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BEHAVIOR OF THREE-DIMENSIONAL REINFORCED CONCRETE BEAM-COLUMN SUBASSEMBLAGES WITH SLABS

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SUMMARY

The experimental study is carried out to clarify the shear resistance behavior of interior beam-column joints in two-way reinforced concrete frames in comparison with the strength equations proposed in Japan and the requirments of ACI Code and New Zealand Standard. Four three-dimensional subassemblages with slab are used as specimens to which lateral cyclic forces are applied in two directions perpendicular to each other. On the basis of the results of experiment and calculation, ultimate strength of joint panels, effectiveness of the panel stiffness in story drift and energy absorption are discussed.

INTRODUCTION

Beam-column joints in space frames are usually subjected to two-way lateral cyclic forces during earthquakes as well as columns of the frames. However, the shear resistance behavior of the beam-column joints has not been defined in spite of their importance. In this paper, the shear resistance behavior is discussed by using the test results of four half-scale interior beam-column joint specimens with two-way beams and slabs. The variations in the specimens were made with the lateral loading direction, the lateral reinforcement in joint and the beam-column depth ratio. A one-way frame joint specimen without transverse beams and slabs of the previous test series is compared with the present specimens.

EXPERIMENTAL WORK

<u>Description of Specimens</u> The specimens were supposed as being a part of a space frame subjected to earthquake forces, and being cut off at six inflection points in four beams and two columns adjacent to an interior joint. The distances from the joint center to the beam-end supports and to the column end loading points were 1500 mm and 875 mm, respectively. The cross-sections of columns were 300 mm x 300 mm in all specimens. The cross-sections of beams were 150 mm x 350 mm in four specimens (X2-1, X2-2, X2-3, and X0-1), and 150 mm x 600 mm in one specimen (X2-4). The thickness of slabs was 60 mm. All the specimens were designed to develop weak-beam strong-column behavior on the assumption that the participating width of slabs in bending strength of the adjasent beams would be their entire width. The details of specimens are shown in Fig.1. The indexes of the properties of the specimens are summarized in Table 1.

The specimens 'X2-1' and 'X2-2' were provided to be wholly identical in detail with each other, of which the only difference was in loading directions, and were the same as ones provided slabs and transverse beams for the specimen 'X0-1'. The joint reinforcement in those three specimens was provided on referring to the joint strength equations proposed in Japan. However, it is insufficient to the requirements for the column-bars confinement in ACI 318-83 and also to the requirements for shear resistance in NZS 3101-1982. The specimen 'X2-3' was provided with a good amount of joint reinforcement of high strength steel according to the requirements of ACI Code or NZ Standard. The specimen 'X2-4' had an oblong joint resulted from deep beams:($h_b/h_c=2$). The arrangement of the joint reinforcement was similar to X2-1. The specimens 'X2-1' and 'X2-4' were loaded laterally at the top of column in two beam-directions perpendicular to each other by changing the direction alternately at every cycle of loading. The specimens 'X2-2' and 'X2-3' were loaded in the same way except that loading directions were ± 45 degree to the beam-directions.

Properties of Materials The mechanical properties of concrete and steel are shown in Table 2. Compressive strength of the concrete was 210 to 238 kgf/cm^2. The longitudinal bars in columns and beams were deformed bars of SD35 (fy = 35 kgf/mm^2). The transverse bars were round bars corresponding to SR30 (fy = 30 kgf/mm^2) except that the high strength steel of 136 kgf/mm^2 in equivalent yield strength for 0.2% residual strain was used in the joint of X2-3.

Instrumentation and Loading The bottom of the column was laterally supported on pin-rollers in two directions and restricted against the horizontal displacement in any direction, while the beam ends were supported on vertical rigid members equipped with universal joints at both ends so as to be free to slide in any horizontal direction. Reversing horizontal bi-directional loads were applied at the top of the column with two actuators installed perpendicular to each other. The controlling history for story displacement was programmed as shown in Fig. 2. The axial column loading was applied with a hydraulic jack through an independent set of yokes and held constant at 35kgf/cm^2 during the tests.

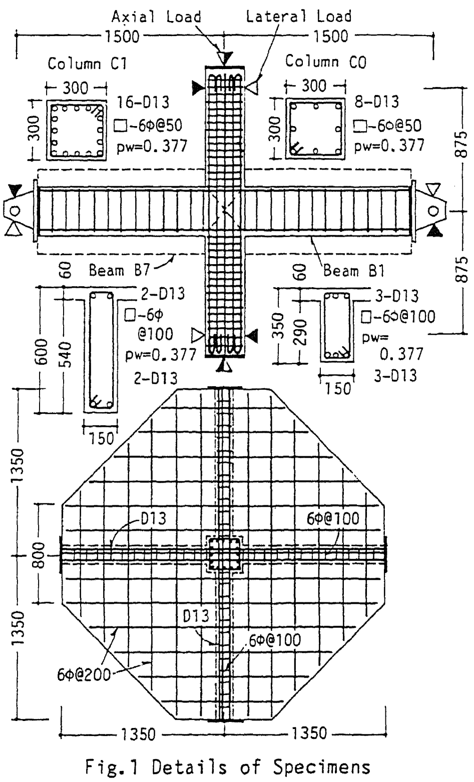


Fig.1 Details of Specimens

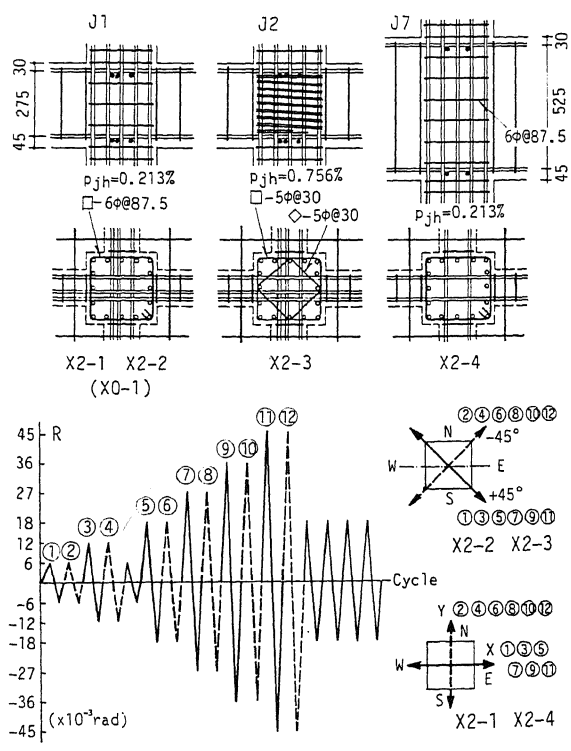


Fig.2 Forced Displacement History

Table 1. Properties of Specimens

Specimens		Types of Members			
Short Name	Full Name	Beam	Column	Joint	Slab
X2-1	JX2B1-XY	B1	C1	J1	exist.
X2-2	JX2B1-45	B1	C1	J1	exist.
X2-3	JX2B1-45PH	B1	C1	J2	exist.
X2-4	JX2B7-XY	B7	C1	J7	exist.
X0-1	JX0B1	B1	C0	J1	non

Column axial load stress is $35 Kgf/cm^2$ in all specimens.

Table 2. Properties of Materials

(a) Concrete

Specimen	Secant Modulus at $\sigma_{CB/3}$ ($10^5 kgf/cm^2$)	Compress. Strength $=\sigma_{CB}$ (kgf/cm^2)	Strain at σ_{CB} (%)
X2-1	1.81	229	0.25
X2-2	1.80	227	0.25
X2-3	1.46	210	0.29
X2-4	1.67	238	0.27
X0-1	2.05	217	0.20

(b) Reinforcement

Bar Size	Yield. Stress (kgf/mm^2)	Frac-ture (kgf/mm^2)	Elonga-tion (%)
D13	37.0	55.3	24.9
6φ 1)	33.9	42.8	23.0
6φ 2)	34.5	42.2	20.2
5φ	136.0	143.4	5.6

Note: 1) Shear reinforcement
2) Slab reinforcement

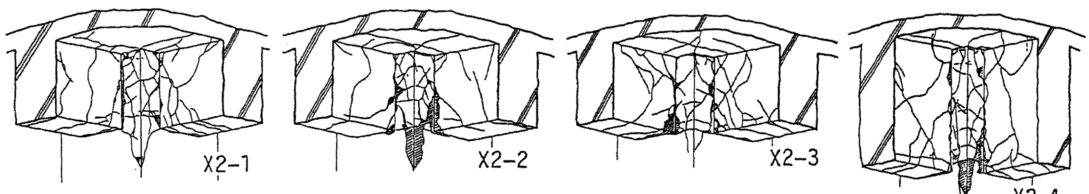


Fig.3 Crack Patterns of Joints after Tests

TEST RESULTS AND DISCUSSION

Crack Patterns and Modes of Failure Occurrence of shear cracks on the faces of the joints, shown in Fig. 3, could not be entirely observed due to the existence of the transverse beams. However, it could be ascertained with abrupt increase of strains in the joint reinforcement. The values of load and story drift angle at the first joint shear cracking obtained from the strain data are shown in Table 3. At the stage of failure, crushing of concrete in the yield hinge region of beams was accompanied by crushing in the compression zones of joints adjacent to the beam ends. Spalling of shell concrete at the corners of joints was also observed.

Strength Tables 3 and 4 show the summary of the test results and the comparison with the calculated values. The observed horizontal shear forces of the joints in the tests were estimated using the following equation:

$$V_{jh} = (M_{b1} + M_{b2})/ j_b - V_c \quad (1)$$

After the yielding of beams, the lateral reinforcement in the joints began yielding except the specimen 'X2-3'. The column shear forces and the joint shear forces at the ultimate strength of each specimen are shown in Table 4. The specimen 'X2-3' attained to the ultimate state without yielding of the lateral joint reinforcement. The test values of the ultimate joint shear force are compared with the values calculated using the empirical equations proposed by Kamimura (Eq. 2) and Koreishi (Eq. 3), and the values specified according to ACI 318-83 Appendix A and NZS 3101-1982 in Table 4.

$$v_{jh} = V_{jh} / b_j j_c = (0.78 - 0.0016 f'_c)\, f'_c + 0.5\, p_w f_y \quad [kgf/cm^2] \quad (2)$$

$$v_{jh} = V_{jh} / b_j j_c = (0.5 - 0.001 f'_c)\, f'_c + 2.7\sqrt{p_w f_y} \quad [kgf/cm^2] \quad (3)$$

where $b_j = (b_c + b_b)/2$, $j_c = 7d_c/8$

As for the specimens loaded in 45° directions, X2-2 and X2-3, the assumption that the normalized interaction curve for bi-directional shear strength is expressed with an arc of circle was adopted in the calculation using the equations (2), (3) and ACI. In the calculation according to NZS, it is permitted to consider that the joint shear strength would be fully effective separately in each of the two principal directions and then the resultant of the two-directional resistances is $\sqrt{2}$ times the uniaxial strength in these cases. However, the contribution of concrete to the shear resistance must be reduced because of dividing the effect of column compression according to the equation:

Table 3. Test Results and Calculated Values

specimen	load direction		panel initial shear cracking						yielding in beams						
			cycle	(1) $_tV_c$	(2) $_tR$	(3) $_tV_{jh}$	(4) $_cV_{jh}$	(5) (3)/(4)	cycle	(6) $_tV_c$	(7) $_tR$	(8) $_cV_{cy1}$	(9) $_cV_{cy2}$	(10) (6)/(8)	(11) (6)/(9)
X2-1	X	+	3	6.33	6.87	36.1	34.7	1.05	3	7.15	8.32	5.40	6.08	1.32	1.18
		−	3	6.00	6.10	34.2		0.99	3	6.49	7.49			1.20	1.07
	Y	+	2	5.64	4.89	31.8		0.92	2	5.64	4.89	5.33	6.02	1.06	0.93
		−	2	5.21	4.69	29.6		0.86	4	5.40	6.66			1.01	0.89
X2-2	+45	+	3	6.91	7.37	39.4	34.4	1.15	3	7.43	8.58	7.59	8.55	0.98	0.87
		−	3	7.65	9.21	43.6		1.27	3	7.65	9.17			1.01	0.89
X2-3	+45	+	3	5.74	5.88	32.7	33.5	0.98	3	8.06	9.25	7.59	8.55	1.06	0.94
		−	3	5.88	6.10	33.4		0.99	3	6.80	7.36			0.90	0.80
X2-4	X	+	3	9.45	14.87	24.1	35.0	0.69	1	9.95	4.65	6.68	7.05	1.49	1.41
		−	7	11.15	9.43	28.5		0.81	1	8.50	2.03			1.27	1.21
	Y	+	4	9.26	6.72	23.5		0.67	2	9.05	4.36	6.68	7.16	1.35	1.26
		−	4	9.73	8.69	24.6		0.72	2	9.05	4.36			1.35	1.26

$_cV$, $_cv$: calculated value
$_tV$, $_tv$, $_tR$: test value
V_c : shearforce of column [tonf]
V_{jh} : shearforce of joint [tonf]
v_{jh} : shear stress of joint [kgf/cm^2]
R : story drift angle [x10^{-3}rad]

$$_cV_{jh} = \frac{1}{b_j \cdot j_b \cdot j_c}(M_b + M_b' - V_c \cdot j_b)$$

where, $j = \frac{7}{8}d$, b_j = column width

$$_cV_{cy} = \{(M_{by} + M_{by}')/2(\ell_b - D_c/2)\} \cdot \ell_b/\ell_c$$

where, $M_{by} = 0.9 \cdot {_ba_t} \cdot {_b\sigma_y} \cdot d$

$M_{by}' = 0.9({_ba_t} \cdot {_b\sigma_y} + {_sa_t} \cdot {_s\sigma_y}) \cdot d$

$_cV_{cy1}$, and $_cV_{cy2}$: calculated from M_{by}' including slab bars within effective width of $0.1\ell_b$ and $0.2\ell_b$, respectively, where ℓ_b = span length of beam.

Table 4. Test Results at Ultimate Stage

Specimen				X2-1	X2-2	X2-3[5)]	X2-4	X0-1
Observed	X or +45	+	$_tV_c$	7.9	9.7	10.3	11.5	6.2
			$_tR$	25.9	17.4	27.5	26.9	32.3
			$_tV_{jh}$	36.7	44.7	47.4	24.7	29.4
		−	$_tV_c$	7.6	9.4	9.8	11.4	5.8
			$_tR$	19.1	27.5	36.3	29.5	32.3
			$_tV_{jh}$	36.3	43.2	45.5	24.6	27.0
Design[1)]	$_DV_{jh}$ [2)]			24.8	35.1	35.1	14.0	25.3
	($_DV_{jh}'$) [3)]			(32.6)	(46.0)	(46.0)	(20.5)	−
Calculated	Kamimura		V_{ch}	50.3	50.2	49.5	50.4	48.9
			V_{sh}	1.9	1.9	8.1	1.9	1.8
			V_{jh}	52.2	52.1	57.6	52.3	50.7
	Koreishi		V_{ch}	32.8	32.8	32.8	33.1	32.6
			V_{sh}	3.9	3.9	7.9	3.9	3.7
			V_{jh}	36.7	36.7	32.8	37.0	36.3
	ACI [4)]		V_{jh}	46.1	45.9	44.2	47.0	33.8
	NZS-3101		V_{ch}	6.6	0	0	6.3	6.9
			V_{sh}	5.7	8.1	34.8	11.5	5.3
			V_{jh}	12.3	8.1	34.8	17.8	12.2

V_{ch}, V_{sh} : Joint shear strength provided by concrete resisting mechanism and by horizontal joint reinforcement, respectively
1) Joint shear strength required beam yielding
2) estimated with slab effective width = $b_w + 0.2\ell_b$
3) estimated with entire slab width
4) Code of ACI 318-1983, $V_{jh} = V_{ch}$
5) calculated with f_y = 4.08 tonf/cm^2

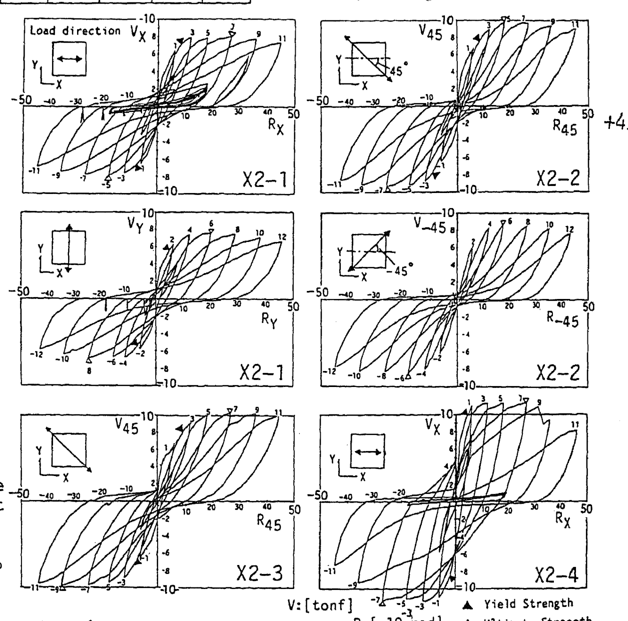


Fig. 4 Lateral Shear Force – Story Drift Angle Relation Curve

$$V_{ch} / b_j h_c = \frac{2}{3}\sqrt{C_j P_e / A_g - f_c' / 10} \qquad [\text{MPa}] \qquad (4)$$

where

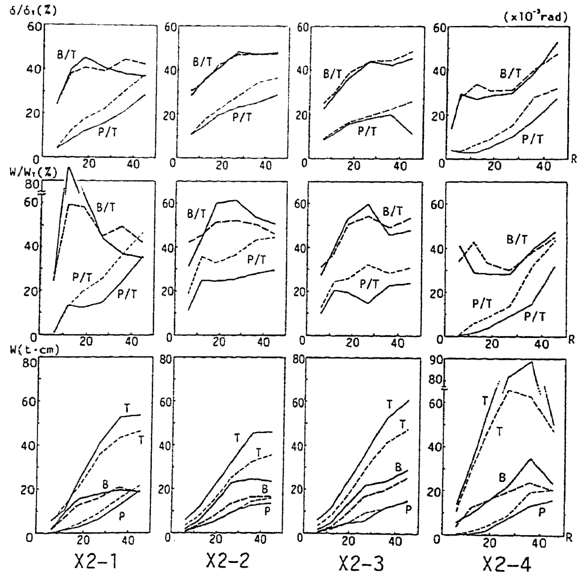
$$C_j = V_{jh} / (V_{jx} + V_{jy})$$

In the case of the present specimens $C_j = 1/2$ and $C_j P_e / A_g < f_c'/10$. The following behavior could be derived from the Table; (1) some enhancement of strength was brought in the space frames in comparison with the one-way frame; (2) the bi-directional shear strength of space frames resulted from the bending strengths of beams and slabs was higher than the principal directional shear strength though less than $\sqrt{2}$ times; (3) heavy reinforcement of high strength steel was not so effective against the expectation from the equations. It might be considered that the enhancement in X2-1 would be caused by the perticipation of the slabs in raising the yield strength of beams and in delaying the deterioration of bond on beam bars due to slippage from joints, and by the participation of the transverse beams in keeping the joint shear stiffness.

Relationship between Load and Deformation Lateral shear force - story drift angle relation curves are shown in Fig. 4, where the lateral shear force 'V' and the story drift angle 'R' express their components in X or Y direction in the specimens X2-1 and X2-4, or in +45°or -45°direction in X2-2 and X2-3. The strength decline after the ultimate strength was the severest in the specimen X2-4 with the oblong joint panel, and was the smallest in the specimen X2-3 with the high lateral joint reinforcement. The shapes of V-R loop at X-directional loading and Y-directional loading were similar to each other, but those at -45° directional loading were slenderer than those at +45°directional loading. The joint shear deformation of X2-3 was slightly small against the other specimens, but increased gradually with the progress of loading cycles without yielding of the joint reinforcement.

Components of Story Drift Caused by Local Deformations Fig. 5 shows the relations between the story drift angle 'R' and the components of the story drift 'δ' or energy absorption 'W'. The variables 'δ' and 'W' caused by the joint panel shear deformation are signified with 'P', and those cased by the beam end rotation are signified with 'B' in every specimen. The vertical axes of the figure express the proportion of component of 'δ' or 'W' to the whole value, or express the constititive values of 'W' in X or Y direction of X2-1 and X2-4 and in +45°or -45°direction of X2-2 and X2-3. The horizontal axes express the story drift angle in each direction. The beam rotation was evaluated by using couples of relative displacements between the column face and the measuring points set on the top and bottom faces of beam ends.

The proportions of story drifts caused by the beam end rotations to the whole story drifts in each specimen were almost same between the X and Y or ° and -45° directions. The proportions of story drifts caused by the joint panel shear deformations in direction Y or -45°were larger than those in direction X or +45°, respectively, except the specimen X2-3.



T= the Whole Values, P= Constitutive Values caused by Joint Panel Shear Deformation, B= Constitutive Values caused by Beam End Rotation, R= Story Drift Angle,
—— at Loading in X or +45° Direction
--- at Loading in Y or -45° Direction

Fig.5 Constitutive Values of Story Drift "δ" and Energy Absorption "W"

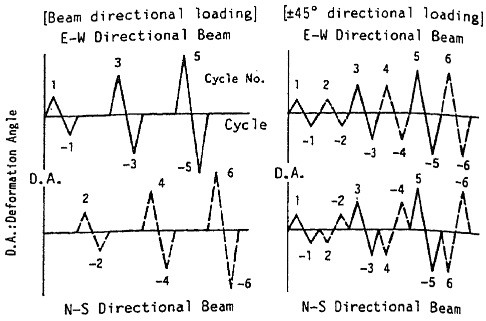


Fig. 6 Loading History of Each Beam

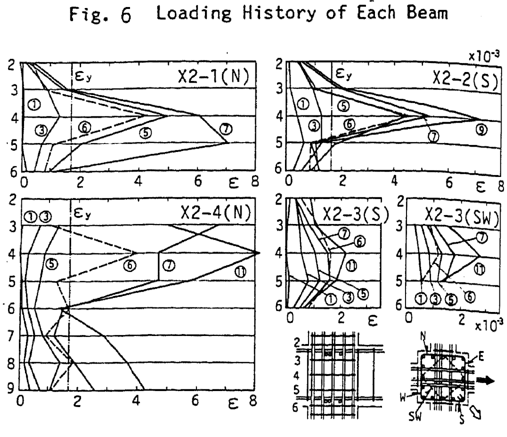


Fig.7 Strain Distribution of Joint Hoops

Energy absorption, W_T, W_P and W_B, were obtained in each cycle of load reversals from loop areas in the relation curves of the shear force and the whole story drift or the component. The whole energy absorption 'W_T' increased after yielding of beam bars in every specimens. W_T at the Y-directional loading of X2-1 or X2-4 became less than W_T at X-directional loading after ultimate strength, that above 15 $\times 10^{-3}$rad. in story drift angle. The inferiority of W_T at -45° directinal loading to W_T at +45°directional loading of X2-2 or X2-3 appeared even in a little range of story drift, because in the north-south beam, duplication of half cyclic loadings in the same sign range were repeated alternately in north and south directions as shown in Fig. 5. As compared with X2-1, W_T of X2-2 was slightly small in any story drift level due to bi-directional loading, but the capacity for energy absorption could be improved by arrangement of high lateral joint reinforcement especially at ultimate stage. The energy absorption in joint panel 'W_P' were almost same in any specimens and in any directional loading, except that W_P in Y-directional loading were larger than W_P in X-directional loading.

<u>Characteristics of lateral joint reinforcement</u> The hoops in the joints of X2-1, X2-2 and X2-4, provided with ordinary steel bars and with low reinforcement bar ratio of 0.213%, yielded at the strain of 0.18% after the beam bar yielding and before the ultimate strength of space frames. The hoops and diamond ties in the joint of X2-3, provided with high strength steel bars and with high reinforcement bar ratio of 0.756%, reached about 0.25% in strain in maximum without yielding. The strains of every hoop in beam-directional loading specimens exhibited different values between the legs arranged in parallel with and in perpendicular to the loading direction, because the parallel legs mainly resisted the shear force in the joint and the perpendicular legs worked as confinement against the compression force. The strain value of the former was from 1.5 to 2 times larger than that of the latter.

REMARKS

(1) Shear cracking stress in the beam-column joint could be assessed with the principal stress concept by assuming the normalized interaction curve of an arc of circle for the bi-directional shear, but some modification should be necessary for the shear cracking stress decline in oblong joints.

(2) The effective width of slab to participate in the bending resistance of beam shall be taken as about one-fifth of span length at the yielding of beams. For the estimation of design shear forces in the joint, the effective width should be taken as the entire width of slab. The effective width of slab subjected to bi-directional loading is smaller than that above mentiond.

(3) The enhancement of joint shear strength by the transverse beams which have yield hinges at the column faces should not be expected for cracking strength.

(4) Calculated values of shear strength of joint panels using the previous equations or according to the requirements of the codes did not show good agreement with the observed values. Further investigation should be necessary.

(5) The yielding of beams prior to injury in joints by shear should not always assure the ductile behavior of frames. The ultimate shear strength or shear stiffness of joint panel after yielding of adjacent beams or columns should be dependent to the intended limit of story drift of frames.

(6) The slippage of top beam bars pulling out from the joint panel was smaller than that of bottom beam bars due to restraint of the slab. In the frames subjected to bi-directional loading, the slippage of beam bars was a little greater comparing with that in the frames subjected to beam directional loading.

(7) The capacity of energy absorption in space frames did not show remarkable difference for any lateral loading directions. However, the amounts of energy absorption were varied with different loading histories.

Reference : Kamimura, T., Strength of Beam-Column Joints in Reinforced Concrete Structures, A.I.J. Annual meeting (in Japanese), (1976)