TIE CONFINING EFFECT ON PLASTIC HINGE IN REINFORCED CONCRETE SHORT COLUMNS UNDER CYCLIC SHEAR LOADINGS

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SUMMARY

Based on the experimental study, shear failure mechanisms of reinforced concrete (RC) short columns are presented and tie reinforcing effect on the brittle failure at the end portion of the member is discussed. For the test specimens used in this study, bond characteristics in main bars, axial load and tie ratios are selected as test parameters. As the results of the analytical and experimental studies, the effects of bond characteristics on the shear failure criteria of RC short columns (bending shear failure and bond splitting failure) are examined and the design considerations of tie confining effect on the bending shear failures are introduced.

INTRODUCTION

Shear failures of reinforced concrete (RC) short columns have been classified into some typical failure criteria based on the experimental studies, and it has been pointed out that the shear span ratio, axial loadings, main bar ratio and tie ratio have significant effect (Ref.1,2). For the members with shear span ratio of about 1.5 to 2.0 loaded by shear reversals, it is known that these members reach the ultimate state in shear failure at the end portions, or, in bond splitting failure at the mid portion, and that in the ultimate state, the bond strength in main bars at the end portions becomes smaller and truss bearing mechanisms are composed (Ref.3,4). The authors have analyzed the shear resisting properties of RC short columns based on the experimental studies of the members with shear span ratio less than 1.5, in which diagonal cracks have significant effect on the ultimate states of the members (Ref.5). Following the former study, in this paper, shear failure criteria of RC short columns with shear span ratio of about 1.5 to 2.0 are discussed based on the experimental study, and a truss theory to analyze the shear failure mechanisms is presented. As the results of this study, shear strength of RC short columns affected by bond characteristics in main bars and tie confining effect to keep ductile behaviour in the ultimate states are discussed mainly qualitatively.

EXPERIMENTAL STUDY

Shear failure mechanisms of reinforced concrete short columns are studied experimentally. Two series of tests are carried out using special specimens in which bond in main bars is removed at the end portions and ordinary specimens. The effects of bond in main bars on the shear resisting properties are discussed comparing the test results of each group of specimens.

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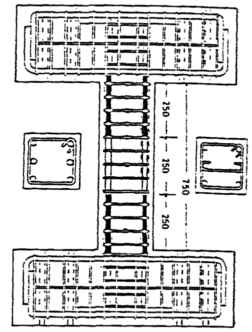
Fig. 1 shows the test specimen. The cross section (=25cm x 25cm), shear span ratio (=1.5) and main bar ratio(Pt=0.96%, 6-D16) are fixed for all specimens and as the test parameters; 1. the magnitude of constant axial force (15t, 30t, 45t), 2. tie ratio (0%, 0.57%, 0.85%, 1.28%) and 3. bond condition at the end portion of the member (ordinary=B, unbonded=U) are used. Table 1 shows the list of test specimens. Names of specimens are given in the order of (bond condition)-(tie ratio)-(axial load). Fig.2 shows the loading apparatus. Constant axial loads are applied by the upper jack and reversible shear loads are applied by the side jacks. Incremental shear loads are applied first by 3 cycles of loading of 3t, 6t, 9t followed by displacement controlled loading of 7.5mm, 15mm, 22.5mm. Horizontal and vertical displacements are measured by dial gauges and strains in ties and main bars are measured by wire strain gauges. Cracks are observed carefully during whole loading processes.



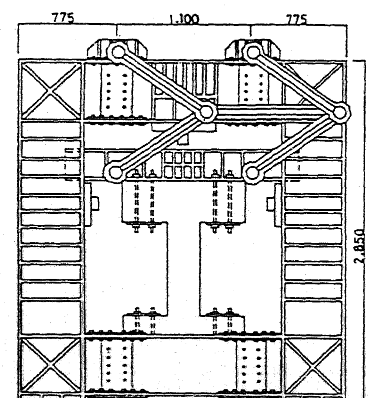
(unit : mm)
Fig.1 Test Specimen

Table 1 List of Test Specimen

B-0.85-15	U-0.85-15
B-1.28-15	U-1.28-15
B-0.00-30	U-0.85-30
B-0.57-30	U-1.28-30
B-0.85-30	U-0.85-45
B-1.28-30	U-1.28-45
B-0.57-45	
B-0.85-45	
B-1.28-45	

Figs.3 show the experimental results of typical crack patterns which appeared up to the loading cycle of 7.5mm displacement. Comparing (B-1.28-15) with (U-1.28-15) in figs.3, bending shear cracks are observed at the ends of the member in the former, while cracks are observed all over the member in the latter. Similar differences exist in the comparison of series B and series U for each condition. In the middle portion of (U-1.28-30), more notable splitting cracks are observed. It can be also pointed out that in series B, the bending cracks develop into the inside of the member and lead to bending shear cracks, and in series U, cracks occur in the core portion of the member independently of bending cracks.

Experimental results outlined above show the effect of bond properties in main bars on the shear failure of RC short columns as follows. For the specimen in which the bond at the end portions of main bars is removed, the middle



(unit : mm)
Fig.2 Loading Apparatus

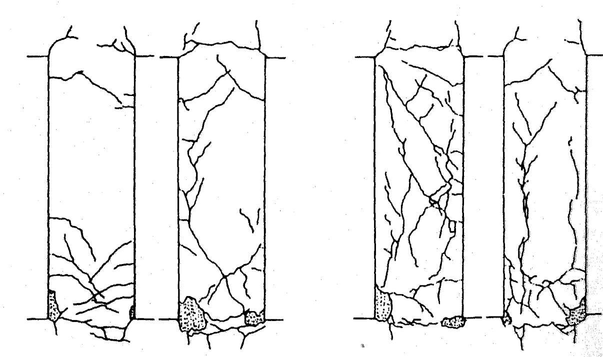


Fig.3 Crack Patterns

portion of the member is affected by shear force more clearly and the member yields in bond splitting failure. On the other hand, for ordinary specimens the effect of shear force in the middle portion is not clear, but the deterioration of the member in the vicinity of bending shear cracks near the ends of the member becomes peculiar. These tendencies described above are explained by considering truss mechanisms composed by main bars as a tension chord and concrete as an inclined strut as shown in Fig.4, that is, for bond removed specimens, the failure of the analogous truss depends on the bond failure near the intersection of strut and tension chord, and, for ordinary specimens, the failure of the member will be caused by the failure of concrete strut due to bending shear cracks.

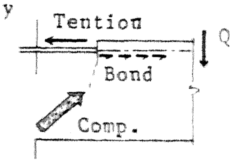


Fig.4 Truss Mechanism

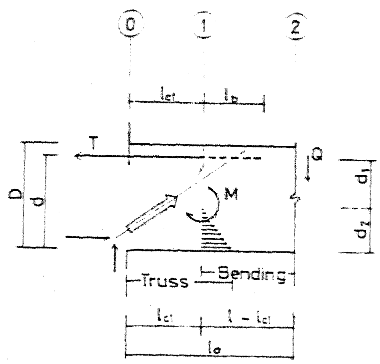
DISCUSSION ON THE SHEAR FAILURE MECHANISMS

Based on truss bearing mechanisms which are deduced from experimental results, and assuming the bond properties of main bars affected by bending cracks, shear failure mechanisms of RC short columns are discussed.

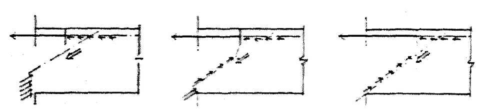
Truss Bearing Mechanisms

Figs.5 show a truss bearing mechanisms composed at the end portion of the member after the bond strength in main bars disappears. In Fig.5(a), point ⓪ shows the fixed end of the member, point ① shows the farthest bending cracks from point ⓪ and point ② shows the mid point. Between the section ⓪ - ①, bond in main bars disappears and the shear loads are resisted by truss mechanism. In this truss mechanism, main bars act as a chord, concrete acts as a strut and these elements are connected by development bond at the middle portion of the member. Thus the shear load Q is given by resisting moment M_1 at point ① as follows,

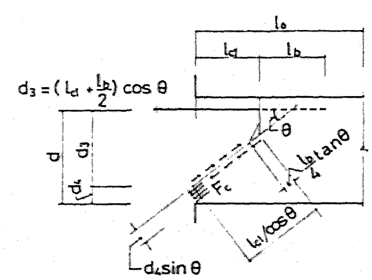
$$Q=\frac{M_1}{\ell_0-\ell_{c1}}\ . \qquad (1)$$



(a) Shear Load and Resisting Forces



(b.1) (b.2) (b.3)
(b) Resisting Patterns



(c) Stress Distribution

Fig.5 Forces Acting in Truss Mechanism

Assuming the bending tensile stress distributions and their variations affected by bending cracks, as shown in Figs.6, truss geometries are given. In Figs.6, bending tensile stresses resisted by concrete are translated to main bars after a bending crack occurs and the development bond acting between main bars and concrete affects the following behaviours. The bending crack at point ① occurs when line segment $\overline{CE}$ in Fig.6(e) becomes equal to the tensile strength of concrete, and the distance of X in Fig.6(e) is given as follows,

$$X = \frac{\ell_0 (N-1)\tau}{\tau_b - \tau}, \qquad (2)$$

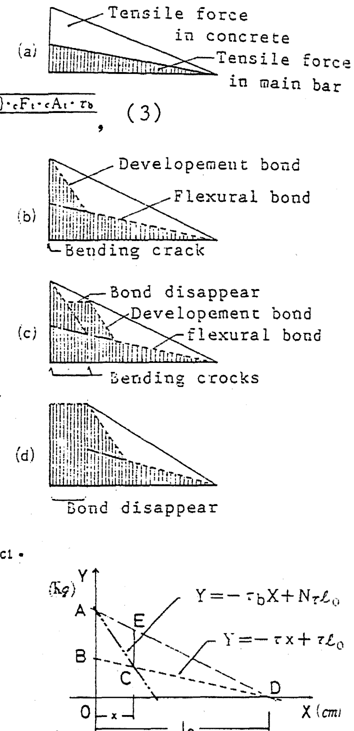


Fig.6 Bending Tensile Stresses

where

$$\tau = \frac{\{{}_cF_t \cdot {}_cA_t + \ell_0 (N-1)\tau_b\} - \sqrt{\{{}_cF_t \cdot {}_cA_t + \ell_0 (N-1)\tau_b\}^2 - 4\{\ell_0 (N-1)^2 + \ell_0 (N-1)\} \cdot {}_cF_t \cdot {}_cA_t \cdot \tau_b}}{2\{\ell_0 (N-1)^2 + \ell_0 (N-1)\}}, \qquad (3)$$

τ_b : strength of development bond,

${}_cF_t$: tensile strength of concrete,

${}_cA_t$: sectional area of concrete resisting tensile force,

N : ratio of tensile forces after bending crack and before bending crack,

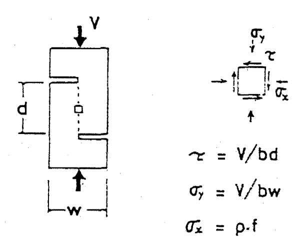
and

$$\{{}_cF_t \cdot {}_cA_t + \ell_0 (N-1)\tau_b\}^2 - 4\{\ell_0 (N-1)^2 + \ell_0 (N-1)\} \cdot {}_cF_t \cdot {}_cA_t \cdot \tau_b > 0.$$

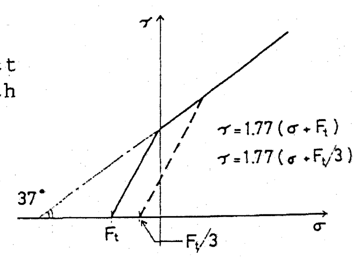
Assuming the bending deformation (rotation) at point 1 is given by pull out elongation in main bars related to development bond at this section, resisting moment M_1 is given provided the section keeps plane under bending moment. Shear force acting on the truss mechanism is given by eq.(1) substituting the maximum value of X by eq.(2) for l_{c1}.

Failure Criteria of Truss Strut Concrete

In analyzing the truss bearing mechanisms, it is assumed that the directions of the compressive forces are determined considering the center of the development bond area and the ultimate state of the member is arrived at by the failure of truss mechanisms and then shear strength is given by one condition among the following conditions; 1. yield in main bar, 2. deterioration of development bond and 3. failure in strut concrete. Failures in strut concrete are assumed as shown in Fig.5(b), where the slip failure along the truss compressive force directions combined with the compressive failure at the fixed end section occurs. Load resisting conditions of the strut concrete shown in Fig.5(b.2) and (b.3) are so called "direct shear" and the bearing strength of strut concrete should be obtained from such experiments as shown in Fig.7(a). In our former studies (Ref.5), the experimental study by Mattock et al. (Ref.6) was applied to analyze direct shear strength of concrete in RC short columns which resists the diagonal compression. The same procedure will be applied to analyze the strength of the strut concrete; that is, the shear stresses acting on the slip surfaces and the direct compressive stresses acting on the sections normal to the slip surfaces are introduced, and the failure criteria for these stresses are analyzed by applying slip failure envelopes (eq.4). After shear slip occurs and tie confining effects are caused, the stress circles are revised by tie confining stresses.



(a) Experiment



(b) Slip Failure Envelopes

Fig.7 Slip Failure Consideration

For the later stress circles, failure envelopes by eq.(5) are applied.

$$\tau=1.77(\sigma+F_t) \qquad (4) \qquad\qquad \tau=1.77(\sigma+F_t/3) \qquad (5)$$

Fig.7(b) shows the failure envelopes given by eqs.(4) and (5).

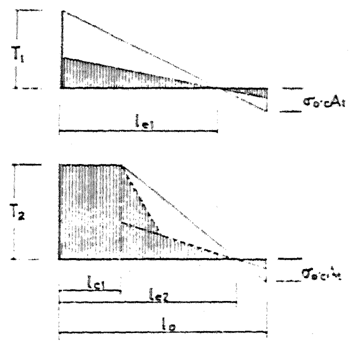
Effect of Axial Loads

In Figs.8, analytical assumptions for the axial stresses are shown. Fig.8(a) shows the bending tensile stress distributions. In this figure, it is shown that the analytical conditions for bending cracks are given by correcting the length of the member by the ratio of axial stress and bending tensile stress. Fig.8(b) shows the axial forces on the strut concrete. The strut concrete may fail in slip under the combined stress condition caused by axial load and truss compressive force. Equivalent lengths of the member for bending crack analysis are given as follows,

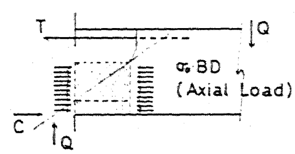
$$\ell_{e1}=\frac{T_1}{T_1+\sigma_0\cdot{}_cA_t}\cdot\ell_0 \ , \qquad (6)$$

$$\ell_{e2}=\frac{T_2}{T_2+\sigma_0\cdot{}_cA_t}\cdot\ell_0+\frac{\sigma_0\cdot{}_cA_t}{T_2+\sigma_0\cdot{}_cA_t}\ell_{c1} \ , \qquad (7)$$

where, $T_1=N_T\cdot l_{c1}$, $T_2=N_T\cdot(l_{e1}-l_{c1})$.



(a) Bending Stress Distribution



(b) Slip Failure Condition

Fig. 8 Effect of Axial Forces

In this condition, stress components are given as follows,

$$\sigma_1=(P_s+\sigma_0\cdot B\cdot D\cdot\cos\theta)/A_2, \ \sigma_2=\sigma_0\cdot B\cdot D\cdot\sin\theta/A_1, \ \tau=(P_s+\sigma_0\cdot B\cdot D\cdot\cos\theta)/A_1, \qquad (8)$$

where, $A_1=(\ell_{c1}/\cos\theta-\frac{\ell_b}{4}\tan\theta)\cdot B$, $A_2=A_1\cdot\tan\theta$.

SHEAR REINFORCEMENT FOR PLASTIC HINGE ZONE

The end portions of the member where truss bearing mechanisms are composed act as plastic hinges if the brittle failure is avoided. In the following, brittle failure criteria of the strut element and tie reinforcing effect to keep ductile properties are presented.

Shear Crack Condition

Geometrical conditions concerning slip failures at the strut element are given as shown in Fig.5(c). The angle between strut and member directions are given by the ratio of tensile force T in the chord (main bar) and shear load Q as such $\tan\theta$ = Q/T and substituting eq.(1) for Q and after some manipulations, $\tan\vartheta$ is given as follows,

$$\tan\theta=(d-\tfrac{1}{3}d_2)/(\ell_0-\ell_{c1}) \ . \qquad (9)$$

As the compressive force acting on the strut is given by $\sqrt{T^2+Q^2}$, a portion of the forces P_c resisted by normal compressive strength is given as follows,

$$Pc = Fc \cdot B \cdot d_4 \sin\theta \,, \qquad (10)$$

where, $d_4 = d - d_3 = d - (\ell_{c1} + \frac{\ell_d}{2})\cos\theta$.

The shear force acting on the strut is given as follows,

$$Ps = \sqrt{T^2 + Q^2} - Pc \qquad (11)$$

Stress components caused by Ps are given as follows (Fig.5.c),

$$\tau = Ps / A_1, \quad \sigma = Ps / A_2 \,, \qquad (12)$$

where, $A_1 = (\ell_{c1}/\cos\theta - \frac{\ell_b}{4}\tan\theta) \cdot B, \quad A_2 = A_1 \cdot \tan\theta$.

Shear failure should occur when the stress circle composed by eq.(10) becomes tangential to the failure envelope (4).

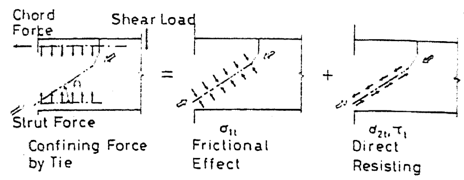


Fig.9 Effect of Tie Reinforcement

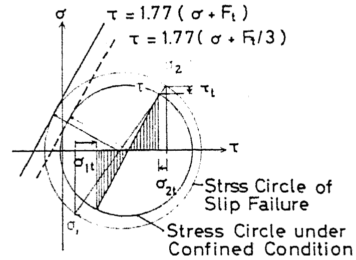


Fig.10 Stress Circle Variation due to Tie Confinement

Reinforcing Effect on the Strut Concrete due to Tie Confining Forces

Fig.9 shows tie reinforcing effect acting on the end portions of the member to confine strut concrete after shear failure. In this condition, as shown in Fig.10, the original stress circle is tangential to the failure envelope given by eq.(4) and the stress circle affected by tie confining stress is tangential to the failure envelope given by eq.(5). The confining stress Pw•fy (kg/cm^2) is given as a root of the following second order equation.

$$3\{\cos^4\theta + 2(2\times1.77^2+1)\cos^3\theta\cdot\sin\theta + \cos^2\theta\cdot\sin^2\theta + 4(1.77^2+1)\sin^4\theta\}\cdot(Pw\cdot fy)^2$$
$$+[\{3\sigma_1 - 3(2\times1.77^2+1) - 2\times1.77^2\times Ft\}\cos^2\theta + \{3(2\times1.77^2+1)\sigma_1 - 3\sigma_2 + 2\times1.77^2\cdot Ft\}\cos\theta\cdot\sin\theta \; - 12(1.77^2+1)\tau\sin^2\theta]\cdot(Pw\cdot fy)$$
$$+3\sigma_1^2 - 6(2\times1.77^2+1)\sigma_1\sigma_2 + 3\sigma_2^2 - 4\times1.77^2(\sigma_1+\sigma_2)\cdot Ft + 12(1.77^2+1)\tau^2 - \frac{4}{3}\times1.77^2 F_t^2 = 0 \qquad (13)$$

RESULTS OF THE ANALYSIS

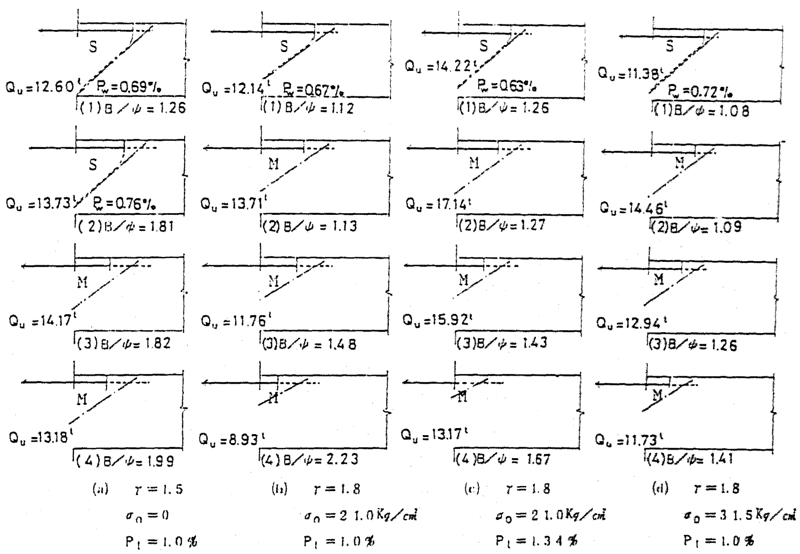
Results of the analysis using eqs.(10) through (13) are shown in Figs.11 and 12. In these figures, bending cracks, development bond areas and bending shear cracks caused by shear slip are presented for the ultimate state of the member. For the member which should fail in bending shear, confining stresses (Pw•fy) which are necessary to keep the ductile behaviours are also presented. Figs.11 (a) through (d) show the ultimate states of members affected by bond characteristics (B/ψ) for each case of axial load, main bar ratio and shear span ratio. Bond strength is chosen as fa=0.4•Fc. As shown in the figures, bending shear failures are observed for smaller B/ψ; B/ψ< 1.81 in Figs.(a), B/ψ< 1.12 in Figs.(b), B/ψ< 1.26 in Figs.(c) and B/ψ< 1.08 in Figs.(d).

It is also observed that the shear strength of the member is affected by bond strength in main bars; that is, for the members which should yield in main bars, shear strength of the member becomes larger with larger bond strength, and for the members which should fail in bending shear, shear strength becomes less with larger bond strength.

Figs.12 shows the border lines of the members which fail in bending shear for each condition of main bar ratio, axial load and shear span ratio. It is shown that the border of bending shear failure for B / ψ is affected by axial load and shear span ratio.

CONCLUSIONS

Based on the experimental study, truss bearing mechanisms are assumed and the shear failure criteria of RC short columns affected by shear span ratio, axial load and B / ψ are discussed. As the results of the analytical study, it



M : Main bar yielding
S : Shear failure

Fig.11 Analytical Results of Failure Condition

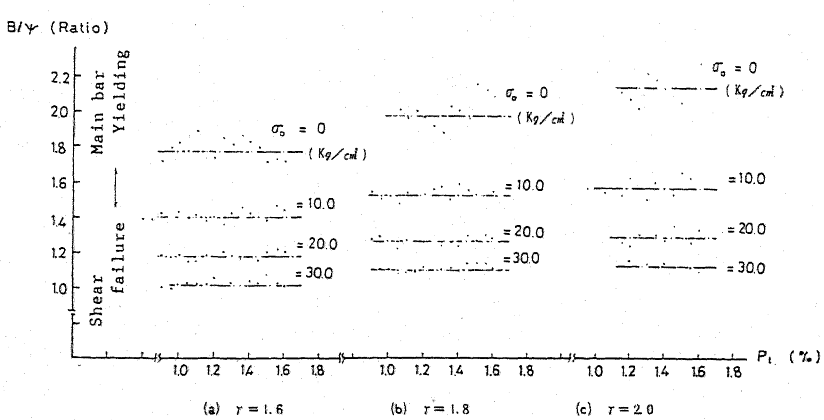


Fig.12 Classification of Causes of Failure

was shown that higher bond strength in main bars contributes to increase the shear strength of the member under the condition in which the member reaches to the ultimate states due to main bar yielding and that, in the other cases in which the member fails in bending shear cracks, higher bond strength in main bars lessens the shear strength of the member. It was also shown that the confining effect by tie reinforcement at the end portions of the member contributes to strengthen the strut concrete for the member which fails in bending shear and keeps the member ductile.

The purpose of this study is to deduce shear failure mechanisms and to describe the behaviours of the member, but, it is not yet satisfactory to verify the analogy and related assumptions. But it could be said that the behaviours of RC short columns explained by truss analogy agree with our knowledge of the former works and the truss mechanisms presented in this paper are considered to be effective to analyze the shear failure criteria of RC short columns of shear span ratio of 1.5 to 2.0.

Acknowledgement

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