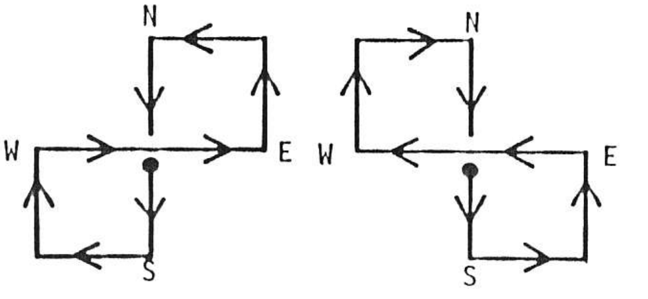


Fig.2 : Loading Apparatus

The deflections of beams and columns relative to the beam-column connection, axial deformation at the top and bottom fiber of beams and beam axial deformation were measured by strain-gauge type displacement transducers. The strain distribution of beam longitudinal reinforcement within and immediately outside the beam-column connection and that of slab reinforcement, the strain of lateral reinforcement within a connection and that of column reinforcement at the critical section were measured by strain gauges. The loads applied by the actuators and beam-end support reactions were measured by load-cells.

2.4 LOADING HISTORY

In first two cycles, specimens were loaded in a longitudinal direction up to the half of an ultimate capacity calculated. Subsequently the yield story drift (Δ_y) was determined and the bi-directional story drift two and four times as large as the yield story drift as shown in Fig.3 was forced. If four times the yield story drift angle was larger than 1/50 rad, the applied story drift history was displaced by the story drift angle of 1/50 rad instead of four times the yield one.

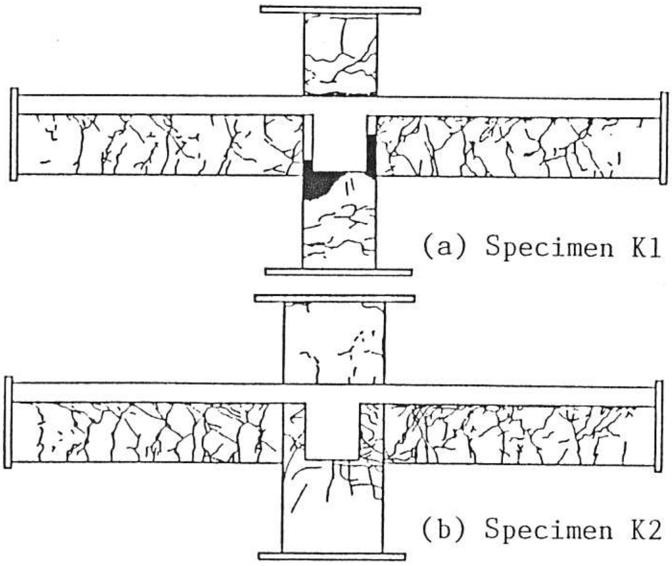


Story Drift $2\Delta_y$ Story Drift $4\Delta_y$

Fig.3 : Story Displacement Path of Loading

3. TEST RESULTS

The column reinforcement of Specimen K1 was observed to yield at a story drift angle of 1/139 rad during a loading in one direction when the beam reinforcement started to yield. The column reinforcement of Specimen K2 was observed to yield at a story drift angle of 1/108 rad during a loading in the two directions after the beam yielding.



(a) Specimen K1

(b) Specimen K2

Fig.4 : Crack Patterns after Test

3.1 CRACK PATTERNS

The crack patterns of the two interior connection specimens K1 and K2 observed at the end of loading are shown in Fig.4.

Specimen K1 developed a single and wide concentrated crack at the critical section and developed hardly any additional cracks in the beams after a story drift angle of 1/50 rad. The shell concrete spalled in the four corners near and within the connection at a story drift angle of 1/25 rad.

On the contrary, Specimen K2 developed fine cracks along the beams after a story drift angle of 1/54 rad. As expected, the bond situation of beam bars was much improved in the connection compared with Specimen K1. Cracks were observed more closely in the slab partially because the beams had to deform more in this specimen compared to the stiff columns.

3.2 HYSTERETIC CHARACTERISTICS

The story shear - story drift relations in the north-south direction are shown in Fig.5. The story drift at yielding was 10.6 mm for Specimen K1 and 6.8 mm for Specimen K2, the difference of which was attributable to the stiffness of the columns. The yield story drift was determined to be 1.33 times the story drift observed at three-quarters of the calculated ultimate load.

A story shear resistance in a direction, although the displacement might be maintained in the direction, could be reduced during the loading in the transverse direction due to the biaxial interaction of resistances. Such phenomenon could be observed between points A and B in Fig.5.

Specimens K1 and K2 showed a pinching hysteresis shape under cyclic load reversals. The equivalent viscous damping ratio is used to quantify the energy dissipating ability of hysteresis loops. The equivalent viscous damping ratio of specimens K1 and K2 were 0.07 and 0.12 respectively at the story drift angle of approximately 1/50 rad. This means that the hysteresis loop in Specimen K2 was fatter than that in specimen K1.

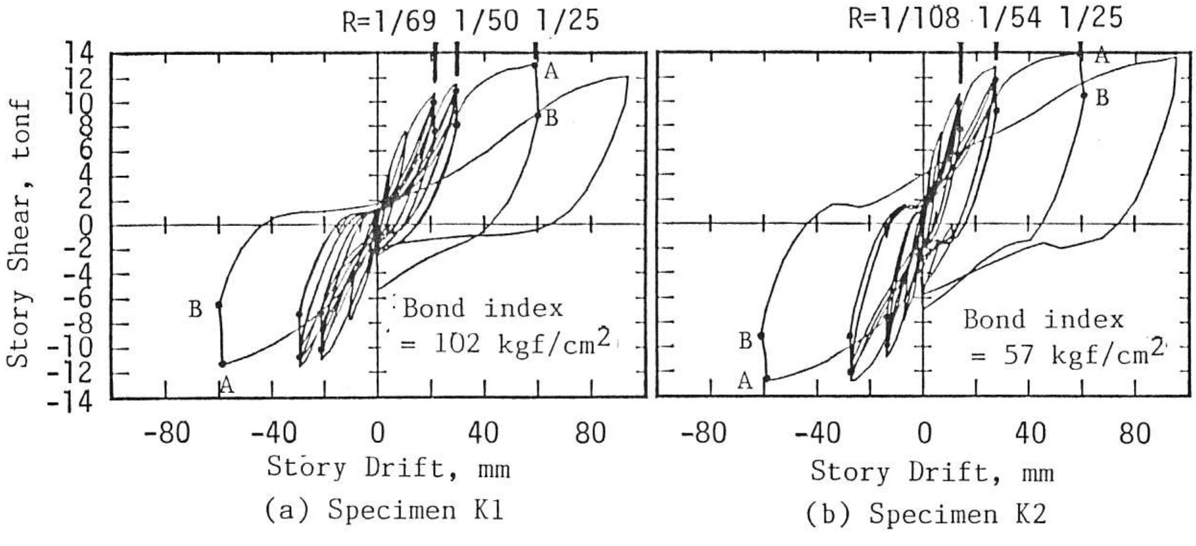


Fig.5 : Story Shear-Story Drift Relations

The behavior of a three-dimensional beam-column connection and a plane connection is compared using the specimens with comparable bond index values and subjected to comparable loading. The bond index value was 57 kgf/cm^2 for Specimen K2, and 52 kgf/cm^2 for a plane beam-column connection specimen tested previously (Ref.1). Since the plane beam-column connection specimen showed a good spindle-shape hysteresis, It is likely that the slab might contribute to the pinching in the shape of hysteretic loops. The total steel area of the top beam bars including all slab bars become 2.5 times that of bottom beam bars. The difference may be considered to have influence on the shape of hysteretic loops.

3.3 DISPLACEMENT CONTRIBUTION

The contribution of parts of a specimen to the story drift was calculated and shown in Fig.6. The contribution of the beam-column connection panel deformation was calculated as the total deflection less the contribution from the beam and column deflections. The deflection of beams for Specimen K2 shares 80 % of the total story drift in contrast to 60 % for Specimen K1. The difference of the beam contribution was caused by the difference in the stiffness of a column. The deformations of the connection and column are considered to have much influence upon a hysteretic behavior in Specimen K1.

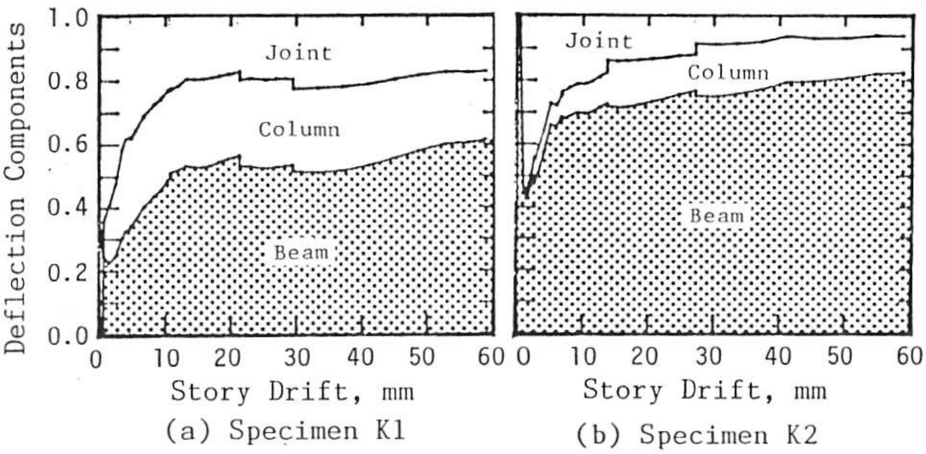


Fig.6 : Deflection Components of Story Drift

The contribution of local rotation in various regions along a beam to the beam deflection was calculated and shown in Fig.7. The rotation was measured over a D/6 distance from a column face, and over successive D/3, D/2 and D distances, where D is a beam overall depth (=300 mm). The four regions are called Region 1 to Region 4 from the column face. The rotation in Region 1 was caused mainly by the pull-out of beam bars from the connection. The deflection component of Region 1 of Specimen K1 reached 70 % of the total beam deflection at a beam deflection of 20 mm, indicating a large pull-out of beam bars from the connection. On the other hand, the deflection component of Region 1 of Specimen K2 was 50 % of the total beam deflection at the same deflection level.

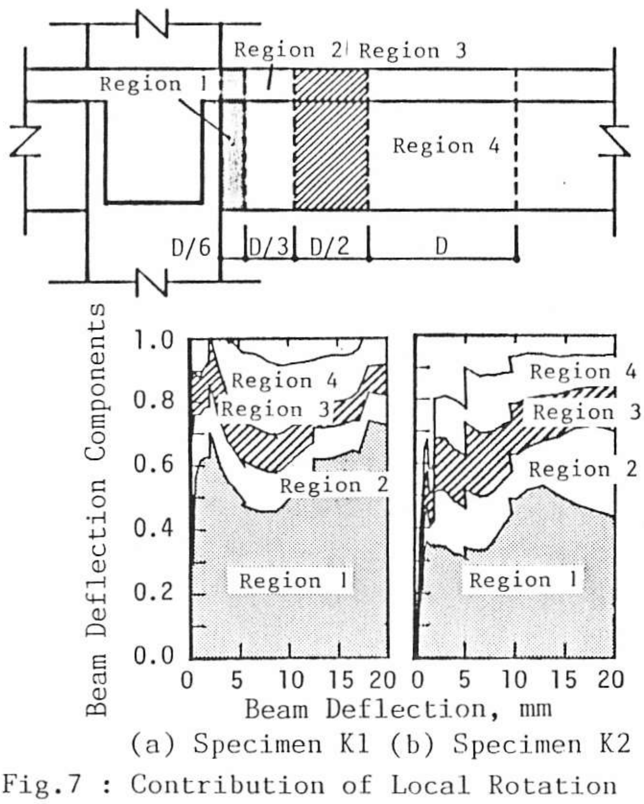


Fig.7 : Contribution of Local Rotation to Beam Deflection

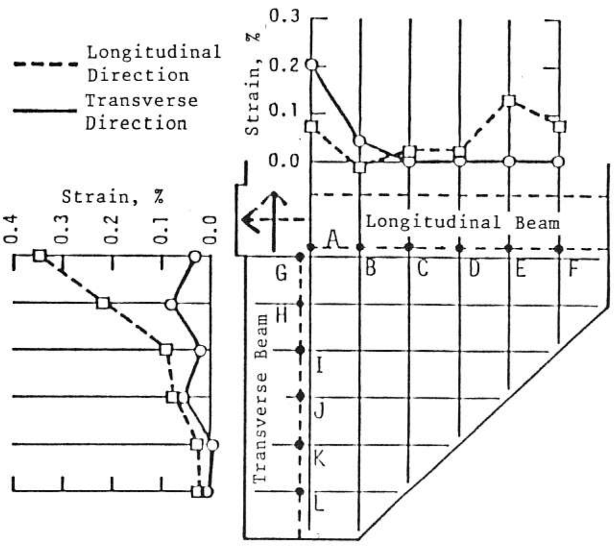


Fig.8 : Strain Distribution of Slab Bars

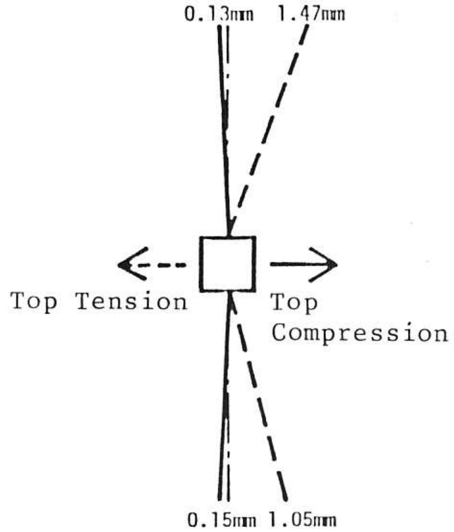


Fig.9 : Horizontal Deflection of Transverse Beams

3.4 BEHAVIOR of TRANSVERSE BEAMS and SLAB

Strain distribution of slab bars in the two directions is shown in Fig.8 at a story drift angle of approximately 1/120 rad. When the story shear was applied only in the longitudinal direction, the slab bars away from the column and parallel to the transverse beam showed tensile strain (locations E and F in Fig.19). On the contrary, during loading only in the transverse direction, the slab bars away from the column (locations K and L) indicated no strain and the slab bars near the column (locations G, H and I) developed tensile strains.

The horizontal deflection of the transverse beams is shown in Fig.9 at a story drift angle of 1/188 rad during the loading in the longitudinal direction. The transverse beams scarcely showed a horizontal deflection when the top of the logitudinal beam was compressed. But when the top fiber of the longitudinal beam was subjected to tensile stress, the transverse beams deflected in the horizontal plane by the tensile force exerted by the slab bars. This horizontal deflection was three or four times larger than that of the interior connection specimens.

For specimen K3, torsional cracks were observed in the transverse beams near the column during the loading in the longitudinal direction. But its width was small so that the transverse beams did not fail in torsion. When all slab bars yielded at the story drift angle of 1/25 rad, the torsional moment around the centroid of a transverse beam was 94.3 tonfxcm (=4.01 tonf/cm^2 x 0.32 cm^2 x 6 rebars x 12.25 cm). On the other hand, the ultimate capacity resisting pure torsion is 214.8 tonfxcm in calculating by the equation of Rangan-McMullen (Ref.3). Considering that the flexure, shear and torsion are act simultaneously, the value is reduced to 198.0 tonfxcm. In this test, the introduced torsional force was small so that torsional failure did not occur.

4. CONCLUDING REMARKS

From the test results, the following conclusions were drawn;

1) The interior beam-column subassemblage with slabs, designed to improve the bond of beam bars within a connection taking into account the beam bar bond index showed a pinching behavior. The bond of beam bars within a connection was considered to be good judging from the strain distribution of beam bars. It is considered that the slab contributed to the pinching in the shape of hysteretic loop.

2) For an exterior beam-column subassemblage with slabs only on one side, the transverse beams more deflected in the horizontal plane by the tensile force introduced by the slab bars than for the interior connection specimens. The transverse beams did not fail in torsion because the torsional resistance was sufficient to prevent the failure by torsion.

ACKNOWLEDGEMENTS

The authors are grateful to the assistance provided by Messrs. Y. Hosokawa and A. Tasai, research associates in Department of Architecture, and the members of Aoyama Laboratory, University of Tokyo, in executing the experimental work.

REFERENCES

1. Kitayama, K., K. Kurusu, S. Otani, and H. Aoyama, "Behavior of Beam-Column Connections with Improved Beam Reinforcement Bond,"Transaction of JCI, Vol 7, 1985, pp.551-558.

2. Kobayashi, K., M. Tamari, S. Otani, and H. Aoyama, "Experimental Study on Reinforced Concrete Beam-Column Subassemblages(in Japanese)," Transaction of 6th JCI Annual Meeting, 1984,pp.653-656.

3. McMullen, A. E. and B. V. Rangan, "Pure Torsion in Rectangular Sections- A Re-Examination," ACI Journal, October, 1978, pp.511-519.