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Ultimate shear force of R/C beam failed in bond splitting

H.Takagi
Meiji University, Japan
H.Matsubara
Fujita Corporation, Japan

ABSTRACT: The authors discuss the effect of high-strength web reinforcementson resistance to splitting bond failure in beam that precedes flexural yielding of the member subjected to bending and shear under a monotonic loading test. This paper describes the following manner, splitting bond failure in the member of the beam, splitting bond strength along bars, shear capacity of the member acompany by splitting bond failure.

1 INTRODUCTION

When a bending shear force is applied, splitting bond failure may occur in such a way that the covering concrete around deformed bars is split by the wedge action of the reinforcing steel node, and bearing strength deteriorate accordingly. This phenomenon, termed splitting bond failure, is attracting attention in many associated fields. In this failure, the bonding force between reinforcing steel and the surrounding concrete is lost by the splitting action. The main reinforcement's stress is thus not transmitted to the concrete, and as a result the truss mechanism, which is an essential component of any reinforced concrete structure, is no longer maintained. Much research on the use of high-strength reinforcing steel is in current progress. The application of high-strength reinforcing steel as stirrups to utilize its high tension capacity, however, decreases the ratio of the stirrup, which in turn increases the likelihood of a splitting bond failure. In this paper, we collectively discuss the features of splitting bond failure and the effect of stirrups on bearing strength and on splitting bond failure.

2 EXPERIMENTAL RESULTS

2.1 Experimental parameters

Table 1 shows the parameters the authors used in the experiments on high-strength stirrups. The table indicates that the experiment consists of three series (A, F, B) in which the cross section, concrete strength, and the amount of main reinforcements are varied. The A-series has a cross section of 20x30 cm, three super high-strength main reinforcements that are subjected tensile force (deformed PC bar of 23 mm diameter and 93755 N/cm^2 of $_s\sigma_y$), and a concrete strength of 2746 N/cm^2. The F and B-series each have a cross section of 20x40 cm and a concrete strength of 3531 N/cm^2. The F-series, like the A-series, has three super high-strength main reinforcements that are subjected to tensile force sufficient to cause a splitting bond failure. The B-series, which is different from the F-series, has five super high-strength main reinforcements that are subjected to tensile force such that a splitting bond failure is prevented. In each series, the amount of stirrups and the strength were made variables and their effects were evaluated. Throughout the whole series, the shear span ratio was determined to be a/D = 2.0.

2.2 Test specimens

Table 2 shows 16 test specimens for this discussion. In the A-series, the stirrup's yield strength $_w\sigma_y$ and diameter were selected as variable factors and the intervals between stirrups were made constant.
In the F and B-series, high-tension steel of 98100 N/cm^2 or more was used as stirrups, and the stirrup diameter and the intervals were changed to provide three different web-reinforcement ratios (p_w). In the A-series, concrete was vertically placed from the upper end of the beam, but in the F and B-series, concrete was horizontally placed from the side face to avoid a difference between the bonding forces in the reinforcements at the upper and lower ends.

2.3 Load application and measurement method

Load was applied in one direction loading of the bending and shear in the antisymmetric moment mode method (Ohno-system). All test specimens were loaded in the same pattern in such a way that a uni directional force less than or equal to 2/3 of the ultimate bearing strength was applied twice, then the load was increased until the specimen failed. In the measurement, the vertical displacement of the beam's center point with respect to the stabs at both side was measured. Also, the strains of the main reinforcements and stirrups were measured.

3 EXPERIMENTAL RESULTS

Table 2 lists the experimental results. Figure 1 shows a typical example of the comparison of the load-deformation curve and cracks at the maximum bearing strength for the F-series and B-series.

3.1 Maximum bearing strength

In every series, if the yield strength (${}_w\sigma_y$) of the stirrups is the same, the greater the web reinforcement ratio (p_w), the greater the maximum bearing strength ; if the web reinforcement ratio (p_w) is the same, the greater the yield strength (${}_w\sigma_y$), the greater the maximum bearing strength. When the F-series and B-series with different numbers of main reinforcements are compare for the same p_w, the B-series with five main reinforcements subjected to tensile force shows that the maximum bearing strength and the amount of deformation are greater than those of the F-series.

3.2 Failure conditions

The main reinforcements were not yielded in any test specimens. In the A and F-series, the maximum bearing strength was reached with splitting bond failure.
In the B series, splitting bond failure rarely occurred and the maximum bearing strength was reached with shearing failure.

4 SPLITTING BOND STRENGTH ALONG BARS

The primary research studies conducted in Japan on splitting bond strength of the deformed bars have been those of Morita and Fujii (Ref.1) and of Shibata and Sakurai (Ref.2). According to Morita's reseach, the splitting bond strength is related to the web reinforcement ratio but is unrelated to the strength while according to Shibata, it relates to the strength of web reinforcement. Figure 2 shows a typical example of the relation between the bond stress (τ_a) in each region of a test beam and the shear force applied to it as observed by our tests. The value of τ_a gradually increases along with increases in shear force. When bond cracks form in a region, the region experiences a decrease in τ_a. The decrease starts from the region near the point of maximum tensile force in each longitudinal bar, but the maximum bond stress in each region does not occur simultaneously. The average value $\tau_{a.av}$ of bond stresses through regions (3)~(7), shown in Figure 3, continues to increase even after bond cracks form in a specific region and the value of $\tau_{a.av}$ in the region turns to decrease. It does not tend to decrease until the ultimate strength of the beam is reached. The maximum value of the bond stress τ_a in each region was almost the same irrespective of the value of ${}_w\sigma_y$ and p_w. The value was estimated to be $0.87\sqrt{f'_c}$ (unit:kgf/cm^2) corresponding to the value proposed by Morita as the maximum bond resistance of a bar which is fully restrained by web reinforcement. The web reinforcement decreases the bond stress gradually after bond crack form in the region. Hence, a maximum value for the average bond stress $\tau_{a.av}$ throughout the bond failure regions increases as a function of the increase in p_w but independent of the strength of the web reinforcement, as shown in Figure 4. The maximum average bond stress can be calculated based on the formula proposed by Morita and Fujii shown later. Muguruma and Watanabe ran tests to study the relationship between the maximum average bond stress for splitting bond failure in columns and values of $p_w \cdot {}_w\sigma_y$ and/or p_w. They concluded that a better correlation was obtained between the maximum average bond stress and the value of p_w than between the bond strength and the value of $p_w \cdot {}_w\sigma_y$ (Ref.3). These two reseach studies indicate that the splitting bond strength along a bar is a function of the web reinforcement ratio p_w and does not depend on the strength of web reinforcement, as stated by Morita. Also, the bond stress does not decrease rapidly after the maximum value is reached, if a larger amount of web reinforcement is applied.

5 SHEAR CAPACITY OF THE BEAM MEMBER CAUSED THE SPLITTING BOND FAILURE

The committee on reinforced concrete structures at the Architectural Institute of Japan (hereafter refered to as AIJ) proposed in 1988 the following new formula to predict the uiltimate shear strength of reinforced concrete beams and columns (Ref.4).

■ $$Q_u = b \cdot j_t \cdot p_w \cdot {}_w\sigma_y \cot\phi + \tan\theta(1-\beta) b \cdot D \cdot \nu f'_c/2 \qquad \text{(UNIT:kgf,cm)} \quad (1)$$

$$\tan\theta = \{\sqrt{[(L/D)^2+1]}\} - L/D \qquad 1)$$

$$\beta = \{(1+\cot^2\phi)\, p_w \cdot {}_w\sigma_y\}/(\nu f'_c) \qquad 2)$$

$$\nu = 0.7 - f'_c/2000 \qquad 3)$$

$\cot\phi$ is the minimum value in the following these equations but may not be smaller than 1.0.

$$\cot\phi \leqq 2.0$$

$$\cot\phi = j_t/(D \cdot \tan\theta) \qquad 4)$$

$$\cot\phi = \sqrt{\nu f'_c/(p_w \cdot {}_w\sigma_y) - 1.0}$$

here, $p_w \cdot {}_w\sigma_y \leqq \nu f'_c/2$

${}_w\sigma_y \leqq 25 \cdot f'_c$

The authors have examined the shear capacity of members with splitting bond failure by modifying the new AIJ formula for shear. The Splitting bond failure strength of a member (${}_fQ_u$) is calculated using the following equations.

■ $${}_fQ_u = {}_aQ + {}_bQ \qquad \text{(UNIT:kgf,cm)} \quad (2)$$

${}_aQ$: arch mechanism strength

${}_bQ$: truss mechanism strength

$${}_aQ = {}_ab/2 \cdot (\sqrt{1+(L/D)^2} - L/D) \cdot D \cdot \nu f'_c$$

$${}_bQ = \tau_{b.av} \cdot \Sigma\phi \cdot j_t$$

$${}_ab = b - {}_rb \qquad 1)$$

$${}_rb = \frac{\tau_{b.av} \cdot \Sigma\phi}{\nu F_c \cdot \sin\phi\cos\phi} \qquad 2)$$

$$\sigma_w = \frac{\tau_{b.av} \cdot \Sigma\phi \cdot \tan\phi}{b \cdot p_w} \qquad 3)$$

$$\tau_{b.av} = k_o \cdot {}_b\tau_u \qquad 4)$$

The shear force carried by the truss mechanism is calculated assuming a splitting bond failure.
The average bond strength $\tau_{b.av}$ is represented by equation 4), where ${}_b\tau_u$ is the bond strength calculeted using the formula below proposed by Morita and Fujii.

$${}_b\tau_u = \left(0.307 b_i + 0.427 + \frac{24.9 k \cdot A_{st}}{s \cdot N \cdot d_b}\right) \cdot \sqrt{f'_c}$$

here, b_i:minimum(b_{si}, b_{ci}, b_{vi})

$$b_{si}=b/(N\cdot d_b)-1 \qquad \rightarrow k=1.0$$
$$b_{ci}=2\cdot((C_s+C_b)/d_b+1)-1 \qquad \rightarrow k=2.0$$
$$b_{vi}=3\cdot(2\cdot C_{min}/d_b+1) \qquad \rightarrow k=0$$

A reduction factor k_o was used to obtain the average bond strength taking into consideration that the maximum bond stress is not reachhed simultaneously in each region. Based on our test results, the value of k_o was taken to be 0.93. Calculation is made in the following manner on the width $_rb$ of diagonal concrete compression struts that are required for the truss mechanism. First, the angle of inclination ϕ of diagonal struts is calculated from formula 3) assuming $\sigma_w={}_w\sigma_y$. It should be confirmed that the calculated value of $\cot\phi$ is within the range given by equation 4) of the formula (1). When $\cot\phi$ is larger than the value given by the equation, splitting bond failure does not occur because the calculated shear strength obtained from formula (1) is smaller than the caluculated splittimg bond strength from formula (2). If the calculated value of ϕ exceeds 45 degrees, ϕ is taken as 45 degrees on the assumption that the web reinfocement does not yield. Figure 5 compares calculated values of splitting bond failure strength $_rQ_u$ and experimental values $Q_{u\cdot exp}$ obtained by the authors. Agreement is excellent whether or not web reinforcement yields. The formula and the tests lead us to the following observations. If the web reinforcement yields, even if the values of p_w and $\tau_{b\cdot av}$ do not change, the strength of splitting bond failure of the member is large if high-strength web reinforcement is used. In this test, the ranges of web reinforcement ratio for which yielding of the web reinforcement was confirmed were $p_w \leq 1.2$ % for $_w\sigma_y=$ 30800 N/cm^2 and $p_w \leq 0.3$ % for $_w\sigma_y=78360$ N/cm^2 where f'_c was 2750 N/cm^2. If less high-strength web reinforcement is used instead of normal-strength web reinforcement where the values of $p_w\cdot{}_w\sigma_y$ are equal, the strength of the splitting bond failure of the member becomes small even if the high-strength bars yield. In general, high-strength web reinforcement is not more effective than normal strength web reinforcement in preventing splitting bond failure in reinforced concrete members.

6 CONCLUSIONS

1) High-strength bars are not specially effective as reinforcements to prevent splitting bond failure in the member.

2) The bond stress in splitting bond failure increases with p_w but it does not depend on the strength of the web reinforcement.

3) The maximum average bond stress along a bar in a member can be calculated based on the formula proposed by Morita and Fujii, in which the strength of web reinforcements is not considered.

4) The ultimate shear force in splitting bond failure of members is large for the same value of p_w if high-strength web reinforcement is used and the web reinforcement yields.

5) If less high-strength web reinforcement is used to achieve an equivalent value of $p_w \cdot {}_w\sigma_y$, the ultimate shear force in the splitting bond failure of a member becomes small even if the high-strength bars yield.

6) The ultimate shear force in splitting bond failure of reinforced concrete members and the limit for effective reinforcement are predicted very well by modifying new AIJ for shear.

NOTATIONS

$_ab$: width of diagonal concrete struts for arch mechanism (cm)
$_rb$: width of diagonal concrete struts for truss mechanism (cm)
b: width of web of a member (cm)
C_s: thickness of cover concrete to hohizontal direction (cm)
C_b: thickness of cover concrete to vertical direction (cm)
d_b: diameter of longitudinal reinforcement (cm)
D: total depth (cm)
j_t: distance between top and bottom of longitudinal bars (cm)
L: clear span length of the member (cm)
s: intervals of shear reinforcement(cm)
A_{st}: cross section area of shear reinforcement in one set(cm^2)
$\Sigma\phi$: total of circumferential length in longitudinal reinforcements (cm)
θ: angle of compressive strut of concrete in arch mechanism to longitudinal axis
ν: effective factor of compressive strength of cracked concrete
N: number of longitudinal tensile reinforcement

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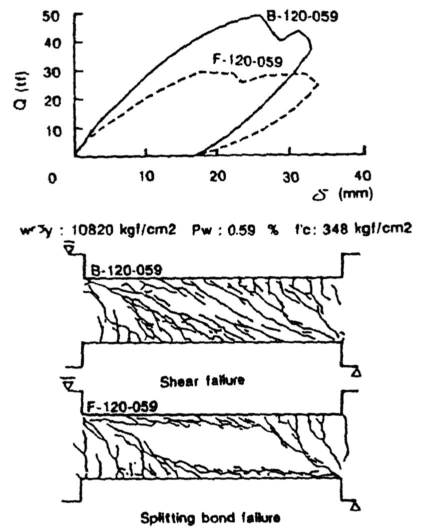

Fig. 1-Comparison of bond splitting failure and shear failure

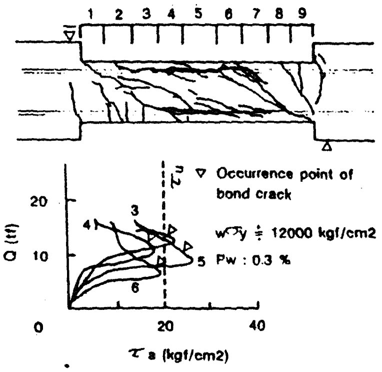

Fig. 2- Relationships between bond stress (τa) and shearing force in each region of a beam

Table 1 Outline of test

Series	Section bxD (cm)	Clear span length (cm)	Effect depth of beam: d (cm)	Strength of concrete: f' (kgf/cm²)	Main reinf. ratio: p_t (%)
A-series	20x30	120	26.0	280	2.39(3-D23)
F-series	20x40	160	36.0	360	1.73(3-D23)
B-series			33.6		3.09(5-D23)

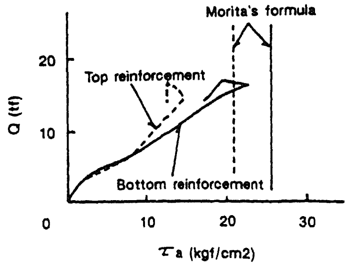

Fig. 3-Relationships between average bond stress (τa .av.) and shearing force

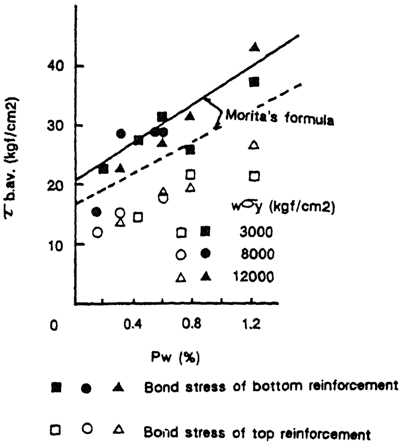

Fig. 4-Relationships between maximum average bond stress (τb .av) and Pw

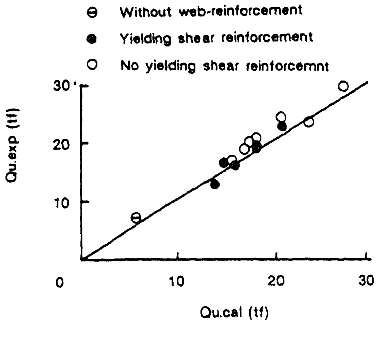

Fig. 5-Comparison between test values of ultimate shear force at splitting bond failure and calculated values by modified AIJ new formula

Table 2 Specimens and results of test

Specimen	Stirrup $_w\sigma_y$ (kgf/cm²)	Stirrup p_w (%)	Stirrup ϕ (mm)	Strength of concrete: f'_c (kgf/cm²)	Max. load : Q_u (tonf)	Failure mode
A-30-043	3260	0.43	6.0	295	15.98	B
A-30-077	3380	0.77	8.0	296	19.04	B
A-30-121	3140	1.21	10.0	299	22.38	B
A-80-015	7610	0.15	3.5	282	12.53	B
A-80-030	7990	0.30	5.0	287	16.38	B
A-80-059	8220	0.59	7.0	273	18.45	B
A-120-030	11710	0.30	5.0	266	16.67	B
A-120-059	11650	0.59	7.0	279	19.73	B
A-120-077	11340	0.77	8.0	288	20.40	B
A-120-121	11580	1.21	10.0	280	23.80	B
F-120-019	10830	0.19	6.0	351	22.98	B
F-120-059	10820	0.59	8.0	353	29.09	B
F-120-121	10870	1.21	10.0	353	43.50	B
B-120-019	10830	0.19	6.0	352	33.05	S
B-120-059	10820	0.59	8.0	354	48.41	S
B-120-121	10870	1.21	10.0	355	55.13	S

* Fuilure mode (B : Splitting bond failure , S : Shear failure)