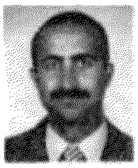
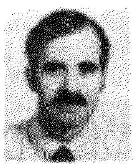
Title no. 82-43

Exterior Reinforced Concrete Beam-to-Column Connections Subjected to Earthquake-Type Loading

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

The experimental results from reversed cyclic loading tests on six exterior reinforced concrete beam-column subassemblies are presented. The primary variables were the ratio of the column-to-beam flexural capacity, the joint shear stress, and the transverse reinforcement in the joint. The specimens were subjected to cyclic load reversals at deflection levels intended to represent the levels that could be obtained during a moderate or severe earthquake. The results are compared with the existing design recommendations. It is concluded that in some cases the present design recommendations could be safely relaxed.

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

Investigating the behavior of reinforced concrete beam-to-column connections has attracted many researchers in the past 20 years. When a reinforced concrete ductile moment-resisting frame is subjected to large seismic lateral forces, the beam-to-column connections must be capable of carrying large shear forces which are accompanied by large deformations. The first behavior studies of beam-to-column connections were conducted at the Portland Cement Association laboratories by Hanson and Conner.[1,2] Since then the problem has been studied by other investigators in the U.S.,[3-6] as well as in Canada,[7] Japan,[8] and New Zealand.[9,10] Although the objectives have varied, the main emphasis of these studies has been to develop guidelines which would insure proper anchorage of beam bars in the joint and provide ductile behavior under repeated cyclic loading.

As a result of these studies, the design philosophy for beam-to-column connections has changed considerably over the past decade. The ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures, published its first design recommendations in 1976.[11] At that time, it was generally assumed that the shear capacity of a connection is the sum of the capacities of the concrete and steel. The contribution of the concrete was a function of the type of loading and the confinement provided for the joint by the transverse beams. The contribution of closed reinforcing hoops was similar to that of shear stirrups in the beams. Additional rules were also provided for detailing joints in seismic regions.

The recommendations of Committee 352 have been undergoing major revisions in the past few years. According to the draft recommendations of ACI Committee 352, which were available at the time this research project was initiated (subsequently referred to as the draft recommendations), the contributions of steel and concrete are not treated separately. It is now believed that a major factor in performance of a connection is the confinement of the concrete in the joint core. It is assumed that the confining steel makes no direct contribution to resisting the shear forces in the joint. According to the draft recommendations, the designer only needs to provide adequate column confining reinforcement through the connection and then limit the nominal joint shear stresses to $12\sqrt{f'_c}$ psi ($1.0\sqrt{f'_c}$ MPa) for external connections. If the ultimate joint shear stresses are found to be higher than the recommended ultimate value, the dimensions of the connection must be increased to reduce the shear stresses rather than providing more transverse reinforcement. Increased transverse reinforcement usually causes increased congestion in the joint.

RESEARCH SIGNIFICANCE

This study investigates the effect of key variables on the behavior of external reinforced concrete beam-to-column connections subjected to earthquake-type loading. The material presented is aimed at engineers who are interested in designing reinforced concrete frame structures capable of resisting earthquake motions.

Received Aug. 19, 1983, and reviewed under Institute publication policies. Pertinent discussion will be published in the May-June 1986 ACI JOURNAL if received by Feb. 1, 1986.

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Table 1 — Physical dimensions and properties of the specimens

Designation	Specimen number 1B	2B	3B	4B	5B	6B
L_c, in.	84.0	84.0	84.0	84.0	87.0	87.0
h_c, in.	11.8	11.8	11.8	11.8	13.4	13.4
d_{1c}, in.	9.6	9.6	9.6	9.6	11.4	11.4
d_{2c}, in.	5.9	5.9	5.9	5.9	6.7	6.7
A_{s1c}*	3#6	4#6	3#6	4#6	4#8	3#6
A_{s2c}*	2#6	2#6	2#6	2#6	2#8	2#6
L_b, in.	60.0	60.0	60.0	60.0	42.0	42.0
h_b, in.	18.9	17.3	18.9	17.3	18.9	18.9
b_b, in.	10.2	10.2	10.2	10.2	11.8	11.8
d_{1b}, in.	16.9	15.4	16.9	15.4	16.9	16.9
d_{2b}, in.	15.0	13.4	15.0	13.4	15.0	15.0
A_{s1b}†	3#7	3#7	3#7	3#7	3#7	3#7
A_{s2b}†	3#6	3#6	3#6	3#6	3#7	2#6
Hoops‡	2	2	3	3	2	2
f'_c, psi	4870	5070	5930	6470	3530	5770

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

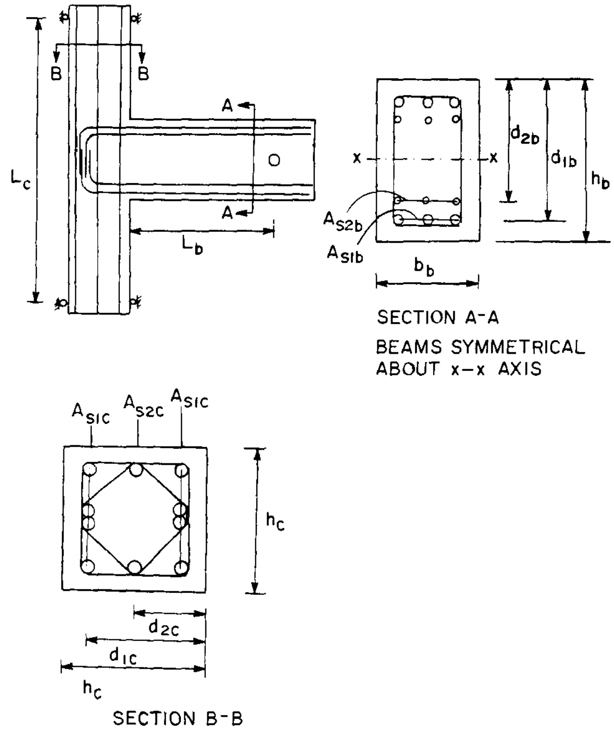

Fig. 1—Configuration and dimension designation for the specimens

TEST PROGRAM

Objective

In many cases the draft recommendations result in congested joints which are very difficult to construct. The main objective of this study was to show that in certain cases, joints reinforced with a smaller percentage of transverse steel than that suggested by the draft recommendations will exhibit satisfactory performance.

Construction of the specimens

Six exterior reinforced concrete beam-to-column connections were constructed and tested. The specimens were designed in the fall of 1978. A detailed description of the specimens is provided in Reference 12. The configuration of these specimens are presented in Fig. 1 and their physical dimensions are listed in Table 1. Adequate shear reinforcement was provided in the beam and columns outside of the joint to prevent shear failure in the beam or the column.

All specimens were cast flat rather than vertical as in actual construction. After the reinforcing cage was assembled and placed inside the formwork, the concrete was placed and internally vibrated. Specimens were moist cured for a period of seven days in the forms. Forms were removed after seven days and the specimens were stored in the laboratory until they were tested.

Primary variables

The effects of the following primary variables on the performance of the connections were studied: (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.

Most building codes recognize the advantages of having stronger columns than beams at every connection. However, the minimum code requirements are usually satisfied by providing flexural strength ratios slightly greater than 1.0. To investigate the effect of this parameter, flexural strength ratios were varied between 1.1 and 2.0.

The joint transverse reinforcement can contribute to the overall connection behavior in two ways. First, it can provide potential shear forces with an upper limit equal to the area of the hoops times its yield stress. Second, the hoops can provide confinement to the joint which is proportional to the number of hoops placed in the joint. As a result, providing a larger number of hoops with a lower yield stress is more advantageous than providing fewer hoops with a larger yield stress, even though the maximum potential shear capacity for both cases may be the same. Except for Specimen 4B, the test specimens had fewer hoops in the joint than were suggested by the draft recommendations. Grade 60 transverse reinforcement was used in this test series.

According to the draft recommendations, the shear stress in an exterior joint is limited to 12 $\sqrt{f'_c}$ psi (1.0 $\sqrt{f'_c}$ MPa). For specimens tested, the joint shear stress varied between 10 $\sqrt{f'_c}$ psi (0.83 $\sqrt{f'_c}$ MPa) and 14 $\sqrt{f'_c}$ psi (1.16 $\sqrt{f'_c}$ MPa). The joint shear stresses were cal-

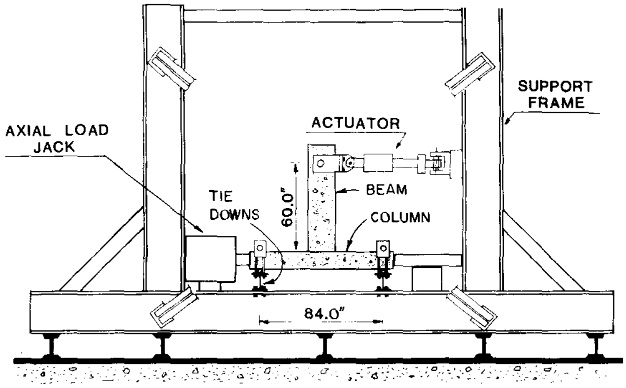

Fig. 2—Testing frame (1 in. = 25.4 mm)

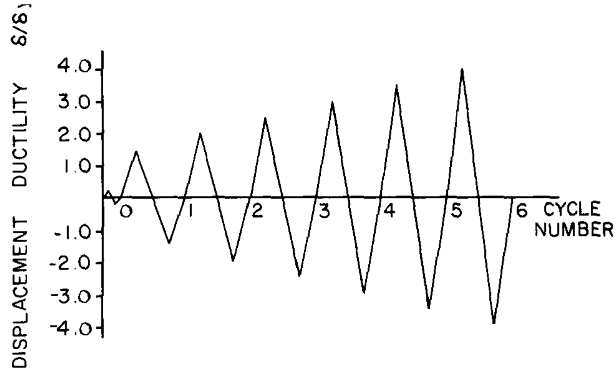

Fig. 4—Loading sequence

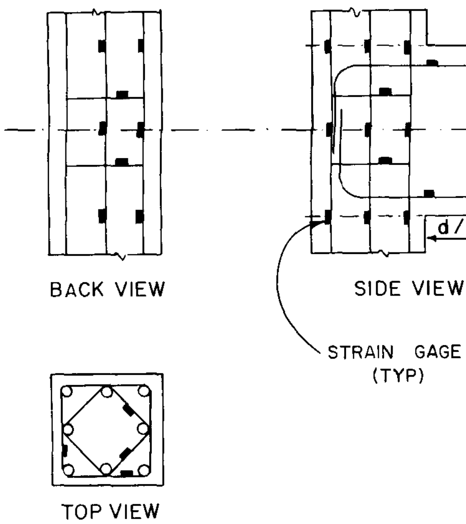

Fig. 3—Location of strain gages

culated when the specimens were designed assuming that strain hardening will increase the tensile stresses in the beam longitudinal reinforcement by ten percent over the measured tensile yield stresses.

Due to the changes in material properties and physical constraints, the final values for the primary variables were slightly different from the original design values. The design values for the primary variables as well as the actual values for each specimen are listed in Table 2.

Materials

A concrete mix using Type I portland cement and coarse aggregate with maximum dimension of ½ in. (13 mm) was used. The slump was kept at about 5 in. (125 mm) for ease of placement. Grade 60 reinforcing bars were used for the column longitudinal and transverse reinforcement including the transverse reinforcement in the joint. Grade 40 steel was used for the longitudinal and shear reinforcement in the beams. The test day compressive strength for the concrete cylinders and the average yield stress for the reinforcing steel are presented in Table 1.

Table 2 — Design and actual values for the primary variables

Specimen number	P/A_g, psi	M_R	$\gamma = \dfrac{V_j}{bh_c\sqrt{f'_c}}$, psi	ρ_t, percent
1B	287	1.1 (1.01)	14.0 (14.2)	1.0 (0.87)
2B	358	1.5 (1.35)	14.0 (14.2)	1.0 (0.98)
3B	358	1.1 (1.07)	14.0 (12.8)	1.5 (1.30)
4B	358	1.5 (1.41)	14.0 (12.5)	1.5 (1.48)
5B	446	2.0 (1.93)	14.0 (15.2)	1.0 (0.78)
6B	380	1.5 (1.56)	10.0 (8.8)	1.0 (0.74)

Note: 1 psi = 0.0069 MPa; $1.0\sqrt{f'_c}$, psi = $0.083\sqrt{f'_c}$, MPa. Numbers outside the parentheses are the design values. Numbers inside the parentheses are the actual values.

Test setup and instrumentation

The specimens were tested in the testing frame shown in Fig. 2. The column portion of the specimens were tied down to the frame. Rollers were provided near the ends of the columns to simulate inflection points, and a hydraulic actuator was used to apply shear forces near the free end of the beam.

Approximately 30 electrical resistance strain gages were attached to the reinforcing bars near the joint region. Locations of the strain gages are shown in Fig. 3. During each cycle of loading, the loading was temporarily stopped at selected points while the strain gages and loads were recorded.

Loading

The column axial load, which was less than 40 percent of the balanced column load, was held constant throughout each test (Table 2). The applied cyclic loading followed the displacement controlled schedule shown in Fig. 4. The specimen was first loaded to its yield displacement. The yield displacement was measured during the test from the plot of the applied load versus the displacement of the load point and corresponded to the point when a significant decrease in the stiffness of the subassemblage occurred. The loading was then continued in the same direction to one and one-half times the yield displacement. The specimen was then unloaded and loaded in the other direction to the same displacement. The maximum displacement for each subsequent cycle of loading was incremented by half of the yield displacement for each cycle after the first cycle of loading.

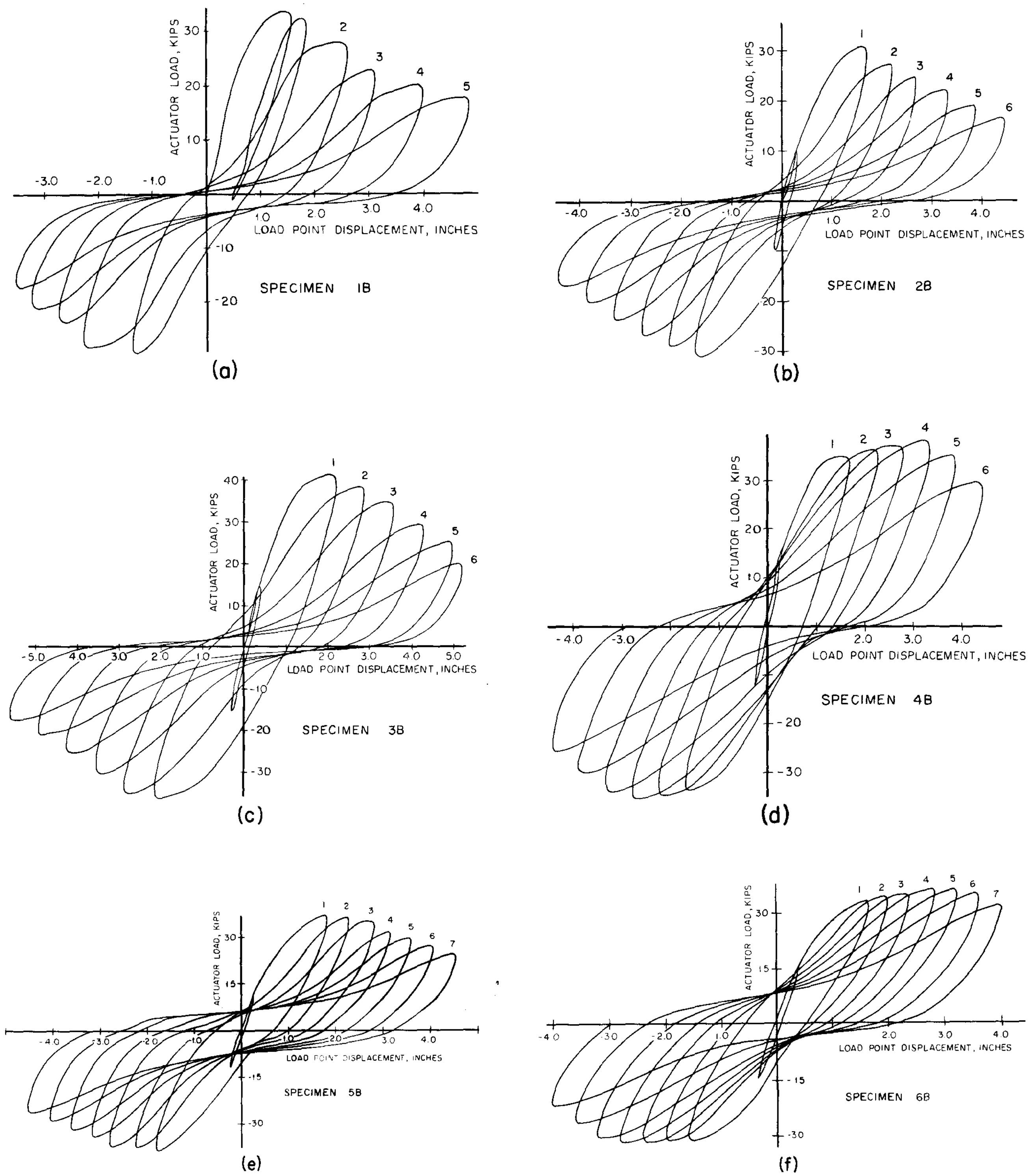

Fig. 5—Load versus deflection response for Specimens 1B through 6B, respectively

DISCUSSION OF TEST RESULTS

Plots of the applied load versus the displacement of the load point for all specimens are shown in Fig. 5(a) through 5(f). To make the comparison of the results easier, the ratio of the maximum load carried by the specimens during each cycle of loading to that of the first cycle is shown in Fig. 6. For calculation of these ratios, the maximum load during each cycle was defined as the average of the maximum positive and maximum negative loads carried by the specimen during that cycle. The load-carrying capacity for Specimens 1B, 2B, and 3B was sharply reduced after the first cycle of loading, while the reduction in load-carrying capacity for Specimen 5B was not as severe. Specimens 4B and 6B maintained their maximum first cycle load for the first four and five cycles of loading, respectively.

Loss of stiffness in subsequent cycles is shown by the "pinching" of the hysteresis loops at midcycle. Two

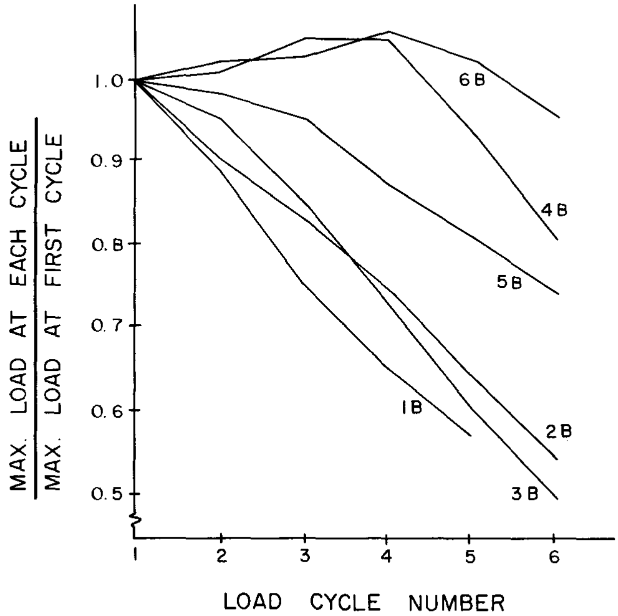

Fig. 6—Cyclic load-carrying capacity of the specimens

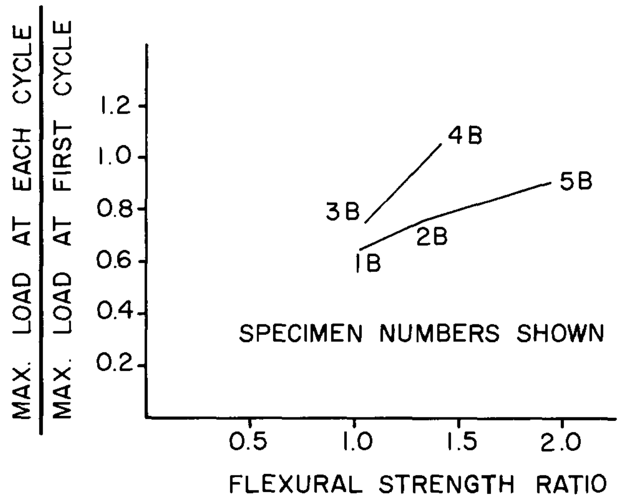

Fig. 7—Effect of the flexural strength ratio on the cyclic load-carrying capacity of the specimens

distinct behaviors were observed with respect to stiffness degradation. The first observation was that after the initial cycle of loading, the stiffness of the specimens was substantially lower when the displacement was near zero due to unclosed beam flexural cracks near the face of the column. The second observation was that the stiffness of the specimens reduced with each additional cycle of loading. This is primarily due to concrete deterioration in and adjacent to joint, slippage of column longitudinal reinforcement, and pullout of the beam longitudinal reinforcement.

Effect of M_R

The flexural strength ratio had a major effect on the location of the flexural hinging in the specimen. All specimens tested in this study had flexural strength ratios greater than 1.0. Therefore, in none of the specimens did flexural hinges form in the columns outside the joint. For specimens which had flexural strength ratios slightly greater than 1.0, most of the damage was concentrated in the joint. In specimens which had flexural strength ratios considerably greater than 1.0, the cracks were distributed more into the beam and away from the joint.

The ratio of the maximum load carried by specimens at the fourth cycle of loading to that of the first cycle of loading for two groups of specimens for which the only change was the flexural strength ratio is plotted in Fig. 7. The improved retention of strength in the beam-column subassembly as the flexural strength ratio is increased is clearly shown in this figure. For flexural strength ratios slightly greater than 1.0, flexural hinges formed in the beam, but spread into the joint. Thus, anchorage of bars was significantly impaired and this led to the pullout of the beam longitudinal steel and slippage of the column longitudinal bars which reduced the load-carrying capacity as well as the stiffness of the specimens.

Effect of ρ_t

In Specimens 3B and 4B, which had larger amounts of transverse reinforcement than Specimens 1B and 2B, respectively, although the concrete in the joint was cracked, it was not crushed. The cover concrete around the joint, however, had spalled off in all specimens by the end of the test. A comparison of the cyclic load-carrying capacity of Specimens 1B and 3B, Fig. 5(a) and (c), respectively, indicates that for specimens that had lower flexural strength ratios, where flexural hinges form in the joint, only a slight improvement can be achieved by providing additional transverse reinforcement. However, in specimens that had larger flexural strength ratios, such as Specimens 2B and 4B, Fig. 5(b) and (d), respectively, the increase in the joint transverse reinforcement made a significant improvement in the overall behavior of the specimen.

Effect of γ

A reduction in the joint shear stress had a distinct effect on the load-carrying capacity of the specimens. As shown in Fig. 5(f), Specimen 6B, which had a joint shear stress of $8.8\sqrt{f_c'}$ psi ($0.73\sqrt{f_c'}$ MPa), maintained its maximum first cycle load through the fifth cycle of loading. However, for Specimen 2B [Fig. 5(b)], which had a joint shear stress of $14.2\sqrt{f_c'}$ psi ($1.18\sqrt{f_c'}$ MPA), the load-carrying capacity deteriorated after the first cycle of loading. These joint shear stresses were calculated using the data from strain gages attached to the beam longitudinal reinforcement which indicated that these bars had yielded in tension.

Examination of the specimens during and at the conclusion of the tests indicated that the specimens that had lower joint shear stresses suffered less damage than

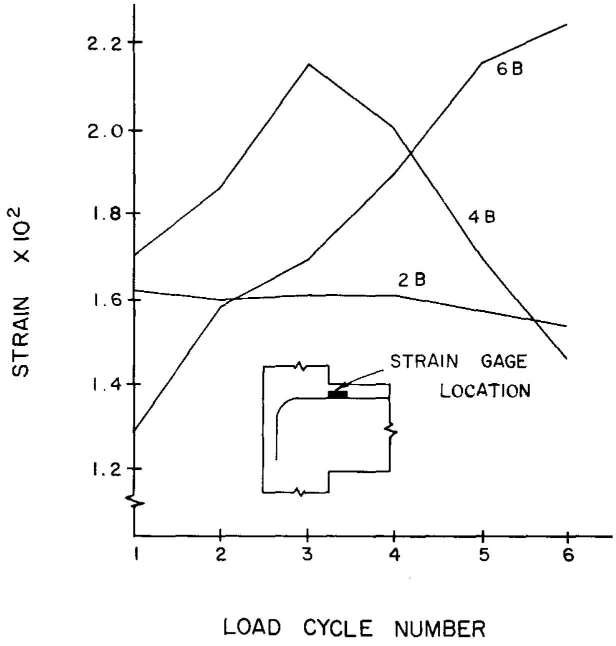

Fig. 8—Maximum strain during each cycle of loading in beam longitudinal reinforcement

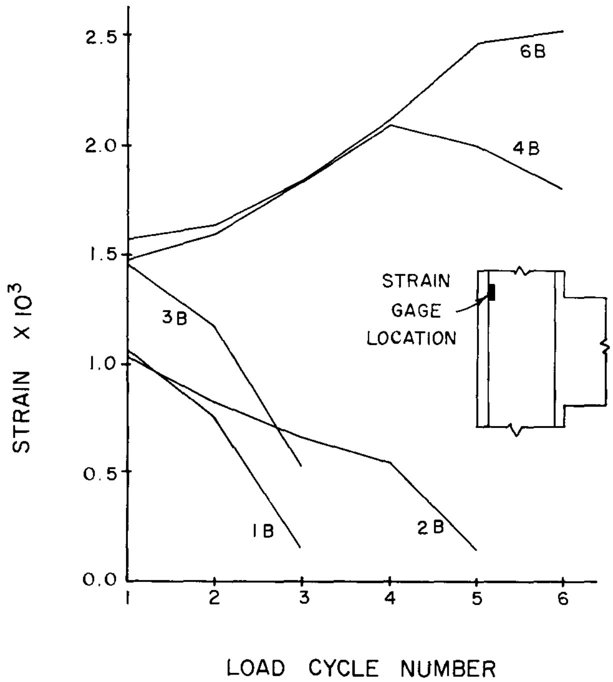

Fig. 9—Maximum strain during each cycle of loading in column longitudinal reinforcement

similar specimens with higher joint shear stresses. Deterioration of the concrete in the joint due to higher shear stresses had an adverse effect on the anchorage conditions of the beam and column longitudinal reinforcement in the joint. As a result, in specimens with higher joint shear stresses, beam bar pullout and column bar slippage started during earlier cycles of loading.

Bar slippage

A major cause for loss of stiffness in the specimens was slippage of the column bars through the joint and pullout of the beam longitudinal reinforcement from the joint. As cracks formed in the joint, the concrete in the joint lost its ability to resist the bond forces exerted by the beam longitudinal reinforcement which was anchored in the joint. The beam bars could then be partially pulled out of the joint as the bar yield stress was developed. In specimens that had low shear stresses in the joint or in specimens with larger flexural strength ratios and thus, with flexural hinges formed in the beam instead of the joint, beam bar pullout did not occur during the first few cycles of loading.

Strain gage measurements were used to determine beam bar pullout. Because the maximum displacement the specimens were subjected to increased with each cycle of loading, the strains in the beam longitudinal reinforcement were expected to increase during each cycle of loading. If the maximum strains during each two consecutive cycles of loading remained the same or decreased, it was concluded that a pullout of the bar had taken place. As shown for Specimen 2B in Fig. 8, due to the excessive damage to the joint, the bar started to pull out after the first cycle of loading. In contrast, the beam longitudinal reinforcement in Specimen 6B maintained adequate anchorage throughout the test due to the low joint shear stresses. Beam bar pullout in Specimen 4B, which had a flexural strength ratio of 1.41, was not recorded until the fourth cycle of loading.

Cracking of the joint also led to slippage of the column longitudinal reinforcement through the joint. In all specimens tested, the column reinforcing steel near the face of the column closest to the beam slipped. However, in specimens which had either a combination of a larger flexural strength ratio and a larger joint transverse reinforcement ratio or lower shear stresses, such as Specimens 4B and 6B, the column bars on the back side of the column either did not slip or slipped only after a large number of cycles of loading. The maximum strains for the same column longitudinal reinforcement in each specimen during each cycle of loading is plotted in Fig. 9. The maximum strains in Specimens 1B, 2B, and 3B started to drop after the first cycle of loading, indicating the slippage of column longitudinal reinforcement. The combination of larger flexural strength ratio and joint transverse reinforcement delayed the slippage of column reinforcement in Specimen 4B until the fifth cycle of loading. In Specimen 6B, which had small joint shear stresses, the column longitudinal reinforcement at the back face of the column did not slip at all.

Based on the hysteretic behavior, strain gage data, and the observation of the damage to the specimens, only the performance of Specimens 4B and 6B were rated satisfactory.

Table 3 — Comparison of design parameters to draft recommendations

Specimen number	M_R	γ*	s_h, in.	l_{dh},† in.	h_b/column bar diameter
1B	1.01 (1.4)	13.2 (12)	4.4 (3.0)	7.4 (8.8)	25.1 (24)
2B	1.35 (1.4)	13.2 (12)	3.9 (3.0)	7.4 (8.6)	23.0 (24)
3B	1.07 (1.4)	12.0 (12)	3.3 (3.0)	7.4 (8.0)	25.1 (24)
4B	1.41 (1.4)	12.2 (12)	3.0 (3.0)	7.4 (7.6)	23.0 (24)
5B	1.93 (1.4)	14.2 (12)	4.3 (3.5)	9.4 (10.3)	18.9 (24)
6B	1.56 (1.4)	8.1 (12)	4.6 (3.5)	9.4 (6.9)	25.1 (24)

Note: 1 in. = 25.4 mm; $1.0\sqrt{f'_c}$ psi = $0.083\sqrt{f'_c}$ MPa. Numbers outside the parentheses are the provided values. Numbers inside the parentheses are required by the draft recommendations.
*Shear stress as a multiple of $\sqrt{f'_c}$, psi.
†$l_{dh} = 0.014\,\alpha f_y d_b/\sqrt{f'_c}$, psi.

COMPARISON TO THE DRAFT RECOMMENDATIONS

The design parameters for these specimens were compared to the draft recommendations of the ASCE-ACI Committee 352. The results are listed in Table 3. For every entry in the table, the draft recommendations of Committee 352 are given in parentheses next to the actual values for the specimens.

Only Specimens 4B, 5B, and 6B satisfied the requirement for minimum flexural strength ratio. The joint shear stress was greater than $12\sqrt{f'_c}$ psi ($1.0\sqrt{f'_c}$ MPa) for all specimens except Specimens 3B and 6B. The requirement limiting the spacing of the hoops within the joint to one-quarter of the column thickness was violated by all specimens except Specimen 4B. As shown in Column 5 of Table 3, only Specimen 6B had adequate development length for the hooked beam bars. The recommendations require the ratio of the beam total depth to column bar diameter to be greater than 24 to inhibit slippage of column longitudinal reinforcement. Column 6 of Table 3 indicates that only Specimens 1B, 3B, and 6B satisfied this requirement.

Specimen 4B, for which all the design parameters were very close to the draft recommendations of Committee 352, demonstrated satisfactory performance. The draft recommendations have separate requirements and limits for the different parameters contributing to the behavior of a joint. However, none of these requirements are relaxed if one or more of the remaining requirements are satisfied more conservatively than the committee recommendations. An example of this can be seen in the behavior of Specimen 6B. The provided hoop spacing was much larger than the draft recommendations. However, because the shear stress and anchorage requirements were met with considerable conservatism, the specimen showed satisfactory behavior. It is therefore necessary that before selecting joint transverse reinforcement, all factors influencing the behavior of the connection should be considered collectively.

CONCLUSIONS

The purpose of this investigation was to study the effect of the parameters influencing the behavior of beam-to-column connections and to determine if in certain cases the existing design guidelines could be relaxed. From the test specimens the following can be concluded:

1. To avoid formation of plastic hinges in the joints, the flexural strength ratio should be no less than 1.4. Larger flexural strength ratios improve the behavior of the connections considerably.
2. The maximum joint shear stress in exterior connections should be limited to $12\sqrt{f'_c}$ psi ($1.0\sqrt{f'_c}$ MPa) to reduce excessive joint damage, column bar slippage, and beam bar pullout. A significant improvement in the behavior of a connection is observed if the joint shear stresses are kept well below this limit.
3. Although slippage of column bars and pullout of beam longitudinal reinforcement is undesirable, there is no need to try to eliminate this problem completely. Specimens which had minor slippage and bar pullout in the later cycles of loading showed a very good overall behavior.
4. Additional transverse reinforcement, when combined with conclusions 1 and 2, does enhance the behavior of the subassemblage. However, the construction of such connections was found to be extremely difficult.
5. The present guidelines for selecting joint transverse reinforcement do result in satisfactory performance of beam-column subassemblies in cases where the flexural strength ratio, joint shear stress, and anchorage requirements are all approximately equal to the limits of the draft recommendations. However, in cases where either the flexural strength ratio, the joint shear stress, or the anchorage requirements are significantly more conservative than the limits of the draft recommendations, the amount of joint transverse reinforcement could be safely reduced.

ACKNOWLEDGMENTS

Support for this research by the National Science Foundation under Grant No. PFR 78-24556 is gratefully acknowledged. The results and conclusions are those of the writers and do not necessarily reflect the views of the sponsor. The writers would also like to thank the technical staff at the Structural Engineering Laboratory of the University of Michigan for their support.

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 outer layer of tension reinforcement in beam, in.^{2}
A_{s2b} = area of inner layer of tension reinforcement in beam, in.^{2}
b = width of column transverse to the direction of shear, in.
b_b = width of beam, in.
d_{1b} = distance from compression face to centroid of outer layer of tension reinforcement in beam, in.
d_{2b} = distance from compression face to centroid of inner layer of tension reinforcement in beam, 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 = total depth of column parallel to the direction of shear, 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.
l_{dh} = development length of hooked bars measured from the face of the column core to back side of the hook, in.
M_R = sum of the flexural capacity of columns to that of beam
P = applied column axial load during test, kips
s_h = center-to-center spacing of hoops, in.
V_j = joint shear force, kips
α = stress multiplier for flexural reinforcement = 1.25 for earthquake loading
γ = joint shear stress as a multiple of $\sqrt{f'_c}$
ρ_t = transverse reinforcement ratio, percent

REFERENCES

1. Hanson, Norman W., and Conner, Harold W., "Seismic Resistance of Reinforced Concrete Beam-Column Joints," *Proceedings,* ASCE, V. 93, ST5, Oct. 1967, pp. 533-560.

2. Hanson, Norman W., "Seismic Resistance of Concrete Frames with Grade 60 Reinforcement," *Proceedings*, ASCE, V. 97, ST6, June 1971, pp. 1685-1700.

3. Lee, Duane L. N.; Wight, James K.; and Hanson, Robert D., "RC Beam-Column Joints under Large Load Reversals," *Proceedings,* ASCE, V. 103, ST12, Dec. 1977, pp. 2337-2350.

4. Meinheit, Donald F., and Jirsa, James O., "Shear Strength of R/C Beam-Column Connections," *Proceedings,* ASCE, V. 107, ST11, Nov. 1981, pp. 2227-2244.

5. Scribner, Charles F., and Wight, James K., "Strength Decay in R/C Beams under Load Reversals," *Proceedings,* ASCE, V. 106, ST4, Apr. 1980, pp. 861-876.

6. Viwathanatepa, S.; Popov, E. P., and Bertero, V. V., "Seismic Behavior of Reinforced Concrete Interior Beam-Column Subassemblages," *Report* No. UCB/EERC-79/14, Earthquake Engineering Research Center, University of California, Berkeley, June 1979, 184 pp.

7. Uzumeri, S. M., and Seckin, M., "Behavior of Reinforced Concrete Beam-Column Joints Subjected to Slow Load Reversals," *Publication* No. 74-05, Department of Civil Engineering, University of Toronto, Mar. 1974, 84 pp.

8. Nakata, S., et al., "Tests of Reinforced Concrete Beam-Column Subassemblages for U.S.-Japan Cooperative Research Program," Building Research Institute, Ministry of Construction, Ibaraki-Pref, Oct. 1980, 112 pp.

9. Paulay, T.; Park, R.; and Priestly, M. J. N., "Reinforced Concrete Beam-Column Joints Under Seismic Action," ACI JOURNAL, *Proceedings* V. 75, No. 11, Nov. 1978, pp. 585-593.

10. Scarpas, A., "The Inelastic Behavior of Earthquake Resistant Reinforced Concrete Exterior Beam-Column Joints," *Report* No. 81-2, Department of Civil Engineering, University of Canterbury, Christchurch, Feb. 1981, 84 pp.

11. ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," (ACI 352R-76), American Concrete Institute, Detroit, 1976, 19 pp.

12. Ehsani, M. R., and Wight, J. K., "Behavior of External Reinforced Concrete Beam to Column Connections Subjected to Earthquake Type Loading," *Report* No. UMEE 82R5, Department of Civil Engineering, University of Michigan, Ann Arbor, July 1982, 243 pp.