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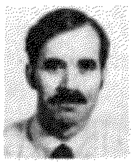
Behavior of Interior Beam-to-Column Connections Under Earthquake-Type Loading

by Ahmad J. Durrani and James K. Wight

Beam-to-column connections designed according to the current recommendations of ACI-ASCE Committee 352[1] may result in a joint which is overly congested with reinforcement, difficult to construct, and uneconomical. The performance of these joints under cyclic loading, however, may not be better than that of lightly reinforced well-detailed joints. The results of an experimental investigation which studied the use of lower amounts of transverse reinforcement than currently recommended are reported. The behavior of internal beam-to-column connections of a ductile moment-resisting frame under an earthquake-type loading are presented along with their design implications. The effect of the amount of joint hoop reinforcement and joint shear stress on strength degradation, loss of stiffness, energy dissipation, shear deformation of the joint, and the slippage of beam and column bars through the joint is examined. The joint shear stress was found to have a pronounced effect on behavior at large ductility levels. The joint hoop reinforcement, on the other hand, was most significant at lower ductility levels. Guidelines are suggested to simplify the design of connections.

Keywords: beams (supports); columns (supports); connections; cyclic loads; earthquake resistant construction; energy dissipation; frames; **joints (junctions)**; reinforced concrete; reinforcing steels; slippage; stiffness; strength; structural design.

During a strong earthquake, beam-to-column connections are subjected to severe reversed cyclic loading. If they are not designed and detailed properly, their performance can significantly affect the overall response of a ductile moment-resisting frame building. To develop the full flexural capacity of beams, which are usually of weaker design than columns, the beam-to-column connection (also referred to as joint) must maintain its strength as well as stiffness during the loading cycles.

The current procedure for the design of beam-to-column connections was recommended by ACI-ASCE Committee 352.[1] These recommendations prescribe the apportionment of total joint shear to concrete and joint hoops similar to the shear design of flexural members. Since these recommendations were developed a large number of beam-to-column connections have been tested in the United States, Canada, and New Zealand. The New Zealand practice[2] requires that the total joint shear be apportioned between that carried by the concrete diagonal strut and that by a truss mechanism consisting of horizontal and vertical stirrups and intermediate column bars. Other studies[3-8] have indicated the significance of joint confinement and joint shear stress on the behavior of joints. Draft revisions to the recommendations of Committee 352* are based on providing adequate confinement to the joint core while limiting the shear stress in the joint to a certain maximum level depending on the type of loading and joint.

The joints designed according to the revised recommendations still require a large amount of hoop reinforcement and may become congested with steel. Added restrictions on the size of beam and column bars to control their slippage through the joint further complicate the design. Recent studies[9,10] indicate that a lesser amount of hoop reinforcement could be used without significantly affecting the performance of joints.

OBJECTIVES AND SCOPE

The main objective of this study was to evaluate the performance of interior joints which have less transverse reinforcement than required by the draft recommendations of Committee 352. Other objectives were to investigate the effect of the level of joint shear stress on strength, stiffness, and energy dissipation of beam-column subassemblages and to examine the slippage of beam and column bars through the joint. To achieve these objectives, three interior beam-to-column subassemblages were designed according to the provisions of Appendix A of the ACI Building Code[11] and tested under reverse cyclic loading.

*"Eighth Draft of Revised Recommendations for Design of Beam-Column Joints in Monolithic Concrete Structures," Minutes of ACI-ASCE Committee 352.

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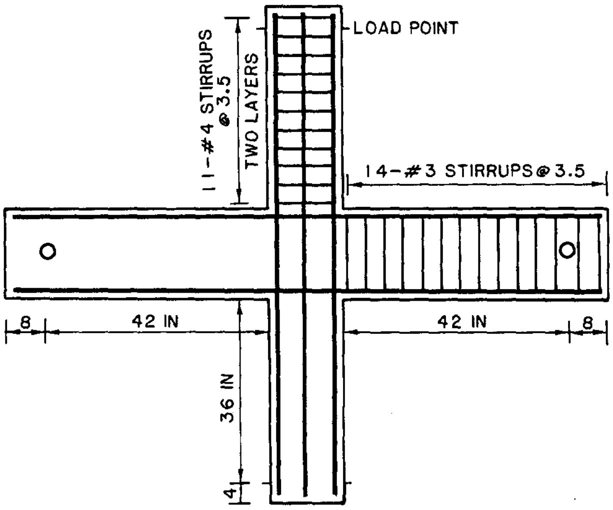

Fig. 1(a) — Beam-column subassemblage (1 in. = 25.4 mm)

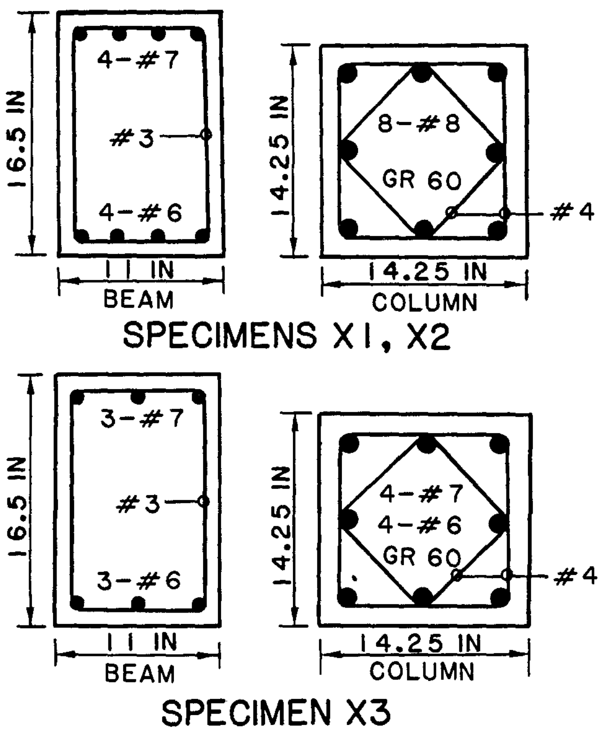

Fig. 1(b) — Beam and column cross sections (1 in. = 25.4 mm)

Table 1 — Ratio of column flexural strength to beam flexural strength M_r

Specimen	Concrete strength, psi	M_r, nominal material strength	M_r, actual material strength
X1	4980	1.89	1.50
X2	4880	1.93	1.55
X3	4500	1.81	1.50

Note: 1000 psi = 6.89 MPa.

RESEARCH SIGNIFICANCE

The research presented in this paper provides structural designers with information on the performance of interior beam-to-column connections which have less joint reinforcement than currently recommended. The data on the strength, stiffness degradation, and energy dissipation characteristics of test specimens for different levels of displacement ductilities is desirable for developing analytical models to predict the nonlinear dynamic response of moment-resisting frames.

DESIGN OF SPECIMENS

The design of these specimens is discussed in terms of nominal ultimate capacities based on both the ACI Code provisions and the theoretical capacities calculated from the actual strength of materials. A typical beam-column subassemblage tested during this investigation is shown in Fig. 1(a). It represents an interior beam-to-column connection isolated at inflection points of the beams and columns. All test specimens were designed to have the same overall dimensions and member sizes [Fig. 1(b)]. The specimens represent beam-to-column joints in the upper stories of a moment-resisting frame structure. The main reinforcement of beams consisted of Grade 40 steel, and the column main reinforcement was of Grade 60 steel. For convenience in fabrication the transverse reinforcement was made of hoops with overlapping legs splice-welded together. The concrete mix was designed for a specified strength of 4000 psi (27.5 MPa) at 28 days using ¾-in. (19-mm) maximum size aggregate and ordinary portland cement. At the time of testing the average age of the specimens was 80 days. The corresponding concrete strengths are listed in Table 1.

The test specimens were designed to have a nominal column-to-beam flexural strength ratio of 1.5. However, the flexural strength ratios based on actual material strengths (Table 1) were almost 20 percent lower than those based on nominal material strengths. This is primarily due to the larger difference between the actual and nominal yield strengths of Grade 40 steel which was used in beams as compared to the Grade 60 steel used in columns. In certain cases, such discrepancy between the design strength and actual strength of beams could result in hinging in columns which is not desirable. The corresponding increase in joint shear stress, however, is only about 5 percent. For comparison purposes, both the nominal and actual capacity of sections were calculated for an ultimate concrete strain of 0.003.

Table 2 — Beam-to-column joint design parameters

Specimen	Joint shear coefficient $\gamma\sqrt{f'_c}$ (psi)	Joint reinforcement: Spacing of hoops,[‡] in.	Joint reinforcement: Number of layers	Joint reinforcement: ρ_h, percent
X1	12.30* (21.8)'	6	2	0.76
X2	12.30 (22.1)	4	3	1.15
X3	9.70 (17.2)	6	2	0.76
1976 recommendations by ACI-ASCE Committee 352	20	4	—	1.53[§]
Revised recommendations by ACI-ASCE Committee 352	15	3.5	—	1.53[§]

*Based on gross column area.
'Based on the effective column area in shear.
‡Two #4 hoops in one layer as shown in Fig. 1(b).
§Governed by maximum spacing requirement.
Note: 1 $\sqrt{f'_c}$ psi = 0.083 $\sqrt{f'_c}$ MPa.

The details of joint hoop reinforcement and the joint shear stress for each specimen are given in Table 2. The joint hoop reinforcement ratio ρ_h varied between 0.75 and 1.15 percent, which is equivalent to one-half and three-quarters, respectively, of the reinforcement recommended by Committee 352. The joint shear stress level varied between 9.7 $\sqrt{f'_c}$ and 12.3 $\sqrt{f'_c}$ psi, based on the gross cross-sectional area of the column. (Note: 1 $\sqrt{f'_c}$ psi = 0.083 $\sqrt{f'_c}$ MPa.

EXPERIMENTAL TEST SETUP AND TEST PROCEDURE

Each specimen was constructed in three stages to simulate the construction sequence of a real building. Electrical resistance strain gages were attached to the reinforcement at locations shown in Fig. 2. Two LDVTs (linear variable differential transformers) were placed diagonally at the beam-to-column joint to measure shear deformation of the joint. The subassemblages were supported in a four-hinged steel frame in the upright position as shown in Fig. 3. The beam ends were constrained to move only in the horizontal direction by Links A and B, which were also instrumented to measure shear in beams. The lateral load representing story shear was applied near the top end of the column through the four-hinged frame by a hydraulic actuator. The gravity load was simulated by a vertical force applied at the column base by a hydraulic jack and was kept constant during each test. This loading arrangement allows the P-Δ effect to take place, which affects the load-carrying capacity of the subassemblage.

The column axial load was applied one day prior to the test to eliminate any immediate creep effect. During the test, reversed cyclic displacements were applied quasi-statically near the top end of the column to simulate the earthquake-type loading. A typical displacement sequence is shown in Fig. 4. The lateral displacements were controlled in terms of displacement ductility μ, which is defined as the ratio of column load-point displacement at any stage during the test to the displacement at initial yield of beam longitudinal reinforcement. For every new loading cycle, the displacement was incremented by one-half the yield displacement. A typical test consisted of seven loading cycles with a maximum displacement ductility of four. This

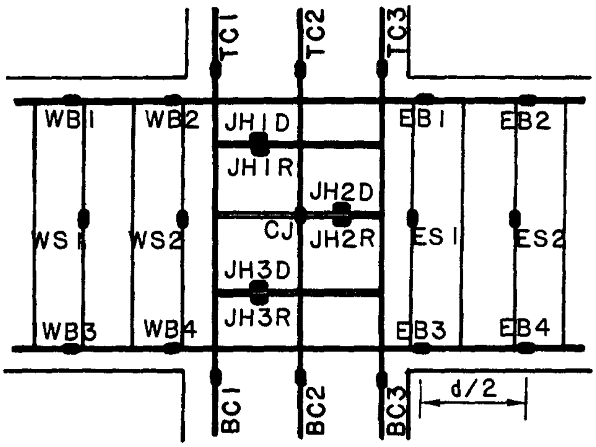

Fig. 2 — Electrical strain gage locations

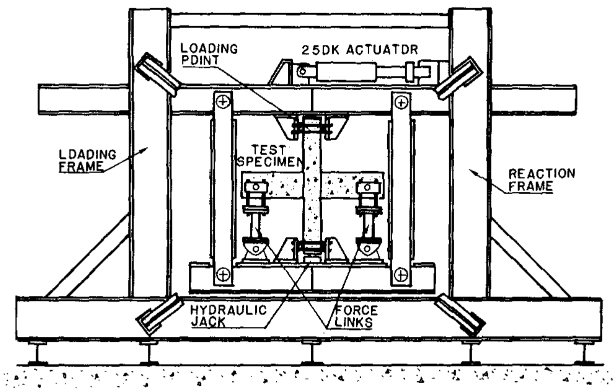

Fig. 3 — Testing frame

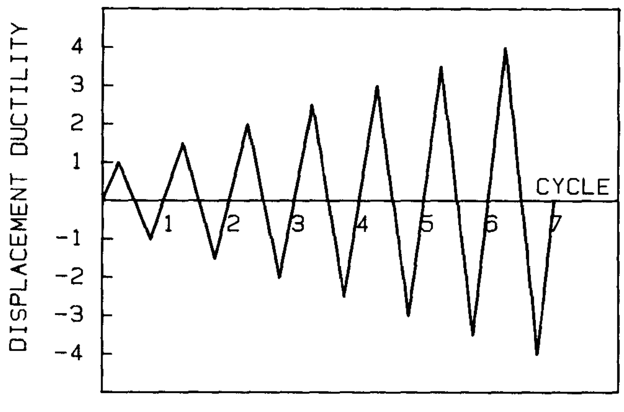

Fig. 4 — Typical displacement routine

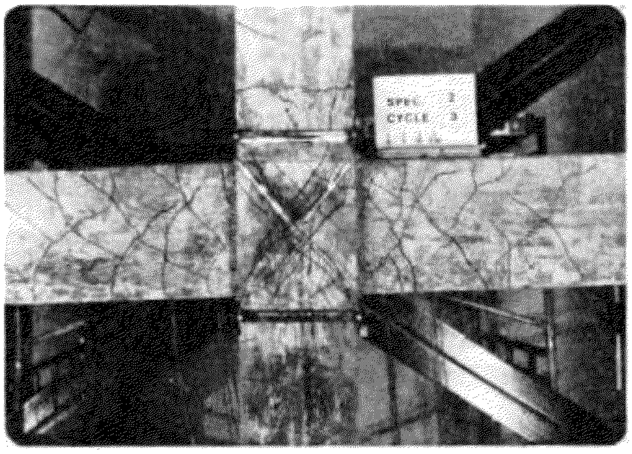

Fig. 5 — Cracking pattern in Specimen X2

level of displacement ductility was selected based on the displacement capacity of the test equipment.

EXPERIMENTAL RESULTS

The results of test specimens are analyzed in terms of (1) P-Δ effect, (2) cracking pattern, (3) strength degradation, (4) loss of stiffness, (5) joint shear deformation, (6) energy dissipation, and (7) the slippage of beam and column bars through the joint. The effect of the joint shear stress level and the joint hoop reinforcement on each of these aspects of behavior is discussed.

P-Δ effect

The P-Δ effect on the lateral load-carrying capacity of a beam-column subassemblage can be considered equivalent to a horizontal force. When the lateral displacement increased in either direction, the P-Δ effect imposed additional force on the subassemblage and thus reduced the net load-carrying capacity. The opposite effect occurred when displacements decreased and thus resulted in an apparent increase in the lateral load capacity. The subassemblages in this study represented upper stories of a frame with column axial loads of the order of 50 kips. The P-Δ effect, therefore, was small and caused a maximum reduction of only 8 percent in strength.

Crack development

Fig. 5 shows a typical cracking pattern in a subassemblage. Except for a few hairline flexural cracks in a region close to the joint, the columns did not experience any significant cracking. The diagonal cracks in the joints appeared in the first loading cycle. In successive cycles their number and severity increased depending on the joint shear stress level and the confinement of the core. The effect of joint reinforcement on the cracking pattern was seen by comparing Specimens X1 and X2. Both specimens had the same level of joint shear stress but had different amounts of joint reinforcement (Table 2). In Specimen X1, which had only two layers of hoops, the center of the joint core began to deteriorate rapidly in the fourth cycle. At the end of the seventh cycle, the concrete between the two layers of hoops spalled and wide cracks extended from the hollow core area toward the corners of the joint. Therefore, most of the inelastic action occurred in the joint, and, in general, the beams experienced limited cracking. Specimen X2 exhibited less joint damage than Specimen X1. The joint core remained well-confined by the hoops and flexural damage was forced over a larger region in the beams. As in the case of Specimen X1, Specimen X3 developed a softer core at the intersection of diagonal cracks. However, due to the lower shear stress level in the joint, severe damage to the core was delayed until the seventh cycle.

Strength degradation

The degradation of load-carrying capacity and stiffness can be seen from the applied column load versus column load-point displacement curves (Fig. 6). Although observed curves represent the behavior of subassemblages as a whole, the marginal design of the joints and the lower column-to-beam flexural strength ratio make them characteristically descriptive of the behavior of a joint. Because of the smaller capacities of its beams and columns, Specimen X3 had a lower strength. For comparison, the strength of each specimen at various cyclic ductility levels was normalized with respect to the strength at first yield of the beam bars. Fig. 7 illustrates the strength reduction of each specimen with respect to the cyclic displacement ductilities (referred to hereafter as ductilities). Specimen X1, which had a lower amount of joint reinforcement and higher level of joint shear stress, began to lose its strength soon after the first two loading cycles. At a ductility level of 3.4, the specimen could carry a load of only 75 percent of its original strength. The relative effects of joint shear stress and joint hoop reinforcement on the strength of a subassemblage can be seen by comparing the response of Specimens X2 and X3 to that of Specimen X1. The well-confined joint core of Specimen X2 helped the beam reinforcement reach strain hardening without any appreciable slippage. As a result, the lateral load resistance of this specimen continuously increased until the fifth cycle. Thereafter, the joint partially lost its confinement due to the yielding of hoops and resulted in a 17 percent loss of strength. Specimen X3, which had a lower joint shear stress, displayed a sustained load-carrying capacity for the first four cycles. The loss of strength at the end of the seventh cycle was only 14 percent of the yield strength and resulted from joint deterioration caused by the loss of adequate confinement.

Loss of stiffness

The load versus deformation curves of all specimens showed distinct pinching of loops for ductility levels greater than two. The average peak to peak stiffness degradation of the specimens is illustrated in Fig. 8. For each specimen the stiffness is shown as a percentage of the initial stiffness. The method for measuring the stiffness is demonstrated for Specimen X1 in Fig. 6(a). Specimen X2, which had the most confinement, experienced a relatively smaller loss of stiffness during the first few cycles. The higher level of joint shear stress and the lower amount of joint hoop reinforcement in Specimen X1 resulted both in the loss of ductility as well as stiffness. Comparing the stiffness degradation of Specimens X2 and X3, the lower level of joint shear stress in Specimen X3 increased its ductility over that of Specimen X2, irrespective of the lower amount of joint confining reinforcement. Similarly, the increased confinement in Specimen X2 improved its ductility over that of Specimen X1. However, the loss of average peak to peak stiffness at the end of the seventh cycle was approximately the same magnitude for all specimens in spite of the different levels of confinement and joint shear stress.

Joint shear deformation

Shear deformation of a joint provides another measure of its performance. Fig. 9 shows the joint shear deformation envelope for positive displacements. Specimen X1, which had the smaller joint confining reinforcement and the higher level of joint shear stress, showed an early excessive shear deformation and stiffness degradation in the joint. Specimen X3, which had the lower level of joint shear stress, had the least joint shear deformation and loss of stiffness. Specimen X2 displayed a behavior that was a combination of X1 and X3. It had the same joint stiffness as that of Specimen X3 for the first few cycles but degraded to the stiffness level of Specimen X1 during the last two cycles.

Energy dissipation

A desirable behavior for a beam-column subassemblage under cyclic loading implies a sufficient amount of energy dissipation without a substantial loss of strength and stiffness. Fig. 10 shows the energy dissipation of each specimen during the loading cycles. Because of the variation in yield load and yield displacement, the total energy dissipated during each loading cycle is normalized with respect to the energy dissipated during the yield cycle. Specimen X3, which had the lower joint shear stress, distinctly displayed a superior behavior. The larger amount of joint reinforcement in Specimen X2 also improved its behavior over Specimen X1. However, the excellent behavior of Specimen X3 clearly indicates that lowering the joint shear stress leads to better behavior than increasing the amount of joint reinforcement.

Slippage of bars

During the cyclic loading of beam-column subassemblages, the beam and column main reinforcement is pulled on one side of the joint and is pushed simultaneously from the opposite side. This action plus the increasing shear distress in the joint core creates a situation where some degree of bar slippage through the joint is inevitable. An important parameter related to the slip of continuous bars through a beam-to-column joint is the ratio of the appropriate joint dimension to reinforcing bar diameter. For the test specimens, the

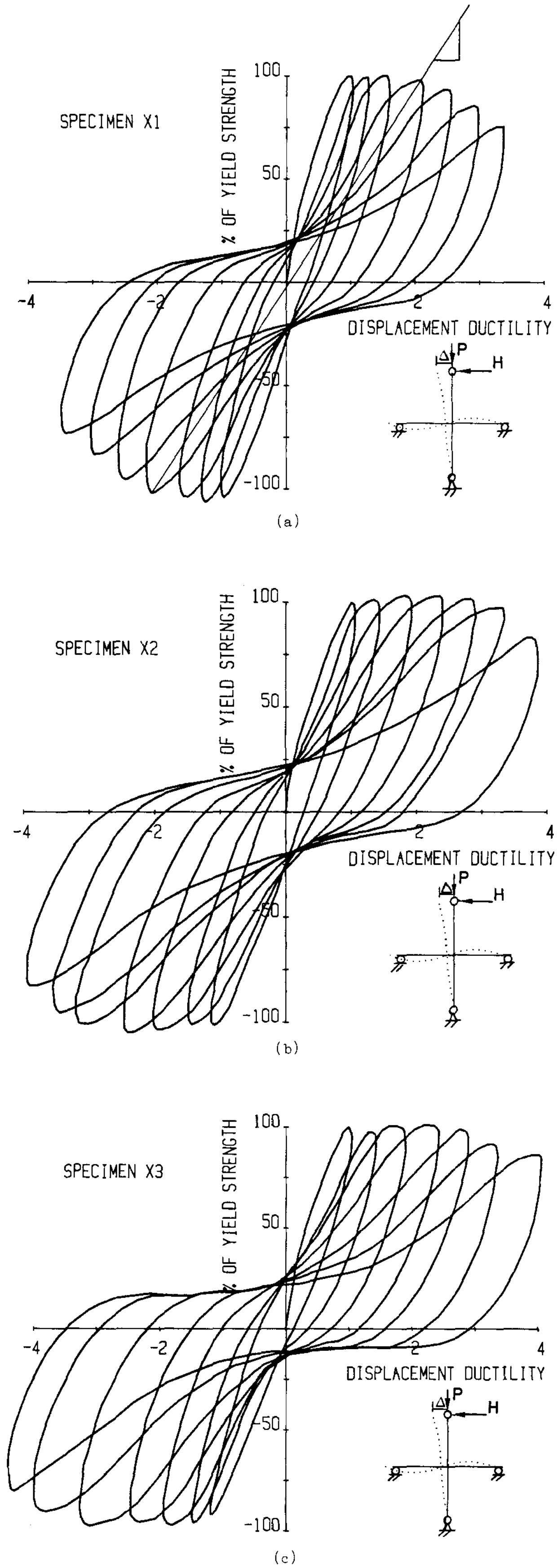

Fig. 6 — Lateral force-displacement curves: (a) Specimen X1, (b) Specimen X2, and (c) Specimen X3

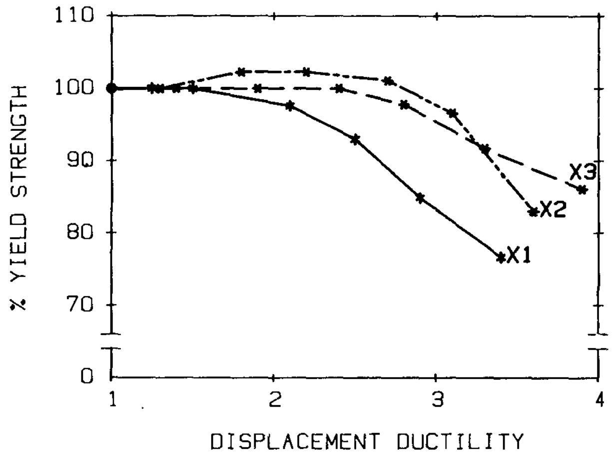

Fig. 7 — Strength degradation of specimens

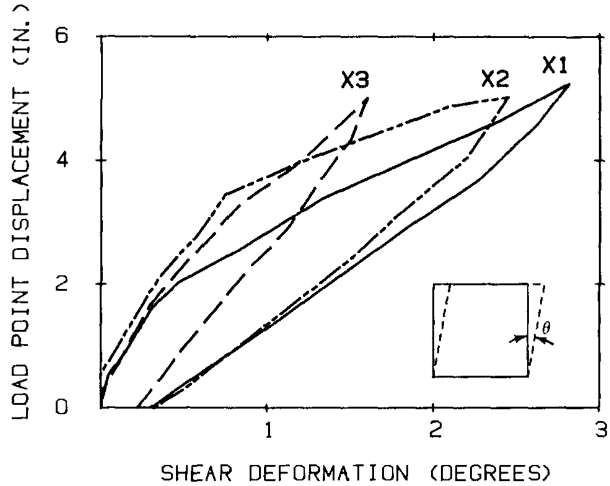

Fig. 9 — Joint shear deformation (1 in. = 25.4 mm)

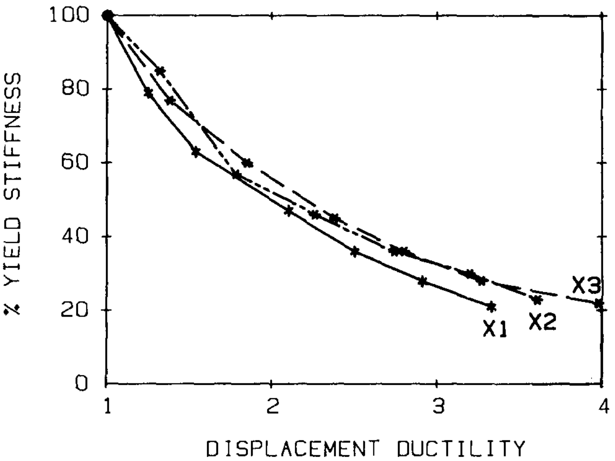

Fig. 8 — Stiffness degradation of specimens

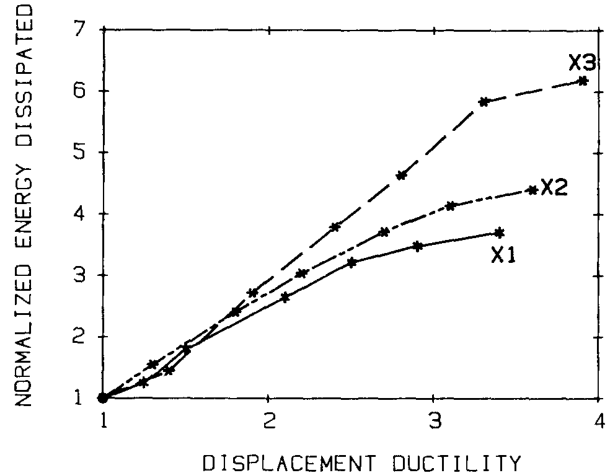

Fig. 10 — Relative energy dissipation

ratio of the total column depth to the beam bar diameter varied from 14 to 16. The ratio of the total beam depth to the column bar diameter varied from 16 to 19. The draft recommendations* indicate that slip is likely to occur at these ratios. In a recent study,[12] a ratio of 35 to 40 is suggested to eliminate completely the slippage of bars through the joint.

The slippage of beam and column bars through the joint was observed qualitatively from the variation of strain in reinforcing bars at the face of joints at locations indicated in Fig. 2. If there is no slippage, the beam and column bars are expected to experience tensile or compressive strain alternately, depending on the direction of cyclic loading. Any deviation from this behavior, particularly a reinforcing bar remaining in tension irrespective of the loading direction, indicates deterioration of bond and thus slippage through the joint. In Specimen X2, the beam main reinforcement experienced very little slippage; for Specimen X3, the slippage was delayed until the sixth cycle. However, in the case of Specimen X1, the beam bars tended to slip during the early loading cycles and after the fourth cycle these bars pulled through the joint easily. Such behavior indicated the significant effect of joint confinement and the level of joint shear stress on the slippage of beam main reinforcement through the joint core.

In all specimens, the column main reinforcement adjacent to the beam-to-column interface showed slippage through the core during the early loading cycles. Although the observed slippage was of a relatively small magnitude, as indicated by the variation in strain, it appeared to be unaffected by the amount of joint reinforcement or the level of joint shear stress. Such behavior could possibly be attributed to the strong column-weak girder design. During the first load cycle the beam main bars yielded at the face of column and a crack which extended approximately through the entire depth of beam formed at the beam-to-column interface. After additional cycles, the deterioration of the beam-to-column interface continued. Such deterioration was observed to extend into the joint and thus

*"Eighth Draft of Revised Recommendations for Design of Beam-Column Joints in Monolithic Concrete Structures," Minutes of ACI-ASCE Committee 352.

seems to affect the bond of column bars in the joint. This reasoning is substantiated by the similar slippage behavior of column reinforcement in all test specimens.

DESIGN RECOMMENDATIONS

Based on the test results of this investigation, the following general guidelines are suggested for a safe, economical, and practical design of internal beam-to-column connections.

1. The same grade of reinforcing steel should be used in beams and columns. A larger difference in the specified and actual yield strengths of beam reinforcing steel compared to the column steel could significantly reduce the column-to-beam flexural strength ratio and may result in column hinging in certain cases.

2. The joint shear stress should be kept as low as possible without compromising the economy of member design.

3. Joint reinforcement should be well distributed in the joint, as indicated by a comparison between Specimens X1 and X2.

4. A joint shear stress of 15 $\sqrt{f_c'}$ psi and joint transverse reinforcement of 1.5 percent should be considered as the upper limits for each parameter. (Note: 1 $\sqrt{f_c'}$ psi = 0.083 $\sqrt{f_c'}$ MPa.)

5. For typical beam-to-column connections, the slippage of beam and column bars through the joint cannot be completely eliminated. Besides the size of bars, slippage of beam bars is also dependent on the joint shear stress level and confinement of the core. Therefore, allowance for member end rotation due to the slippage of bars should be made in the nonlinear dynamic analysis of frames.

6. A minimum design of joint transverse reinforcement would have a reinforcement percentage greater than or equal to 0.75 percent with at least three layers of hoops.

CONCLUSIONS

The following conclusions are drawn from the tests on beam-to-column connections subjected to reversed cyclic loading:

1. The joint shear stress appears to have a significant effect on strength and stiffness of subassemblages at larger ductility levels (greater than two). A lower joint shear stress helps stabilize the loss of strength and stiffness irrespective of the amount of reinforcement in the joint.

2. The joint hoop reinforcement is more effective for lower ductility levels (less than two). It helps to maintain the strength of a subassemblage more than it helps to reduce the loss of stiffness.

3. The shear stress in a joint affects the joint shear deformation, the slippage of beam and column bars, and thus the pinching of hysteresis loops. Lower joint shear stress seems to be more effective in reducing the joint shear deformation and slippage of bars than the addition of more joint reinforcement.

4. For higher energy dissipation, a combination of a lower joint shear stress and a moderate amount of joint reinforcement was found to be more effective than a combination of a higher shear stress level and a heavily reinforced joint.

5. A minimum column-to-beam flexural strength ratio of 1.5 was found to be suitable for design.

ACKNOWLEDGMENT

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NOTATION

f_c' = standard 28-day cylinder strength
P = column axial load
H = applied lateral load
γ = joint shear stress coefficient
Δ = column load-point displacement
ρ_h = effective area of transverse reinforcement per layer divided by the product of column width and spacing between layers
θ = joint shear deformation

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