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EARTHQUAKE RESISTANT DESIGN CRITERIA FOR REINFORCED CONCRETE INTERIOR BEAM-COLUMN JOINTS

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ABSTRACT

The tests of reinforced concrete (R/C) interior beam-column joints in a plane frame were carried out with high joint shear and poor bond along beam reinforcement, varying the lateral reinforcement detail in the joint. The levels of joint shear stress input and allowable bond deterioration along beam bars within a joint are examined. Additional test data including plane joint specimens with high strength concrete were studied. The lateral reinforcement is claimed to confine the core concrete in the joint rather than to resist joint shear.

1. INTRODUCTION

The shear transfer mechanism of a reinforced concrete beam-column joint changes with the bond deterioration along the beam reinforcement in the joint of a frame structure, especially after beam flexural yielding. When the beam bar stresses can be transfered to the joint concrete by bond, diagonal compressive stresses distribute uniformly within the joint panel (the "truss" mechanism). In this case, the joint lateral reinforcement carries tensile stresses and resists joint shear. However, the truss mechanism diminishes with bond deterioration along the beam reinforcement, and the principal role of the lateral reinforcement becomes to confine the cracked joint core concrete.

The effect of bond deterioration along the beam reinforcement on earthquake response was studied by nonlinear analyses of frame structures (Ref.1). The bond deterioration increased the maximum response amplitudes, but by small amount, although the number of large-amplitude oscillations increased. Therefore, some bond deterioration might be permitted. With the bond deterioration, the compressive strut stresses increases in the main diagonal of the joint panel to cause shear compression failure. Therefore, it was proposed to limit the joint shear stress by Eq.(1) in order to prevent the shear compression failure to a story drift angle greater than 1/25 rad after beam flexural yielding (Ref.1);

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$$v_u \ / \ f_c' \leq 0.25 \tag{1}$$

in which v_u is the maximum joint shear stress, and f_c' is the concrete compressive strength. The effective joint area to resist shear is defined by the column depth and the average of the beam and column widths.

The value of 0.25 in Eq.(1) may require re-examination by the following reasons;

(1) Shear failure of a beam-column joint was scarcely observed in many laboratory tests up to a story drift angle of 1/50 rad, which is an arbitrarily defined allowable drift limit of reinforced concrete frames.

(2) Joint shear resistance did not decay abruptly in beam-column specimens although the specimens appeared to fail in joint shear after beam flexural yielding.

This paper discusses the acceptable level of joint shear input and the required amount of joint lateral reinforcement to confine the core concrete on the basis of the test results of the plane beam-column joint specimens and the recent test results of plane joint specimens.

2. LIMITATION OF BEAM BAR BOND INDEX

The average bond stress u_b over the column width for simultaneous yielding of the beam reinforcement in tension and compression at the two faces of the joint divided by the square root of the concrete compressive strength, called beam bar bond index, is used to indicate the possibility of bond degradation along the beam reinforcement;

$$u_b \ / \sqrt{f_c'} = f_y \ (\ d_b \ / \ h_c \ ) \ / \ 2\sqrt{f_c'} \tag{2}$$

where f_y: yield strength of beam bars in kgf/cm^2, d_b: diameter of beam bars, h_c: column width and f_c': concrete compressive strength in kgf/cm^2.

From the results of earthquake response analyses (Ref.1), the effect of hysteresis energy dissipating capacity on the response was found relatively small for a range of equivalent viscous damping ratio h_{eq} from 0.10 to 0.25 defined at ductility factor of 4.0, where h_{eq} : ratio of the dissipated energy within half a cycle to 2π times the strain energy at the peak of an equivalent linearly elastic system. Therefore, some bond deterioration of beam bars within a joint may be tolerated.

The beam bar bond index $u_b/\sqrt{f_c'}$ and the equivalent viscous damping ratio h_{eq} at a story drift angle of 1/50 rad are compared in Fig.1 for the plane beam-column joints tested previously in Japan. These specimens developed beam

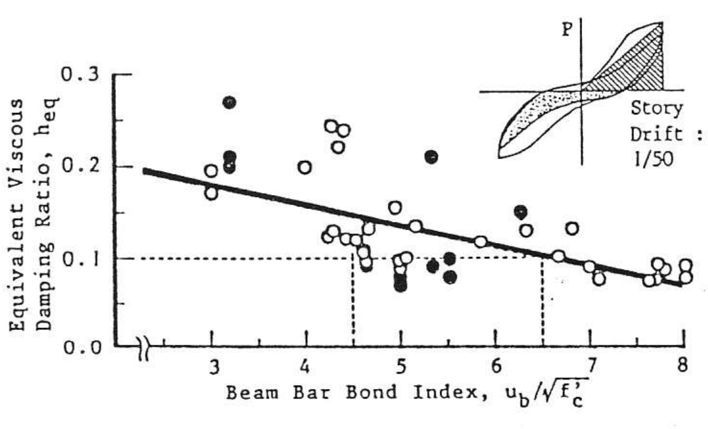


Fig. 1 Equivalent Viscous Damping Ratio-Beam Bar Bond Index Relation

flexural yielding and the damage by shear did not concentrate in a joint core concrete up to a story drift angle of 1/50 rad. The axial stress of 0.07 f_c' to 0.32 f_c' for a column gross section had little influence on the fatness of a hysteresis shape. The solid line was derived from the least squares method to fit the data. Concrete compressive strength f_c' was greater than 270 kgf/cm^2 for specimens in open symbols. The h_{eq} values decrease with an increasing beam bar bond index. If an allowable deformation level is taken to be a story drift angle of 1/50 rad, the beam bar bond index must satisfy Eq.(3) to ensure the equivalent viscous damping ratio of 0.10, as indicated in the earthquake response analyses.

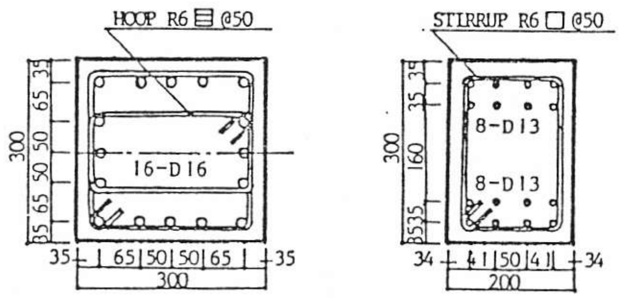
$$u_b /\sqrt{f_c'} \leqq 4.5 \tag{3}$$

Substituting $u_b/\ f_c'$ in Eq.(2) into Eq.(3), the required ratio of the column width to beam bar diameter is obtained as follows;

$$h_c / d_b \geqq f_y / (9\sqrt{f_c'}) \tag{4}$$

3. TEST PROGRAMME

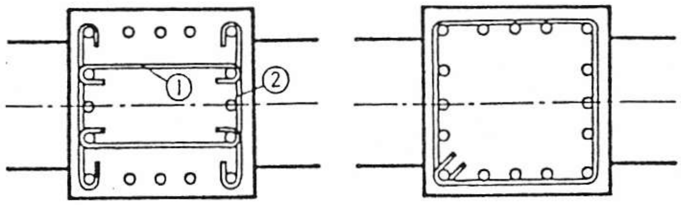
Two half-scale R/C interior beam-column joints (Specimens B1 and B2), removed from a plane frame by cutting off the beams and columns at arbitrarily assumed inflection points under lateral loading, were tested without transverse beams. The specimens were designed to develop beam flexural yielding prior to column yielding. Dimensions of specimens were common; 200x300 mm in a beam and 300x300 mm in a column as shown in Fig.2. The beam reinforcement, equal in amount at top and bottom, passed through the joint. The beam bar bond index was 5.1 using the actual yield strength of the beam bar. Therefore, the bond along beam bars was expected to deteriorate within the joint. Plain bars were used as lateral reinforcement within the joint to eliminate the stress transfer by bond action. Joint lateral reinforcment ratio was 0.35 %, which is defined as the total cross-sectional area of the lateral reinforcement between the beam top and bottom bars divided by the column width and the distance of (7/8)d, d: beam effective depth.



(1) Column (2) Beam

Fig. 2 Member Sections (unit : mm)

Detail of the joint lateral reinforcement was varied as shown in Fig.3. Legged ties were used in Specimen B1 to identify the strains associated with shear resistance and those associated with confinement of joint core concrete. Usual closed hoops were placed within the joint of Specimen B2. Ties parallel to the loading direction, indicated by circle 1 in Fig.3, can resist joint shear in the truss mechanism, whereas ties indicated by circle 2 restrain the expansion of the core concrete normal to the loading direction. The action of confinement by a closed joint hoop is illustrated in Fig.4. Radial pressure in the joint core concrete pushes out the corner column reinforcing bars. The diagonal force is balanced with tensile forces in the joint hoop supporting the corner bars. The tie



(1) Specimen B1 (2) Specimen B2

Fig. 3 Detail in Joint Reinforcement

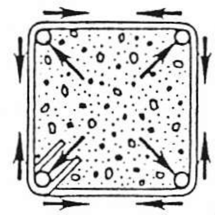


Fig. 4 Confinement Action by Closed Hoop

(circle 1) supporting an intermediate column bar is not affected by this confining action because the tie of circle 2 perpendicular to that of circle 1 is hooked at a different column bar.

Compressive strength of concrete was 250 kgf/cm^2, and splitting tensile strength was 26 kgf/cm^2. Yield strength of reinforcement was 3580 kgf/cm^2 for D16 bars used in columns, 3780 kgf/cm^2 for D13 bars used in beams, 4980 kgf/cm^2 for R6 bars (plain bar) for beam and column shear reinforcement and 2400 kgf/cm^2 for R6 bars for joint lateral reinforcement.

Loading apparatus is shown in Fig.5. The constant column axial stress of 20 kgf/cm^2 was applied. The distance from the column center to the beam-end support was 1,350 mm, and the distance from the beam center to the bottom support or to the top horizontal loading point was 735 mm. Specimens were loaded as follows; one cycle each at a story drift angle of 1/400 rad, 1/200 rad, two cycles at a story drift angle of 1/100 rad, four cycles at a story drift angle of 1/75 rad, two cycles at a story drift angle of 1/50 rad, and one cycle at a story drift angle of 1/25 rad.

4. TEST RESULTS

The beam-column joints of the two specimens did not fail in shear up to the story drift angle of 1/50 rad. However, the contribution of the joint shear deformation became approximately 50 % of the total story drift

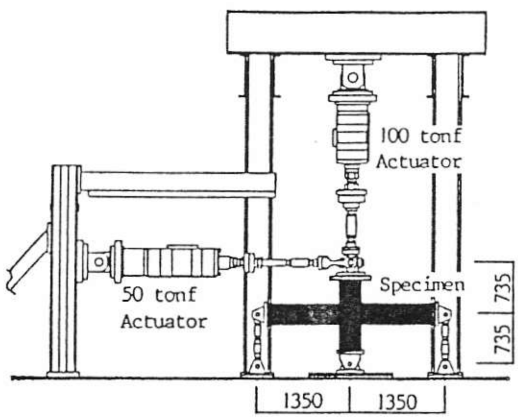
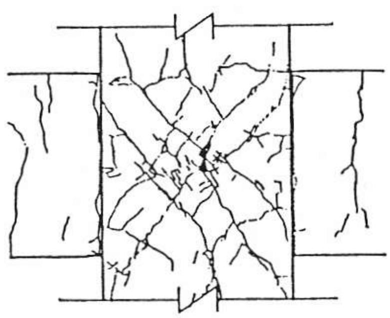


Fig. 5 Loading Apparatus



Specimen B2

Fig. 6 Crack Pattern (after 1/50 rad)

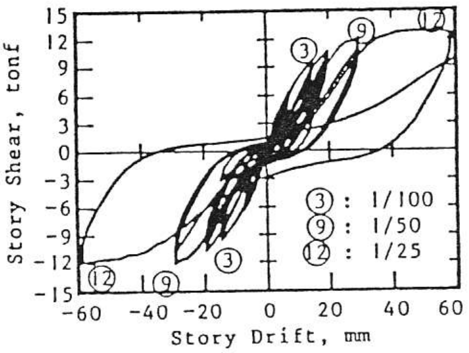


Fig. 7 Story Shear-Drift Relation

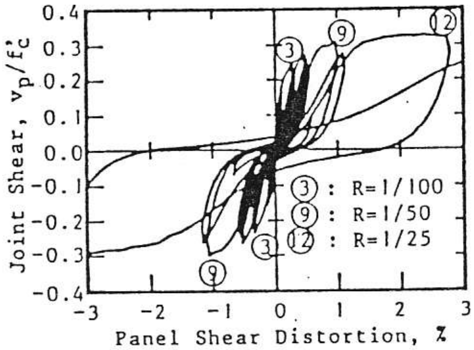


Fig. 8 Joint Shear-Distortion Angle Relation

at a story drift angle of 1/25 rad, and the damage was observed to concentrate in the joint panel region due to high shear. The beam longitudinal bars in the outer-most layer yielded at a story drift angle of 1/75 rad, and those in a intermediate layer at a story drift angle of 1/50 rad. The beam flexural resistance did not reach the ultimate capacity calculated assuming that plane section remains plane. The column reinforcement did not yield up to a story drift angle of 1/50 rad. Overall behavior was similar in Specimens B1 and B2 except for the strain distribution of the joint lateral reinforcement.

Crack pattern in the joint of Specimen B2 observed at a story drift angle of 1/50 rad is shown in Fig.6. Fine diagonal cracks developed by severe compressive stresses near the joint center. Diagonal shear cracks occurred sparsely due to poor bond transfer from the beam reinforcement to the joint core concrete.

Story shear-drift relation of Specimen B2 is shown in Fig.7. A pinched hysteresis shape was caused by both the shear distress of the joint core concrete and the bond deterioration along the beam bars. The resistance of beams did not degrade even at a story drift angle of 1/25 rad. Equivalent viscous damping ratio (the index indicating fatness of hysteresis loops) was 0.07 in Specimen B1 and 0.08 in Specimen B2 in the second cycle at a story drift angle of 1/50 rad, smaller than that of Specimen J1 (Ref.2) with bond deterioration along the beam reinforcement.

Joint shear stress normalized by concrete compressive strength-distortion angle relation of Specimen B2 is shown in Fig.8. Shear distortion angle was calculated using the panel diagonal elongation over a gauge length of 276 mm, with the inclination of 46.5 degrees to the horizontal axis. Joint shear stress of 0.31 f_c', shear distortion angle of 1 %, and average diagonal strain of the panel concrete of 0.25 % were obtained at a story drift angle of 1/50 rad. Principal compressive strain of the panel concrete reached the strain at a

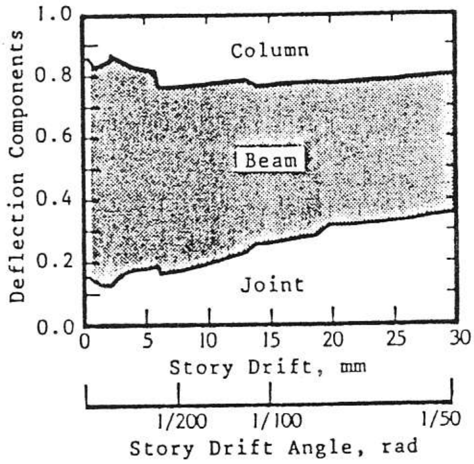


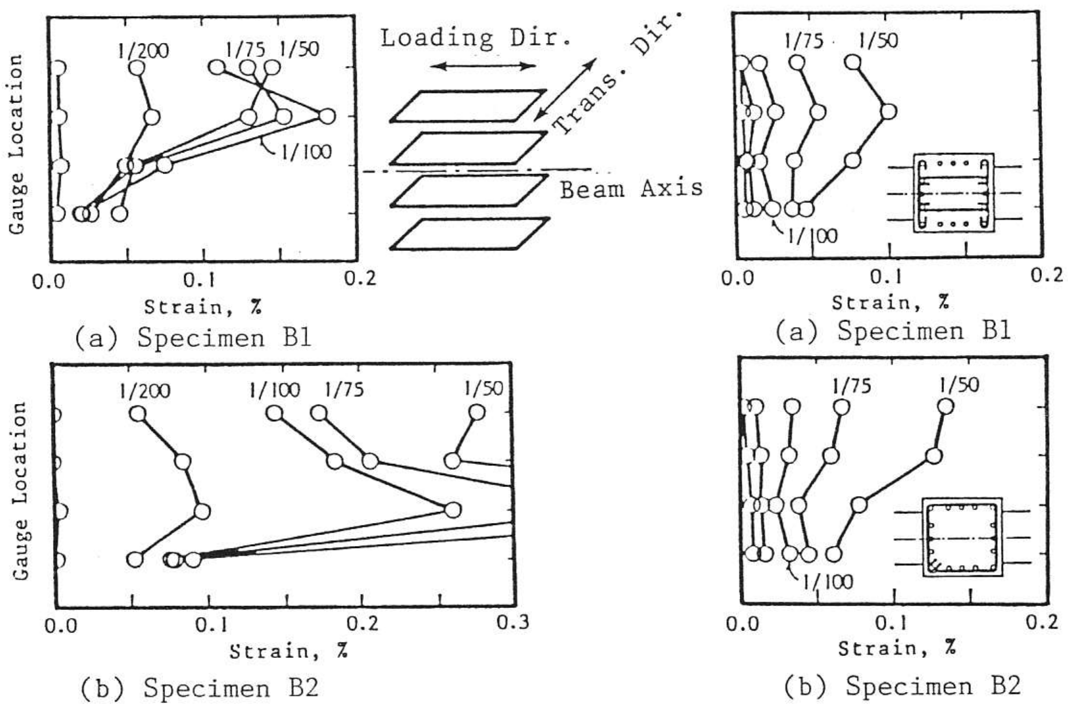
Fig. 9 Deflection Components of Story Drift

peak stress in the stress-strain curve of concrete. Shear distortion angle at a story drift angle of 1/25 rad was greater than twice that at a story drift angle of 1/50 rad.

The contribution of various parts of Specimen B2 to a story drift was calculated and shown in Fig.9. Ratio of joint shear deformation to a story drift increased with the deflection of beams, and reached 40 %, comparable to that of the beam deflections.

The strains in lateral reinforcement within the joint of Specimens B1 and B2 are shown in Fig.10. Yield strain was defined as 0.2 % arbitrarily. Strains in the loading direction of Specimen B1 became almost constant at story drift angles greater than 1/100 rad, and did not reach the yield strain. Hence the contribution of the sub-strut mechanism to the joint shear resistance decreased with bond deterioration along the beam reinforcement, increasing the part carried by the diagonal concrete strut. On the other hand, strains in the loading direction of Specimen B2 increased with a story drift. Note that the influence of confinement of the joint core concrete was eliminated by using legged ties as the lateral reinforcement parallel to loading direction.

Strains orthogonal to the loading direction in Specimens B1 and B2 increased with the story drift, but the ties perpendicular to the loading direction did not yield up to the story drift angle of 1/50 rad. Therefore, the amount of the lateral reinforcement provided in Specimens B1 and B2, i.e., 0.35 % is sufficient to confine the joint core concrete.



(1) Loading Direction (2) Orthogonal Direction

Fig. 10 Strains in Joint Lateral Reinforcement

5. DISCUSSION OF TEST RESULTS

The performance of Specimens B1 and B2 was judged satisfactory because the joint panel did not fail in shear up to a story drift angle of 1/50 rad. The influence of joint shear distortion as large as approximately 1 % should be studied on earthquake responses in frame structures.

The joint shear stress of 0.3 f_c' in Specimens B1 and B2 is not permitted as reasons below; (1) the story drift at the beam yielding increases due to the large shear deformation in the joint panel, (2) energy dissipation ability decreases in the beam hinging regions under earthquake excitations, (3) the soft story mechanism may develop, and (4) repair of a damaged joint is difficult. Joint lateral reinforcement of 0.4 % was concluded sufficient to confine the core concrete.

The limitation in shear stress of an actual joint with transverse beams and slabs may be different from that of a plane joint as mentioned above. Shear stress in a joint during the uni-directional loading is normalized by the concrete compressive strength f_c' and is shown in Fig. 11 for Specimen B2 and three-dimensional joint specimens K1 and K3 (Ref.3) subjected to bi-directional loading. Effective slab width on beam flexural resistance increased with a story drift, and the shear stress reached as high as 0.38 f_c' and 0.35 f_c' in Specimens K1 and K3 at a story drift angle of 1/25 rad without failing in the joint by shear. The ratio of joint shear deformation to a story drift was 20 % in the three-dimensional specimens. Therefore, the limitation in shear stress might be lightened in a three-dimensional joint than that in a plane joint.

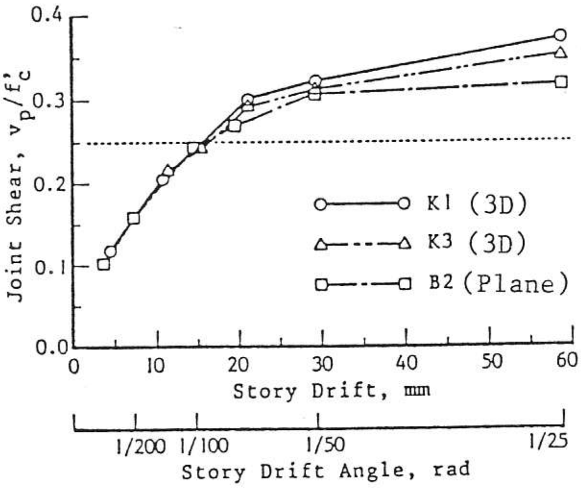


Fig. 11 Shear Stress into Joint

6. CONCLUDING REMARKS

Limitation in joint shear stress and required amount of lateral reinforcement in a reinforced concrete beam-column joint were studied through the test. Allowable bond deterioration along the beam reinforcement within a joint was examined. Design provisions were suggested as follows;

$$v_u \;/\; f_c' \leq 0.25 \tag{5}$$

$$h_c \;/\; d_b \geq f_y \;/\; (9\sqrt{f_c'}) \tag{6}$$

A minimum joint lateral reinforcement ratio of 0.4 % is recommended. Lateral reinforcement with close spacing and small cross-sectional area should be placed to confine the joint core concrete. Permissible shear stress in a joint with slabs and transverse beams may be increased from that in a plane joint.

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