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BI-DIRECTIONAL RESPONSE OF SLAB-BEAM-COLUMN CONNECTIONS USING HIGH STRENGTH MATERIALS

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SUMMARY

This paper reports on a study conducted to evaluate the behavior of slab-beam-column connections constructed with high strength materials. Large scale interior specimens were tested under complex bidirectional loadings. Results of the tests were used to determine the influence of high strength concrete and high strength steel on (a) joint shear strength, (b) specimen stiffness, (c) bidirectional loading effects and (d) effectiveness of the slab.

INTRODUCTION

A series of six full-scale reinforced concrete slab-beam-column connections were tested under cyclic bidirectional loading at the Ferguson Structural Engineering Laboratory of the University of Texas at Austin. The first three specimens were part of a cooperative program conducted by researchers in the United States, New Zealand, China (P.R.C.) and Japan. Three other specimens were constructed using high strength concrete and high strength steel.

The objective of this paper is to report on tests conducted on the high strength specimens (specimens J4, J5 and J6). Tests on the first three specimens (J1, J2 and J3) are reported in detail elsewhere (Ref.1).

SPECIMEN DESCRIPTION

The specimens were designed using recommendations of the ACI 318 Building Code (Ref. 2) and ACI 352 Committee Report (Ref. 3). Beam reinforcement was chosen to produce high joint shear forces. Column reinforcement was designed so the column bidirectional moment capacity was higher than the beam moment capacity. Slab top reinforcement was #3 bars at 12 in. spacing and #3 bottom bars were placed every 24 in.

Specimens J2, J4, J5 and J6 were interior joints with the same dimensions (Ref. 1). The reinforcement used is shown in Table 1. Specimens J2 and J4 were constructed with normal-strength concrete, and high-strength concrete was used in specimens J5 and J6. High-strength steel (Grade 75) and welded wire fabric reinforcing cages were used in specimens J4 and J6. Specimen J2 was the control specimen (normal strength concrete, Grade 60 steel).

Joint transverse reinforcement was the same as column transverse reinforcement with at least three stirrups located in the joint between beam bars. The use of welded wire fabric as transverse reinforcement for beams and columns significantly improved quality control and productivity during construction of reinforcing cages. The concrete casting procedure is described in Ref. 1. Table 2 shows the 28-day concrete cylinder compressive strength and Table 3 gives the tensile yield and ultimate strengths of the reinforcing steel.

TESTING PROGRAM

The cyclic loading was applied through a program of interstory drift angles as shown in Fig. 1. East-west was the primary loading direction, with bidirectional cycles at the 2% drift level (cycles 7 and 8) and 4% drift level (cycles 11 and 12).

TEST RESULTS

Crack Patterns Figure 2 shows the final crack pattern for specimen J6. Cracks developed in the beams, column, and top and bottom of the slab. Horizontal flexural cracks and inclined shear cracks predominated in the column. Specimens J5 and J6 showed vertical column cracks as shown on the east view of J6 (Fig.2). The beams showed mostly flexural and inclined shear cracks, some propagating from flexural cracks at the bottom of the slab. Loading at higher drift levels caused torsional cracks in the transverse beams near the column.

Cracking and spalling of concrete was most extensive around the joint. Significant spalling occurred at 4% drift levels. Specimens J2 and J4 showed more spalling of the joint concrete than J5 and J6. High-strength concrete joints showed minor distress up to cycle 8. Cracking in specimens J5 and J6 increased significantly during cycle 9 (4% drift). In all specimens spalling exposed rebars in the joint area.

Story Shear - Drift Angle Relations In Fig. 3, story shear is plotted against interstory drift angle for the EW loading of specimens J4 and J5. Comparable specimens J2 and J6 showed similar behavior to J4 and J5 respectively. The hysteretic response showed considerable pinching, especially in the high-strength concrete specimens. Vertical segments at 0%, 2% and 4% drift levels are due to bidirectional interaction as loading or unloading in one direction lowered the story shear in the other direction.

Peak-to-Peak Stiffness Peak-to-peak stiffnesses are shown in Table 4 for cycles 1, 3, 5, 7, 9 and 11 (EW loading). Specimens J5 and J6 (high-strength concrete specimens) were always stiffer than J2 and J4, especially at higher drift levels. Loss of stiffness in J2 and J4 was greater up to cycle 9 compared with loss of stiffness in specimens J5 and J6.

Joint Shear Strength Table 5 shows maximum measured story shears for unidirectional and bidirectional loadings. Maximum story shears occurred at 4% drift, except for bidirectional maxima for specimens J4 and J5 which occurred at 2% drift. Story shears calculated based on References 3, 4 and 5 are also shown. Calculated bidirectional story shears were based on circular interaction curves using calculated uniaxial shear strengths.

All specimens resisted story shears higher than any of the calculated values. Ratios of measured to calculated story shears are shown in parenthesis in Table 5. The tests conducted on these 4 specimens indicate that current ACI and AIJ recommendations can be used to approximate the shear strength of interior joints in buildings constructed with high strength materials.

Interstory Drift Angle Components Figure 4 shows the relative contribution of beam, column and joint deformations to interstory drift for specimens J4 and J5 (EW loading). Although the beam component was largest, the joint component increased steadily, indicating joint shear distress. The high-strength concrete specimens (J5 and J6) showed a slight decrease in the joint contribution during cycle 9 which was due to the increase in the beam component as stiffness was lost due to sudden spalling and crushing of the concrete around the joint region.

Slab Influence Table 6 shows maximum beam moments measured at the column face compared with calculated moment capacities for positive and negative loading. For calculated moment capacities the effective slab width was taken as 60% of the total slab width. In negative bending, all slab reinforcement within the effective slab width was considered effective. Negative T-beam moment capacities are in very good agreement with measured values.

CONCLUSIONS

The following conclusions were obtained from tests completed on beam-column-slab interior connections constructed using normal and high-strength materials.

1. All specimens reached story shears higher than calculated values based on ACI, AIJ and NZS Design Recommendations. Bidirectional strength exceeded measured unidirectional strength. Considerable pinching occurred in the story shear-drift angle curves, especially for the high-strength concrete specimens.
2. High-strength concrete specimens were stiffer and showed minor joint distress at 2% drift levels. Joint shear strength under bidirectional loading resulted in about the same ratios of measured to calculated shear strength as unidirectional loading. From these two tests it would appear that current equations for joint shear can be applied to high-strength materials.
3. Beam moment capacities, assuming the effective slab width to be 60% of the total slab width showed very good agreement with test results.

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4. "Standard for Structural Calculation of Steel Reinforced Concrete Structures", Architectural Institute of Japan, 1975.
5. "Code of Practice for the Design of Concrete Structures", Standards Association of New Zealand, Wellington, 1982.

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Table 1 - Specimen Details

Specimen	Design Strength		Beam Reinforcement			Column Reinforcement	
			Longitudinal		Transverse	Longitudinal	Transverse*
	f'c (psi)	fy (ksi)	Top	Bottom			
J2	4,000	60	4#8 / 2#8	2#6 / 4#6	2- #4 @4"	16 #9	3- #4 @4"
J4	4,000	75	4 #8	4#7	3- #3 @4"	16 #9	3- #3 @3"
J5	12,000	60	4#10 / 2#10	2#10 / 4#8	3- #4 @3.5"	20 #10	4- #4 @2"
J6	12,000	75	4#9 / 2#9	4 #9	3- #4 @4"	20 #10	4- #4 @2.5"

* Transverse reinforcement also used in joint

Table 2 - 28-Day Concrete Compressive Strength (psi)

	J2	J4	J5	J6
Slab, Beams, Joint, Lower Column	3,700 (4,010)	4,800 (4,590)	10,310 (11,300)	11,860 (13,360)
Upper Column	3,780	4,420 (4,220)	12,720 (13,800)	10,040 (10,200)

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

Table 3 - Reinforcing Steel Strength

Grade	Bar	Diameter (in.)	Yield Strength (ksi)	Ultimate Strength (ksi)
60	#3	0.38	80.8	118.2
	#4	0.50	79.7	111.9
	#6	0.75	74.2	108.9
	#7	0.80	65.6	101.4
	#8	1.00	67.2	106.0
	#9	1.13	66.6	106.1
	#10	1.25	78.8	108.0
75	#3WWF*	0.38	84.8	90.0
	#4WWF*	0.50	82.6	99.0
	#7	0.75	79.5	120.0
	#8	1.00	80.0	108.0
	#9	1.13	75.8	119.2
	#10	1.25	81.4	121.8

* Welded Wire Fabric Cage Reinforcement

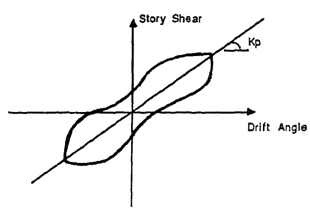

Table 4 - Peak-to-Peak Stiffness (EW Direction)

Cycle	Drift Angle	J2	J4	J5	J6
1	0.5%	0.594	0.549	0.796	0.731
3	1%	0.416	0.411	0.657	0.606
5	2%	0.309	0.301	0.550	0.505
7	2% BI	0.272	0.271	0.518	0.476
9	4%	0.174	0.166	0.306	0.282
11	4% BI	0.146	0.131	0.261	0.242

Stiffnesses in kips/rad

Table 5 - Measured Maximum Story Shear Compared with Calculated Story Shear

		J2		J4		J5		J6	
		Uniaxial	Biaxial	Uniaxial	Biaxial	Uniaxial	Biaxial	Uniaxial	Biaxial
Measured Story Shear (kips)		72.0	73.4 [EW = 37.6] [NS = 63.0]	67.1	69.6 [EW= 42.65] [NS = 55.0]	123.9	136.9 [EW = 86.9] [NS = 105.8]	114.7	129.7 [EW = 72.4] [NS = 107.6]
Calculated Story Shear (kips)	a) ACI 352	48.7 (1.48)	48.7 (1.51)	54.6 (1.23)	54.6 (1.27)	80.2 (1.54)	80.2 (1.72)	88.1 (1.30)	88.1 (1.47)
	b) AIJ-SRC	45.6 (1.58)	45.6 (1.61)	47.0 (1.43)	47.0 (1.47)	68.3 (1.81)	68.3 (2.02)	70.4 (1.64)	70.4 (1.85)
	c) NZS-3101	15.3 (4.70)	15.3 (4.79)	12.5 (5.35)	12.5 (5.52)	20.0 (6.18)	20.0 (6.88)	21.0 (5.47)	21.0 (6.16)

( ) Ratio of measured story shear to calculated story shear

Table 6 - Measured Maximum Beam Moments vs. Calculated Moment Capacity

Beam		Positive Bending					Negative Bending				
		M_{test} (k-in)	R_{calc} (k-in)	M_{test}/R_{calc}	T_{calc} (k-in)	M_{test}/T_{calc}	M_{test} (k-in)	R_{calc} (k-in)	M_{test}/R_{calc}	T_{calc} (k-in)	M_{test}/T_{calc}
J2	East	4,161	2,766	1.50	3,024	1.38	6,156	4,379	1.41	5,885	1.05
	West	3,962		1.43		1.31	6,478		1.48		1.10
	North	3,785	2,969	1.27	3,227	1.17	5,533	4,060	1.36	5,619	0.98
	South	3,778		1.27		1.17	4,936		1.22		0.88
J4	East	3,861	2,809	1.37	3,023	1.28	6,110	3,975	1.54	5,828	1.05
	West	3,604		1.28		1.19	6,048		1.52		1.04
	North	3,698	3,047	1.21	3,261	1.13	4,689	3,659	1.28	5,524	0.85
	South	3,855		1.27		1.18	4,820		1.32		0.87
J5	East	7,599	5,462	1.39	5,816	1.31	10,382	7,739	1.34	9,785	1.06
	West	7,139		1.31		1.23	10,707		1.38		1.09
	North	7,630	5,891	1.30	6,244	1.22	8,970	6,985	1.28	9,042	0.99
	South	7,509		1.27		1.20	8,867		1.27		0.98
J6	East	7,000	4,562	1.53	4,747	1.47	10,136	6,915	1.47	9,081	1.12
	West	6,197		1.36		1.31	9,955		1.44		1.10
	North	6,877	5,017	1.37	5,202	1.32	8,715	6,309	1.38	8,488	1.03
	South	7,188		1.43		1.38	8,570		1.36		1.01

M_{test} : measured maximum beam moment at joint end

R_{calc} : calculated moment capacity as rectangular beam

T_{calc} : calculated moment capacity as T-beam (effective slab width = 60% of slab width)

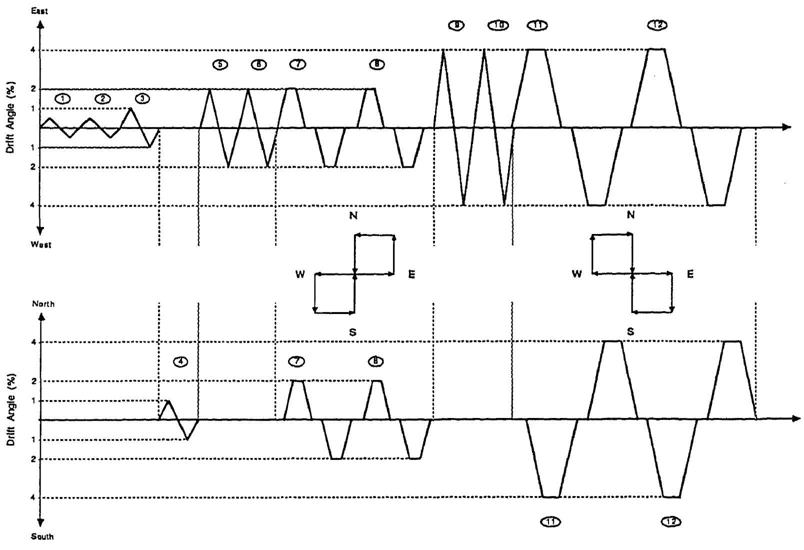

Figure 1 - Loading History

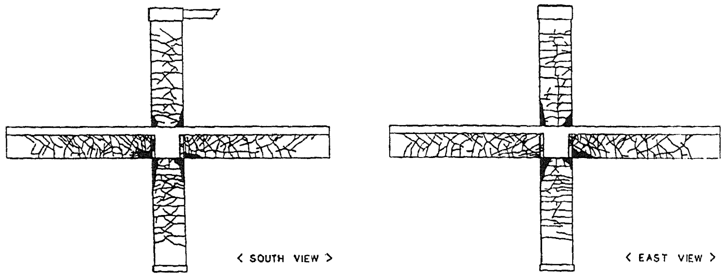

Figure 2 - Crack Patterns (Specimen J6)

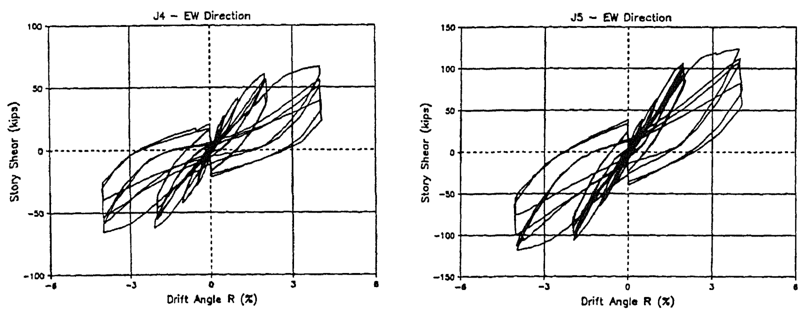

Figure 3 - Story Shear vs Drift Angle

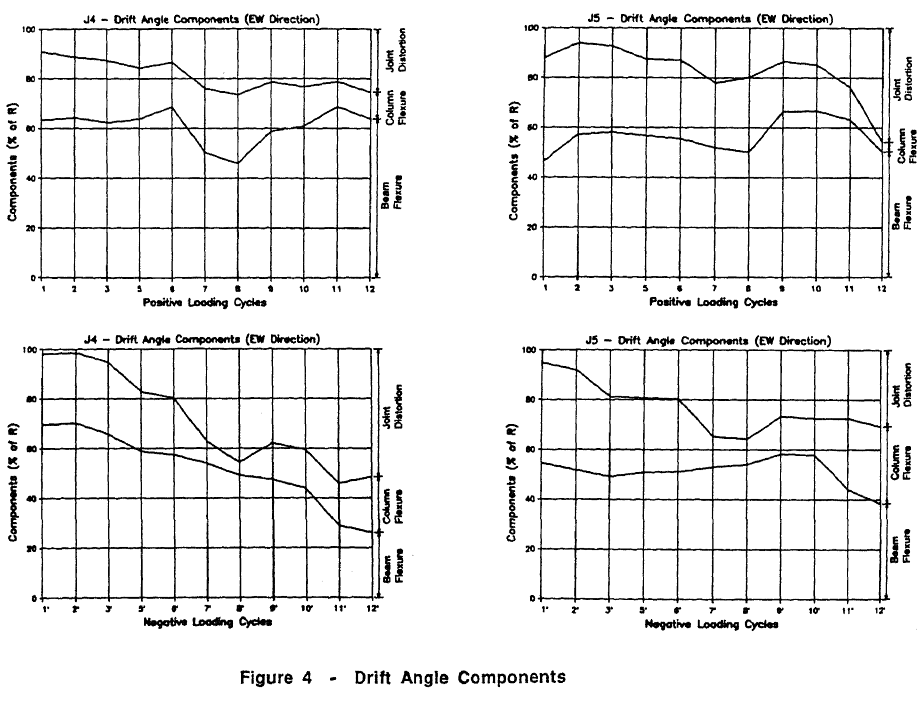

Figure 4 - Drift Angle Components