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K. Kitayama

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BASIC TESTS ON BOND ALONG HIGH-STRENGTH BEAM LONGITUDINAL BARS THROUGH HIGH-STRENGTH R/C BEAM-COLUMN JOINT

Kazuhiro KITAYAMA[1], Makoto FUKUOKA[2],
Keiji ETO[3] and Takashi FUJITA[4]

ABSTRACT

High-strength concrete panels reinforced by high-strength longitudinal bars were tested by a simple method developed herein to study the bond characteristics along the beam reinforcement within an interior beam-column joint. The bond stress reached the maximum value for panels constructed with the high-strength materials when a diagonal shear crack occurred across the beam bar, and the bond transfer subsequently decayed with the beam bar yielding. On the contrary, the bond deterioration for panels made of the ordinary-strength materials was caused by the yielding of the beam reinforcement.

1. INTRODUCTION

The use of higher strength concrete and steel is necessary to build high-rise reinforced concrete structures with the adoption of an ultimate strength design procedure depending on the ductility. The reduction of column dimensions derived from the use of high-strength materials causes the increase in an input shear stress to a beam-column joint and the severe bond along the beam reinforcement within a joint. Hence the tests of beam-column joints using complicated loading apparatus are necessitated. In this study the stress field within a joint was simulated by a simple test to investigate the bond transfer from the high-strength beam bar to the high-strength concrete.

2. TEST PROGRAM

Four reinforced concrete square panels (called PJ1 to PJ4) sliced from the half-scale interior beam-column joint were tested, whose dimensions were 300 x 300 mm with thickness of 110 mm. Concrete blocks were added to a panel to represent the compression stress block acting in the beam critical section by the flexure. Reinforcement details are shown in Fig. 1 and the properties of specimens in Table 1. The beam and column longitudinal deformed bars were welded to the anchorage plates. The legged ties with 180 degree hooks at both ends were used as the joint lateral reinforcement. Concrete compressive strength, beam

1) Department of Architecture, Faculty of Engineering, Chiba University
2) Structural Design Division, Oka Sekkei Ltd.
3) Sato-Gumi Co. Ltd.
4) Building Construction Division, Nippon Steel Corporation

bar yield strength and the amount of joint lateral bars were varied in this test. The beam bar diameter of 16 mm was common in all specimens. Specimens PJ1 and PJ2 consisted of both high–strength concrete of 875 kgf/cm^2 and high yield strength beam bars of 8140 kgf/cm^2, whereas specimens PJ3 and PJ4 of both ordinary–strength concrete of 264 kgf/cm^2 and usual yield strength beam bars of 3460 kgf/cm^2. Concrete properties are shown in Table 2. The water to cement ratio in high–strength concrete was 25 percent and super water–reducer was mixed. The concrete was cast in a horizontal position using metal form.

The loading apparatus is shown in Fig. 2. Top and bottom beam reinforcing bars were pulled monotonically by two jacks. Reacting forces are developed in the column bars connected to reaction frame by prestressing bars to prevent a specimen from rotating. The compressive forces acting on both beam critical sections, which contribute to a joint shear in an actual beam–column joint, were neglected to facilitate the tests. Therefore a joint shear into a specimen was approximately one–half times smaller than that of an actual interior joint.

The loads applied by the jacks were measured by load–cells. The strain distributions along the beam and column reinforcement and the strain of joint lateral reinforcement were measured by strain gauges. The horizontal, vertical and diagonal deformations of a panel were measured by displacement transducers.

Table 1 : Properties of Specimens

Specimen	PJ1 / PJ2	PJ3 / PJ4
Beam Bar	D16	D16
Yield Strength	8140 kgf/cm^2	3460 kgf/cm^2
Column Bar	D19	D16
Yield Strength	7610 kgf/cm^2	8140 kgf/cm^2
Panel Reinf.	2–φ6	2–φ6
Sets	1 / 3	1 / 3
Ratio	0.22 / 0.68 %	0.22 / 0.68 %
Yield Strength	2535 kgf/cm^2	2535 kgf/cm^2

Table 2 : Material Properties of Concrete (unit in kgf/cm^2)

Specimen	Compressive Strength	Tensile Strength	Secant Modulus*
PJ1 PJ2	875	58.0	396,000
PJ3 PJ4	264	20.5	257,000

* Secant modulus at one–quarter of the compressive strength

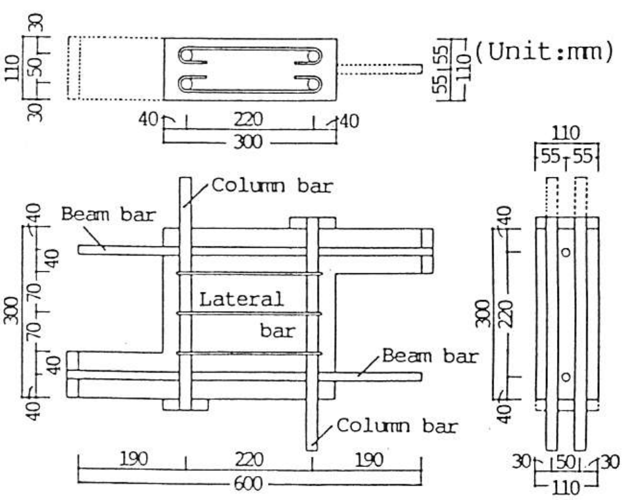
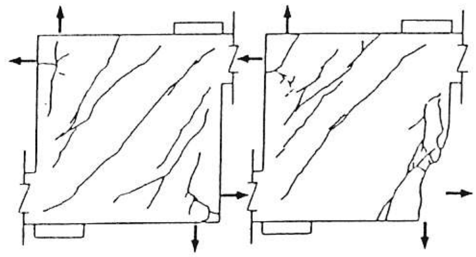


Fig. 1 : Reinforcement Details

3. TEST RESULTS

3.1 GENERAL RESULTS AND OBSERVATIONS

The crack patterns for all specimens at the end of loading are shown in Fig.3. Several diagonal shear cracks occurred in the left–up and right–down region of a panel. The beam bars yielded near the loaded end prior to diagonal cracking in the panel in specimens PJ3 and PJ4 constructed with ordinary–strength concrete and steel. Shear crack was developed along the main diagonal of panels in specimens PJ1 and PJ2 constructed with high–strength concrete and steel.



(a) Specimen PJ1 (b) Specimen PJ2

(c) Specimen PJ3 (d) Specimen PJ4

Fig. 3 : Crack Patterns after Test

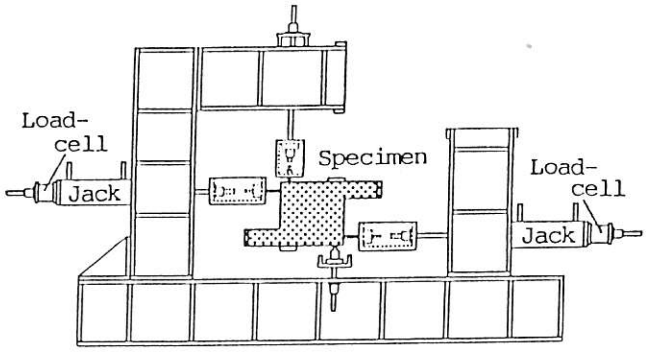


Fig. 2 : Loading Apparatus

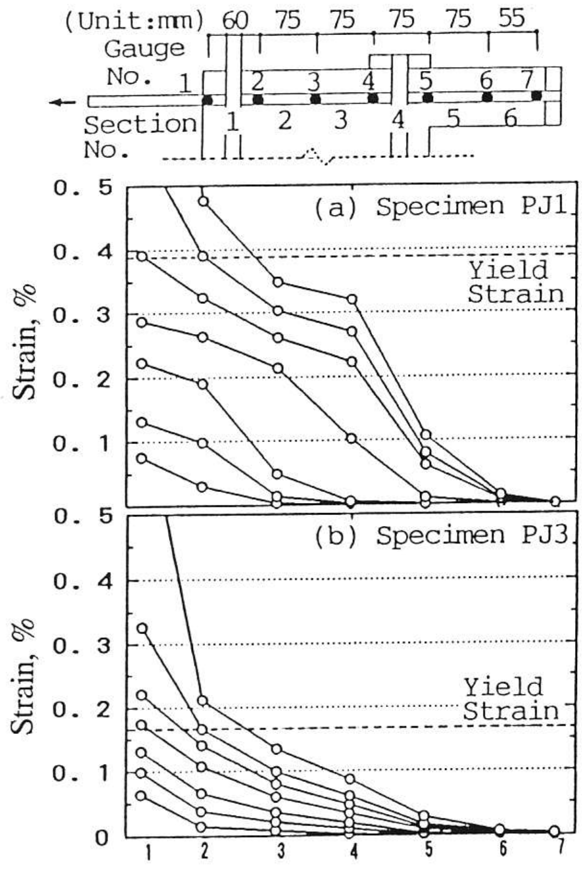
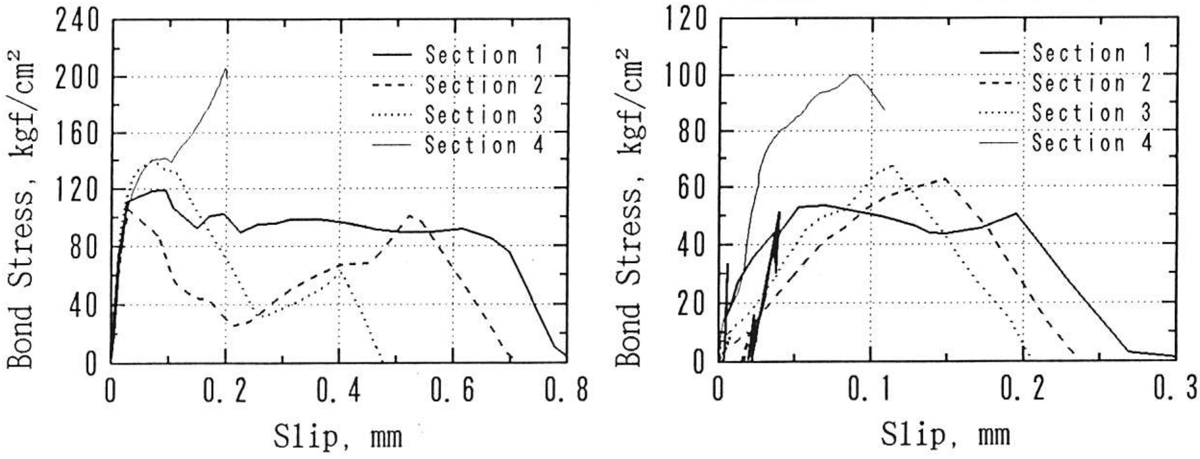


Fig. 4 : Strain Distributions along Beam Bar

The strain distributions along a beam bar are shown in Fig. 4 for specimens PJ1 and PJ3. The location of the steep slope of strains in specimen PJ1 moved from the loaded end to the center of a panel with the increase in a load. Joint lateral reinforcement in specimens PJ1 and PJ2 yielded, but that of specimens PJ3 and PJ4 did not. The column bars remained elastic for all specimens. The influence of the amount of the joint lateral reinforcement was not observed on the crack pattern and the bond transfer along the beam bars.

3.2 BOND ALONG BEAM REINFORCEMENT

The bond stress–slip relations for respective sections divided by the successive strain gauges along the beam bar are shown in Fig. 5 for specimens PJ2 and PJ4. The i-th section denotes the portion between the i-th strain gauge and the (i+1)-th strain gauge as shown in Fig. 4. The bond stress was calculated as the difference of bar stresses in two adjacent gauge points. The slip was obtained by the integration of strains along the beam reinforcement from the anchored end to the center of each section. The maximum bond stresses in the first and second sections were a little smaller than that in the third section for panels with high–strength materials in Fig. 5(a). On the other hand, the bond stresses were substantially same from the first to third sections for panels with ordinary–strength concrete in Fig. 5(b). The bond stress in the fourth section for all specimens was quite larger than those in other sections since the concrete surrounding the fourth section was compressed by the reacting force from the anchor plate of column bars induced by the bond deterioration along the column bars.



(a) Specimen PJ2 (High-Strength) (b) Specimen PJ4 (Ordinary-Strength)

Fig. 5 : Bond Stress – Slip Relations

The maximum bond stress–corresponding slip relations from the first to the fourth sections are shown in Fig. 6 for all specimens. The maximum bond stresses for specimens PJ1 and PJ2 with high-strength concrete were larger than those at the equal slip for specimens PJ3 and PJ4 with ordinary-strength concrete. Note that these maximum bond stresses are not necessarily the inherent bond strength because the bond stresses in many sections reached the maximum value due to the diagonal cracking by shear in a panel or the yielding of the beam reinforcement as mentioned after. The maximum bond stresses for high-strength concrete specimens increased with slip, whereas those for ordinary-strength concrete specimens kept almost constant regardless of the slip.

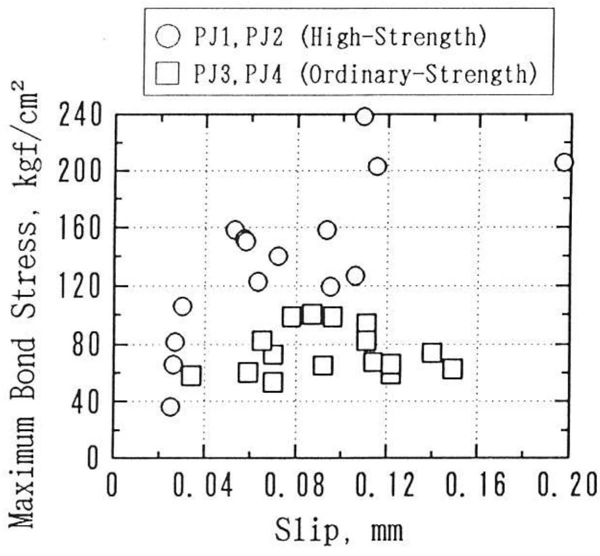


Fig. 6 : Maximum Bond Stress – Slip Relations

The bond stress–slip relations for specimens PJ2 and PJ4 are shown in Fig. 7 in the manner of separating each section. Yielding of the beam bar and the diagonal shear cracking across the beam bar in the section are indicated in figures. The initial stiffness of high-strength concrete specimens was greater than that of ordinary-strength concrete specimens. For high-strength materials specimens a remarkable decrease in the bond stress was caused by shear cracking diagonally crossing the beam reinforcement. However the bond stress ascended gradually during the elasticity of the beam bar, and eventually decayed with the beam bar yielding. On the contrary, for ordinary-strength materials specimens the maximum bond stresses except for the first section were derived from the beam bar yielding. The diagonal cracks in a panel affected little the bond transfer.

Thus diagonal shear cracks had an important effect on the bond transfer from a beam high-strength reinforcing bar to surrounding high-strength concrete. This points out that the bond deterioration along beam bars may occur before yielding in actual interior beam-column joints made of high-strength materials.

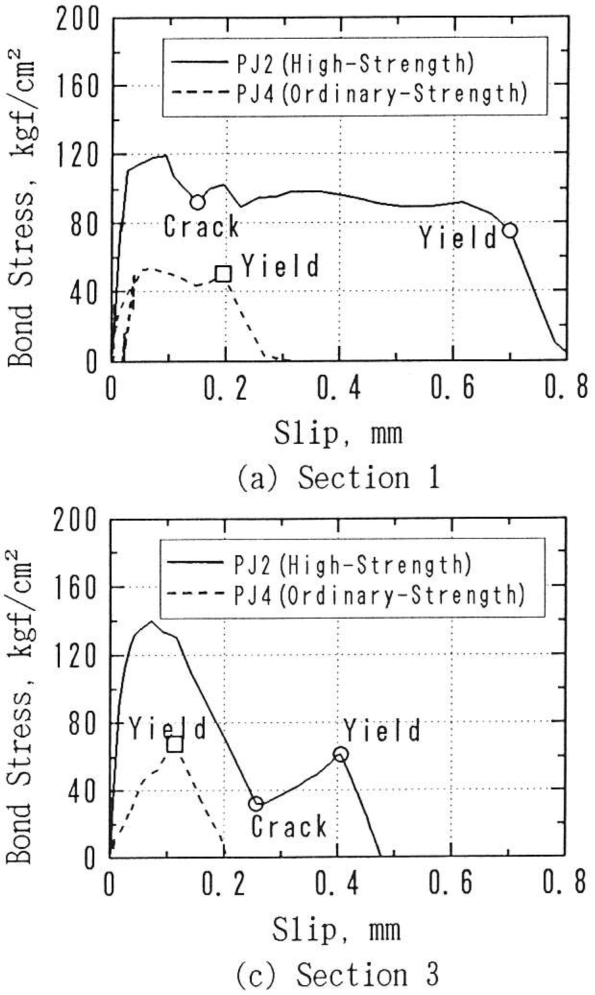


Fig. 7 : Bond Stress – Slip Relations in Each Section

3.3 COMPARISON WITH INTERIOR BEAM–COLUMN JOINT TEST

The bond stress–slip relations along the bottom beam reinforcement are shown in Fig. 8 for interior beam–column subassemblages tested in the University of Tokyo; the dimensions were 200 x 300 mm in beams, and 300 x 300 mm in columns. One was constructed with the high–strength concrete of 870 kgf/cm^2 and the 19 mm diameter beam bars with high yield strength of 7870 kgf/cm^2 (specimen I6 in Ref. 1), and the other was constructed with the ordinary–strength concrete of 245 kgf/cm^2 and the 13 mm diameter beam bars with yield strength of 4090 kgf/cm^2 (specimen J2 in Ref. 2). The bond stress in one–third of a column depth adjoining the beam critical section with the bottom fiber in tension

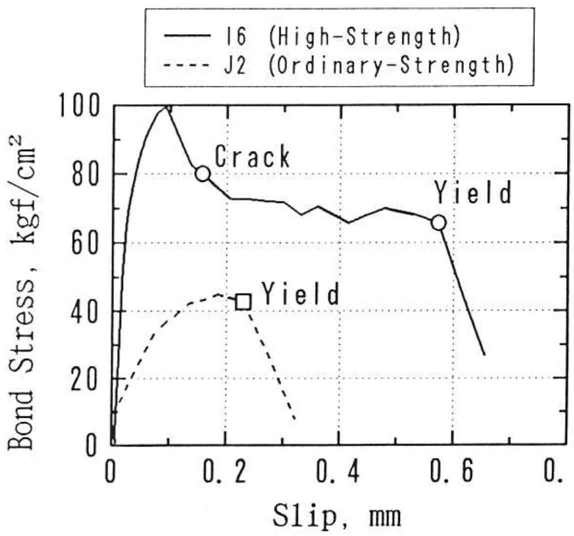


Fig. 8 : Bond Stress – Slip Relations in Beam–Column Joint Specimens

was used. The slip was calculated by the integration of strains along the beam reinforcement within a joint. The bond transfer declined due to the diagonal cracking for the beam–column joint with high–strength materials, whereas due to the yielding of a beam bar for the joint with ordinary–strength materials. Bond characteristics along the beam bar passing through the beam–column joints, hence, was quite similar to that obtained by panel tests.

4. CONCLUDING REMARKS

The panel tests reported in this paper were able to simulate the bond transfer along the beam reinforcement within an interior beam–column joint. Initial stiffness in bond–slip relations for the specimens with high–strength materials was greater than that for the specimens with ordinary–strength materials. The bond force reached the maximum value for panels with the high–strength materials when a diagonal shear crack intersected the beam bar, and the bond force subsequently decayed with the beam bar yielding. On the contrary, the bond deterioration for panels made of the ordinary–strength materials was not caused by the cracks, but caused by the yielding of the beam reinforcement. Thus the event reducing the bond transfer ability along the beam reinforcement is different by the strength of steel and concrete for beam–column joints in frames forming the beam collapse mechanism. The influence of the amount of the joint lateral reinforcement was not observed on a crack pattern and the bond transfer along the beam bars.

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REFERENCES

1. Lee, S., T. Fujita, K. Kitayama and S. Otani, "Anchorage of Beam Reinforcement in High–Strength Reinforced Concrete Interior Beam–Column Joints," Transactions of the Japan Concrete Institute (JCI), Vol.13, 1991, December.

2. Kobayashi, Y., M. Tamari, S. Otani, and H. Aoyama, "Experimental Study on Reinforced Concrete Beam–Column Subassemblages (in Japanese)," Transaction of Sixth JCI Annual Meeting, 1984, pp.653–656.