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Strength of Embedded Type Corner Steel Column Base

Résistance d'un pied de poteau d'angle en acier encastré

Widerstand einbetonierter Eckstützen aus Stahl

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The mechanical properties of the corner steel column base embedded in concrete are not yet fully clarified. So, the experiments were conducted using models to clarify these mechanical properties.
Table 1 indicates the test program and Fig. 1 shows the outline of the test. Tie spacing was taken as the test variable.
Cold formed square steel tube was used for the columns to which alternating positive and negative lateral forces were repeatedly applied by hydraulic jack. Fig. 2 shows the load (Q) - deformation (δ) relations. Fig. 3 shows the envelope curves for these Q-δ relations. The test results are indicated in Table 3. In all of the specimens, the ultimate strength of the column bases was governed by the shear fracture of end concrete. The test results indicated that as the ties were spaced closer, the ultimate strength and ductility increased and the degree of decrease in maximum strength due to repeated loading became small. The initial rigidity, however, remained unchanged.
For calculation of the ultimate strength, the fracture planes of concrete due to punching shear were assumed for each of the positive and negative loadings as shown in Fig. 4. [1] It was assumed that the capacity of concrete against punching shear was to be obtained firstly by developing these areas on the end wall of foundation beam and then by multiplying such developed areas (Su and Sℓ) by $\sqrt{Fc}$ where Fc was concrete strength. ($Rcu = Su\sqrt{Fc}$ and $Rc\ell = S\ell\sqrt{Fc}$) However, since the ultimate strength increases as the tie spacing becomes shorter, the capacity formula should take into account the effects of reinforcement as well. For the purpose of this paper, the capacity in which the effect of reinforcement is considered is taken as the capacity (Rcu or Rcℓ) of concrete against punching shear fracture plus the capacity (Rhu or Rhℓ) of that part of reinforcement which is located above or below the neutral axis of the embedded portion of the steel column.
As for the load-resistance mechanism of the embedded portion of the column, three cases were assumed as indicated in Fig. 5. Then, the bearing stress resultants (Bu and Bℓ) caused in portions above and below the neutral axis by the assumed lateral force were calculated for the aforesaid three cases. For each case, the ultimate strength was calculated by equating it: (I) to the capacity of concrete against punching shear fracture ($Bu = Rcu$ and $B\ell = Rc\ell$); and (II) to the aforesaid capacity plus the capacity of the reinforcement ($Bu = Rcu + Rhu$ and $B\ell = Rc\ell + Rh\ell$). Since the tie bars did not yield in the present experiment, the actual stresses were used for calculation. The ratios of the values obtained by the experiments to the calculated values are indicated in Fig. 6. The experiment results clearly indicate that, where the effects of reinforcement are taken into account, the values calculated in the way as shown in Case 2 (see Fig. 5) approximately coincide with the experiment values, which means that the load causing the fracture of concrete covering outside the corner column base can be estimated from such calculated values.

Table 1 Test program

No.	Specimen Symbol	L (mm)	L/D	E (mm)	E/D	‡1 (mm)	‡2 H, F	‡3 M, R
1	CH-RHN □'1 16	250	2.50	100	1.00	—	H	R
2	CH-RH75 □'1 16	〃	〃	〃	〃	75	〃	〃
3	CH-RH50 □'1 16	〃	〃	〃	〃	50	〃	〃
4	CH-RH25 □'1 16	〃	〃	〃	〃	25	〃	〃

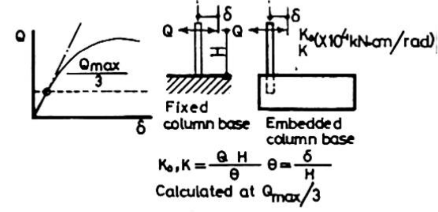
Table 3 Test results

No.	Specimen Symbol	Qy (kN)	Qp (kN)	Qmax (kN) (+)	Qmax (kN) (-)	Qmax/Qp (+)	Qmax/Qp (-)	Ko ($\times10^4$ kN·cm/rad)	K ($\times10^4$ kN·cm/rad) (+)	K (-)	K/Ko (+)	K/Ko (-)
1	CH-RHN □'1 16	53.0	63.9	37.6	25.6	0.59	0.40	33.6	17.9	17.1	0.532	0.508
2	CH-RH75 □'1 16	〃	〃	61.5	42.0	0.96	0.66	〃	18.1	17.7	0.539	0.528
3	CH-RH50 □'1 16	〃	〃	63.0	42.5	0.99	0.67	〃	17.4	16.7	0.517	0.499
4	CH-RH25 □'1 16	〃	〃	74.0	61.2	1.16	0.96	〃	17.9	17.2	0.533	0.513

Table 2 Mechanical properties of steel column and concrete

		σy (N/mm²)	σmax (N/mm²)	E ($\times10^5$ N/mm²)	El. (%)
Column	□-100×100×6.0	458	509	2.18	22
Tie	D6	479	522	1.92	21
Concrete		—	24.3	0.217	—

‡1 Tie Spacing
‡2 Hollow or Filled
‡3 Monotonic or Repeated
No anchor bolt
No axial load



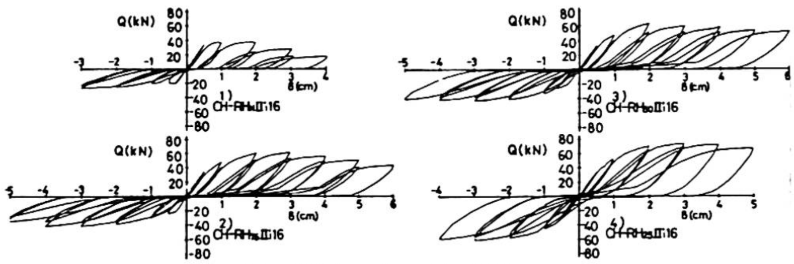


Fig. 2 Load(Q)−deformation(δ) curves

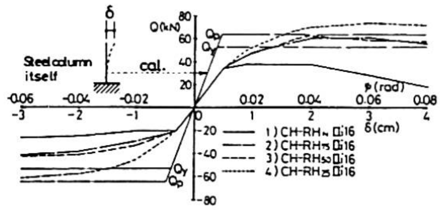


Fig. 3 Envelope curves of Q-δ relations

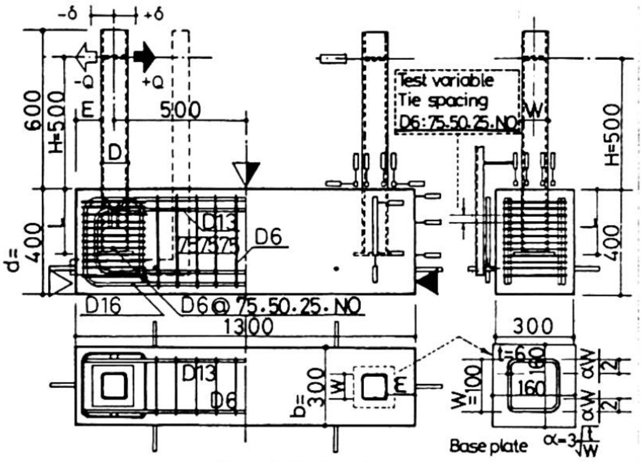
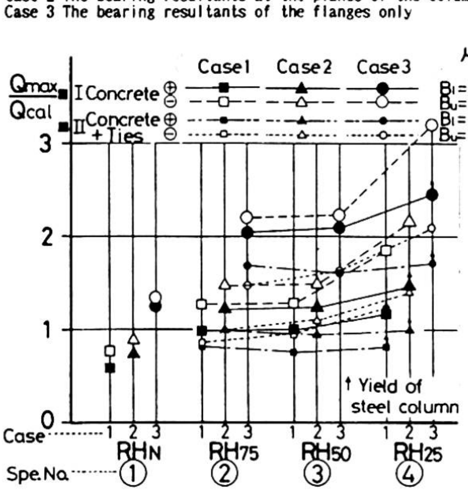
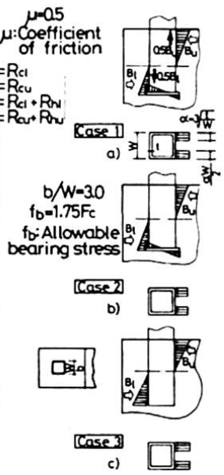


Fig. 1 Test set-up

Case I The bearing resultants and the frictional forces at the planes of the column flanges and the bearing resultants of the base plate
Case 2 The bearing resultants at the planes of the column flanges and the base plate
Case 3 The bearing resultants of the flanges only





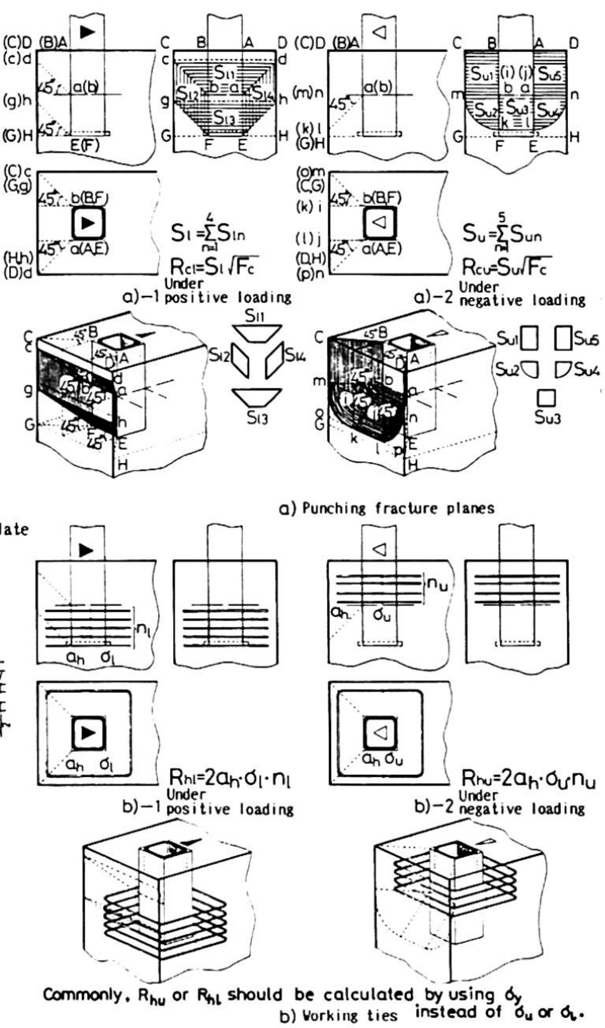


Fig. 6 Ratios of test values to calculated capacities

Fig. 5 Assumption of three resistance conditions

Fig. 4 Assumption of punching fracture planes and working ties

REFERENCE

1. MORITA K. et al., Experimental Studies on the Ultimate Strength of the Embedded Type Steel Column-to-Footing Connections. Trans. of AIJ, Jan. 1985.