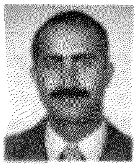
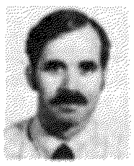
Title no. 82-17

Effect of Transverse Beams and Slab on Behavior of Reinforced Concrete Beam-to-Column Connections

by M. R. Ehsani and J. K. Wight

Experimental results on the tests of six reinforced concrete beam-column subassemblies with transverse beams and slab are presented. The primary variables were the ratio of the column flexural capacity to that of the beam and slab, the joint shear stress, and the transverse reinforcement in the joint. The specimens were subjected to inelastic cyclic loading. The results are compared to the behavior of similar specimens without transverse beams and slab.

Keywords: beam-column frame; **beams (supports); columns (supports);** concrete slabs; connections; cyclic loads; **earthquake resistant structures;** hinges (structural); joints (junctions); reinforced concrete; **shear properties;** structural analysis.

To obtain better understanding of reinforced concrete structures subjected to earthquake forces, many researchers have studied the behavior of beam-to-column connections. The first tests of reinforced concrete beam to column connections were conducted in the early 1960s by Hanson and Conner[1,2] and have been used since as a benchmark for later studies. Hanson and Conner concluded that designing shear reinforcement for a connection according to the equations developed for reinforced concrete beams would result in satisfactory performance of the connection under cyclic loading. This problem has been investigated by other researchers in the United States,[3,4] as well as in Canada[5] and New Zealand.[6]

The results of the above studies were used by the ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures, to develop its first design guidelines in 1976.[7] These guidelines were based on the assumption that the concrete and transverse reinforcement in the joint act collectively to resist the shear forces in the joint. A sufficient amount of transverse reinforcement was to be provided to resist any shear stresses beyond the shear capacity of the concrete in the joint. ACI-ASCE Committee 352 is in the process of developing new design recommendations using the more recent test results.[8-13]

Although the attempt of Committee 352 has been to avoid congestion of confinement reinforcement in the joint, in many cases joints designed according to these guidelines are congested and difficult to construct. The recommendations of the committee are based primarily on the tests of isolated subassemblies, i.e., beams and columns which did not have any transverse beams and slab. However, in most reinforced concrete frames, where transverse beams and slab are present, the confinement provided by the transverse beam should not be ignored. Except for one study of interior connections tested by Meinheit and Jirsa,[8] none of the earlier tests included transverse beams and slabs. However, the transverse beams in Meinheit and Jirsa's specimens were not loaded during the tests.

RESEARCH SIGNIFICANCE

The research results reported in this paper are intended to clarify the effect of key variables on the behavior of reinforced concrete beam-to-column connections under large load reversals. An attempt was made to model an entire beam-column subassembly by including a floor slab and transverse beam. This research is aimed at scientists and engineers interested in seeking a feasible earthquake resistant design procedure for beam to column connections in reinforced concrete frame structures.

EXPERIMENTAL STUDY

Objective

The primary objective of this study was to investigate the behavior of beam-column subassemblies having transverse beams and slab.

Construction of the specimens

Six exterior reinforced concrete beam to column subassemblages with transverse beams and slabs were constructed and tested. The configuration of these speci-

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mens is shown in Fig. 1, and the dimensions and the reinforcement details are presented in Table 1. Some of the top beam reinforcement was placed in the slab and thus was anchored outside the confined column core. The beams and columns were properly designed to prevent any shear failure in these elements.

Specimens were cast upright to simulate construction procedure in the field. Average test-day strength for the concrete cylinders and a summary of the yield stresses for the reinforcing steel is given in Table 1.

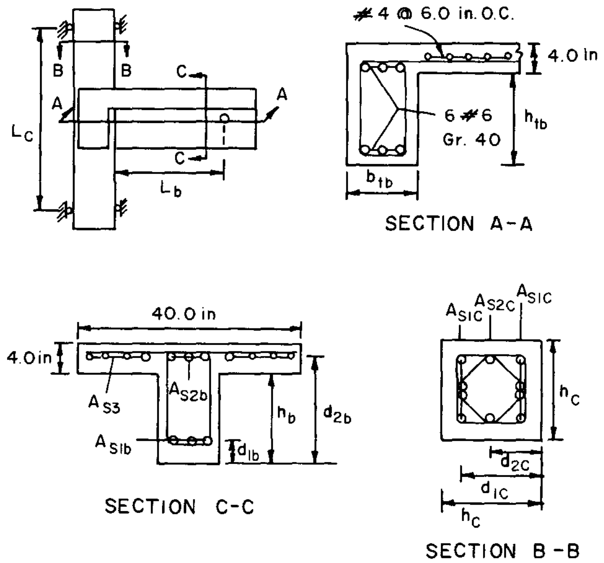

Fig. 1—Configuration and dimension designation for the specimens (1 in. = 25.4 mm)

Primary variables

The parameters investigated include (1) the flexural strength ratio M_R, defined as the sum of the flexural capacities of the columns to that of the beam, (2) the percentage of transverse reinforcement used within the joint, ρ_t, and (3) the shear stress in the joint as a multiple of $\sqrt{f'_c}$ defined as γ.

In addition, for every specimen with transverse beams and slab discussed in this paper, a companion specimen with the same design parameters but without transverse beams and slab was tested. Details of the specimens without transverse beams and slab, which are given in Reference 14, are not included in this paper.

Flexural strength ratio — The original ACI-ASCE Committee 352 recommendations[7] called for flexural strength ratios greater than 1.0. The specimens in this test series were designed to have flexural strength ratios of 1.1, or 1.5, or 2.0.

Transverse reinforcement — Specimens were constructed with either two or three sets of reinforcement at the joints. Each set of hoops consisted of a square tie enclosing all column longitudinal bars and a diamond-shaped tie enclosing only the intermediate column longitudinal reinforcement as shown in Fig. 1.

Joint shear stress — The design joint shear stresses varied between $10\sqrt{f'_c}$ and $14\sqrt{f'_c}$ psi ($0.83\sqrt{f'_c}$ and $1.16\sqrt{f'_c}$ MPa). In calculating the joint shear stresses, the actual yield stresses for the beam and slab longitudinal reinforcement were increased by 10 percent to account for the strain hardening effects.

Prior to testing of the specimens, when calculating the flexural strength ratio and joint shear stresses, it

Table 1 — Physical dimensions and properties of the specimens

Designation	Specimen number					
	1S	2S	3S	4S	5S	6S
L_c, in.	87.0	87.0	87.0	87.0	87.0	87.0
h_c, in.	11.8	11.8	11.8	11.8	13.4	13.4
d_{1c}, in	9.8	9.8	9.8	9.8	11.4	11.4
d_{2c}, in.	5.9	5.9	5.9	5.9	6.7	6.7
A_{s1c}*	3#6	3#6	4#6	4#6	4#8	3#6
A_{s2c}*	2#6	2#6	2#6	2#6	2#8	2#6
L_b, in.	42.0	42.0	42.0	42.0	42.0	42.0
h_b, in.	14.9	14.9	13.3	13.3	14.9	14.9
b_b, in.	10.2	10.2	10.2	10.2	11.8	11.8
d_{1b}, in.	2.0	2.0	2.0	2.0	2.0	2.0
d_{2b}, in.	16.9	16.9	15.4	15.4	16.9	16.9
A_{s1b}†	3#6	3#6	3#6	3#6	3#7	3#7
A_{s2b}†	3#6	3#6	3#6	3#6	3#7	3#7
A_{s3}*	1#6, 3#4	1#6, 3#4	1#6, 3#4	1#6, 3#4	1#7, 3#4	4#4
h_{tb}, in.	14.9	14.9	13.3	13.3	14.9	14.9
b_{tb}, in.	10.2	10.2	10.2	10.2	11.8	11.8
Hoops‡	2	3	2	3	2	2
f'_c, psi:						
lower column	4240	3940	3950	3760	3490	3630
beams + slab	6180	5730	4200	4260	3470	5090
upper column	4390	3910	3930	3900	3470	5090

Note: 1 in. = 25.4 mm; 1 psi = 0.0069 MPa.
*Summary of column steel yield strengths, in ksi; bar size #6 = 71.0, #8 = 60.0.
†Summary of beams and slab steel yield strengths, in ksi; bar size #4 = 51.0, #6 = 50.0, #7 = 48.0.
‡Number of sets of #4 hoops in the joint with yield stress of 63.4 ksi.

Table 2 — Design and actual values for the primary variables

Specimen number	P/A_g, psi	M_R*	$\frac{V_j^†}{bh_c\sqrt{f'_c}}$, psi	ρ_t, percent	h_b/column bar diameter
1S	358	1.1 (0.89)	14.0 (10.9)	0.8 (0.77)	25.1
2S	358	1.1 (0.87)	14.0 (11.3)	1.2 (1.16)	25.1
3S	358	1.5 (1.17)	14.0 (13.5)	0.8 (0.86)	23.0
4S	358	1.5 (1.16)	14.0 (13.4)	1.2 (1.30)	23.0
5S	446	2.0 (1.58)	14.0 (14.4)	0.8 (0.68)	18.9
6S	380	1.5 (1.17)	10.0 (9.1)	0.8 (0.68)	25.1

Note: 1 psi = 0.0069 MPa; $1.0\sqrt{f'_c}$, psi = $0.083\sqrt{f'_c}$, MPa. Numbers outside parentheses are the design values; numbers inside are the actual values.
*Assuming that all slab longitudinal reinforcement is effective in tension.
†Assuming that only the two slab longitudinal reinforcing bars on each side of the main beam contribute to the joint shear.

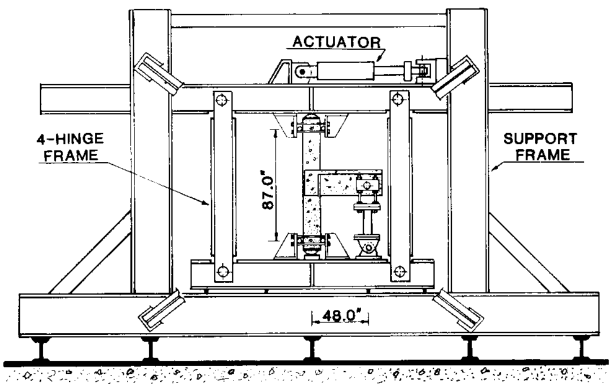

Fig. 2—Testing frame (1 in. = 25.4 mm)

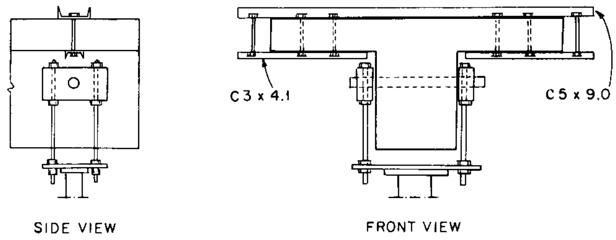

Fig. 3—Slab load point stiffeners

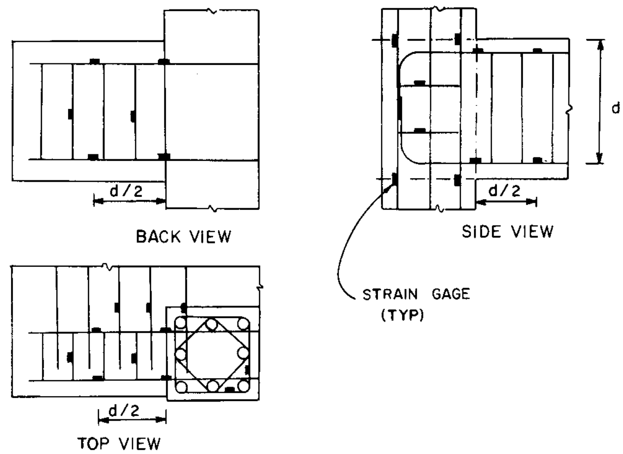

Fig. 4—Location of strain gages in the specimens

was assumed that only the first two longitudinal slab reinforcements adjacent to the main beam should be included. During the tests, however, it was observed that all slab longitudinal reinforcement yielded. The yielding of the longitudinal bars gradually spread across the width of the slab. Data from the strain gages indicated that the first and the second longitudinal bars away from the main beam yielded during the second and third cycles of loading, respectively. There were not any strain gages attached to the third and fourth longitudinal bars in the slab. However, based on the visual inspection of the crack widths, it was concluded that these bars yielded during the fourth or fifth cycles of loading. Therefore, a second flexural strength ratio was calculated, assuming all longitudinal reinforcement in the slab to be effective in tension. This resulted in lower M_R values than did the original design.

Because the two longitudinal reinforcing bars near the edge of the slab were too far from the joint, the joint shear stresses were calculated using the original assumption that only two longitudinal reinforcing bars on each side of the beam contribute to the joint shear stresses. The design and actual values for the specimens are presented in Table 2.

Test setup and instrumentation

The specimens were tested in the frame shown in Fig. 2. The frame consisted of a 4-hinge steel frame supported by a larger steel reaction frame. The lower beam of the 4-hinge frame was bolted to the support frame. Pin-ended columns supported the top beam of the 4-hinge frame, which was free to move horizontally in the plane of the frame.

The ends of the column portions of the specimens were tied to the steel brackets on the frame. Rollers were provided to simulate points of inflection. A specially constructed force link supported the free end of the beam. The end of the slab was stiffened externally, as shown in Fig. 3, so that the beam shear force would be distributed over the entire width of the slab. The actuator on the top beam of the 4-hinge frame was used to apply shear forces at the assumed top inflection point in the column.

Approximately thirty electrical resistance strain gages were attached to the reinforcing bars near the joint as shown in Fig. 4. During each cycle of loading, the loading was temporarily stopped while the strain gage measurements were automatically recorded.

Loading sequence

During each test, the column axial load, which was less than 40 percent of the column balanced axial load, remained constant. The ratio of the axial load to the cross-sectional area for all columns is listed in Table 2. The reversing shear forces were applied to the top column inflection point according to the displacement controlled schedule shown in Fig. 5. The specimen was first loaded in the positive direction (slab in tension) to its yield displacement. The yield displacement was determined by observing a flattening of the plot of the applied load versus the load point displacement. The specimen was then unloaded and displaced in the negative direction to the negative yield displacement, noting that the negative yield displacement was much smaller than the yield displacement in the positive direction. For each subsequent cycle of loading, the maximum displacement was increased by one-quarter of the yield displacement.

EXPERIMENTAL RESULTS

General behavior

Plots of the applied load versus the load point displacement for all specimens are presented in Fig. 6(a) through 6(f). In all specimens, flexural cracks were observed in the beam and slab near the column. These cracks extended at least a distance equal to one and a half times the depth of the beam from the face of the column. Starting with the second cycle of loading, torsional cracks were formed on the back of the specimen at midheight of the transverse beams. As shown in Fig. 7, these cracks followed a spiral path and terminated at

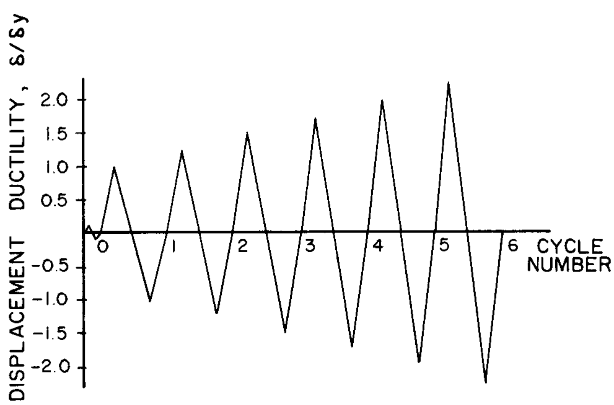

Fig. 5—Loading history

the front face of the column after crossing a short section of the slab.

After the second cycle of loading, one or two flexural cracks at the bottom of the beam near the column

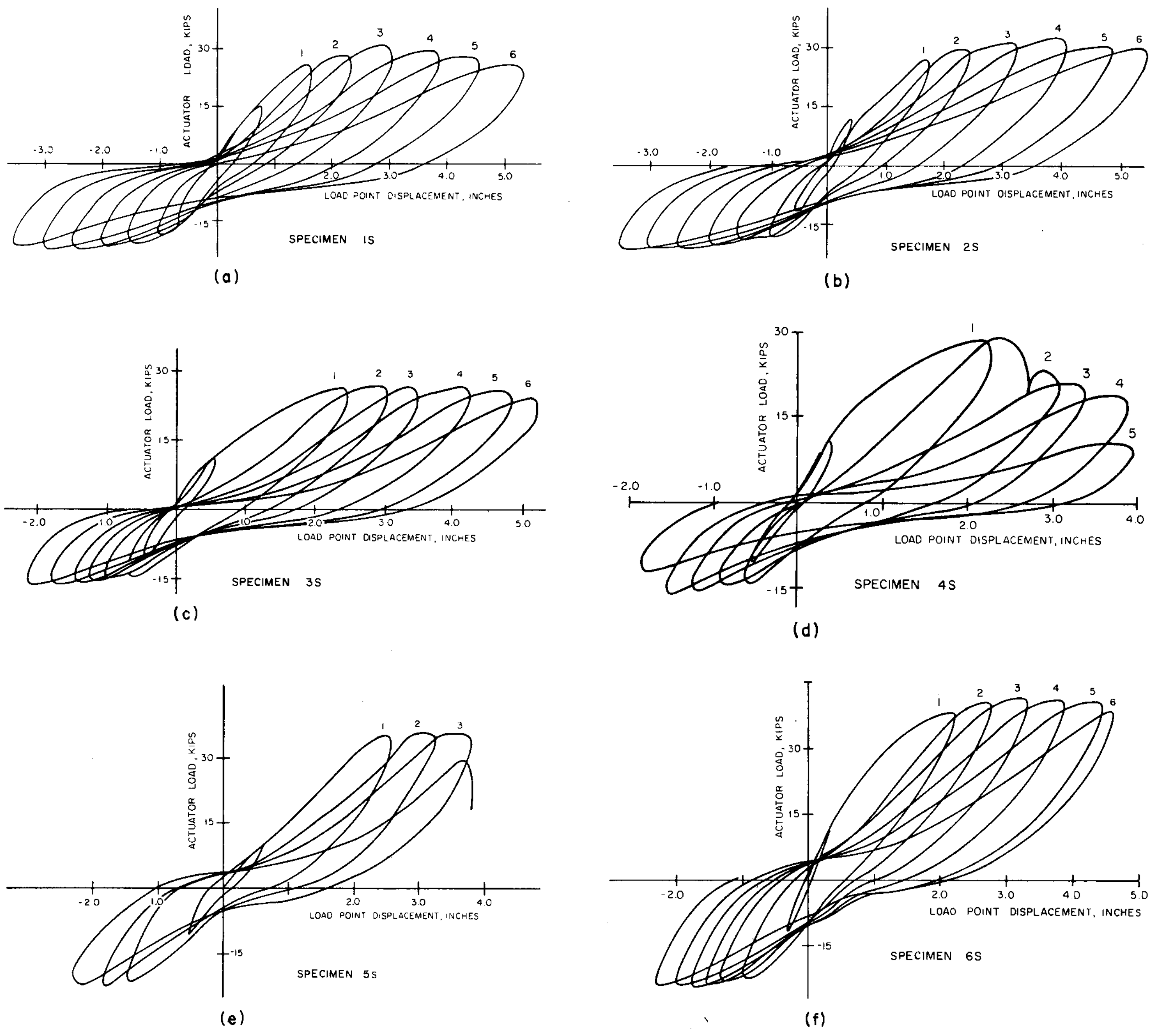

Fig. 6—Load versus deflection response for Specimens 1S through 6S, respectively

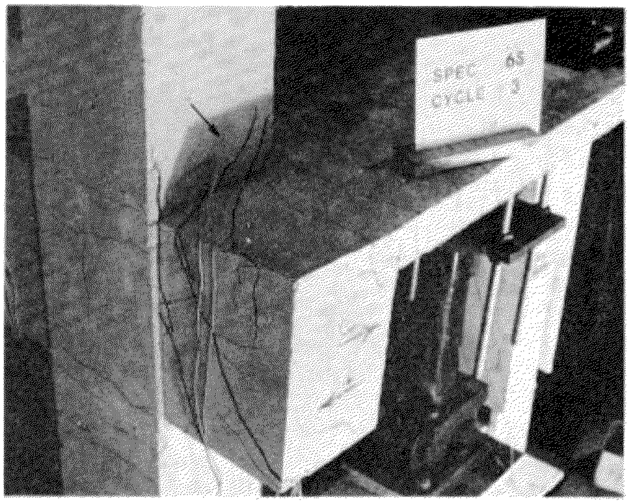

Fig. 7—Torsional cracks in the transverse beams and column of Specimen 6S

remained open during both directions of loading as shown in Fig. 8. The full depth of cracks caused a noticeable unequal loss of stiffness or "pinching" of the hysteresis loops in the two directions of loading. This behavior was due to the unequal amount of longitudinal steel in the slab and the beam and has been observed by other researchers.[3,11] For all specimens, the area of the longitudinal reinforcement at the bottom of the beam was much less than that at the top of the beam and slab. During the positive half-cycles of loading (slab in tension), flexural cracks formed in the slab and propagated into the beam. When the loading direction was reversed, the tensile force at the bottom of the beam could not create a large enough compression force in the slab to close these cracks. Therefore, the newly formed flexural cracks at the bottom of the beam joined the cracks at the top of the beam. After additional cycles of loading, the width of these cracks increased considerably and as a result the stiffness of the subassemblage was reduced.

Specimen 4S failed near the free end of the beam during the second cycle of loading. The slab longitudinal reinforcement directly above the web of the main beam was accidentally bent into the web of the main beam near the beam loading point. Therefore, the beam and the slab were separated near the beam loading point. Due to this failure, most of the damage in the subsequent cycles of loading was concentrated near the beam end. Specimen 5S was accidentally loaded in the testing frame before the specimen was properly tied to the frame. Several cracks were observed near the beam loading point. Although as a precautionary measure the region near the free end of the beam was externally tied to the slab, the stiffness of the specimen was considerably lower in the first cycle, and the specimen failed near the beam loading point during the fourth cycle of loading.

Based on the hysteretic curves of the subassemblies, behavior of Specimens 1S, 2S, and 6S was determined to be satisfactory. These three specimens were capable

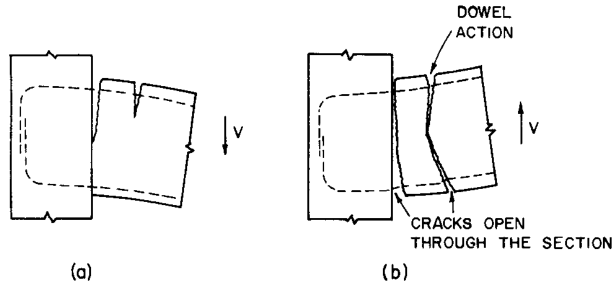

Fig. 8—Opening and closing of flexural cracks during loading in (a) the positive direction and (b) the negative direction

of carrying larger loads than that of the first cycle for all cycles of loading. Specimen 3S maintained its maximum first cycle load for the first four cycles of loading. Due to the premature failure at the beam loading point, no conclusive observation could be made for Specimens 4S and 5S.

Bar slippage

A major cause of the loss of stiffness for beam-column subassemblies is the slippage of the column longitudinal reinforcement through the joint and the pullout of the beam longitudinal reinforcement from the joint. Adequate confinement of the joint will result in smaller shear cracks in the joint which will help reduce bar pullout and slippage.

Due to the improved confinement provided by the transverse beams, pullout of the beam longitudinal bars was not recorded for any of the specimens tested. However, several column longitudinal bars did slip through the joint. Strains measured just above the joint on the column longitudinal reinforcement during the first two cycles of loading for Specimen 1S are shown in Fig. 9. During the first half-cycle of loading, the strains at the indicated location were expected to continue along the dashed line to indicate an increase in the compressive strains.

At the bottom of the connection, this same column reinforcing bar was subjected to tensile strains which would have been very near or above the yield strain because the design moment ratio for this specimen was 1.1. Due to the high bond stresses created by the loading conditions on this bar, it is assumed that a bond failure occurred and the bar slipped within the joint. Therefore, the strains started to decrease at the indicated locations as the load increased.

The maximum measured strain at the same location on the column longitudinal reinforcement during each positive half-cycle of loading (slab in tension) for each specimen is shown in Fig. 10. These strains should remain in the positive region of the plot to indicate compression in the bars. Also, because the displacements increase in every cycle of loading, the measured strains are expected to increase. If the strains remain the same or decrease, a slippage of the reinforcement has taken place. As shown in Fig. 10, the column longitudinal reinforcement in Specimens 1S and 3S started

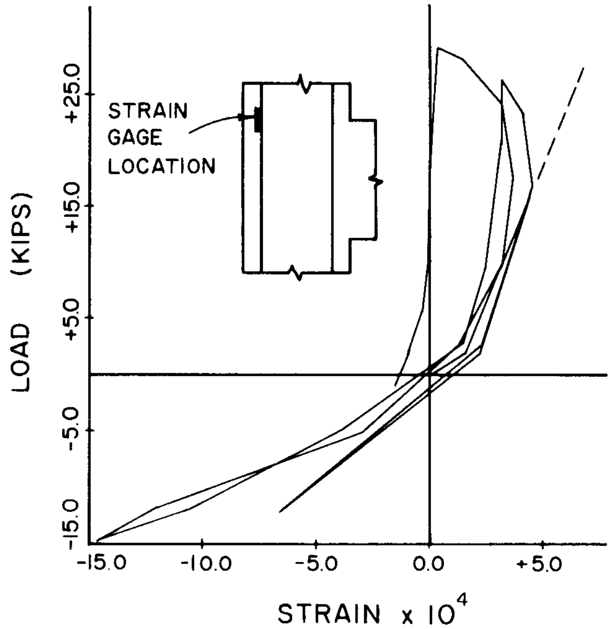

Fig. 9—Strain gage data indicating the slippage of column longitudinal reinforcement in Specimen 1S

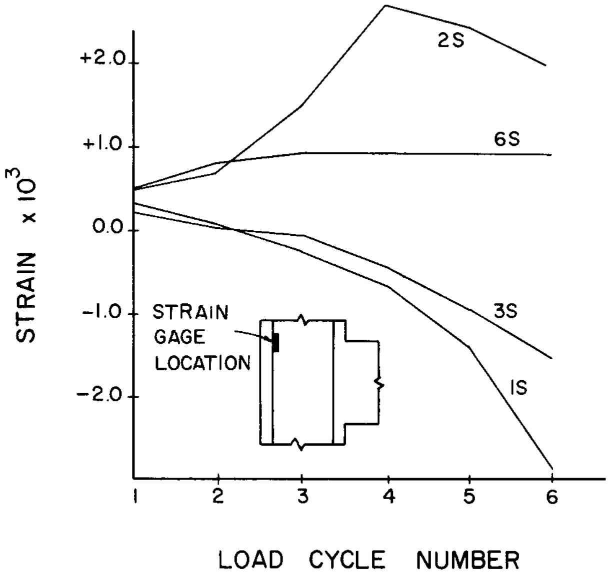

Fig. 10—Maximum strain during each cycle of loading in column longitudinal reinforcement

to slip after the first cycle of loading, and tensile strains were recorded after the second cycle of loading. Specimens 2S and 6S, respectively, showed a slight amount of slippage after the fourth and the third cycles of loading. The ratios of joint depth to column bar diameter are given in column 5 of Table 2.

Effect of M_R

Due to the yielding of all slab longitudinal reinforcement, the actual flexural strength ratios for the specimens were lower than the design values. In Specimens 1S and 2S, where the flexural strength ratios were smaller than 1.0, flexural hinges were formed in the upper column near the slab. As shown in Fig. 11, the concrete in the column above the joint was crushed. In Specimen 3S, which had a flexural strength ratio slightly greater than 1.0 and relatively high joint shear stresses, the flexural hinges were formed in the joint. Specimen 6S had the same flexural strength ratios as Specimen 3S, but due to the lower joint shear stresses, the flexural cracks extended into the beam for a distance of approximately twice the depth of the beam from the face of the column.

The hysteretic behavior of Specimens 1S, 2S, and 6S was superior to that of Specimen 3S. Based on this observation, it is evident that as far as the cyclic load-carrying capacity of the specimen is concerned, specimens for which the flexural hinging occurs outside of the joint region (in the beam or column) demonstrate a more stable behavior than the specimens for which the majority of the damage is concentrated in the joint. Column hinging, however, may cause structural stability problems.

Effect of ρ_t

Transverse reinforcement in the joint is needed to resist the shear forces and to provide confinement for the

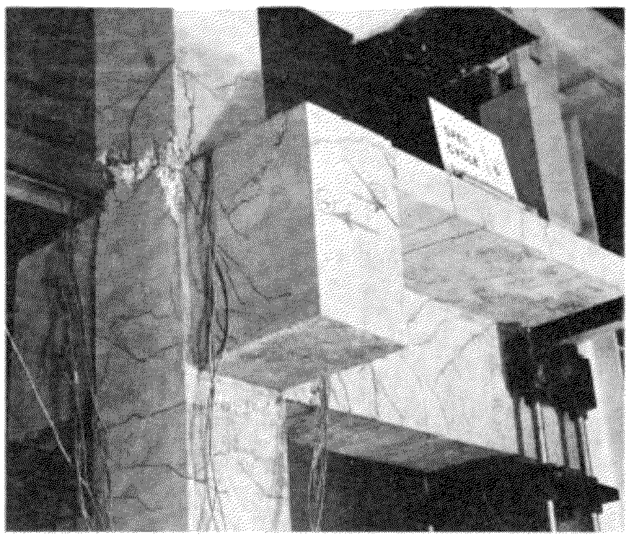

Fig. 11—Flexural hinging in the column of Specimen 2S

concrete in the joint. The presence of transverse beams, which are not directly loaded, also enhances the joint confinement. The only difference in the values for the design parameters for Specimens 1S and 2S was the higher amount of joint transverse reinforcement in Specimen 2S. Although in tests of specimens without transverse beams and slab a significant improvement in the behavior of subassemblies with more joint transverse reinforcement was observed,[14] the behavior of Specimens 1S and 2S were practically the same. It was thus concluded that, due to the confining effects of the transverse beams, the increase in the joint transverse reinforcement ratio did not significantly improve the behavior of the subassemblage.

The major benefit from the additional hoop in the joint of Specimen 2S was the delay of bar slippage. The additional set of hoops limited the width of the shear cracks in the joint core. As shown in Fig. 10, this resulted in a delay of the column bar slippage in Specimen 2S as compared to Specimen 1S.

Effect of γ

Specimens 3S and 6S were designed to have the same values for the primary variables except that the joint shear stress in Specimen 6S was smaller than that in Specimen 3S. Due to the presence of the transverse beams, visual comparison of the joint damage for these two specimens was impossible. However, specimen 6S had a more stable hysteretic behavior. The load carried by Specimen 6S during the fifth cycle of loading was 106 percent of that of the first cycle of loading, while Specimen 3S was capable of carrying 96 percent of its first cycle load during the fifth cycle of loading.

This improvement in behavior was not as noticeable as for the case of specimens without the transverse beams and slab.[14] In specimens without the transverse beams, the joint shear forces are resisted by the core of the joint. For specimens with transverse beams and slab, some of the beam longitudinal bars contributing to the joint shear forces are anchored in the transverse beams. Although the shear stresses are calculated assuming only the core of the joint to be effective in resisting the joint shear stresses, the true area resisting these shear stresses should include a certain portion of the transverse beams near the joint. As a result, a reduction in the design joint shear stress in specimens with transverse beams and slab does not result in as significant an improvement in the overall behavior as it would for specimens without the transverse beams and slab.

Lower joint shear stresses did delay the column bar slippage through the joint. As shown in Fig. 10, the column longitudinal reinforcement in Specimen 3S started to slip after the first cycle of loading while the same bar in Specimen 6S, which had lower shear stresses, slipped very little after the third cycle of loading.

Effect of transverse beams and slab

The behavior of these specimens was compared to six additional specimens without the transverse beams and slab but which were designed with the same values for the primary variables. The complete description of the behavior of the specimens without the transverse beams and slab is presented elsewhere.[14]

The hysteresis diagrams for the specimens with transverse beams and slab demonstrated unequal pinching during the positive and negative half cycles of loading. This was primarily due to the presence of flexural cracks at the bottom of the main beam near the column, which remained open throughout the test.

Due to the test setup used, the transverse beams were subjected to a combination of bending and torsional loading. This behavior was verified by the data from the strain gages attached to the longitudinal and shear reinforcement in the transverse beam. This caused additional cracking of the concrete near the joint. On the other hand, transverse beams provided additional confinement for the joint. The net result of these two behaviors was beneficial and the confinement of the joint in specimens with transverse beams and slab improved significantly over similar specimens without the transverse beams and slab.

The strains measured in the joint transverse reinforcement in specimens with transverse beams and slab were lower than those in similar specimens without transverse beams and slab. This observation indicates that, in specimens with transverse beams and slab, some of the joint shear stresses are resisted by the concrete outside the joint core, thus leaving a smaller percentage of the shear stresses to be resisted by the hoops.

The improved confinement of the joint in specimens with transverse beams and slab limited the width of the shear cracks in the joint. This resulted in an elimination of the beam bar pullout in all specimens which had transverse beams and slab. Beam bar pullout was recorded, however, for all specimens without transverse beams and slab.[14] Slippage of the column longitudinal reinforcement through the joint was observed in specimens with and without the transverse beams and slab.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this investigation was to evaluate the effects of the flexural strength ratio, joint shear stresses, joint transverse reinforcement ratio, and the presence of transverse beams and slabs in beam-column subassemblies. The following conclusions are based on the results of the specimens tested:

1. Subassemblages, in which the flexural hinges are formed outside of the joint (in the beam or column), exhibit a more stable behavior than those for which flexural hinges are formed in the joint. Column hinging may, however, cause structural instability.

2. The flexural strength ratio M_R at the connections is reduced significantly due to the contribution of the slab longitudinal reinforcement. It is recommended that to insure flexural hinging in the beam the flexural strength ratio should be no less than 1.2. Furthermore, when calculating the flexural strength ratio, the slab longitudinal steel over a region at least equal to the width of the beam on each side of the main beam must be considered effective.

3. The effective width of slabs in tension is not well defined. Athough in the specimens tested the entire width of the slab was found to be effective in tension, a more detailed study of this problem is required. Information is also needed on accurate assessment of the shear forces and shear stresses in connections where transverse beams and slabs are present.

4. The presence of transverse beams, which are not loaded directly, considerably improves the joint behavior.

5. An increase in the amount of joint transverse reinforcement in specimens with the transverse beams and slab did not improve the overall behavior of the specimens as much as it did for the specimens without transverse beams and slab.

6. The presence of transverse beams helped eliminate the beam bar pullout. However, slippage of column

longitudinal reinforcement was observed in specimens with and without transverse beams and slab.

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NOTATION

A_g = cross-sectional area of column, in.^{2}
A_{s1c} = area of tension reinforcement in column, in.^{2}
A_{s2c} = area of intermediate longitudinal reinforcement in column, in.^{2}
A_{s1b} = area of tension reinforcement in beam, in.^{2}
A_{s2b} = area of slab longitudinal reinforcement directly above the web portion of main beam, in.^{2}
A_{s3} = area of slab longitudinal reinforcement on each side of main beam, in.^{2}
b = width of column, in.
b_b = width of main beam, in.
b_{tb} = width of transverse beam, in.
d_{1b} = distance from compression face to centroid of compression reinforcement in beam, in.
d_{2b} = distance from compression face to centroid of slab longitudinal reinforcement, in.
d_{1c} = distance from compression face to centroid of tension reinforcement in column, in.
d_{2c} = distance from compression face to centroid of intermediate longitudinal reinforcement in column, in.
f'_c = compressive strength of concrete, psi
h_b = total depth of beam, in.
h_c = total depth of column parallel to the direction of shear, in.
h_{tb} = total height of transverse beam, in.
L_b = length of beam section of specimen between the beam loading point and the front face of column, in.
L_c = length of column portion of specimens held between simple supports, in.
M_R = flexural strength ratio, sum of the flexural capacities of column sections at a joint to that of the beams
P = applied column axial load during test, kips
V_j = joint shear force, kips
γ = joint shear stress as a multiple of $\sqrt{f'_c}$
ρ_t = transverse reinforcement ratio, percent

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