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EFFECTIVE UTILIZATION OF HIGH TENSION STEEL BAR AS SHEAR REINFORCEMENT OF RC MEMBERS

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SUMMARY

Many cases are found where the high tension steel bar called ULBON is used as shear reinforcement in a critical shear design of RC members. Recently, it is officially approved[1)] that the full strength of ULBON can be evaluated when the ultimate shear strength of RC members is estimated in a bearing capacity design method enforced in Japan. In this study, the experiments were introduced, which were carried out to find the condition that the strength of high tension steel would be more effectively utilized. The experimental results including that of past studies were synthetically discussed and the design procedure was proposed.

INTRODUCTION

High tension steel bar with yield strength over 13000kg/cm^2 (186ksi) has been applied to many RC structures as shear reinforcement. Over hundreds of specimens were tested to investigate the shear reinforcing effect and the ductility under reversed cyclic loading. As a result, the permissible tensile stress under temporary loading 6000kg/cm^2 and the upper limit of shear reinforcement ratio 0.6% have been officially approved. Recently those are requested to be increased, because the critical shear design is often needed in a design of such a high-rise RC buildings. The objective of this study is to investigate the condition that the strength of high tension steel will be more effectively utilized and to propose the estimation method of the ultimate shear strength of RC members laterally reinforced by high tension steel bar in a bearing capacity design.

EXPERIMENT OF BEAMS

The beam specimens are listed in Table 1. The parameters are the diameter of shear reinforcement and the strength of concrete. The detail of specimen is shown in Fig.1. The cross section (b x D) is 18cm x 40cm and the shear span to depth ratio (a/D) is 1.5 in all specimens. The specimens were longitudinally reinforced by five deformed bars of 22mm in nominal diameter both in tensile and compressive side, whose yield strength was enhanced up to 8140kg/cm^2 to make shear failure precede. The shear reinforcement was shaped spirally with 10cm intervals. The same value of load (P) was monotonously applied on both of right and left stubs and the antisymmetric bending moment condition was produced as shown in Fig.2.

The shear failure occurred in all specimens before yielding of longitudinal bars. The relations between the ultimate shear strength(τ_u) and $P_w \cdot {}_w\sigma_y$ are shown in

Table 1 Characteristics of Beam Specimens

No.	Specimen's name	D_s *1 (mm)	P_w *2 (%)	${}_w\sigma_y$ *3 (kg/cm^2)	$P_w \cdot {}_w\sigma_y$ (kg/cm^2)	${}_c\sigma_B$ *4 (kg/cm^2)	Q_{uexp} *5 (ton)	τ_{uexp} *6 (kg/cm^2)
1	B-210- 0		0.0	0	0.0	208	11.6	21.7
2	B-210- 6.0	6.0	0.31	13600	42.7	208	24.6	46.0
3	B-210- 7.4	7.4	0.44	14500	64.4	208	28.5	53.1
4	B-210- 9.2	9.2	0.71	14300	101.7	208	32.8	61.3
5	B-210-11.0	11.0	1.00	14600	146.0	208	36.3	67.8
6	B-360- 0		0.0	0	0.0	383	17.6	32.9
7	B-360- 4.1	4.1	0.15	14200	20.9	383	30.8	57.5
8	B-360- 5.1	5.1	0.23	14500	32.9	383	35.5	66.3
9	B-360- 6.0	6.0	0.31	13600	42.7	383	37.3	69.7
10	B-360- 7.4	7.4	0.44	14500	64.4	383	37.5	70.0
11	B-360- 9.2	9.2	0.71	14300	101.7	383	46.9	87.6
12	B-360-11.0	11.0	1.00	14600	146.0	383	52.0	97.1
13	B-570- 0		0.0	0	0.0	549	19.4	36.2
14	B-570- 4.1	4.1	0.15	14200	20.9	549	30.6	57.1
15	B-570- 6.0	6.0	0.31	13600	42.7	549	42.5	79.4
16	B-570- 7.4	7.4	0.44	14500	64.4	549	49.5	92.4
17	B-570- 9.2	9.2	0.71	14300	101.7	549	56.0	104.6
18	B-570-11.0	11.0	1.00	14600	146.0	549	60.5	113.0

*1 Diameter of shear reinforcing bar
*2 Shear reinforcement ratio
*3 Yield strength of shear reinforcement (0.2% offset)
*4 Compressive strength of concrete cylinder (10cmφ x 20cm)
*5 Maximum shear force in experiment
*6 Shear stress at maximum shear force (Q_{uexp}/bj, j=29.8cm)

Fig.3. Here, P_w is the shear reinforcement ratio and ${}_w\sigma_y$ is the yield strength of shear reinforcement. τ_u increases according to the increase of $P_w \cdot {}_w\sigma_y$. The ratio of increase of τ_u is, however, in slow with the increase of $P_w \cdot {}_w\sigma_y$. Figure 4 shows the strain distribution of shear reinforcement at shear failure in case of B-570 series. The yielding of shear reinforcement was observed in B-570-4.1 and B-570-6.0, but not observed in B-570-9.2 and B-570-11.0. In case of B-570-7.4, the shear reinforcement was just before yielding. In the region of small $P_w \cdot {}_w\sigma_y$, the bending shear crack appeared at right and left end of the beam, and then, they developed to the splitting crack along the longitudinal bars. The splitting cracks came to open and ultimately shear reinforcement yielded. In the region of large $P_w \cdot {}_w\sigma_y$, the opening of the splitting crack was resisted by shear reinforcement. Ultimately the diagonal crack appeared and the shear compression failure of concrete occurred. That is, in the region of small $P_w \cdot {}_w\sigma_y$, the full strength of high tension shear reinforcement was displayed at shear failure and in the region of large $P_w \cdot {}_w\sigma_y$ over some value, the shear compression failure of concrete goes ahead without full working of high tension shear reinforcement. In this experiment, it was judged that such boundary value of $P_w \cdot {}_w\sigma_y$ was increased and the strength of high tension steel could be more effectively utilized by using high strength concrete.

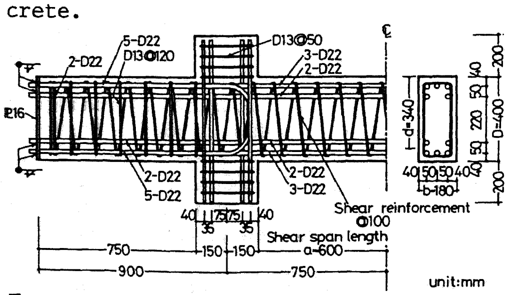


Fig. 1 Detail of Beam Specimen

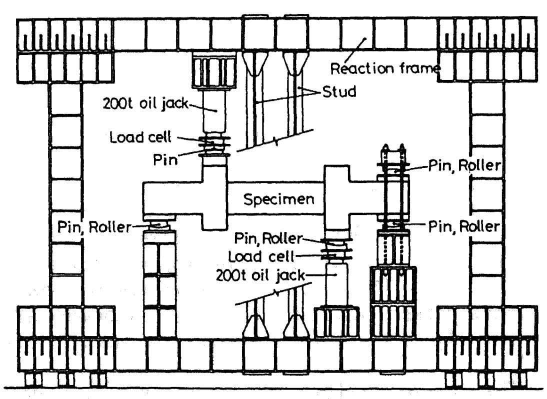


Fig. 2 Loading Apparatus for Beam Specimen

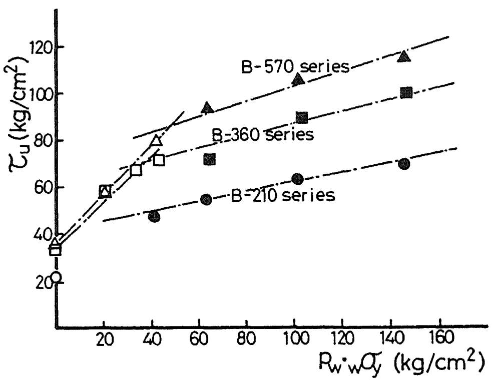


Fig. 3 $\tau_u - P_w \cdot {}_w\sigma_y$ Relations of Beams

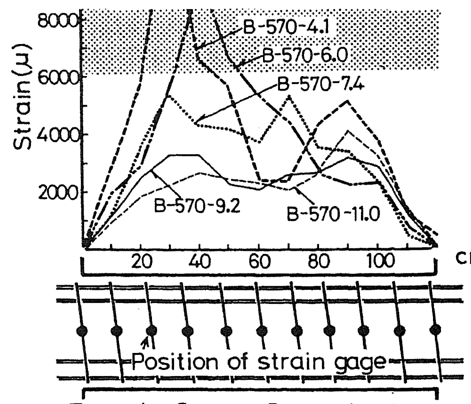


Fig. 4 Strain Distribution on Shear Reinforcement

EXPERIMENT OF COLUMNS

It was found that the shear compression failure should be avoided and high strength concrete should be used in order to utilize the strength of high tension steel more effectively in case of beams. However, in case of columns it is expected that the axial force encourages the shear compression failure of concrete. Then, the effect of axial force was experimentally discussed.

The column specimens are shown in detail in Fig.5 and are listed in Table 2. All the specimens had same cross section (b x D=15cm x 27cm), and same shear span to depth ratio (1.5). The interval of shear reinforcement was fixed to be 7.36cm. Five longitudinal deformed bars of 13mm in nominal diameter, whose yield strength was enhanced up to 8790kg/cm^2 , were placed on both compressive and tensile side of the columns. Figure 6 and Fig.7 show the loading apparatus and the measuring

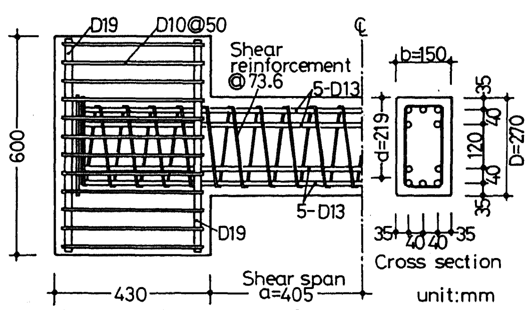


Fig. 5 Detail of Column Specimen

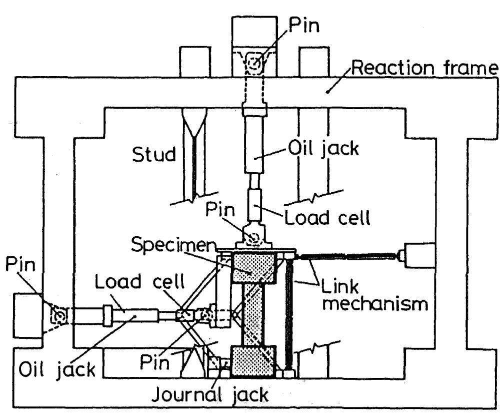
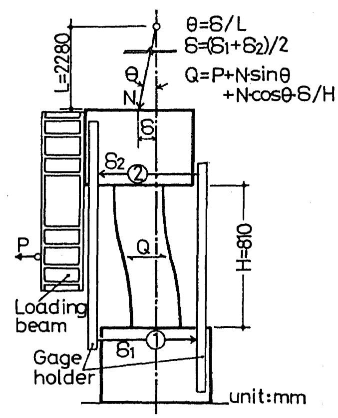


Fig. 6 Loading Apparatus for Column Specimen



L : distance between the pin joints at both ends of the oil jack for axial force
H : height of column
θ : inclination angle of applied axial force to the vertical axis of column

Fig. 7 Measuring System

system respectively. The axial force (N) was applied by oil jack with pin-joints on both ends. The horizontal force (P) was monotonously introduced also by oil jack. Shear force (Q) was counted as the sum of the horizontal force (P), the horizontal component of the axial force ($N \cdot \sin\theta$) and the equivalent horizontal force ($N \cdot \cos\theta \cdot \delta/H$) by the additional bending moment caused by axial load as indicated in Fig.7. The parameters were the compressive axial stress σ_o (N/bD, N:applied axial force) and $P_w \cdot {}_w\sigma_y$.

The test results are listed in Table 2 and the relations between τ_u and $P_w \cdot {}_w\sigma_y$ are shown in Fig.8. The specimens whose shear reinforcement yielded at shear failure and did not yield, were plotted by the open marks and the solid marks respectively. The region of $P_w \cdot {}_w\sigma_y$, where the shear reinforcement yield, tends to be small, as the compressive axial stress increased. However the increase of τ_u by axial force is remarkable especially at the smaller $P_w \cdot {}_w\sigma_y$. While

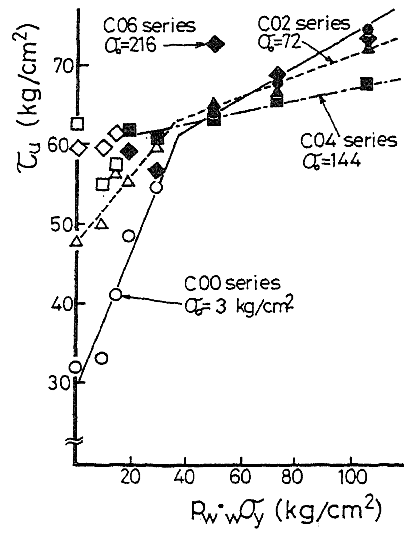


Fig. 8 $\tau_u - P_w \cdot {}_w\sigma_y$ Relations of Columns

Table 2 Characteristics of Column Specimens

No.	Specimen's name	σ_o *1 (kg/cm²)	D_s *2 (mm)	P_w *3 (%)	${}_w\sigma_y$ *4 (kg/cm²)	$P_w \cdot {}_w\sigma_y$ (kg/cm²)	${}_c\sigma_B$ *5 (kg/cm²)	Q_{uexp} *6 (ton)	τ_{uexp} *7 (kg/cm²)
1	C00-0	3		0.0		0.0	353	9.2	32.0
2	C00-32N	3	3.2	0.15	6370	9.3	331	9.5	33.1
3	C00-40N	3	4.0	0.23	6230	14.2	331	11.9	41.3
4	C00-32H	3	3.2	0.15	13100	19.1	331	13.9	48.4
5	C00-40H	3	4.0	0.23	13000	29.6	331	15.8	54.8
6	C00-50H	3	5.0	0.36	14000	49.8	353	18.2	63.4
7	C00-60H	3	6.0	0.51	14200	72.2	353	19.5	67.9
8	C00-74H	3	7.4	0.72	14500	105.0	353	21.4	74.4
9	C02-0	72		0.0		0.0	367	13.7	47.7
10	C02-32N	72	3.2	0.15	6370	9.3	352	14.3	49.9
11	C02-40N	72	4.0	0.23	6230	14.2	352	16.2	56.4
12	C02-32H	72	3.2	0.15	13100	19.1	352	15.9	55.4
13	C02-40H	72	4.0	0.23	13000	29.6	352	17.2	59.8
14	C02-50H	72	5.0	0.36	14000	49.8	367	18.7	65.2
15	C02-60H	72	6.0	0.51	14200	72.7	367	19.2	66.8
16	C02-74H	72	7.4	0.72	14500	105.0	367	20.8	72.2
17	C04-0	144		0.0		0.0	346	18.0	62.6
18	C04-32N	144	3.2	0.15	6370	9.3	366	15.8	55.1
19	C04-40N	144	4.0	0.23	6230	14.2	366	16.5	57.4
20	C04-32H	144	3.2	0.15	13100	19.1	366	17.8	61.9
21	C04-40H	144	4.0	0.23	13000	29.6	366	17.5	60.9
22	C04-50H	144	5.0	0.36	14000	49.8	346	18.2	63.3
23	C04-60H	144	6.0	0.51	14200	72.7	346	18.8	65.4
24	C04-74H	144	7.4	0.72	14500	105.0	346	19.4	67.5
25	C06-0	216		0.0		42.7	358	17.2	59.7
26	C06-32N	216	3.2	0.15	6370	64.4	295	17.2	59.7
27	C06-40N	216	4.0	0.23	6230	101.7	295	17.7	61.6
28	C06-32H	216	3.2	0.15	13100	146.0	295	17.0	59.3
29	C06-40H	216	4.0	0.23	13000	0.0	295	16.3	56.8
30	C06-50H	216	5.0	0.36	14000	20.9	358	20.9	72.6
31	C06-60H	216	6.0	0.51	14200	42.7	358	19.6	68.3
32	C06-74H	216	7.4	0.72	14500	64.4	358	21.1	73.4

*1 Compressive axial stress (N/bD, N:Applied axial force)
*2 Diameter of shear reinforcing bar
*3 Shear reinforcement ratio
*4 Yield strength of shear reinforcement (0.2% offset)
*5 Compressive strength of concrete cylinder (10cmφ x 20cm)
*6 Maximum shear force in experiment
*7 Shear stress at maximum shear force (Q_{uexp}/bj, j=19.2cm)

the region of $P_w \cdot {}_w\sigma_y$, where the shear compression failure occurred, tended to be large, as the compressive axial stress increased. The slope of increasing of τ_u due to the increase of $P_w \cdot {}_w\sigma_y$ became to be more gentle than in the case of beams. Usually the ultimate shear strength of RC members with axial force can be evaluated as the sum of τ_u of the member without axial force and the contribution of axial force. In this experiment, it can be find that the contribution of axial force is variable with the value of $P_w \cdot {}_w\sigma_y$.

SHEAR DESIGN UNDER TEMPORARY LOADING

Usually the shear stress of beams and columns must be under the allowable shear stress by Eq.(1), in the allowable stress design method enforced in Japan as the first step aseismic design.

$$\tau_A = \alpha \cdot f_s + 0.5\ {}_wf_t\ (P_w - 0.002) \tag{1}$$

τ_A : allowable shear stress of beams and columns in unit of kg/cm^2
α : coefficient by shear span to effective depth ratio $(1 \leq \alpha \leq 2)$
f_s : allowable shear stress of concrete in unit of kg/cm^2
${}_wf_t$: allowable tensile stress of shear reinforcement in unit of kg/cm^2

Here, the ordinary strength steel whose yield strength is around 3000 kg/cm^2 (43ksi) is usually used and the upper limit of shear reinforcement ratio(P_{wmax}) is fixed at 1.2%. The allowable tensile stress of high tension steel(ULBON) has been officially approved at 6000kg/cm^2 (86ksi) and P_{wmax} at 0.6%, assuming that the shear reinforcing effect of high tension steel is same as that of the ordinary strength steel. The product of P_{wmax} and ${}_wf_t$ approximately corresponds to the boundary value of $P_w \cdot {}_w\sigma_y$ of B-210 series in Fig.4, which separates the shear compression failure and the shear tension failure region. If P_{wmax} can be determined in the region of $P_w \cdot {}_w\sigma_y$ where the shear tension failure occurs and the shear reinforcement displays its full strength, Fig.4 indicates that P_{wmax} can be improved up to 0.8% to 1.0% by using high strength concrete in case of beams. From the experimental results of columns, the axial force encourages the shear compression failure and the region of $P_w \cdot {}_w\sigma_y$, where the full strength of shear reinforcement is displayed, becomes small. So, if P_{wmax} will be determined by the philosophy as same as that concerning beams, P_{wmax} should not be improved by using high strength concrete in case of columns. The increase of τ_u by axial force, however, should be properly estimated according to the value of $P_w \cdot {}_w\sigma_y$.

ULTIMATE SHEAR STRENGTH DESIGN

The design formula for the ultimate shear strength should be given based on the experimental and theoretical equation which well agrees with the the actual shear strength of RC members. At this stage, the estimation about the ultimate shear strength of RC members with high tension shear reinforcement has not been fixed and the usual idiomatic design formula must be correspondingly applied. Equation (2) is one of the authorized formulae in the bearing capacity design enforced in Japan and is generally called Arakawa's formula[4].

$$\tau_u = \frac{0.053\ P_t^{0.23}\ (F_c + 180)}{M/Qd + 0.12} + 2.7\sqrt{P_w \cdot {}_w\sigma_y} \tag{2}$$

τ_u : ultimate shear strength of beams and columns in unit of kg/cm^2
P_t : longitudinal reinforcement ratio
F_c : compressive strength of concrete in unit of kg/cm^2
M/Qd : shear span to effective depth ratio
${}_w\sigma_y$: yield strength of shear reinforcement in unit of kg/cm^2

The experimental results of over 130 specimens with high tension shear reinforcement including the specimens in past studies[2),3)] as well as the beam and column specimens in this study, are compared with the value given by Eq.(2) in Fig.9. The parameters of the plotted specimens are as follows.

Depth of section (cm)	25.0 - 40.0
Effective depth (cm)	21.9 - 36.0
Width of section (cm)	13.0 - 36.0
Shear span to effective depth ratio	1.14 - 2.75
Tensile longitudinal reinforcement ratio (%)	0.36 - 3.20
Shear reinforcement ratio (%)	0.10 - 1.15
Concrete strength (kg/cm^2)	163 - 549
Axial stress to concrete strength ratio	0.01 - 0.73

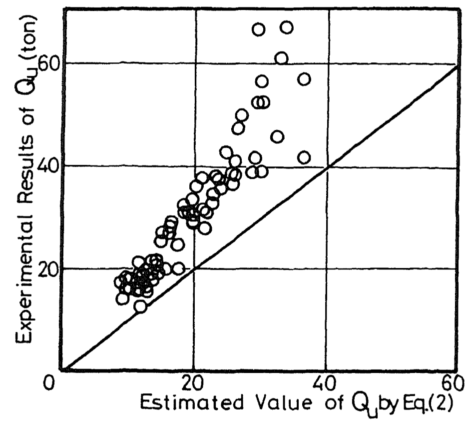


Fig. 9 Experimental Result of Qu and the Estimated Value by Eq.(2)

The ultimate shear strength of all the specimens is greater than the calculated value by Eq.(2). So it is found that Eq.(2) can produce a safety side value of the ultimate shear strength of RC members including columns as well as beams. Based on the results of Fig.9, it is officially approved[1)] that the yield strength of high tension steel bar (ULBON) can be evaluated in full when Eq.2 is applied for the ultimate shear capacity of beams and columns in the bearing capacity design of RC structures. However, it should be noted that there is a few column specimens whose concrete strength is under $270kg/cm^2$ in Fig.9.

CONCLUSION REMARKS

(1) The strength of high tension steel can be more effectively utilized with combination of high strength concrete.
(2) Axial force improves the ultimate shear strength in the shear tension failure region of $P_w \cdot {}_w\sigma_y$. In the region of large $P_w \cdot {}_w\sigma_y$, however, axial force encourages the diagonal shear compression failure of concrete.
(3) Under the temporary loading, the upper limit of shear reinforcement ratio could be improved by using high strength concrete.
(4) At this stage, the ultimate shear capacity of both beams and columns can be estimated in safety side by Eq.(2). And it has been officially approved that the full strength of high tension steel (ULBON) can be evaluated in Eq.(2).
(5) A critical shear design such as of high-rise RC building could be performed by using high tension steel bar with combination of high strength concrete.

REFERENCES

1. NETUREN Co., Ltd., Design standard for using high tension steel ULBON as shear reinforcement in reinforced concrete beams and columns, (1988)
2. Fukuhara, M., Kokusho, S. and Eigawa, T., "Experimental studies of the ultimate shear strength and deformation capacity in reinforced concrete members with high tension shear reinforcement," Proceedings of AICAP Symposium, Rome, Vol.2, 157-164, (1979).
3. Fukuhara, M., Wada, A. and Kokusho, S., "Shear strength of reinforced concrete beams with high tension shear reinforcement," Proceedings of IABSE 12th Congress, Vancouver, (1984)
4. Arakawa, T., "Allowable shear stress and shear reinforcing in reinforced concrete beams," Journal of JCI, Vol.8, No.7, (1970)