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STRENGTH AND RIGIDITY OF VERY SHALLOW TYPE STEEL COLUMN-TO-FOOTING CONNECTIONS

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SUMMARY

This paper presents strength and rigidity of very shallow type steel column-to-footing connections with adequate reinforcement subjected to cyclic reversed bending moments and shearing forces as well as axial column forces. The experimental results showed that the significant progresses of the ultimate strength and inelastic deformation capacity can be achieved by employing the proposed stiffening method to the connections, even if the column bases were covered by very shallow embedding concrete. Based on the results, an analytical model to estimate the behaviors of the very shallow connections whose embedding heights are about 0.6 times the column depths, has been proposed.

INTRODUCTION

The column-to-footing connections in the steel structures, if designed as the rigid connections, would be the most crucial regions, since the connections have to carry safely not only the column axial forces but also the bending moments and the shearing forces to the foundations of the structures, in the event of earthquakes. However, in the case of heavy industrial buildings such as steam power plants and other energy plants, because the large sectional members are necessarily used for the columns, it can be resulted in that the connections are composed of very thick plates and the anchor bolts with comparatively large cross sections. Consequently, it can be recognized that the connecting methods of the column bases with the footing concrete become very complicated and the higher construction techniques have to be required in order to secure the sufficient mechanical continuity between the components.

Such consideration can lead that the developments of the more practical design and easier construction methods are necessitated for the large scale column-to-footing connections. A few researchers have experimentally investigated about the shallow embedded type steel column-to-footing connections with rigid double base plates and the additional concrete cover whose depths were over about 0.8 times the column depths, and have reported that the connections may be regarded as the rigid connections (Ref. 1). In the experiment, however, the behaviors of the connections with shallower embedment and thick single base plates have not been referred to. For this reason, the objectives of this paper are focussed to propose an adequate stiffening method of the very shallow type connections whose embedding heights are about 0.6 times the column depths, and to make clear the stiffening effect on the strength and rigidity of the connections from the experimental investigation.

THE FIRST STEP OF EXPERIMENT

Experimental Procedures The dimensions of the specimens were scaled down to about a quarter of actual subassemblages in the typical energy power plant structures. The connections were designed as the exposed type connections and any contribution of the additional reinforced concrete cover to the overall strength of the connections was not accounted. In this design procedure, it was assumed that the anchor bolts located at the tension sides can only resist and the base plates can not yield due to the bending moments, prior to the yielding of the anchor bolts and to attainment of the compressive strength of the concrete. The four different types of specimens named as A to D-Type were prepared for the experiment. The material used and the design conditions of the specimens are tabulated in Table 1 and the detailed configurations are shown in Figs.1(a) to (c). The reinforcement ratio of the additional concrete cover was set up at a constant value of about 0.36% for three types of specimens except of B-Type, and the transverse reinforcement ratio of the concrete column stub was kept constant at about 0.25% for all types of specimens.

Table 1 Specimens and Materials

Type of Specimen	Base Plate (mm)	Anchor Bolt (mm)	Reinforcement of Additional Concrete Cover
A	550x430x28	6-φ39	Existence
B	550x430x28	6-φ39	Not Existence
C	550x430x22	6-φ25	Existence
D	550x430x22	6-φ16 *	Existence

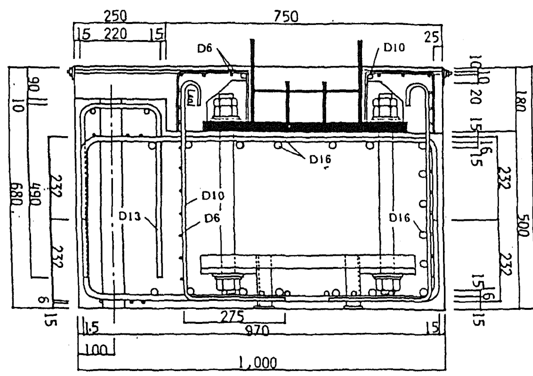
Material

Steel: SM50A
Anchor Bolt: S25C(φ39, φ25), SS41(φ16)
Reinforcing Bar: SD35(D10 to D16), SD30(D6)
Concrete: FC210

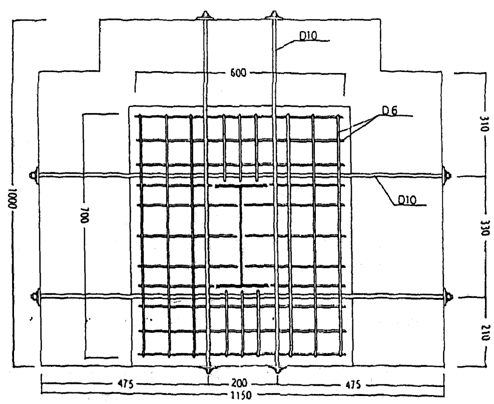
Column Size

BH-300x150x9x10

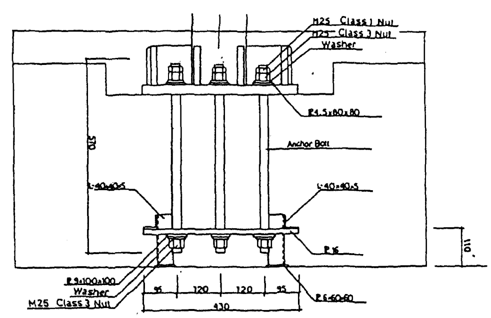
* Epoxy Resin Anchor Bolt



(a) Longitudinal Section



(b) Plan



(c) Transverse Section

Fig. 1 Details of Specimens (A and C-Type) (Unit: mm)

Horizontal loads which generated bending moments and shearing forces at the column-to-footing connections, were applied to the column tips at the points 1.18 m apart from the column bases. Since the tip deflections included the effects due to the rigid body rotations and the lateral distortions caused by slippage of the specimens as well as the flexural and the shear distortions of the concrete bodies, the net horizontal deflections were obtained by subtracting the horizontal components of the above incidental deflections from the measured tip deflections.

The Results of Experiment Fig.2 shows the horizontal load-net deflection relation for C-Type specimen, and the envelopes of the load-deflection relations for four types are compared with each other, in Fig.3. In the figures, the solid lines through the origins represent the initial elastic stiffnesses calculated in the assumption that the tip deflections of the cantilever steel columns are composed of both flexural and shear distortions. It can be recognized that the experimental curves at the initial states are very close to the calculated elastic stiffnesses, and that any distinct deterioration has not been found out for A to C-Type specimens. The non-linear portions in the experimental curves were mainly caused by the fact that the bending moments of the column cross sections at the bottoms arrived at the yield moment, and subsequently full plastification occurred over the cross sections as the horizontal forces continued to increase. These experiments were terminated only after the occurrence of local buckling of the column flanges and the subsequent decrease of load carrying capacities being observed. Remarkable differencies in restoring force characteristics can not be observed beween A and B-Types, eventhough small hair cracks on the additional concrete cover were visible due to the lack of reinforcement in B-Type specimen. On the contrary, D-Type specimen showed poor strength and rigidity, and distinct deterioration due to cyclic reversed loadings.

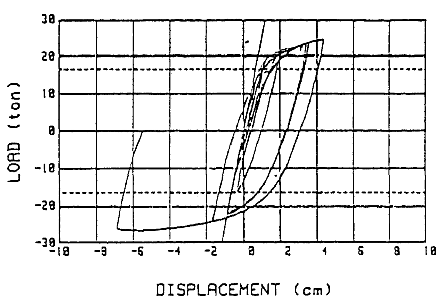


Fig. 2 Horizontal Load-Displacement Curves (C-Type)

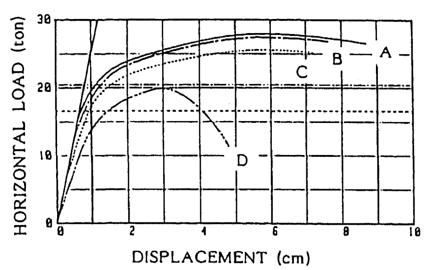


Fig. 3 Enveropes of Load-Displacement Curves

The load stages indicated by dottted and dot-dashed lines in Figs.2 and 3, are corresponding to the specific values of the horizontal loads at which the bending moments of the cross sections at the column bases have just arrived at the yield moment and the fully plastic moment, respectively. It has been recognized that the connections except of D-Type can exhibit the sufficient lateral resistant capability against certain degrees of the severe horizontal loads where the column sections have become the full plastic regions.

THE SECOND STEP OF EXPERIMENT

Experimental Procedures From the results of the first step of experiment, it has been demonstrated that the connections with adequate stiffening can exhibit the satisfactory uplift on the strength and rigidity against cyclic reversed bending moments and shearing forces, even if the connections were designed as the exposed types which possessed at least the ultimate strengths of 0.7 times the column plastic moments. Further, the second step of experiment was carried out in order to clarify the resistant capability of the connections under complex loadings taking into consideration the effect of axial forces. E and F-Type specimens were used for this experiment.

The column sections of two types were stiffened using the plates of 22 mm thickness but identical in shape to the sections of the former four types, in order to prevent any reduction of the column flexural strength due to local buckling phenomenon. The connection details like as the dimensions of the base plates and the anchor bolts, and as the reinforcement were identical among C, E and F-Types. However, for E and F-Types, additional loading conditions were conducted. Namely, the former was tested under cyclic reversed horizontal loadings as well as a constant column axial force of about 0.2 times the compressive yield strength of the column, and the applied forces for the latter specimen were varied as follows.

$$
\begin{aligned}
&N = N_0 + \Delta N \\
&\Delta N = 5H/4 \\
&-0.1 \leqq N/N_y \leqq 0.4
\end{aligned}
\tag{1}
$$

where N denotes the applied axial force, N_0 is a constant of 0.15 times of the column yield load N_y, and ΔN is the increment of the axial force varying with the horizontal force H. The relation between N and H in Eq.(1), was determined so as to reflect the typical loading conditions of the exterior column bases in the case when the actual steam power plant structures would be subjected to both gravity and seismic loads.

The Results of Experiment The relationships between horizontal net deflections and corresponding applied loads are graphically shown in Figs. 4 and 5, for E and F-Type specimens, respectively. In the figures, the representative reference values are also shown, with exception of that the specific horizontal loads corresponding to the yield moments and fully plastic moments are calculated by using the column dimensions of A to D-Types and taking account for the column axial forces. From the figures, it has been recognized that the sufficient ultimate strength and fairly well deformation capacity of the connections can be also achieved under combined loadings.

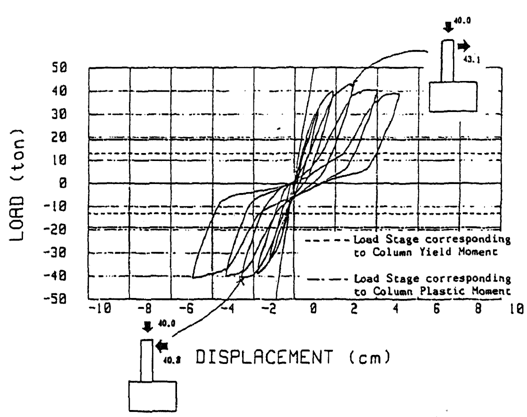


Fig. 4 Horizontal Load-Displacement Curves (E-Type)

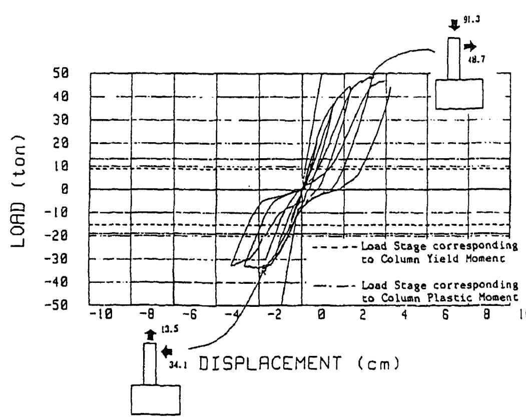
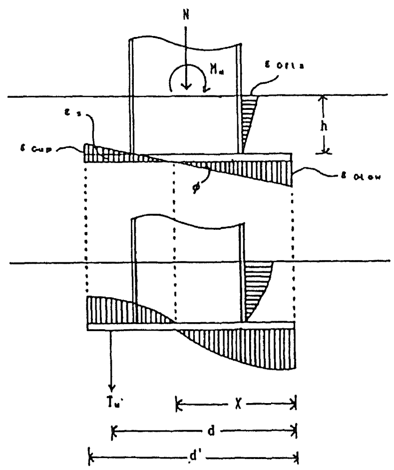
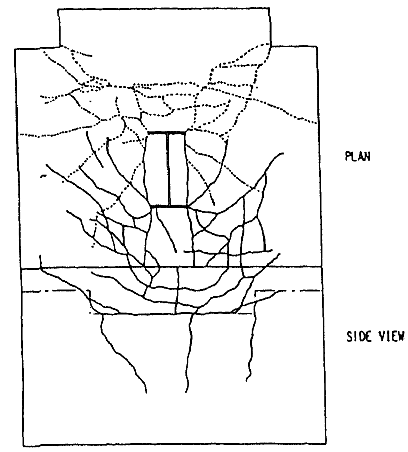


Fig. 5 Horizontal Load-Displacement Curves (F-Type)

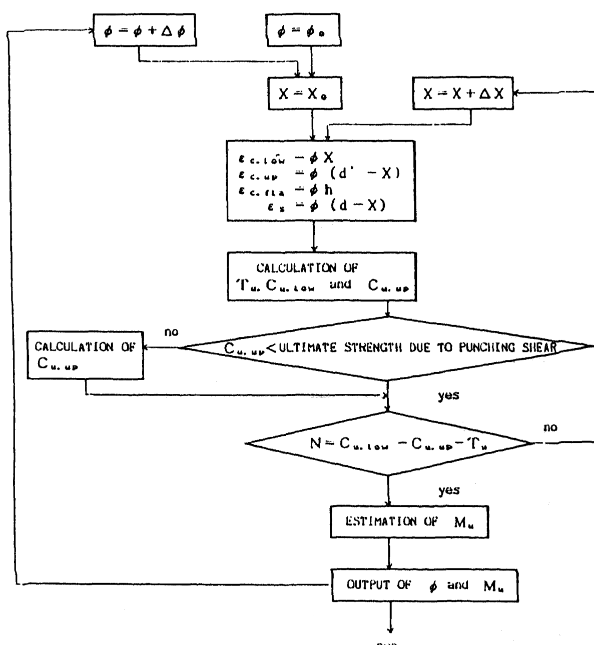
ANALYTICAL MODEL

Estimate of Ultimate Strength From the results of the experiments, it can be derived that the ultimate strength and the rigidity of very shallow embedded type connection have to be assessed taking into consideration the effects of additional concrete cover as well as the transverse reinforcement of the additional concrete cover and longitudinal reinforcement of the concrete column stub. The resisting mechanism of the connection was modeled as shown in the upper figure of Fig.6. In this analysis, the following assumptions are made.
1) Plane cross section remains plane and normal to the center line after bending.
2) The stress strain relation of the concrete can be expressed by an well known exponential function as shown in Fig.6.





TYPECAL CRACK PATTERN (E-TYPE)



Notations

- Mu: Bending moment
- N: Column axial load
- Tu: Reaction of anchor bolts in tension
- Cu.low: Reaction of footing concrete
- Cu.up: Reaction of additional concrete cover
- X: Neutral axis
- d': Length of base plate
- d: Distance from axis of anchor bolts in tension to the edge of base plate
- b: Width of base plate
- h: Distance from upper face of base plate to the face of additional concrete cover
- εs: Strain of anchor bolts in tension
- εc.up: Strain of additional concrete cover at the edge of base plate
- εc.low: Strain of footing concrete at the edge of base plate
- εc.fla: Strain of additional concrete cover at the face of column flange
- Es: Young's modulus of anchor bolt
- As: Cross sectional area of anchor bolts in tension
- $a_t\sigma_y$: Resistant capacity of longitudinal reinforcement of column stub across the punching shear failure surface
- $a_s\sigma_y/\sqrt{3}$: Resistant capacity of main re-inforcement of additional concrete cover across the punching shear failure surface

$$T_u = E_s\,\varepsilon_s\,A_s \quad (\leq \sigma_y A_s)$$

$$C_{u.low} = \frac{b}{\phi}\int_0^{\varepsilon_{c.low}} \sigma(\varepsilon)\,d\varepsilon$$

$$C_{u.up} = \frac{b}{\phi}\int_0^{\varepsilon_{c.up}} \sigma(\varepsilon)\,d\varepsilon$$

$$\sigma(\varepsilon) = 6.75F_c\left(\exp\left(-0.812\frac{\varepsilon}{\varepsilon_0}\right) - \exp\left(-1.218\frac{\varepsilon}{\varepsilon_0}\right)\right)$$

AFTER PUNCHING SHEAR FAILURE OF COVER CONCRETE

$$C_{u.up} = a_t\,\sigma_y + a_s\,\frac{\sigma_y}{\sqrt{3}}$$

ULTIMATE STRENGTH OF ADDITIONAL CONCRETE COVER DUE TO PUNCHING SHEAR

$Cu = An\sqrt{Fc}$

An: Horizontal projection area of additional concrete cover

Fc: Compressive strength of concrete

Fig. 6 Flow Chart for Estimating Ultimate Strength of Very Shallow Type Connection

3) The shear capacity of the additional concrete cover can be estimated by dividing into following two parts, namely, the shear resistance of the concrete alone until the attainment of the punching shear failure, and the sum of the shear resistance of the main reinforcement of the additional concrete cover and the tensile resistance of the longitudinal reinforcement of the concrete column stub after the failure of the additional concrete cover.

The ultimate flexural strength of the connection can be calculated by following the set order of the flow chart in Fig.6. Initially, let ϕ be the curvature at the section. Since the distance from the neutral axis X is determined to satisfy the equilibrium condition with respect to the axial forces for the curvature ϕ, the corresponding bending moment can be calculated. As ϕ increases, the magnitude of the bending stress also increases at the upper concrete portion of the base plate, but the increase is terminated at the time when the upper concrete block have broken down due to the occurrence of punching shear failure along 45 degree lines from the ends of the base plate as shown in the upper figure of Fig.6. After the occurrence of the failure of the concrete, the reinforcement across the failure surface can only resist against further increase of the curvature. The punching shear failure of the additional concrete cover can occur when the shearing stress at the failure surface have just arrived at the square root of the compressive strength of the concrete.

Result of Analysis The aforementioned analytical procedure was applied to estimate the ultimate strengths of E and F-Types. The results are tabulated together with the experimental results in Table 2. It can be recognized that the proposed analytical model explains the experimental results fairly well.

Table 2 Comparison between Experimental and Analytical Results with respect to the Ultimate Strength

Type of Specimen	Direction of Loading	Column Axial Force (ton)	Experimental Result (tm)	Analytical Result(tm)
E	+	40.0	50.9	46.5
	-	40.0	48.1	46.5
F	+	91.3	57.5	51.6
	-	-13.5	40.2	41.2

CONCLUSIONS

The following conclusions have been drawn from the results of the experimental and the analytical investigations concerning with the stiffening effect for the very shallow embedded type steel column-to-footing connection subject to earthquake like loading.

1) Significant progress of the ultimate strength and fairly well ductility can be achieved by applying the proposed stiffening method to the connecton, if the connection would be designed as the exposed type which would possess the ultimate strength over at least 0.7 times the strength of the steel column.
2) The analytical model can assess fairly well the ultimate strength of the very shallow embedded type steel column-to-footing connection.

ACKNOWLEDGMENT

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REFERENCE

1. Morita, K. et al., "The Ultimate Strength of the Shallow Embedded Type Steel Column-to-Footing Connections," J. of Structural and Construction Eng., Trans. of AIJ, No.365, 76-86, (1986) (in Japanese)."