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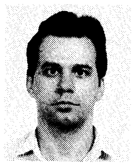
Evaluation of Joint-Shear Provisions for Interior Beam-Column-Slab Connections Using High-Strength Materials



by Gilson N. Guimaraes, Michael E. Kreger, and James O. Jirsa

Current provisions for design of beam-column-slab connections are based primarily on results of tests of connections constructed with concrete strengths not exceeding 6000 psi and reinforcement with nominal yield strengths of 60 ksi or less. Results of tests conducted on four interior beam-column-slab connections constructed with combinations of normal and high-strength concrete and reinforcement are presented, and existing joint-shear provisions are evaluated for use in design of connections constructed with high-strength materials.

Keywords: beams (supports); columns (supports); **connections;** cyclic loads; earthquakes; high-strength concretes; high-strength steels; **joints (junctions);** reinforced concrete; reinforcing steels; **shear strength;** slabs; welded wire fabric.

During a strong earthquake, beam-column-slab connections can experience severe reversed cyclic loads. If the joints in a moment-resisting frame do not possess adequate strength, the overall strength and stiffness of the frame may be adversely affected. Current ACI Building Code (ACI 318-89)[1] recommendations and ACI-ASCE Committee 352 recommendations (ACI 352 R-85)[2] for design of beam-column connections were developed primarily from results of tests conducted on specimens constructed with concrete strengths less than 6000 psi and steel nominal yield strengths of 60 ksi or less.

Production of high-strength concretes with compressive strengths exceeding 12,000 psi is now technically and economically feasible in commercial ready-mix concrete plants. It is not envisioned that complete frame systems will be constructed in the future using concrete with a 12,000 psi compressive strength. However, because columns have already been constructed with concrete strengths exceeding 12,000 psi, it is likely that concrete strengths exceeding 6000 psi will be used in joints in the future. To evaluate current joint-shear strength provisions for a large range of concrete strengths, nominal compressive strengths as high as 12,000 psi were included in this study of interior beam-column-slab connections. In addition, use of high-strength reinforcement and welded wire fabric (nominal yield stress of 75 and 80 ksi, respectively) has been shown to increase productivity and reduce labor costs on construction sites. One of the concerns about high-strength welded wire fabric is whether it will perform satisfactorily if the areas are proportioned using the actual yield capacity rather than the limiting value of 60 ksi specified in some codes. In this study, high-strength welded wire fabric was used in conjunction with normal and high-strength concrete in two of the four beam-column-slab specimens.

RESEARCH SIGNIFICANCE

The test results presented in this paper will be used to evaluate current ACI Building Code (ACI 318-89) and Committee 352 (ACI 352 R-85) joint-shear strength provisions for use in design of connections constructed with high-strength materials.

EXPERIMENTAL PROGRAM

Four reinforced concrete beam-column-slab connections were tested under cyclic bidirectional loading.[3] The specimens were constructed using combinations of normal and high-strength materials. Table 1 includes the nominal concrete compressive strengths and reinforcing steel grades used in designing the specimens. The actual strengths were somewhat higher. All column ties and beam-shear reinforcement in the specimens with 75 ksi steel were fabricated using welded wire fabric (nominal yield strength of 80 ksi). The four tests discussed in this paper were part of a larger testing

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*ACI member **Gilson N. Guimaraes** is an assistant professor in the Department of Civil Engineering at the Universidade Federal de Goias in Brazil. He received his MS and PhD degrees from the University of Texas at Austin in 1985 and 1988, respectively.*

*ACI member **Michael E. Kreger** is an associate professor of civil engineering at The University of Texas at Austin. He is Chairman of ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures, and is a member of ACI Committees 215, Fatigue of Concrete; 318-C, Standard Building Code—Analysis, Serviceability, and Safety; and 368, Earthquake-Resisting Concrete Structural Elements and Systems.*

***James O. Jirsa**, FACI, holds the Janet S. Cockrell Centennial Chair in Engineering at The University of Texas at Austin. He serves on ACI Committees 318, Standard Building Code, and 408, Bond and Development of Reinforcement, and is past member of the ACI Board of Direction.*

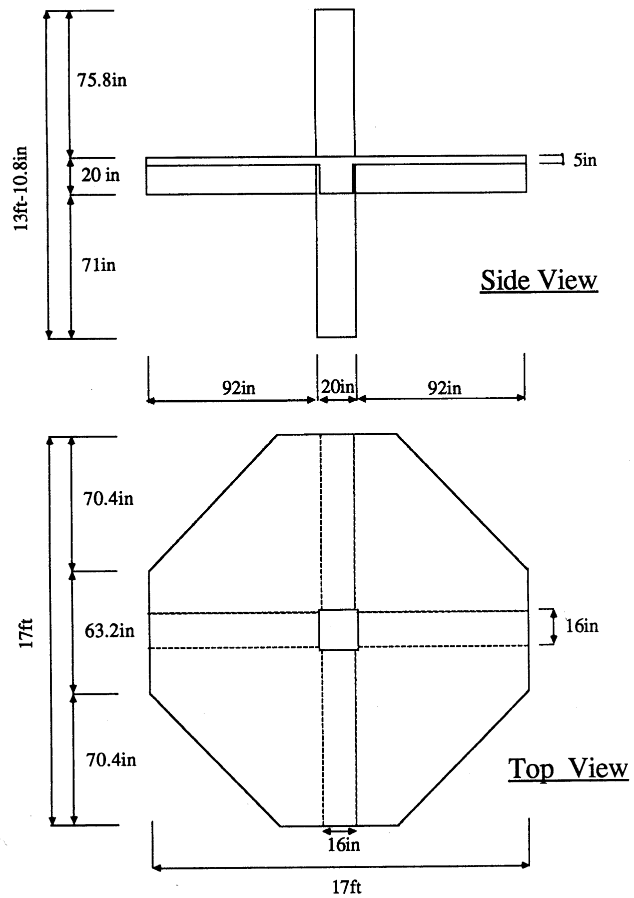

Fig. 1—Specimen dimensions

program composed of six beam-column-slab specimens. Specimen notation associated with the four tests described here is not in sequence, to preserve the original designations (J2, J4, J5, and J6). Results of the two tests not included here (an exterior joint and an interior joint with beams in one direction only) are reported by Kurose et al.[4]

All specimens were large-scale interior connections with transverse beams, as shown in Fig. 1. To evaluate the shear strength of the joint, beam and column cross sections and reinforcement were chosen so that shear stresses in the joint would reach strength design limits according to ACI Building Code recommendations.

Bottom beam reinforcement was approximately half of top reinforcement. In some specimens, beam top and bottom reinforcement consisted of six reinforcing bars placed in two layers. Table 1 shows the reinforcement used and the reinforcement ratios. Column longitudinal reinforcement was chosen to provide the column with bending moment capacity at least 20 percent greater than the beam moment capacities under bidirectional loading. High reinforcement ratios in the column were needed to obtain the necessary flexural capacity without affecting the joint dimensions. Increasing the column flexural capacity by increasing the cross sectional area would also increase the joint-shear strength, and consequently, the joint would not fail in shear. Note that joint failure was desired for this experimental investigation, but in the design of a frame located in a seismic zone a failure mode involving beam hinging would be desired.

Details of the transverse reinforcement in the column and joint were chosen to satisfy the requirements of Chapter 21 of the ACI Building Code. Five sets of perimeter hoops and crossties were placed within the joint area. Six sets were used in Specimen J4. One set was always placed above the uppermost longitudinal reinforcing bars and another was placed below the lowest longitudinal bars. Therefore, three layers were placed in the region between top and bottom beam bars (four layers in Specimen J4). Welded wire fabric was used for all transverse reinforcement (including the joint region) in high-strength steel specimens.

All closed ties terminated in standard 135-deg hooks.

Table 1 — Specimen details

Specimen no.	Nominal design strength			Beam reinforcement			Column reinforcement		Slab reinforcement	
	f'_c, psi	f_y, ksi longitudinal	f_y, ksi transverse	Longitudinal Top	Longitudinal Bottom	Transverse	Longitudinal	Transverse	Top	Bottom
J2	4000	60	60	6 #8 (1.5%)	6 #6 (0.8%)	2-#4 @ 4 in.	16 #9 (4.0%)	3-#4 @ 4 in.	#3 @ 12 in. (0.2%)	#3 @ 24 in. (0.1%)
J4	4000	75	80	4 #8 (1.0%)	4 #7 (0.8%)	3-D11 @ 4 in.	16 #9 (4.0%)	3-D11 @ 3 in.	#3 @ 12 in. (0.2%)	#3 @ 24 in. (0.1%)
J5	12,000	60	60	6 #10 (2.3%)	2 #10 4 #8 (1.8%)	3-#4 @ 3.5 in.	20 #10 (6.2%)	4-#4 @ 2 in.	#3 @ 12 in. (0.2%)	#3 @ 24 in. (0.1%)
J6	12,000	75	80	6 #9 (1.9%)	4 #9 (1.3%)	3-D20 @ 4 in.	20 #10 (6.2%)	4-D20 @ 2.5 in.	#3 @ 12 in. (0.2%)	#3 @ 24 in. (0.1%)

Notes:
1. Beam longitudinal reinforcement consisted of six bars placed in two layers.
2. Steel reinforcement ratio (shown in parentheses) based on gross cross-sectional area.
3. Slab reinforcement was Grade 60 steel for all specimens.
4. Joint transverse reinforcement same as column transverse reinforcement.

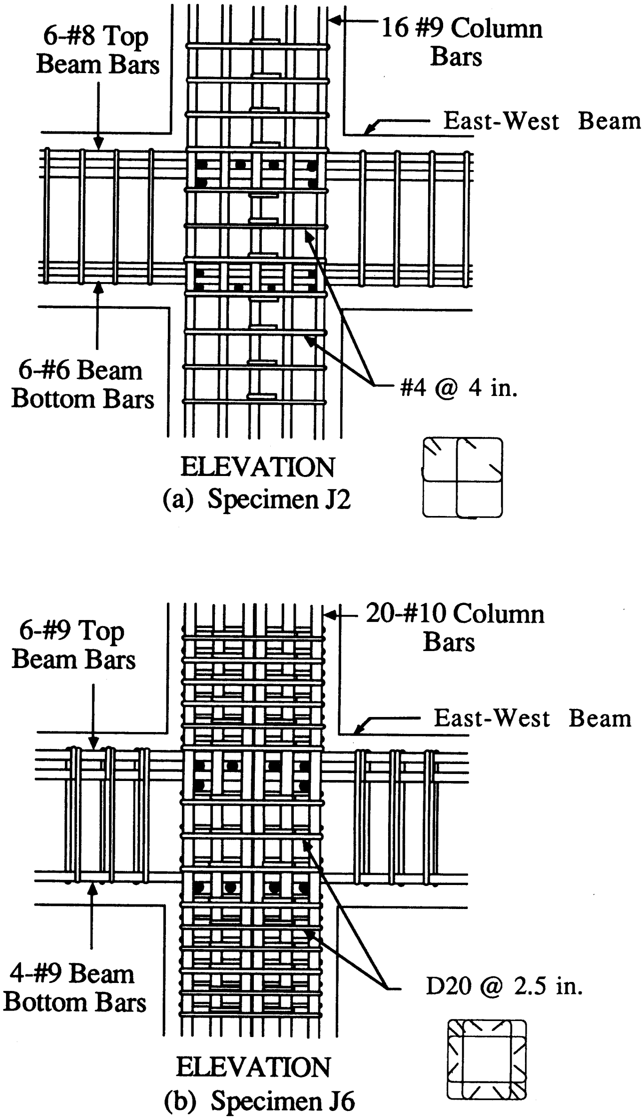

Fig. 2—Joint details

Fig. 3—Comparisons of column crossties

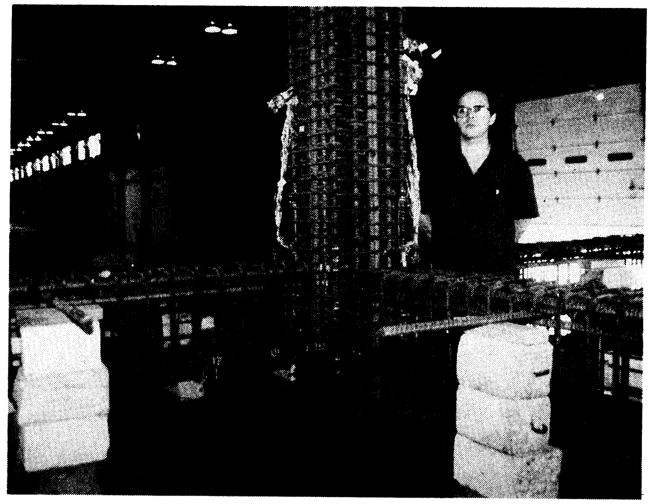

Fig. 4—Welded wire fabric reinforcing cage for Specimen J6

Crossties fabricated from Grade 60 bars had a 135-deg hook at one end and a 90-deg hook at the opposite end. During cage fabrication, the 90- and 135-deg hooks were alternated from one layer to the next. Welded wire fabric crossties had 135-deg hooks at both ends. Specimens J2 and J4 had two crossties in each layer, and Specimens J5 and J6 had four crossties in each layer.

Fig. 2(a) shows the reinforcement details used in Specimen J2, and Fig. 2(b) shows the reinforcement details used in Specimen J6 where welded wire fabric was used for transverse reinforcement.

The quality of reinforcement cages constructed with welded wire fabric (WWF) was substantially higher than the quality of cages constructed with Grade 60 reinforcing bars because closer tolerances were maintained during bending of the welded wire fabric. Differences in fabrication quality are illustrated through comparisons of five crossties in Fig. 3. The crosstie shown on the extreme left was cut from a WWF cage and was typical of all WWF crossties. The other ties were fabricated by a local supplier using Grade 60 deformed bars. The large variation in dimensions of fabricated Grade 60 bars had a negative influence on the quality of constructed cages and productivity in assembly of reinforcing cages. However, ties having correct dimensions were selected for use in the joint regions of Specimens J2 and J5, so that the behavior of critical joint regions would not be adversely affected by poorly fabricated reinforcement. The close tolerances associated with WWF are illustrated by the photograph of the reinforcing cage for specimen J6 (Fig. 4).

Welded wire fabric reinforcing cages for Specimens J4 and J6 were assembled in less time and with greater ease than the reinforcing cages of Grade 60 bars because substantially fewer pieces had to be tied into place, and because dimensions of ties and crossties were carefully controlled during fabrication. Fig. 5 and 6 illustrate the assembly sequence for column and beam WWF gages. The column cage was fabricated horizontally. Two column longitudinal bars were supported at their ends, then crossties were tied to the bars as shown in Fig. 5(a). Crossties in the orthogonal direction were then tied in place, as shown in Fig. 5(b). The WWF that composed half of the outer ties was placed in position,

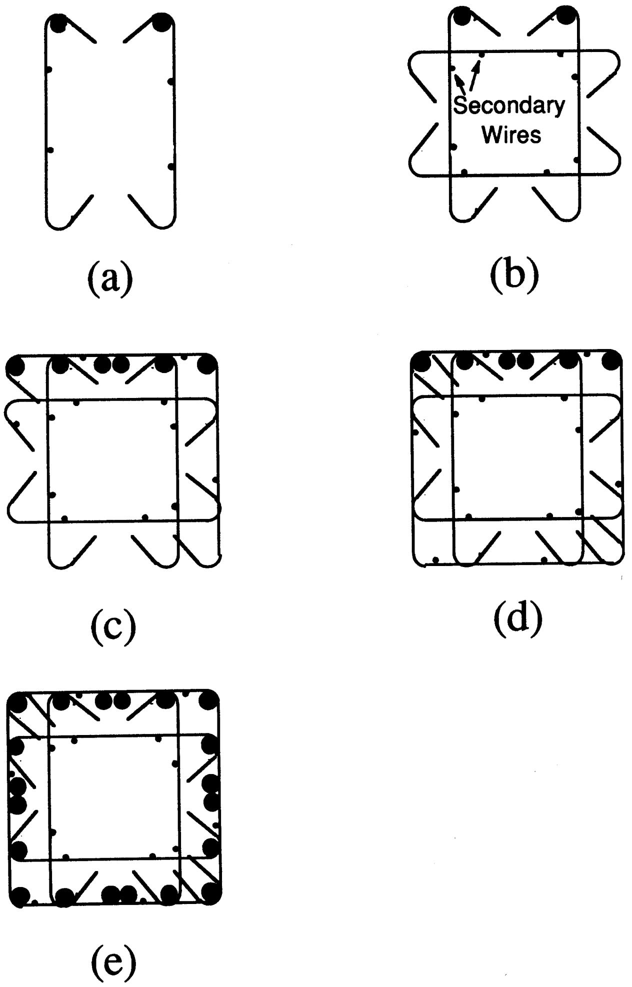

Fig. 5—Welded wire fabric column cage assembly sequence for Specimen J6

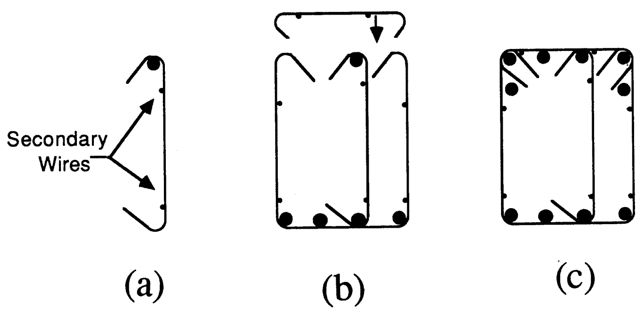

Fig. 6—Welded wire fabric beam cage assembly sequence for Specimen J6

then four longitudinal bars were threaded into place and secured [Fig. 5(c)]. The other half of the outer ties was temporarily secured to crossties [Fig. 5(d)], then the remaining 14 longitudinal bars were threaded into place and secured as shown in Fig. 5(e). The completed column cage was erected, and beam reinforcement was assembled in a similar manner as shown in Fig. 6(a) through (c).

The secondary fabric wires noted in Fig 5 and 6 were not considered as part of the longitudinal reinforcement. However, the position of secondary wires in WWF cages is very important in the assembly of cages. Poor positioning of secondary wires will make assembly of reinforcing cages extremely difficult or impossible.

Table 2 — 28-day concrete compressive strength, psi

	J2	J4	J5	J6
Slabs, beams, joint, lower column	3700 (4010)	4800 (4590)	10,310 (11,300)	11,860 (13,360)
Upper column	3780	4420 (4220)	12,720 (13,800)	10,040 (10,200)

( ) Compressive strength at the time of testing, psi.

Table 3 — Reinforcing steel strength

Grade	Bar	Yield strength, ksi	Tensile strength, ksi
60	#3	80.8	118
	#4	79.7	111
	#6	74.2	108
	#7	65.6	101
	#8	67.2	106
	#9	66.6	106
	#10	78.8	108
80	#3WWF*	84.8	90
	#4WWF*	82.6	99
75	#7	79.5	120
	#8	80.0	108
	#9	75.8	119
	#10	81.4	121

*Welded wire fabric cage reinforcement.

The 5-in. thick slab was reinforced with #3 Grade 60 bars placed in top (12-in. spacing) and bottom (24-in. spacing) layers in both directions. All slab bars were continuous over the beams. Concrete cover was 1½ in. for beam and column reinforcement and ¾ in. for slab bars. Tables 2 and 3 show the 28-day concrete compressive strengths (determined from 6 x 12-in. cylinders) and reinforcing steel strengths.

Fig. 7 illustrates the loading setup used in the investigation. The lower end of the column was connected at the base by grouting it to a steel box which rested on a spherical contact bearing that was connected to the floor. The spherical bearing allowed rotation of the steel box about any axis and restricted all horizontal movement. The upper end of the column was grouted to another steel box which was connected to the reaction wall through square steel tubes and threaded bars. Because the restraining system at the top of the column was inherently flexible in the north-south direction (parallel to the strong wall), displacement measurements were corrected for rigid body motion of the specimen in that direction. The upper connection allowed rotation of the column end about any axis contained in a horizontal plane. Additional details can be found in Reference 3.

Each specimen was loaded by displacing beam tips equal distances in opposite directions while holding column ends fixed. The interstory drift angle was then defined as the sum of beam tip displacements divided by the beam span. The displacement-controlled loading history is shown in Fig. 8. Specimens were loaded to prescribed drift levels in the east-west and/or north-south directions. East-west was the primary loading direction (perpendicular to the reaction wall). This loading program was used in the series of specimens tested as part of the U.S.-Japan-New Zealand-China Cooperative Research Project described in detail in Reference 4. Since control specimen J2 was part of that series, the same loading pattern was used for the specimens constructed with high-strength materials.

During Cycles 1, 2, 3, 5, 6, 9, and 10, specimens were loaded in the E-W direction only. During Cycle 4, unidirectional loading was applied in the N-S direction. Bidirectional cycles occurred at the 2 percent drift level (Cycles 7 and 8) and at the 4 percent drift level (Cycles 11 and 12). During bidirectional load cycles, the specimen was first displaced to the prescribed drift level in the E-W direction (a, in Fig. 8), then while maintaining the E-W displacement, the specimen was displaced to the required drift in the N-S direction (b). After reaching the maximum drift, displacements were first reduced to zero in the E-W direction (c) then in the N-S direction (d). The bidirectional loading in the opposite quadrant was performed in the same manner.

TEST RESULTS

Behavior of test specimens is presented through story shear-drift angle relations and story-shear orbits. The contributions of joint-shear deformations and beam and column deformations to total drift for each specimen are examined to provide an indication of the role of each element in responding to large deformations. Finally, the test results are compared with current joint shear-strength provisions.

Overall behavior of specimens is shown using story shear-versus-drift angle relations. Drift angle R, plotted in figures that follow, is defined as the sum of beam tip displacements $(\Delta_i + \Delta_j)$ divided by the beam span L expressed as a percentage (see the inset on Fig. 9 and 10). Uniaxial story shear is computed from beam reactions by multiplying the sum of beam reactions $P_i + P_j$ by the ratio of half the beam span to column height $L/2H$. Fig. 9 shows the E-W component of story shear-versus-drift angle for Specimens J2, J4, J5, and J6. Fig. 10 shows the N-S component of story shear-versus-drift angle for Specimens J2 and J5. Overall behavior of Specimens J2 and J5 in both directions was very similar to that of Specimens J4 and J6, respectively. Maximum story shear for unidirectional loading always occurred during the positive loading portion of Cycle 9 (4 percent drift in E-W direction). Maximum unidirectional story shears for 2 percent drift did not differ from values measured for 4 percent drift by more than 15 percent. Test results indicated that the behavior of specimens with high-strength steel was almost identical to those with normal strength steel. Since the behavior of Specimens J2 and J5 was similar to Specimens J4 and J6, respectively, differences in behavior between normal and high-strength concrete specimens will be emphasized.

The vertical segments in Fig. 9 and 10 at 0, 2, and 4

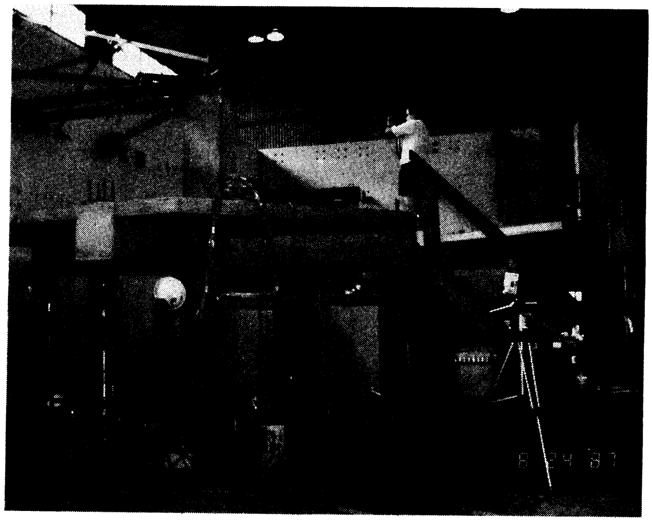

Fig. 7—Test setup

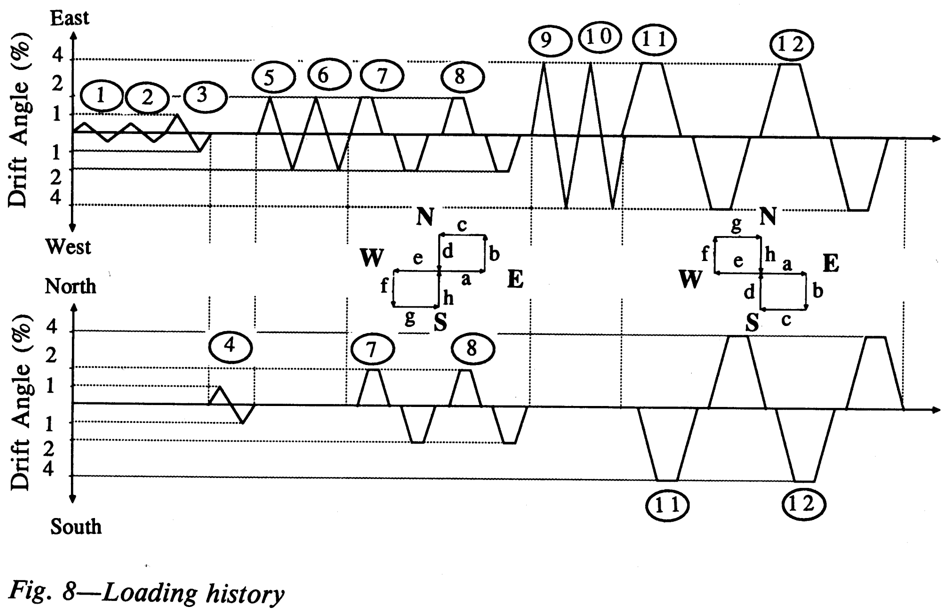

Fig. 8—Loading history

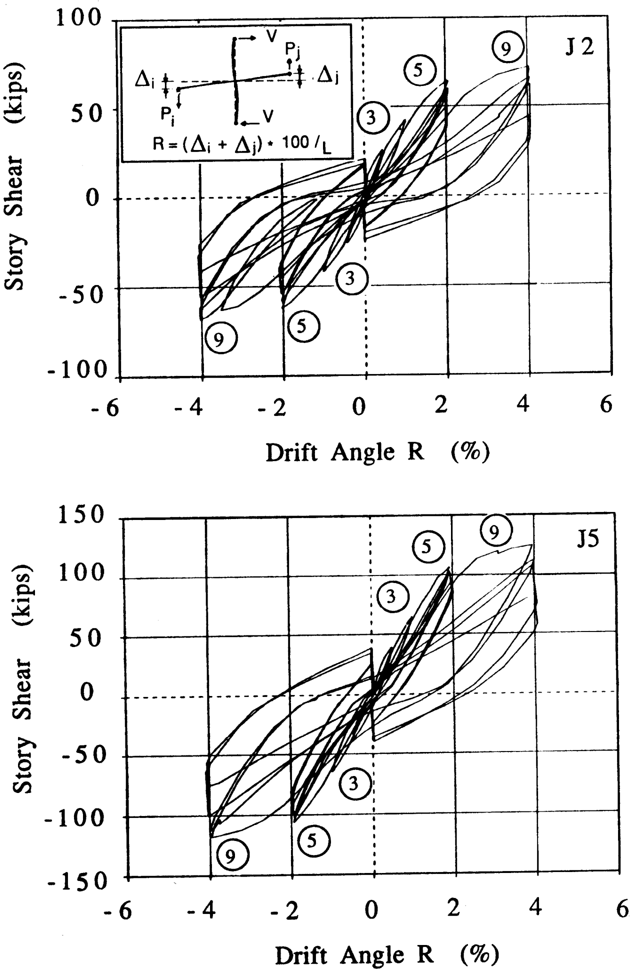

Fig. 9—Story shear-versus-drift angle (E-W direction)

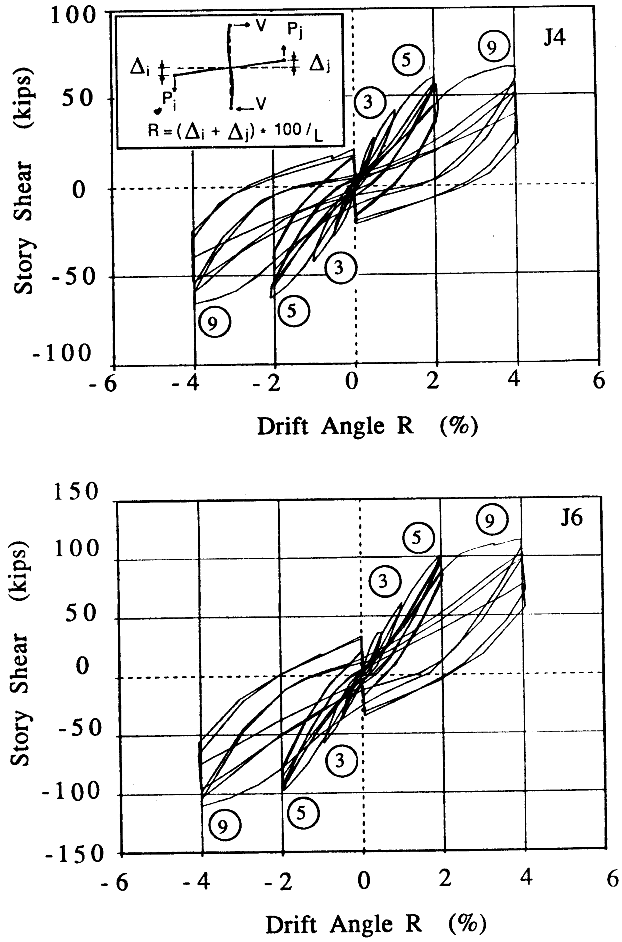

Fig. 9(b)—Story shear-versus-drift angle (E-W direction)

percent drift cycles are due to loading interaction. Loading or unloading in one direction decreased the load in the other direction. A large drop in the story shear was observed in the last bidirectional cycle for each specimen. The loss of strength indicated failure.

A plot of measured E-W and N-S story shears provides a means of assessing bidirectional response. Such plots occasionally have been referred to as story shear orbits. Fig. 11 contains story shear orbits for Specimens J2 and J5. An initial examination of the orbits indicates that during bidirectional load cycles, change in loading direction resulted in change in story shear in the orthogonal direction, even though drift remained constant in the orthogonal direction at peaks of bidirectional cycles (2 and 4 percent drift) and at zero drift (The same change in story shear is shown in Fig. 9 and 10 by the vertical offsets). Further examination indicates that maximum bidirectional story shears (represented in Fig. 11 by the largest distance between the origin and any ordered pair of story shears) occurred during Cycle 7 (2 percent drift) for Specimen J5 and Cycle 11 (4 percent drift) for Specimen J2. However, peak story shears for 2 and 4 percent drift did not differ by more than 5 percent. Similar results were observed for J4 and J6. A small decrease in story shear occurred during the second loading cycle at a given drift level. Larger reductions in story shear were observed for 4 percent bidirectional drift cycles (Quadrants 2 and 4 in Fig. 11), and were associated with the failure just noted.

It can be shown that specimen strengths were limited by joint shear, and not by flexure or shear in the beams and columns. This is accomplished by examining the three major components that comprise the total drift response of each specimen: beam flexural response, column flexural response, and joint-shear distortion. The magnitude of each component was determined from displacement transducer readings.

Fig. 12 shows the beam, column, and joint contributions to story drift angle for Specimens J2, J4, J5, and J6. Each contribution is expressed as a percentage of the total drift angle. Plots for E-W and N-S loading cycles are shown for each specimen. For each loading cycle, the percent contribution shown is an average of values calculated at positive and negative peak deformations. In general, joint contributions increased and column and beam contributions decreased as the drift angle increased during testing. The largest contribution (50 to 80 percent) was by the beams. The percent contribution of the column decreased, but the absolute deformation contributed by the column remained nearly constant; it simply represented a smaller fraction of an increasing drift angle. The formation of plastic hinges in E-W beams of Specimens J4, J5, and J6 was indi-

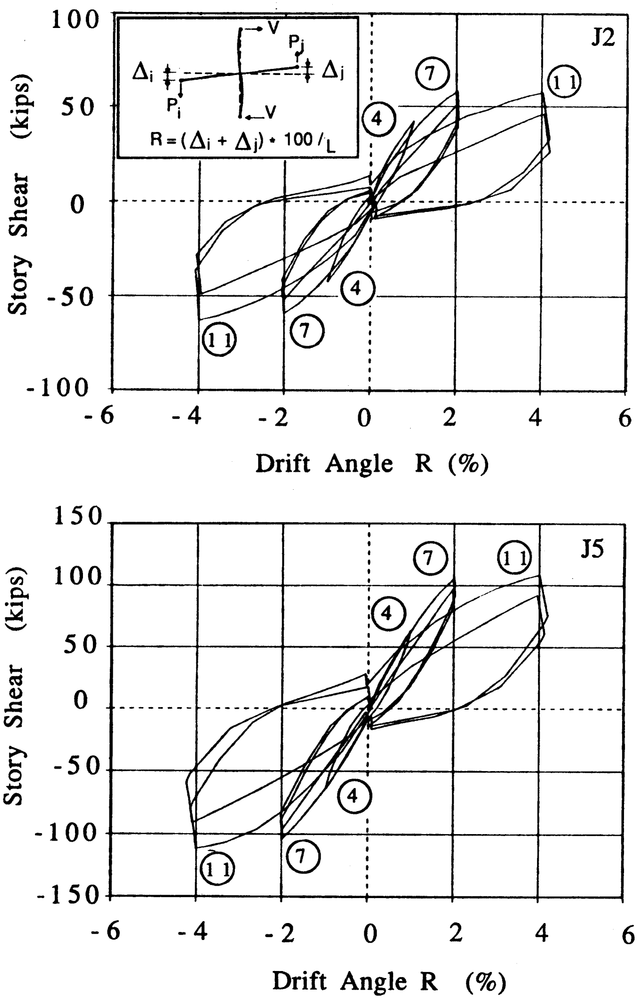

Fig. 10—*Story shear-versus-drift angle (N-S direction)*

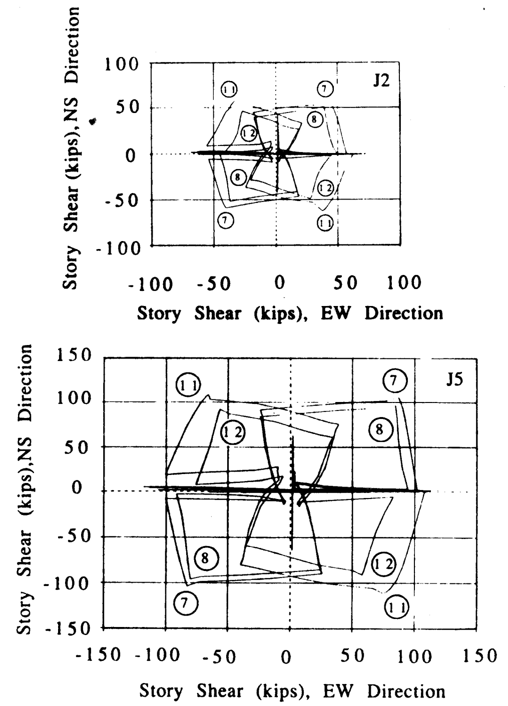

Fig. 11—Story-shear orbits

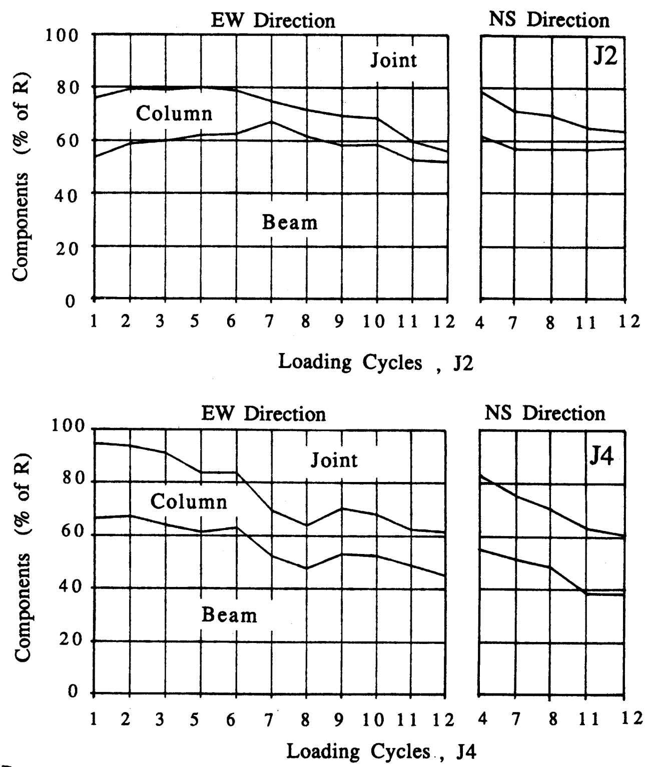

Fig. 12(a)—Beam, column, and joint contributions to interstory drift angle

cated by the increase in beam contribution from Cycle 8 to Cycle 9 during E-W loading. In Specimens J4 and J5, beam hinging was accompanied by a relative decrease in the joint contribution, a trend which was not observed in Specimen J6. Beam hinging is not indicated in Fig. 12 for Specimen J2, although hinges were observed to have formed in beams. Instead, changes in behavior appear to have been dominated more by increasing distress in the joint. Although beam hinges formed in all specimens, it is significant that joint contributions to total drift continued to increase through remaining cycles, strongly indicating that the joint-shear strength for each specimen had been reached.

EVALUATION OF JOINT DESIGN PROVISIONS

Measured joint-shear strengths are compared with values calculated using the ACI Building Code (ACI 318-89)[1] and ACI-ASCE Committee 352 (ACI 352 R-85)[2] provisions. Related provisions used in the design of joints, vis a vis, high-strength materials are also considered.

Joint shear strength

Maximum story shears for 2 and 4 percent uniaxial and bidirectional drift response are compiled in Table 4. To evaluate the joint strength provisions, measured story shears were converted to shears acting on a horizontal plane through the joint (joint shears). Measured unidirectional joint shear components are listed in Table 5, along with unidirectional joint shear strengths

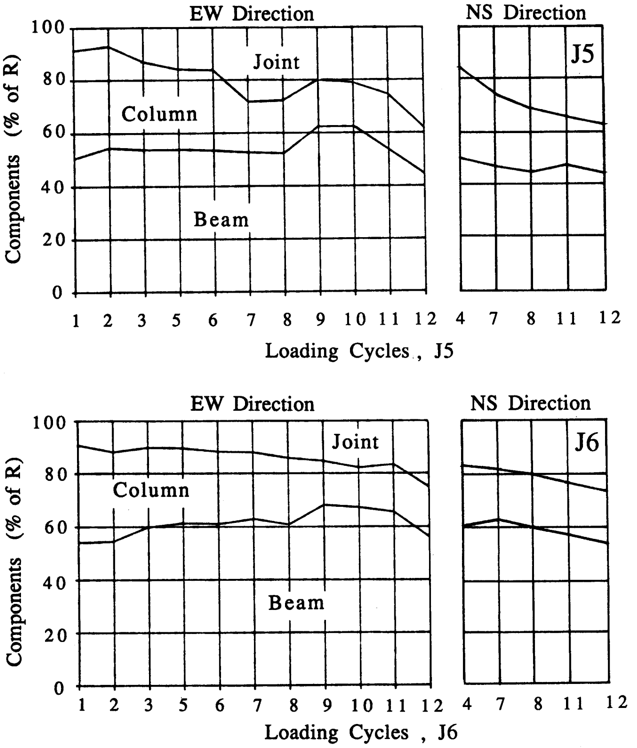

Fig. 12(b)—Beam, column, and joint contributions to interstory drift angle

which were calculated using the ACI 352 recommendations. For interior joints, shear strength was calculated as

$$V_n = 20\sqrt{f'_c}\, b_j h$$

where

V_n = unidirectional joint shear strength
f'_c = concrete compressive strength, psi
b_j = effective joint width (average of the beam and column widths in inches)
h = column depth in direction of loading, in.

The data in Table 5 indicate that because of interaction between response in the two primary loading directions, specimens did not maintain computed unidirectional joint strengths during all bidirectional load cycles. There are no recommendations in the ACI Building Code or ACI 352 recommendations[1,2] for bidirectional story-shear capacity. A simple model that yields a conservative estimate of measured joint strengths is an elliptical interaction curve based on the calculated unidirectional joint shear strengths. For the specimens in this study, the elliptical interaction curve reverts to a circular interaction curve because joint dimensions associated with the two loading directions are the same. The observed bidirectional strength can be determined using the square root of the sum of the squares of the E-W and N-S shear values.

Unidirectional and bidirectional joint-shear maxima for 2 percent drift are compared with the proposed interaction curve in Fig. 13. Data plotted in Fig. 13 have been normalized using $\sqrt{f'_c}\, b_j h$ to facilitate comparisons between data. Ratios of measured to computed joint shear strength are also listed in Table 6 for both unidirectional and bidirectional response. The ratios of measured to calculated strength are typically higher for bidirectional loading, indicating that perhaps a less conservative (but not necessarily simpler) interaction relationship could be developed.

Joint confinement

High-strength welded wire cages were used as transverse reinforcement in Specimens J4 and J6, while Grade 60 bars were used as transverse joint reinforce-

Table 4 — Measured maximum story shear

		Specimen J2		Specimen J4		Specimen J5		Specimen J6	
		Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional
Measured story shear, kips	2% drift	64.3	EW = 42.7 NS = 58.1	62.9	EW = 42.6 NS = 55.0	106.7	EW = 86.9 NS = 105.8	101.4	EW = 85.0 NS = 96.4
	4% drift	72.0	EW = 37.6 NS = 63.0	67.1	EW = 32.0 NS = 57.8	123.9	EW = 74.8 NS = 111.5	114.7	EW = 72.4 NS = 107.6

Table 5 — Measured and calculated joint-shear strength

		Specimen J2		Specimen J4		Specimen J5		Specimen J6	
		Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional
Measured joint shear, kips	2% drift	602	EW = 400 NS = 544 (675)	562	EW = 381 NS = 491 (621)	1018	EW = 829 NS = 1021 (1315)	958	EW = 803 NS = 906 (1211)
	4% drift	674	EW = 352 NS = 590 (687)	599	EW = 286 NS = 516 (590)	1182	EW = 714 NS = 1076 (1291)	1083	EW = 684 NS = 1011 (1221)
Calculated joint shear, kips		456	EW = 456 NS = 456	488	EW = 488 NS = 488	765	EW = 765 NS = 765	832	EW = 832 NS = 832

( ) Bidirectional response based on square root of sum of squares of unidirectional response.

ment in Specimens J2 and J5. Size, number, and spacing of joint ties were determined according to the following ACI 318 recommendation[1]

$$A_{sh} = 0.12\, sh_c \frac{f'_c}{f_{yh}}$$

where

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties)
s = tie spacing
h_c = size of column core measured center-to-center of confining reinforcement
f_{yh} = yield strength of transverse reinforcement

Design yield strengths of 60 and 80 ksi were used for proportioning the normal and high-strength reinforcement.

Strain histories for ties within the joint demonstrated that both normal and high-strength reinforcement were effective for providing confinement in the joints of normal and high-strength concrete specimens as indicated by strains in ties exceeding yield levels and measured joint shear strengths exceeding calculated values. A typical story shear versus strain history for a crosstie in Specimen J4 (high-strength tie) is shown in Fig. 14.

SUMMARY

The primary objective of this study was to examine the behavior of interior beam-column-slab specimens constructed with high-strength materials. Therefore, the four specimens included in this study were designed so joint-shear stresses reached design values specified in ACI 318 and ACI 352 documents. Observations and conclusions are summarized in the following:

1. All specimens failed in joint shear at 4 percent drift after formation of beam hinges adjacent to the column.

2. All specimens resisted unidirectional joint shears higher than those calculated using current ACI 318 and ACI 352 design recommendations for joint-shear strength.

3. Maximum unidirectional joint shears were obtained during 4 percent drift cycles. However, maximum joint shears obtained during 2 percent drift cycles did not differ from those obtained at 4 percent drift by more than 15 percent and were still higher than calculated joint shears. Bidirectional loading resulted in practically no difference between joint-shear maxima obtained during 2 and 4 percent drift cycles.

4. Bidirectional joint-shear strength for interior beam-column-slab specimens constructed with normal or high-strength concrete is conservatively predicted using an elliptical interaction curve based on unidirectional joint-shear strengths recommended by ACI 318 and ACI 352.

5. Both normal and high-strength transverse rein-

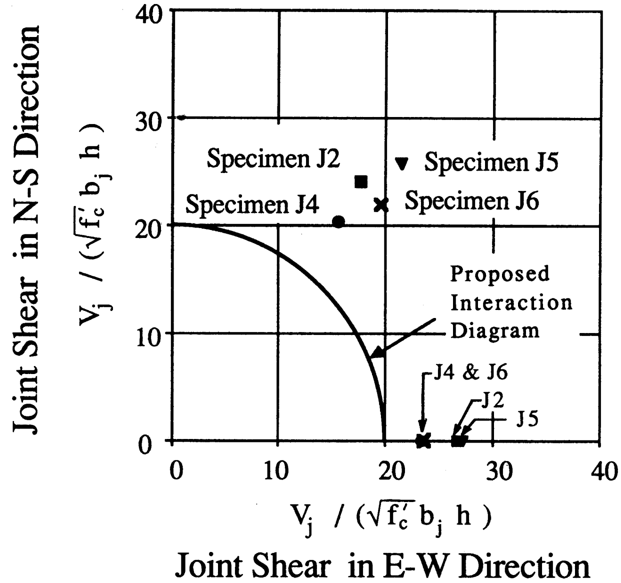

Fig. 13—Joint-shear maxima at 2 percent drift

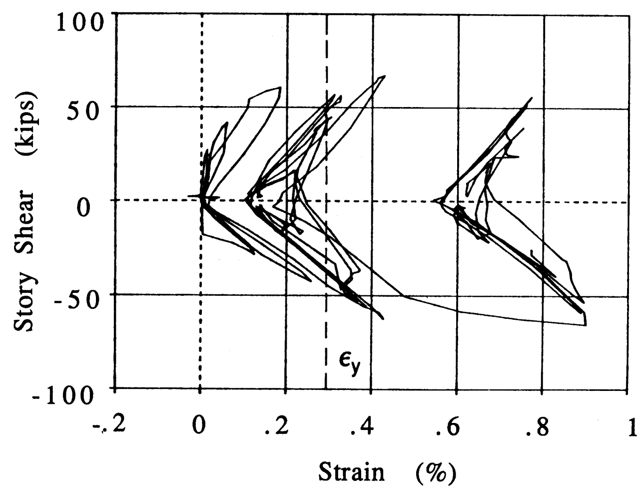

Fig. 14—Story shear versus strain for joint crosstie (Specimen J4)

Table 6 — Comparison of measured and calculated joint-shear strengths

		Specimen J2		Specimen J4		Specimen J5		Specimen J6	
		Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional	Unidirectional	Bidirectional
Ratio of measured to calculated strengths	2% drift	1.32	1.48	1.15	1.27	1.33	1.72	1.15	1.46
	4% drift	1.48	1.51	1.23	1.21	1.55	1.69	1.30	1.47

forcement provided adequate confinement for joints. Size, number, and spacing of ties were determined according to ACI 318 recommendations.

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CONVERSION FACTORS

1 in. = 25.4 mm
1 ksi = 6.90 MPa
1 kip-ft = 1360 N-m
1 kip = 4450 N

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89)," American Concrete Institute, Detroit, 1989, 353 pp.

2. ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," (ACI 352 R-85), *ACI Structural Journal,* Vol. 82, No. 3, May-June 1985, pp. 266-283.

3. Guimaraes, G. N.; Kreger, M. E.; and Jirsa, J. O., "Reinforced Concrete Frame Connections Constructed Using High Strength Materials," *Report* No. 89-1, Phil M. Ferguson Structural Engineering Laboratory, The University of Texas at Austin, Aug., 1989.

4. Kurose, Y.; Guimaraes, G. N.; Liu, Z.; Kreger, M. E.; and Jirsa, J. O., "Study of Reinforced Concrete Beam Column Joints Under Uniaxial and Biaxial Loading," *Report* No. 88-2, Phil M. Ferguson Structural Engineering Laboratory, The University of Texas at Austin, Dec., 1988.