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ANCHORAGE BEHAVIOR OF 90-DEGREE HOOKED BEAM BARS IN REINFORCED CONCRETE BEAM-COLUMN JOINTS

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ABSTRACT

In the author's previous paper, failure modes of beam bar anchorage with 90-degree bend used in reinforced concrete beam-column joints were classified into three types: a side split failure, a local compression failure and a raking-out failure, and an equation evaluating anchorage strength for raking-out failure was proposed. In this paper, the evaluation was modified in order to be applied accurately to the factors of column axial stress and cover concrete thickness, based on experimental results.

KEYWORDS

Anchorage failure; Anchorage strength; 90-degree hook; Beam-column joint; Development length; Raking-out failure; Column axial stress; Joint reinforcement Concrete compressive strength; Cover concrete thickness.

INTRODUCTION

When a main bar of a reinforced concrete member is anchored into an adjacent member, the bar end is usually arranged with a 90-degree bend in case the adjacent member has a short depth, as connections of girder to exterior column, beam to exterior girder, slab to wall, wall to transverse wall and so on. Resistance of bar anchorage with a 90-degree bend is shared by the straight portion and bent portion of the bar. The resistance at the bent portion increases in accordance with tensile stress level of the bar and thereby the concrete surrounding the bend may possibly fail if it is weeker than the bar tensile strength. Such concrete resistance surrounding the bend may be various among the connection ways mentioned above.

This paper focuses on the anchorage behavior of 90-degree hooked beam bars in exterior beam-column joints. In structural design, it has to be avoided that reinforced concrete exterior beam-column joints fail in shear and also in anchorage of beam bars. It is well known that the anchorage strength of 90-degree hook is increased by using a large development length of beam bar, a large radius of bar bend and a large thickness of cover concrete in a joint. However, the large development length can not be arranged in a corner column or at inside beam bars in double layer arrangement, the large radius of bar bend disturbs practical set-up of transverse beam bars and the large thickness of cover concrete is difficult for using of wide beams.

In the previous paper (Joh, Goto & Shibata, 1993), the authors classified the anchorage failure of 90-degree hooked bars in beam-column joints into three modes: side split failure, local compression failure and raking-out failure, and proposed the equations to evaluate the anchorage strengths for these failure modes. The equation for the raking-out failure mode, which was named first by the authors, was estimated by using the factors of concrete strength, joint lateral reinforcement, column axial force and so on. The purpose of this paper is to establish an anchorage strength evaluation for the raking out failure mode of 90-degree bar bend in a beam-column joint which has more various influence on the strength. Therefore, experimental studies to elucidate unknown factors influencing the anchorage strengths were carried out.

CLASSIFICATION OF ANCHORAGE FAILURE MODES

Failure modes of beam bar anchorage with a 90-degree bend in a beam-column joint are classified into three types as shown in Fig. 1 based on the author's and others' previous experimental test results (e.g. Pinc, Watkins & Jirsa, 1977). One is a side split failure as concrete covers beside bar bents in a joint spall out with a dish shape individually at the both sides in the joint. Another is a local compression failure as a small part of concrete just inside bar bend crushes individually at each beam bar. The other is a raking-out failure as a concrete block inside a L-shape bar layer is raked out toward to the beam side caused by providing of many beam bars and/or short development length in the joint, and all beam bars lost their resistance at same time.

The raking-out failure can occur even if neither the side split failure is prevented by thick cover concrete nor the local compression failure is prevented by a large bend radius. This failure mode is similar to shear failure of a beam-column joint in relation curve of total bar force and bar displacement. However, the shear failure is caused by compression failure of diagonal concrete strut in a joint. In contrast, the raking-out failure is independent of the compression failure. The raking-out failure is different from so-called cone-shaped failure because the crack plane of the raking-out failure runs across the overall joint width and one part of the cracks appears along the bar bend and tail. There are few studies on the behavior of this failure mode (Nishiyama & Minami, 1986). The previous paper (Joh, Goto & Shibata, 1993) shows the detail of the three failure modes and the evaluation of anchorage strengths for the three modes.

Many structural design codes specify minimum requirements of development length, radius of bend and thickness of side-cover concrete according to strengths of materials and bar diameter in order to avoid anchorage failure of the raking out mode, local compression mode and side split mode, respectively. However it becomes difficult to satisfy these requirements in a beam bar anchorage with a 90-degree bend in a beam-column joint. Especially this difficulty appears in a joint at lower stories of a tall building, in a cast-in-situ concrete interior joint between precast beams, or at inner layer bars of multi-bar-layer arrangement in a joint.

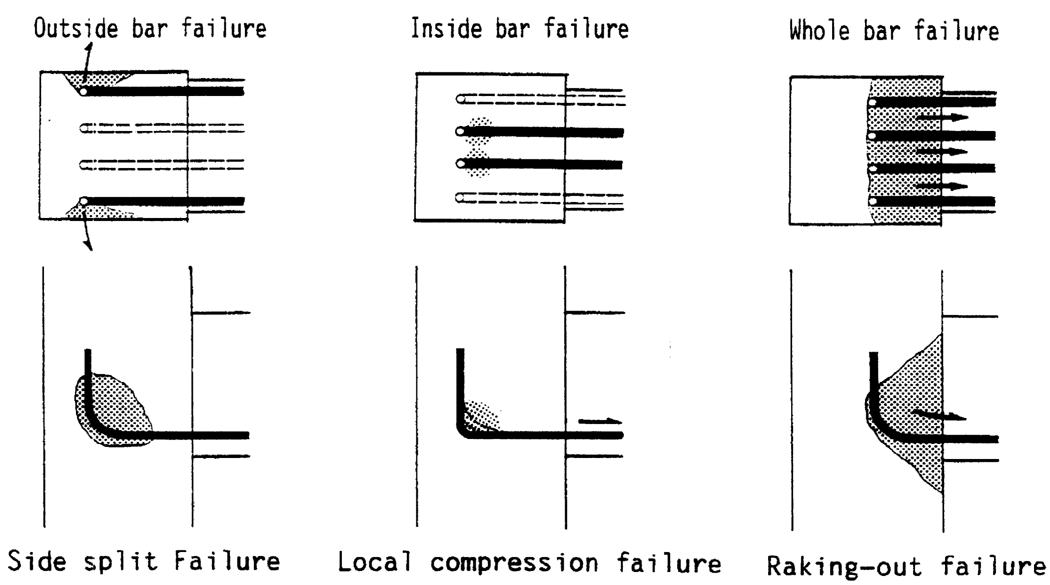

Fig. 1 Classification of anchorage failure modes

EXPERIMENTAL WORK

Test Specimens

Specimens had an exterior beam-column joint form with no beam concrete nor beam longitudinal bar in the compression side to simplify their production as shown in Fig. 2. Beam longitudinal bars in the tension side had identical characteristics; (1) development length L_{dh} (distance from beam end to tail center) was 200 mm equaled to a half of column depth, (2) inside radius of the bar bend was 3 d_b (d_b: nominal bar diameter of 19 mm), (3) tail length after the bar bend was 12 d_b, (4) spacing of bar (distance between bar centers) was 57 mm equaled to 3 d_b, (5) number of bars was four with single layer, (6) the bars were a high-strength threaded deformed bar. These bars were intersected at the beam-column joint portion in the column with a story height of 1750 mm, which was about half scale of an actual structure. The other identical configuration of the specimens was column depth = 400 mm, moment arm of beam = 328 mm and lateral reinforcement ratio p_w in joint = 0.2%. In the previous paper, the effect of these unvaried factors on the anchorage behavior was already investigated.

The specimens consisted of two test series including three variables as shown in Table 1. The first series (hereafter referred as LA5 series) focused on the effect of thickness of cover concrete C_o (distance between column side face and center of beam corner bar). The second series (hereafter referred as LA8 series) focused on the effect of column axial stress σ_o combined concrete compressive strength σ_B (column axial stress ratio $\mu = \sigma_o / \sigma_B$). The prototype specimen LA1-1 had C_o = 64.5 mm (a joint width b_j = 300 mm), σ_o = 0 and σ_B = 300 kg/cm^2. The thickness of cover concrete was the minimum value obtained when beam bars were passed inside of column corner bars and was calculated so

Table 1. Variations of specimens

Specimen	Axial stress ratio	Concrete comp. strength
LA1-1*	0 %	300kg/cm^2
LA8-1	22 %	400 〃
LA8-2	33 %	400 〃
LA8-3	8 %	300 〃
LA8-4	26 %	300 〃
LA8-5	3 %	300 〃
LA8-6	15 %	300 〃
LA8-7	20 %	600 〃
LA8-8	6 %	600 〃

Specimen	Cover conc. thick.
LA1-1*	64.5 mm
LA5-1	89.5 mm
LA5-2	114.5 mm
LA5-3	133.0 mm
LA5-4	200.0 mm
LA5-5	264.5 mm

*prototype specimen

Table 2. Properties of materials

Diameter of bar	σ_y kg/cm^2	ε_y μ	σ_{MAX} kg/cm^2	ε_B %
D19	6830	3380	9000	12.8
D16	3750	2160	5600	22.5
6 ϕ	3450	1760	4670	23.7

Specimen	σ_B kg/cm^2	ε_u μ	$E_{1/3}$ x10^{5}kg/cm^2	$E_{2/3}$ x10^{5}kg/cm^2
LA1-1	315	2750	2.18	1.96
LA8-5	260	2600	2.26	1.88
LA8-8	566	2970	3.00	2.65

$E_{1/3}$, $E_{2/3}$: Young's modulas

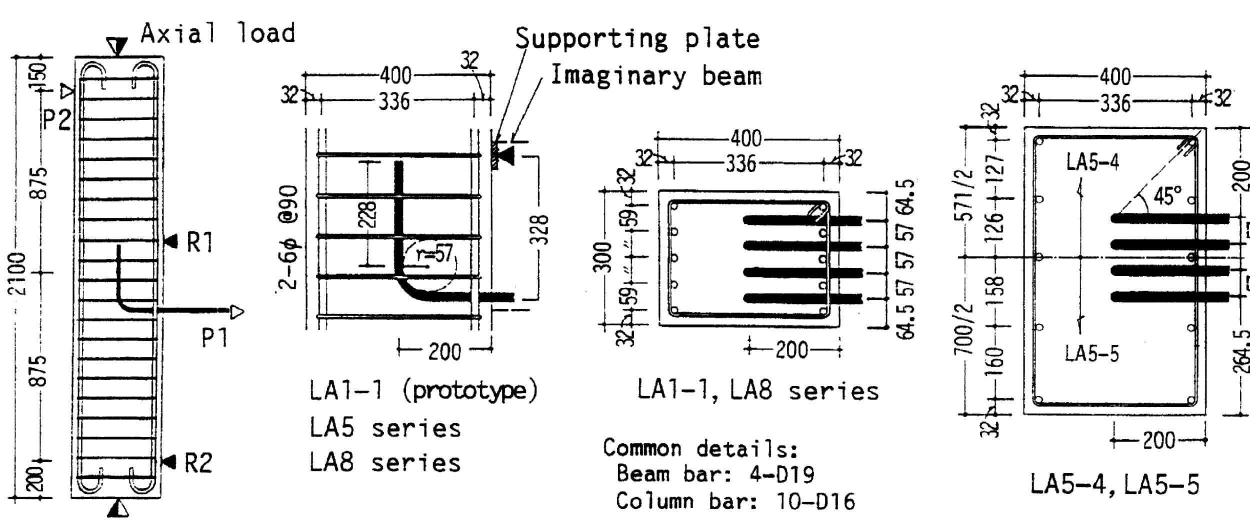

Fig. 2 Details of specimens

as not to fail in the side split mode. Some details are shown in Fig. 2. Mechanical properties of materials are shown in Table 2. The aggregate was crushed-stone with a maximum size of 13 mm matched with the scale of specimens. High strength beam bars were used in all specimens to avoid their yielding, but column bars and hoops were normal.

Instrumentation and Loading

Tensile load P_1 was applied horizontally on the beam bars by a 200 tonf oil-jack. Reaction R_1 was supported at the compression zone of imaginary beam cross section by a steel plate with its height of one-fifth beam depth, and reaction R_2 was supported at the bottom of column. The other load P_2 was applied on the top of column by a 50 tonf oil-jack controlled so as to generate the same shear force in both columns. The four beam bars were controlled so as to distribute the same pull-out displacement in order to simulate actual beam bar conditions, consequently tensile loads were slightly different from each other. LA8 series specimens were subjected to constant axial force vertically by another oil jack using a pair of loading steel beams on the top and bottom of the column and four tie rods between the loading beams.

EXPERIMENTAL RESULTS AND DISCUSSION

Behavior of Cracks and Failure

Column Axial Stress: Fig. 3 shows schematic crack patterns at the final loading stage of some specimens and thick lines express the cracks opened severely at maximum strength stage (hereafter referred failure crack or failure plane). The crack patterns were different in each specimen, but the common failure plane in all specimens consisted of three main cracks: a sloped crack which appeared from the bar bend portion into the lower column with inclinations of 30-degrees to 60-degrees; a vertical crack along the bar tail; and a diagonal crack running toward the compression zone in the joint. In all specimens, the concrete block with a shape of trapezoidal beam formed by these three cracks was raked out and the anchorage failed finally without yielding in beam bars.

The angles of cracks on the side faces of lower column and joint became steeper with increase of column axial force. In case of the column axial stress ratio under 8 % (Fig. 3a & 3b) and more than 8% (Fig. 3c), the failure cracks appeared in the lower column and in the joint, respectively. Every cases failed in slippage on the cracks between the trapezoidal block and the column or joint concrete.

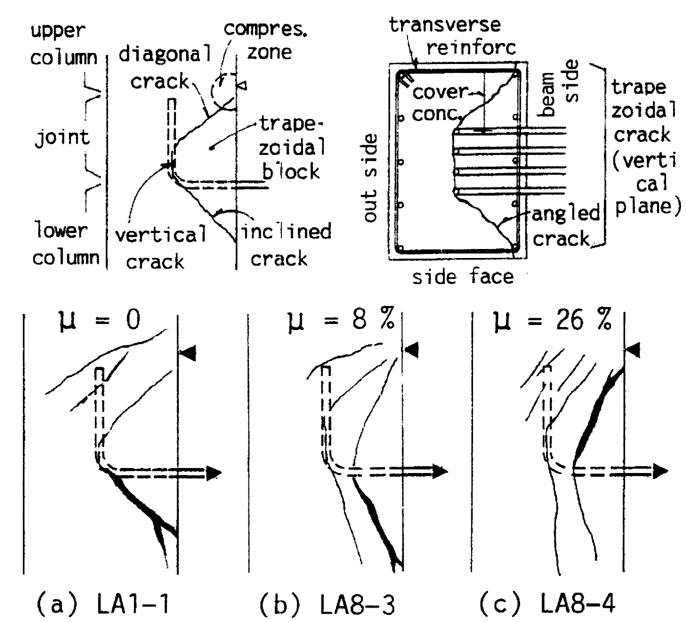

Fig. 3 Examples of crack patterns of LA8 series

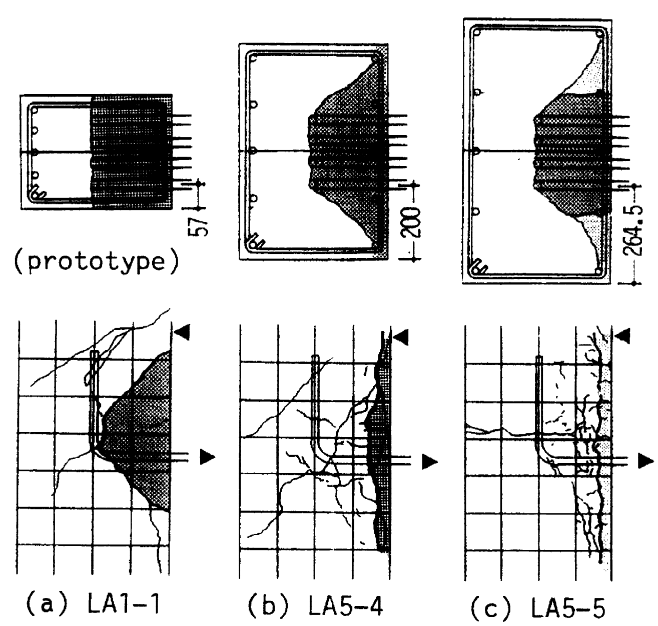

Fig. 4 Examples of crack patterns of LA5 series

Thickness of Side Cover Concrete: Fig. 4 shows crack patterns at the final loading stage appeared on a column side face and in a horizontal cross section at the beam bar level. The crack zone on the column face was restricted to narrow portion along the column edge near beam side according to increase of the thickness of cover concrete, and the failure crack pattern changed from a deep trapezoid to a shallow one. The failure crack in the cross section appeared on a approximately straight line in case of small thickness like the prototype specimen. However, the failure crack changed to a trapezoidal shape with angles of about 40-degree according to increase of the thickness as shown in Figs. 4b & 4c. These angled failure cracks in the beam-column joint which had the thickness of side cover concrete larger than the development length L_{dh}, as Specimen LA5-5, crossed the column face of beam side. Such crack pattern supposed that the lateral joint reinforcement would reduce the effect on anchorage strength.

Evaluation of Anchorage Strength with Raking-Out Failure

Authors' Previous Equation: The authors previously proposed the evaluation of anchorage strength with raking-out failure. Main influence factors are considered to yield an anchorage strength ${}_{cal}T_u$ in the following equation:

$$ {}_{cal}T_u = T_c + T_w \qquad \text{(Eq. 1)} $$

where, $T_c = 2\, L_{dh} \cdot b_e \cdot \sigma_t\, (1 + 6.32\, \sigma_o / \sigma_B) / \sin\theta$

$T_w = k_w \cdot a_w \cdot \sigma_{wy}$

$L_{dh} = L + r + d_b/2 = l_{dh} - d_b$

a_w : total sectional area of lateral reinforcement passing through failure planes

b_e : effective joint width $= b_j - n \cdot d_b$, n : number of beam bars

k_w : coefficient of effective lateral reinforcement = 0.7

σ_t : concrete tensile strength = sqrt σ_B

σ_o : column axial stress, but not larger than $\sigma_B/6$

σ_{wy}: yield stress of lateral reinforcement

θ : strut angle

[unit]: hereafter used [tonf] to force, [kgf/cm^2] to stress and [cm] to length

It could be assumed that the failure plane had appeared along two lines with had 45-degree angles of elevation and depression from the intersection of axes of beam bar and tail bar as shown in Fig. 5, and that the anchorage strength consisted of concrete and

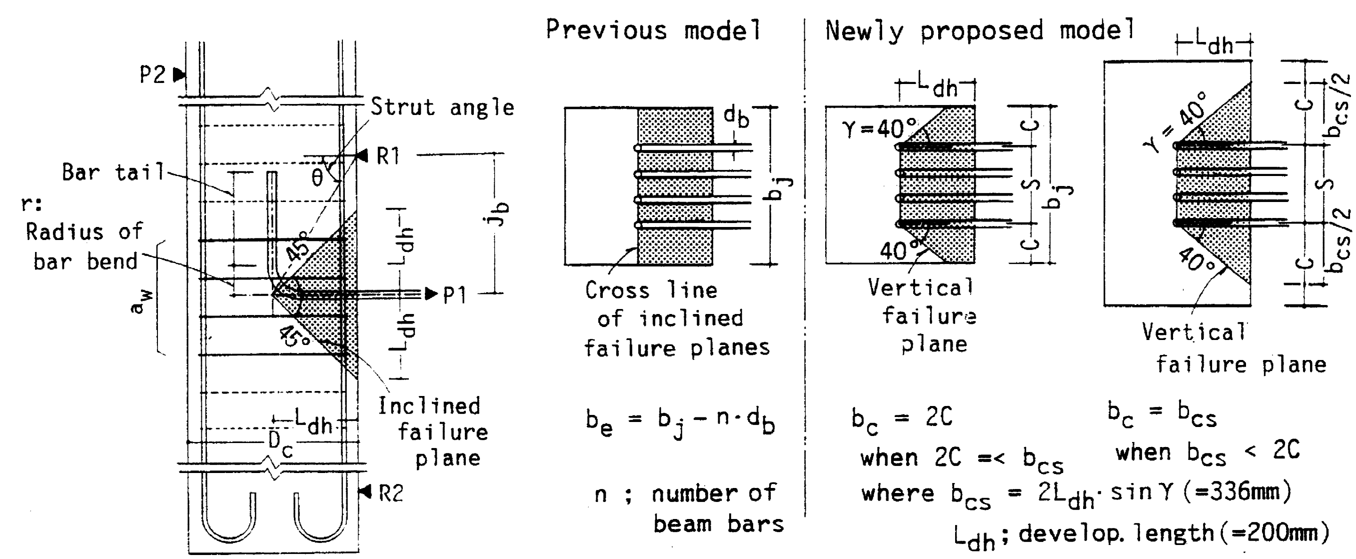

Fig. 5 Model of failure planes and definition of strut angle

reinforcement resistances at the failure plane. The concrete resistance could be evaluated as the horizontal contribution of cracking strength along the failure planes, and it was equal to the tensile strength perpendicular to the vertical plane with a height of twice L_{dh}. The concrete resistance T_c was divided by sin because the component of shear resistance transmitted directly to the compression zone through the diagonal concrete strut increased with the decrease of strut angle. The reinforcement resistance could be evaluated as the total force generated in the lateral reinforcement passing through the failure planes. Effective coefficient k_w was used to express the average stress of reinforcement provided within the failure planes. The additional concrete resistance was assumed to increase proportionally to σ_o up to $\sigma_o = \sigma_B/6$, and was constant when σ_o was larger than $\sigma_B/6$. However, this assumption was not so accurate because it was resulted from only two specimens.

Effect of Column Axial Stress: Test results shown in Table 3 say that the column axial stress σ_o increased the anchorage strength T_u, but the strengths had no difference among the specimens with similar concrete strength when they subjected to the axial stress larger than some value. This means there is a superior limit of column axial stress in strength increasing. It was estimated that the superior limit σ_{os} was $0.08\,\sigma_B$, which was the minimum ratio of σ_o/σ_B among the specimens reached σ_{os} obviously. Specimens LA8-5 and LA8-8 did not reach such superior limit.

It was defined that an experienced strength increasing coefficient by axial stress, ${}_{exp}k_N$, was the ratio of observed strength ${}_{exp}T_u$ to calculated one without axial stress, ${}_{cal}T_{uo}$. This calculated strength was obtained from Eq. 3 which was led from Eq. 1 by substituting zero into σ_o and actual values into the variables except σ_B. The values of ${}_{exp}k_N$ and σ_{oe} had a linear relation as shown in Fig. 6 and the relation could be expressed by Eq. 4, where effective column axial stress σ_{oe} was column axial stress σ_o when σ_o was not larger than $0.08\,\sigma_B$ and was $0.08\,\sigma_B$ when σ_o was larger than $0.08\,\sigma_B$.

Tat Table 3. Test results of LA8 series at ultimate stage

Specimen	σ_B kgf/cm^2	σ_o kgf/cm^2	σ_{os} kgf/cm^2	σ_{oe} kgf/cm^2	${}_{exp}T_u$ tonf	${}_{cal}T_{uo}$ tonf	${}_{exp}k_N$	${}_{cal}k_N$	${}_{exp}k_N$ / ${}_{cal}k_N$
LA8-1	380	85.1	30.4	30.4	46.2	28.0	1.65	1.63	1.01
LA8-2	398	130.8	31.8	31.8	47.2	28.5	1.66	1.66	1.00
LA8-3	305	25.5	24.4	24.4	38.3	25.7	1.49	1.51	0.99
LA8-4	292	74.9	23.4	23.4	38.2	25.3	1.51	1.49	1.01
LA8-5	260	8.7	20.8	8.7	31.0	24.2	1.28	1.17	1.09
LA8-6	279	42.6	22.3	22.3	36.8	24.9	1.48	1.47	1.01
LA8-7	544	108.8	43.5	43.5	62.0	32.2	1.92	1.91	1.01
LA8-8	567	34.0	45.4	34.0	52.5	32.8	1.60	1.68	0.95

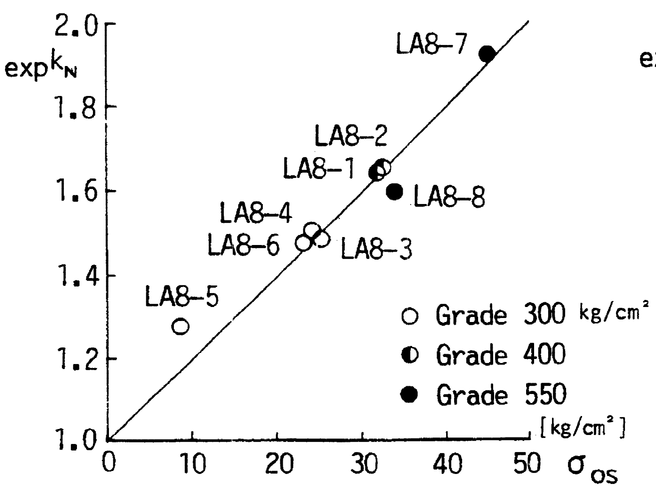

Fig. 6 Relation of ${}_{exp}k_N$ vs. σ_{os}

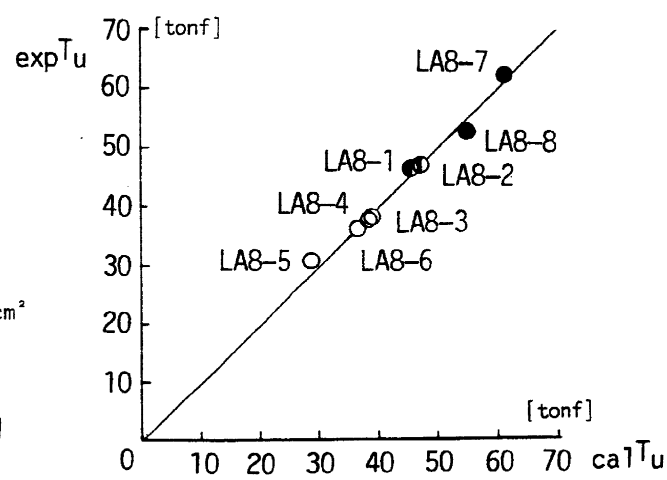

Fig. 7 Experimental vs. calculated results

$$_{cal}T_u = {}_{cal}k_N \cdot {}_{cal}T_{uo} \quad \text{(Eq. 2)}$$

$$_{cal}T_{uo} = T_c + T_w = 1.11\ \mathrm{sqrt}\ \sigma_B + 6.34 \quad \text{(Eq. 3)}$$

$$_{cal}k_N = 1 + 0.020\,\sigma_{oe}, \quad \text{where } \sigma_{oe} = [\,\sigma_o,\ 0.08\,\sigma_B]\mathrm{min} \quad \text{(Eq. 4)}$$

Fig. 7 shows the relationship between $_{cal}T_u$ and $_{exp}T_u$ and the calculated values estimated by Eq. 4 were in very nice agreement with the experiments.

Effect of Thickness of Cover Concrete: The test results of LA5 series and the prototype LA1-1 are shown in Table 4. In order to cancel the difference of concrete strengths among the specimens, the modified anchorage strength $_{exp}T_{um}$ corresponding to the concrete strength 315kgf/cm² of prototype LA1-1 were calculated by Eq. 1' led from Eq. 1. It was assumed that the vertical failure planes in the cover concrete appeared with an angle of 40-degree shown in Fig. 5b. Whenever these failure planes crossed the side faces of column as the specimens from LA1-1 to LA5-3 had the total thickness b_c less than the calculated effective total thickness $_{cal}b_{ce}$ of 33.6 cm ($=L_{dh}\sin 40°$), the calculated strength contribution of lateral joint reinforcement, $_{cal}T_w$, was considered as being constant because total number of the reinforcement passing through the entire (vertical and inclined) failure planes was much the same. The observed total tension forces of such joint reinforcement, $_{exp}T_w$, were nearly constant, and the mean value was 6.53 tonf and was close to $_{cal}T_w$ of 6.34 tonf calculated by Eq. 3.

Fig. 8 shows that $_{exp}T_{um}$ increased linearly according to increase of b_c under $_{cal}b_{ce}$ because the strength contribution of concrete T_c increased. By applying a regression analysis to this behavior, Eq. 5 expressing $_{cal}T_{um}$ could be obtained. However not only

Table 4. Test results on effect of cover concrete thickness at ultimate stage

Specimen	σ_B kgf/cm²	b_c cm	$_{exp}T_w$ tonf	$_{exp}T_u$ tonf	$_{exp}T_{um}$ tonf	$_{cal}T_{um}$ tonf	$_{exp}T_{um}$ / $_{cal}T_{um}$
LA1-1	315	12.9	5.79	23.8	23.8	24.8	0.96
LA5-1	335	17.9	6.90	32.1	31.4	27.8	1.13
LA5-2	355	22.9	5.03	27.9	26.3	30.8	0.85
LA5-3	320	26.6	8.40	35.1	34.9	33.1	1.05
LA5-4	251	40.0	2.41	31.6	35.2	34.2	1.03
LA5-5	238	52.9	-0.02	26.5	30.9	31.8	0.97

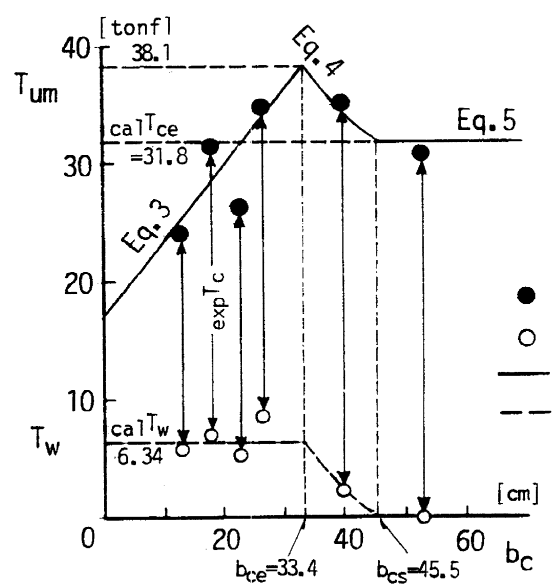

Fig. 8 Effect of b_c on anchorage strength

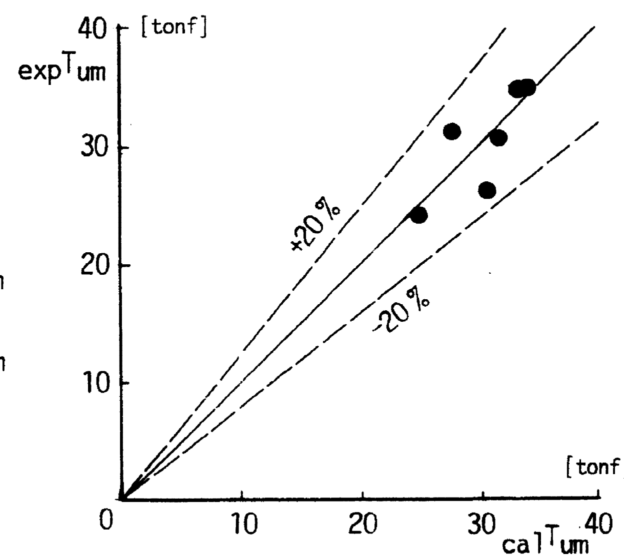

Fig. 9 Experimental vs. calculated results

${}_{exp}T_{um}$ but also ${}_{exp}T_w$ decreased according to increase of b_c in range larger than ${}_{cal}b_{ce}$. The differences between ${}_{exp}T_{um}$ and ${}_{exp}T_w$ for Specimens LA5-4 & -5 were almost constant and the average difference was 31.8 tonf. This means concrete contribution ${}_{exp}T_{cs}$ reached the superior limit of 31.8 tonf. Since the maximum anchorage strength could be estimated at 38.1 tonf which was the sum of ${}_{exp}T_{cs}$ 31.8 tonf and ${}_{exp}T_w$ 6.3 tonf, the observed effective total thickness ${}_{exp}b_{ce}$ of 33.4 cm was obtained as the value at which the maximum strength appeared on Eq. 5. The value of ${}_{cal}b_{ce}$ was almost the same as ${}_{exp}b_{ce}$, and it proved appropriateness of the assumption on formation of the vertical failure planes. The reduction of ${}_{cal}T_w$ according to increase of b_c was led as a function passing at a observed value of LA5-4 and at the coordinates (33.4 cm, 6.34 tonf) and ${}_{cal}T_w$ was estimated to be zero in range larger than b_{co} of 45.5 cm at which this function crossed the axis of abscissa. Finally the reduction of ${}_{cal}T_{um}$ was expressed by the sum of this function and ${}_{exp}T_{cs}$ of 31.8 tonf, shown as Eq. 6.

$${}_{exp}T_{um} = {}_{exp}T_u - 1.11(\mathrm{sqrt}\ \sigma_B - \mathrm{sqrt}\ 315)\ b_j/30 \quad \text{(Eq. 1')}$$

$${}_{cal}T_{um} = 17.1 + 0.6\ b_c, \quad \text{when } b_c =< 33.4 \text{ cm} \quad \text{(Eq. 5)}$$

$${}_{cal}T_{um} = (796/b_c) + 14.3, \quad \text{when } 33.4 < b_c < 45.5 \quad \text{(Eq. 6)}$$

$${}_{cal}T_{um} = 31.8, \quad \text{when } 45.5 =< b_c \quad \text{(Eq. 7)}$$

Fig. 8 shows the relation of the total cover concrete thickness vs. the calculated or observed values of modified total tension force and their contributions of joint lateral reinforcement. The calculated values expressed by Eqs. 5 - 7 corresponded to the experienced values and the ratio of experienced values to calculated ones in T_{um} ranged from 0.85 to 1.13 as shown in Table 4 and also in Fig. 9.

CONCLUSIONS

Column type specimens with 90-degree hooked beam bars in their beam-column joint portions were prepared to modify the authors' previous evaluation of the anchorage strength with the raking-out failure. From the experimental results obtained by the loading tests of the specimens with three variables: column axial stress; thickness of side cover concrete and concrete compressive strength, the conclusions are remarked as followings:

1)The anchorage strength increased severely according to increase of column axial stress. However the effect of the column axial stress on the strengthening had a superior limit and the limit was 0.08 times concrete compressive strength.

2)The modification method for the effect of column axial stress was proposed. It can be applied accurately in range of the axial stress ratio less than about one third.

3)The failure crack with a trapezoidal shape in horizontal cross section appeared at around the beam bar level in case of large thickness of cover concrete. The angled failure plane along the legs of trapezoid had about 40-degree against beam bar, therefore the effect of lateral joint reinforcement on the strength reduced when the angled failure plane crossed the column face of beam side.

4)The relations of the thickness of cover concrete vs. the contributions of concrete or joint reinforcement to anchorage strength was clarified. In the future study, these relations has to be generalized and to be incorporated in the evaluation.

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