# Behavior of high-strength R/C beam-column joints

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ABSTRACT: Reinforced concrete interior beam-column subassemblages with high-strength concrete and high-strength steel (a three-dimensional joint without slabs and three plane joints) were tested to study the bond transfer along beam reinforcement and the shear resistance of a joint subjected to uni-directional and bi-directional reversed loads. Bond index proposed in the paper is shown to evaluate appropriately bond resistance of beam reinforcement within a joint, even for high-strength concrete. Bond transfer along beam reinforcement within a joint was further deteriorated under bi-directional loading. Uni-directional shear resistance in a joint was enhanced by confinement action from even precracked transverse beams. Moreover, joint shear resistance under bi-directional loading reached as high as 1.3 times the joint shear strength of a plane frame.

## 1 INTRODUCTION

High-strength concrete and high-strength steel of a large diameter are used in high-rise reinforced concrete (R/C) structures. However, reduced column dimensions result in lower beam-column joint resistance. Therefore, the performance of interior beam-column subassemblages constructed with high-strength materials was studied through test; i.e., shear resistance of a joint under uni-directional or bi-directional loading and bond deterioration along the beam longitudinal reinforcement.

## 2 TEST PROGRAM

### 2.1 *Specimens*

Five half-scale R/C interior beam-column joints (called I-series) were tested; a three-dimensional joint without slabs constructed with high-strength concrete and steel (Specimen I2), three plane joints with high-strength materials (Specimens I1, I5 and I6) and a plane joint with ordinary-strength materials (Specimen I3). The properties of specimens are listed in Table 1. Member sections are shown in Fig. 1. The dimensions of beams (200 x 300 mm) and columns (300 x 300 mm) were common in five specimens. The distance from the column center to the beam-end support was 1350 mm, and the distance from bottom support to the top horizontal loading point was 1470 mm. The beams in Specimens I1, I2, I5 and I6 were reinforced by bars of a yield strength from 770 to 800 MPa, and the concrete of compressive strength from 85 to 99 MPa. The specimens were cast in upright position.

The beam and column reinforcement details were common in Specimens I1 and I2 to compare the shear resistance of a joint subjected to uni-directional and bi-directional load reversals.

The beam bar diameter was varied in Specimens I5

Table 1 Properties of specimens

| Specimen | I1 | I2 | I3 | I5 | I6 |
|---|---|---|---|---|---|
| (a) Longitudinal Beam | | | | | |
| Top Bars | 8-D16 | 8-D16 | 6-D16 | 6-D13 | 3-D19 |
| $a_t$(cm$^2$) | 15.92 | 15.92 | 11.94 | 7.62 | 8.61 |
| $p_t$(%) | 3.34 | 3.34 | 2.44 | 1.54 | 1.66 |
| Bottom Bars | 8-D16 | 8-D16 | 4-D16 | 3-D13 | 2-D19 |
| $a_t$(cm$^2$) | 15.92 | 15.92 | 7.96 | 3.81 | 5.74 |
| $p_t$(%) | 3.34 | 3.65 | 1.53 | 0.73 | 1.10 |
| Stirrups | 2-U6.4 | 2-U6.4 | 2-D6 | 2-D6 | 2-D6 |
| @(cm),$p_w$(%) | 3.5,0.86 | 3.5,0.86 | 4.0,0.80 | 4.0,0.80 | 4.0,0.80 |
| (b) Transverse Beam | | | | | |
| top Bars | (none) | 8-D16 | (none) | (none) | (none) |
| $a_t$(cm$^2$) | | 15.92 | | | |
| $p_t$(%) | | 3.65 | | | |
| Bottom Bars | | 8-D16 | | | |
| $a_t$(cm$^2$) | | 15.92 | | | |
| $p_t$(%) | | 3.34 | | | |
| Stirrups | | 2-U6.4 | | | |
| @(cm),$p_w$(%) | | 3.5,0.86 | | | |
| (c) Column | | | | | |
| Total Bars | 16-D19 | 16-D19 | 16-D16 | 16-D16 | 16-D16 |
| $a_g$(cm$^2$) | 45.92 | 45.92 | 31.84 | 31.84 | 31.84 |
| $p_g$(%) | 5.10 | 5.10 | 3.54 | 3.54 | 3.54 |
| Hoops | 2-U6.4 | 2-U6.4 | 4-D6 | 4-D6 | 4-D6 |
| @(cm),$p_w$(%) | 4.0,0.50 | 4.0,0.50 | 5.0,0.85 | 5.0,0.85 | 5.0,0.85 |
| Load(tonf) | 32.4 | 32.4 | 9.7 | 18.0 | 18.0 |
| (d) Joint | | | | | |
| Hoops | 3-R6 | 4-R6 | 3-R6 | 4-R5.5 | 4-R5.5 |
| sets | 3 | 2 | 3 | 3 | 3 |
| $a_w$(cm$^2$) | 2.54 | 2.26 | 2.54 | 2.85 | 2.85 |
| $p_{jh}$(%) | 0.41 | 0.39 | 0.37 | 0.42 | 0.42 |

Note D : deformed bar, U : super-high-strength deformed bar, and R : plain bar

$a_t$ : total area of tensile reinforcement,
$p_t$ : tensile reinforcement ratio,
$a_g$ : total area of longitudinal reinforcement,
$p_g$ : gross reinforcement ratio,
$a_w$ : total area of lateral reinforcement placed between top and bottom beam reinforcement in joint, and
$p_{jh}$ : lateral reinforcement ratio in joint.

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Table 2 Material properties

(a) Concrete ( unit in MPa )

| Specimen | Compressive Strength | Tensile Strength | Secant Modulus *1 |
|---|---|---|---|
| I1, I2 | 98.8 | 4.2 | $3.9 \times 10^4$ |
| I3 | 41.4 | 3.1 | $3.3 \times 10^4$ |
| I5, I6 | 85.4 | 4.3 | $4.1 \times 10^4$ |

*1 Secant modulus at one-quarter of the compressive strength

(b) Steel ( unit in MPa )

| Size (component in specimen) | Yield Strength | Tensile Strength |
|---|---|---|
| D19 (I1, I2 column longitu. bar) | 746.5 | 806.4 |
| D19 (I6 beam longitudinal bar) | 772.0 | 834.8 |
| D16 (I1, I2 beam longitu. bar) | 798.5 | 860.3 |
| D16 (I3 beam and column longitu.) | 361.0 | 535.6 |
| D16 (I5, I6 column longitu. bar) | 533.7 | 684.7 |
| D13 (I5 beam longitudinal bar) | 769.1 | 819.1 |
| U6.4(I1, I2 shear reinforce. bar) | 1308.7 | 1398.9 |
| D6 (I3 shear reinforcement bar) | 358.1 | 482.7 |
| D6 (I5, I6 shear reinforce. bar) | 395.3 | 517.0 |
| R6 (I1, I2, I3 joint reinf. bar) | 360.0 | 431.6 |
| R5.5(I5, I6 joint reinforce. bar) | 250.2 | 377.7 |

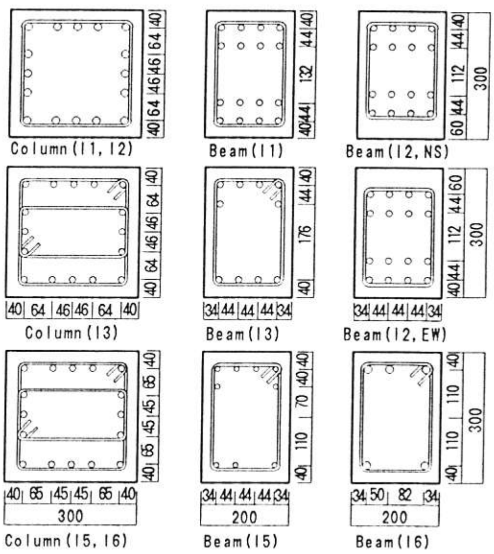
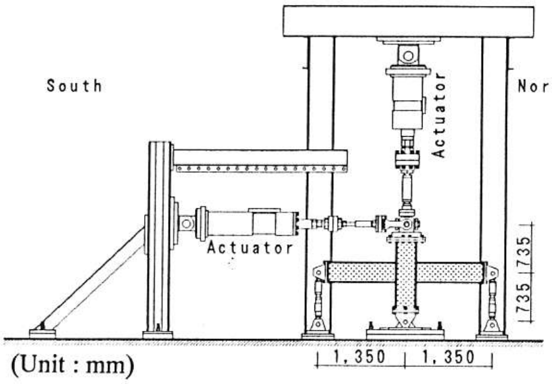



Fig. 1 Member sections

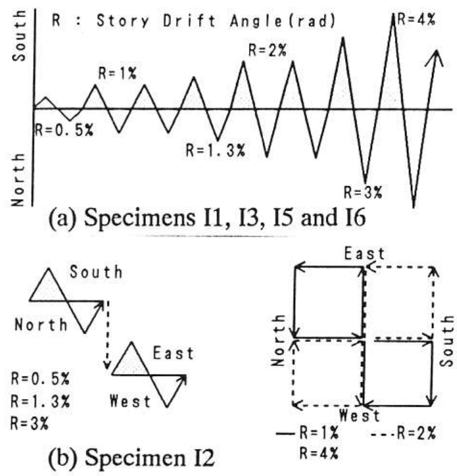
and I6 to investigate a bond strength along the beam bar within the joint. The input joint shear stress was restricted to 0.15 times the concrete compressive strength; the effective joint area to resist shear was defined by the column depth and the average of the beam and column widths.

The joint lateral reinforcement of approximately 0.4 % was placed between the beam top and bottom bars in




(Unit : mm)

Fig. 2 Loading apparatus




(a) Specimens I1, I3, I5 and I6

(b) Specimen I2

Fig. 3 Loading histories and paths

all specimens. The columns in all specimens were designed to remain elastic.

Material properties are listed in Table 2. Silica-fume was added and the water to cement ratio of 28 % was chosen to attain higher strength of the concrete.

## 2.2 *Loading method*

Loading apparatus is shown in Fig. 2. Bi-directional loads to a three-dimensional Specimen I2 and uni-directional load to plane Specimens I1, I3, I5 and I6 were applied with constant column axial load. Loading histories and loading paths at the top of a column under bi-directional load reversals are shown in Fig. 3. Story shear was defined as the horizontal force corrected for the P-Delta effect.

# 3 TEST RESULTS

## 3.1 *General observations*

Crack patterns after test are shown in Fig. 4 and story shear – story drift relations in Fig. 5.

Beam bars in an outer layer in plane Specimen I1 yielded at a story drift angle of 3 %. A joint was damaged severely with concrete spalling at a story drift angle of 4 %. The joint shear resistance was reduced

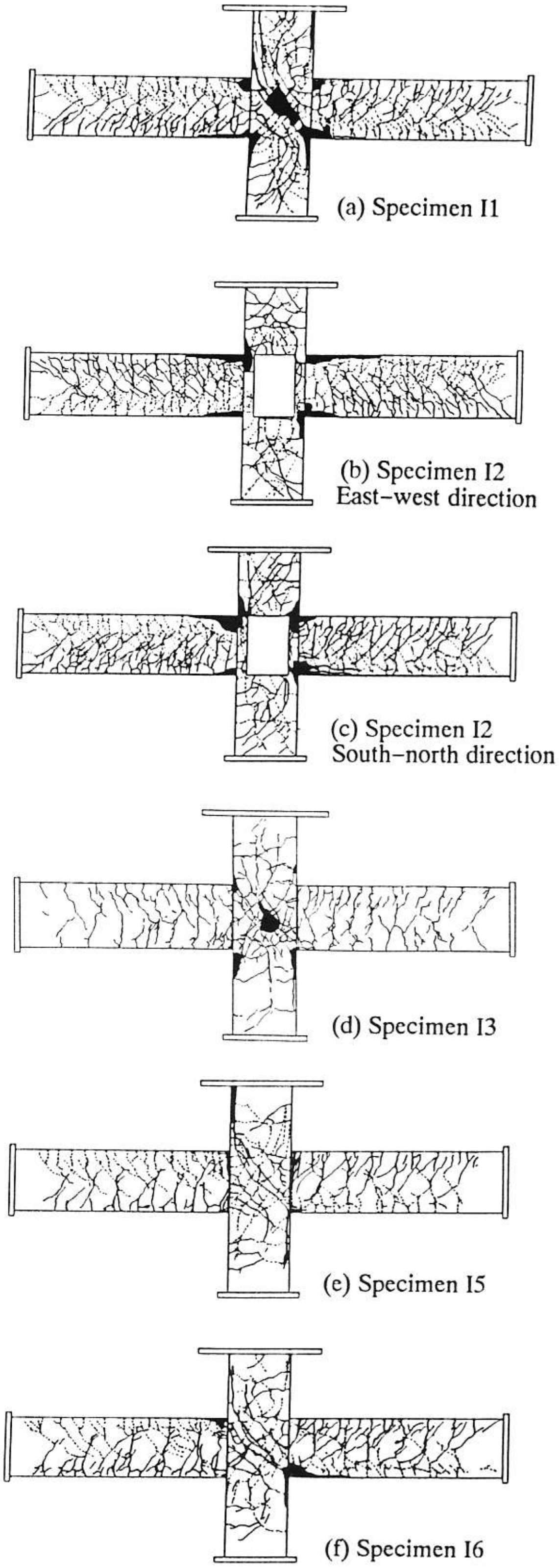



Fig. 4 Crack patterns after test

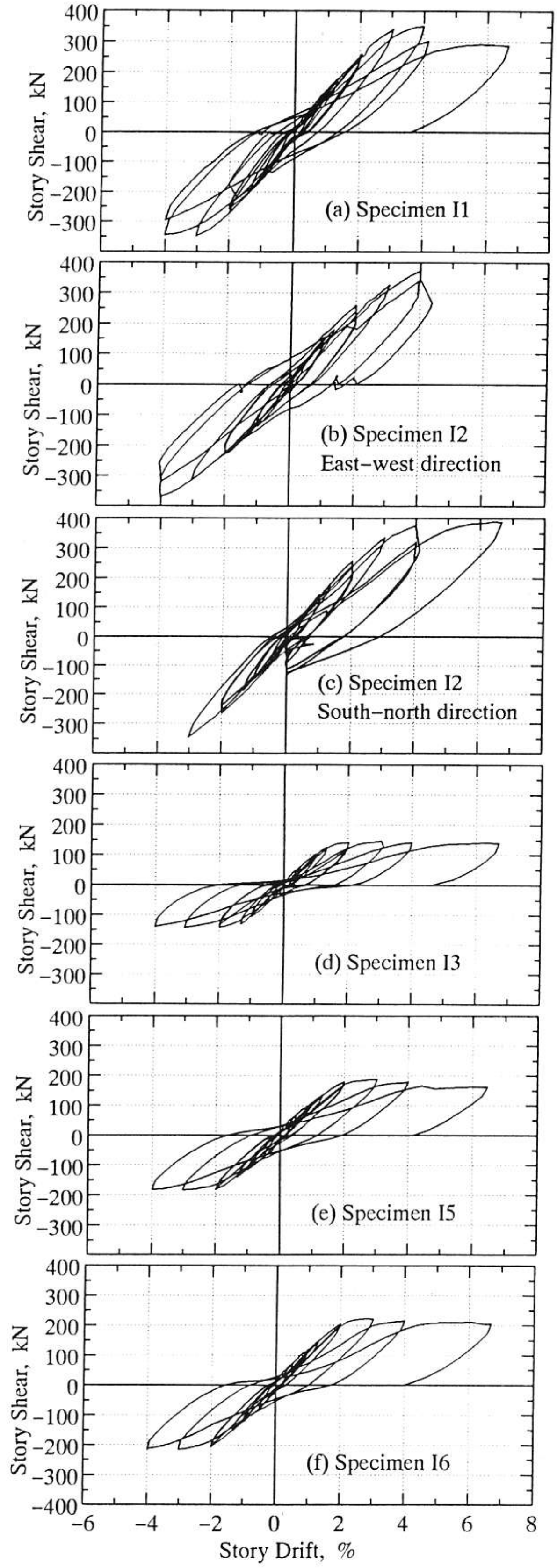



Fig. 5 Story shear – story drift relations

by crushing of joint core concrete in diagonal compression.

On the contrary, three-dimensional joint of Specimen I2 did not fail in shear even under large bi-directional loading. Specimen I2 failed in flexural compression at the beam ends. Beam bars yielded in an outer layer at a story drift angle of 3 to 4 % and in an intermediate layer at a story drift angle of 4 to 6 %. Story shear increased after beam yielding.

Beam bars in Specimen I3 with ordinary-strength concrete yielded after a story drift angle of 1 %, and the joint eventually failed in shear at a story drift angle of 4 %.

Beam bars yielded at a story drift angle of 1.3 to 2 % in Specimen I5 and 1.6 to 2.2 % in Specimen I6, reaching the joint shear stress of 0.15 $\sigma_B$ ; where $\sigma_B$ : concrete compressive strength. Concrete at the beam compressive regions adjacent to column faces crushed and spalled during a cycle of story drift angle of 3 %. Diagonal cracks were observed in the joints, but excessive damage was not concentrated.

### 3.2 *Joint shear deformation*

Joint shear stress – shear distortion relations are shown in Fig. 6. Joint shear distortion was measured in plane specimens by two transducers mounted in diagonal directions on a joint panel, and was computed in a three-dimensional Specimen I2 as the story drift less the contribution from the beam and column deflections. Joint shear distortion in Specimens I1 and I3 increased with story drift, indicating the distress in a joint panel. The second stiffness after diagonal cracking in a joint shear – distortion relation was larger in joints with high-strength concrete (Specimens I1, I2, I5 and I6) than that with ordinary-strength concrete (Specimen I3). The second stiffness was almost similar in Specimens I1 and I2. This suggests that the transverse beams with crack opening at the column faces did not contribute to the enhancement of the second stiffness in a joint shear – distortion relation.

The joint shear distortion contributed to approximately 40 % of the story drift at a story drift angle of 4 % in Specimens I1 and I3 in contrast with 10 to 17 % in Specimens I2, I5 and I6 in which the beam deflection dominated the total story drift.

Joint lateral reinforcement parallel to a loading direction yielded in Specimens I1 and I2 at a joint shear distortion of 0.5 to 0.6 %, which was 2 times greater than that of 0.2 to 0.3 % in Specimens I5 and I6. The difference depended on; 1) the magnitude of yield strain in joint lateral reinforcement, i.e., 0.21 % in Specimens I1 and I2 in contrast with 0.13 % in Specimens I5 and I6, and 2) the amount of column intermediate longitudinal reinforcement which restrained diagonal crack opening in a joint panel. Lateral reinforcement orthogonal to a loading direction, which confined the joint core concrete, also yielded in all specimens during load reversals at a story drift angle of approximately 3 %.

### 3.3 *Bond along beam reinforcement*

Local bond stresses along a beam bar within the center one-third of a column depth, $\tau_l$, are shown in Fig. 7 for Specimens I3, I5 and I6. The bond stress was computed from the difference of beam reinforcement stresses. The

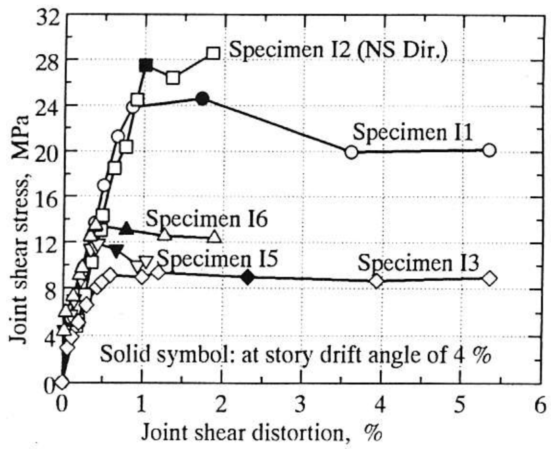



Fig. 6 Joint shear stress – shear distortion relations

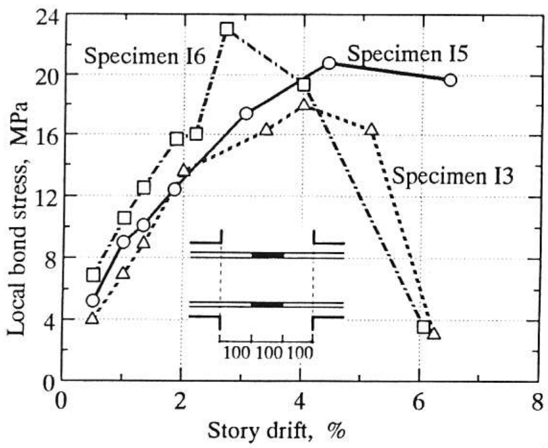



Fig. 7 Local bond stress within joint

local bond stress $\tau_l$ increased with a story drift in Specimen I5 and good bond was maintained during the test. The value of $\tau_l$ in Specimen I6 also increased but decayed after a story drift angle of 2.7 %, indicating the bond deterioration. The maximum value of $\tau_l$ in Specimen I3 made of ordinary-strength concrete was smaller than those in the two specimens with high-strength concrete. However, the bond stress normalized by the square root of concrete compression strength was similar between Specimens I3 and I6.

An average bond stress $u_b$, computed for simultaneous tensile and compressive yielding of beam reinforcement at the opposite column faces, is expressed as follows;

$$u_b = (1 + \gamma)(\sigma_y / 4)(d_b / D) \tag{1}$$

where, $\gamma$ ( $\leq$ 1.0) : a ratio of compressive to tensile bar areas, $\sigma_y$ : yield strength of a beam bar, $d_b$ : diameter of a beam bar and D : column depth. Measured average bond stress $\tau_{av}$ within a joint is related with the index $u_b$ in Fig. 8 obtained from interior beam-column joint tests at the University of Tokyo (Refs. 1 – 3). Solid symbols represent specimens in which average bond

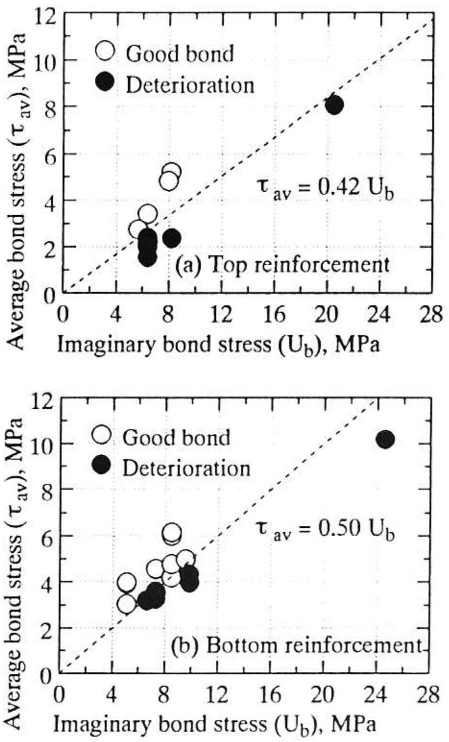



Fig. 8 Measured bond stress $\tau_{av}$ – $u_b$ relations

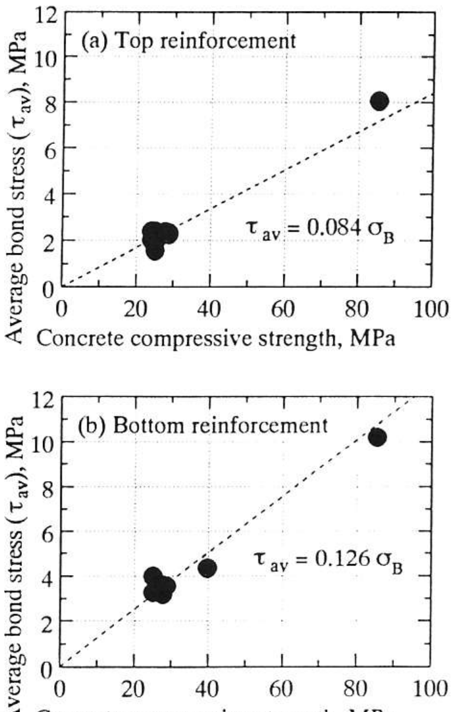



Fig. 9 Measured bond stress $\tau_{av}$ –
concrete compressive strength $\sigma_B$ relations

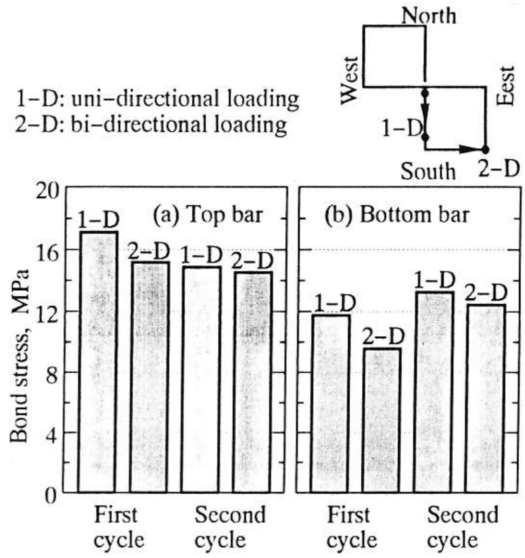



Fig. 10 Bond under uni- and bi-directional loading

stress $\tau_{av}$ deteriorated either before beam bar yielding or after yielding, and open symbols represent specimens with good bond even after beam bar yielding. Average bond stress $\tau_{av}$ did not decay before beam yielding, when average bond stress $\tau_{av}$ of top reinforcement is larger than $0.42\ u_b$, and when that of bottom reinforcement is larger than $0.5\ u_b$.

An average bond stress $\tau_{av}$ – concrete compressive strength $\sigma_B$ relationship is shown in Fig. 9. The average bond stress $\tau_{av}$ is proportional to concrete compressive strength $\sigma_B$, i.e., average bond stress $\tau_{av}$ of top bars is estimated by $0.084\ \sigma_B$, and that of bottom bars by $0.126\ \sigma_B$.

From the two relations above, the average bond stress $\tau_{av}$ within a joint will not deteriorate before beam yielding if the bond index $u_b / \sigma_B$ of top reinforcement is smaller than 0.2, and that of bottom reinforcement smaller than 0.25.

A bond stress along north-south direction beam reinforcement at the center one-third of a column width in Specimen I2 is shown in Fig. 10 under bi-directional loading at a story drift angle of 4 %, comparing with that at the same story shear under uni-directional loading. The bond hardly deteriorated even under bi-directional loading when the beam bar strain exceeded slightly the yield strain.

3.4 *Joint shear resistance*

Story drift – joint shear stress relations normalized by concrete compressive strength $\sigma_B$ are shown in Fig. 11 for a plane Specimen I1 and a three-dimensional Specimen I2 subjected to uni-directional and bi-directional loading. The joint shear and story drift under bi-directional loading were computed as the square root of the sum of the squares of shears and drifts in respective directions.

Joint shear stress in Specimen I1 decreased after reaching the maximum value of $0.25\ \sigma_B$, accompanied with the severe damage in the joint panel. On the contrary, uni-directional joint shear stress in Specimen I2

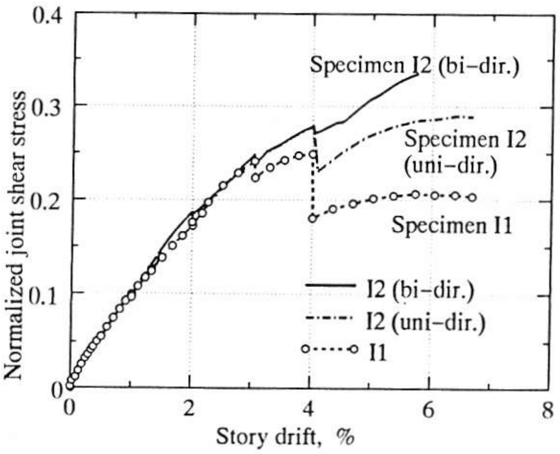



Fig. 11 Normalized joint shear stress

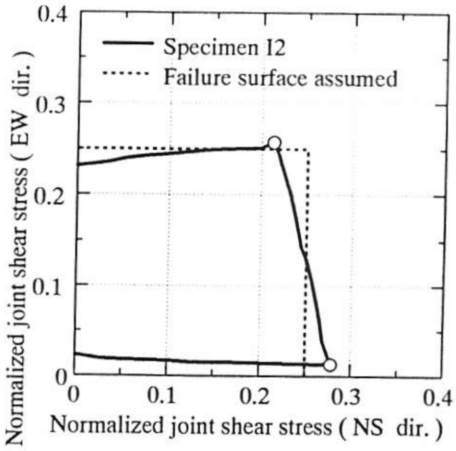



Fig. 12 Joint shear resistance under bi-directional loading

increased to 0.29 $\sigma_B$. The transverse beams, even loaded cyclically, could enhanced joint shear strength at least 1.2 times more than that of a plane joint. This enhancement ratio was almost equal to that observed in a joint of ordinary-strength concrete (Ref. 4).

Bi-directional joint shear resistance in Specimen I2 reached as high as 0.33 $\sigma_B$. However, the joint did not fail in shear because of the confinement effect by the transverse beams. The orbit of a joint shear resistance under bi-directional loading at a story dirft angle of 4 % is shown in Fig. 12. The biaxial interaction surface of a joint shear resistance was assumed to form two orthogonal lines at the shear stress of 0.25 $\sigma_B$ as shown by broken lines. The joint shear stress in north-south direction decreased slightly under bi-directional loading by the biaxial interaction in column resistance. Therefore, shear failure in the joint with high-strength concrete under bi-directional load reversals could be avoided by limiting independently the input shear to 0.25 $\sigma_B$ in respective directions.

## 4 CONCLUSIONS

The following conclusions were drawn from the test results of interior beam-column joints with high-strength materials;

(1) The bond along the beam reinforcement within a joint was good up to a story drift angle of 3 to 4 % (ductility factor of 1.3 to 2.0) regardless of the diameter of a beam bar.

(2) Average bond stress $\tau_{av}$ within the joint even made of high-strength concrete will not deteriorate before beam bar yielding if the bond index $u_b / \sigma_B$ of top reinforcement is smaller than 0.2, and that of bottom reinforcement smaller than 0.25.

(3) Bi-directional load reversals did not influence the bond transfer along beam bars within a joint even if the beam bar strain was slightly more than yield strain.

(4) Joint in a plane frame failed in shear prior to beam yielding at a shear stress of 0.25 $\sigma_B$, which was approximately 0.8 times smaller than the joint shear strength of ordinary-strength concrete in recent works (Ref. 5).

(5) The uni-directional shear strength of a joint with transverse beams loaded to flexural yielding was enhanced to 1.2 times that of a plane joint.

(6) The joint shear strength under bi-directional loading was increased to 1.3 times that of a plane joint by confining effect of transverse beams.

(7) A joint can be kept sound even under bi-directional cyclic loading if the design shear in orthogonal directions is limited to 0.25 $\sigma_B$ in each direction.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