Connections Using Cast Steel T-Stubs in Composite Structures

Assemblages utilisant des souches en forme de T pour les structures mixtes

Verwendung von Gussstahl T-Stücken in Verbundtragwerken

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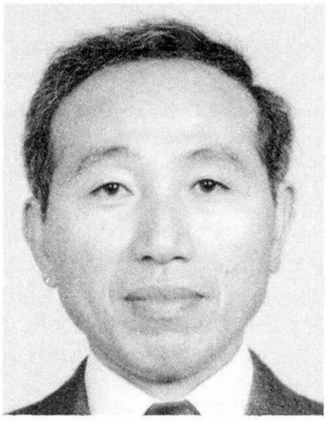

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SUMMARY
The aim of this study is to investigate the structural behaviour under seismic loading of beam-to-column connections in composite structures, in which steel beams are connected to concrete-encased steel columns by high-strength bolts using cast steel T-stubs. For this purpose, lateral load tests were conducted on beam-to-column subassemblages of this composite structure and the ultimate flexural strength of the connections was investigated both experimentally and theoretically.

RÉSUMÉ
Le but de l'étude présentée ici est d'examiner le comportement sous charges sismiques d'assemblages poutres-colonnes dans les constructions mixtes; la poutre en acier est liée à travers le béton de la colonne par une souche en forme de T, en acier moulé et par des boulons HR. Ce type d'assemblage a été testé, avec des charges latérales, et la résistance ultime à la flexion a été déterminée expérimentalement et par calcul.

ZUSAMMENFASSUNG
Das Ziel dieser Studie ist die Untersuchung des Tragverhaltens einer Träger-Stützen Verbindung in einer Verbundkonstruktion unter seismischer Belastung. Dabei werden die Stahlträger mit den ausbetonierten Stahlstützen mittels hochfester Schrauben unter Verwendung von Gussstahl T-Stücken verbunden. An Träger-Stützen Elementen des Verbundtragwerks wurden Biegeversuche durchgeführt, und die Traglast dieser Verbindung wurde experimentell und theoretisch ermittelt.

1. INTRODUCTION

Composite structures have been applied lately not only to low- and middle-rise buildings but also high-rise buildings for the advantages of structural rationality and economy in building construction in Japan.

A composite structure system, which consists of concrete encased steel column and steel or composite beam, has been developed. The most distinctive feature of this system is that the high-strength bolted T-stub connection method using cast steel attachments called HISPLIT is adopted in the connection of steel beam and steel member of the column. HISPLIT is an improved version of ordinary T-stubs; the thickness of T-stub flange is tapered from stub center to edges, and thus the prying force due to the flexural deflection of the T-stub flange is minimized. It has been confirmed experimentally that the structural properties of the HISPLIT connection in steel frames well enable it to fulfil the function of a moment resisting rigid connection. However, an out-of-plane bending force acts on the flange of the H-shaped steel column because the lateral stiffener plates in the steel column are generally eliminated in the HISPLIT connection system, and so the resisting capacity of the flange of the H-shaped column controls the strength and rigidity of the connection considerably.

Previous to the application of the HISPLIT connection system to the composite structure, the tension test was carried out to investigate the fundamental behaviour of beam flange-to-column connection being covered by concrete and the theoretical prediction method of the pull-out strength of the connection based on the test results has been suggested in Ref. [1].

This paper investigates experimentally the structural behaviour of beam-to-column connection of this composite structure under seismic loading. It also presents the prediction method of ultimate flexural strength of such connection and compares this prediction with the results of the experiments.

2. OUTLINE OF THE COMPOSITE STRUCTURE SYSTEM

An outline of the system is shown in Fig.1. The steel column is rolled H-shaped steel of considerably small cross section compared with the size of concrete encasement. By means of this small section the restraining effect of the covering concrete on the structural properties of the connection is enhanced; moreover greater productivity and cost-saving can be achieved. To increase the restraining effect, tie-bars are used in addition to ordinary square hoops in the column. Floor slab comprising a corrugated metal deck and concrete, is connected to the beam by means of stud connectors to form a composite beam. The steel column and steel beam are connected by high-strength bolts using HISPLIT.

This system, known as the KAJIMA MIXED-STRUCTURE SYSTEM, or KM system, has already been used in more than 50 low-rise and middle-rise buildings and has contributed greatly to cutting construction time and costs.

3. LATERAL LOAD TEST OF BEAM-TO-COLUMN SUBASSEMBLAGE

3.1 Test method

Lateral load test was performed using full-scale beam-to-column subassemblage specimens as shown in Fig.2 to observe the structural behaviour of beam-to-column connections in KM system under seismic loading. As shown in Table 1, in three specimens the beam was connected to the strong-axis of the column (strong-axis specimens), and in another to the weak-axis (weak-axis specimen). In the test of strong-axis specimens, the kind of beams and the method of reinforcement of the connection were chosen as the experimental parameters.

All specimens were designed so that their maximum strength could be determined from the ultimate flexural strength of the connection, which was calculated using Eqs. 1, 2, 3 and 4 proposed in Ref. [1].

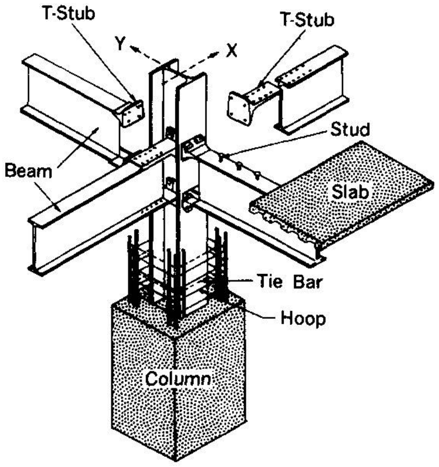


Fig. 1 KM system

Specimen No.	Beam	Reinforcement of joint panel
CX-O	Composite beam	Hoops and tie-bars
SX-O	Steel beam	Hoops and tie-bars
CX-P	Composite beam	Plates
CY-O	Composite beam	Hoops and tie-bars

Table 1 List of Test Specimen

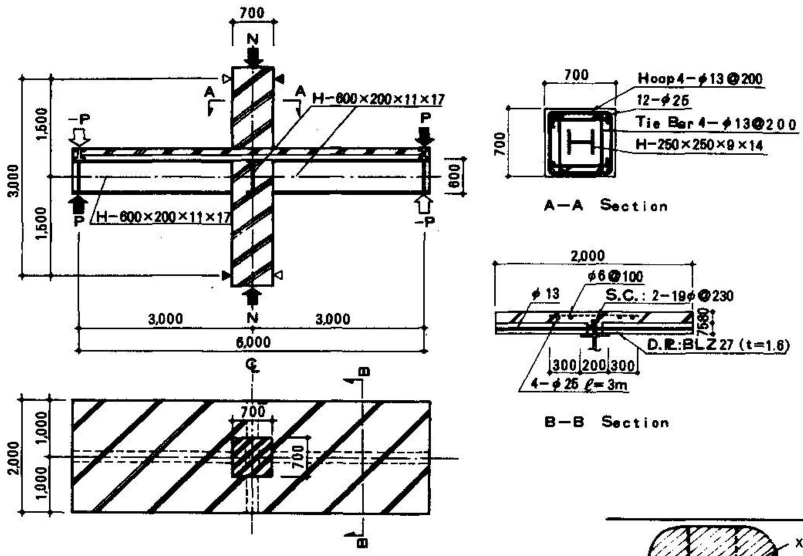
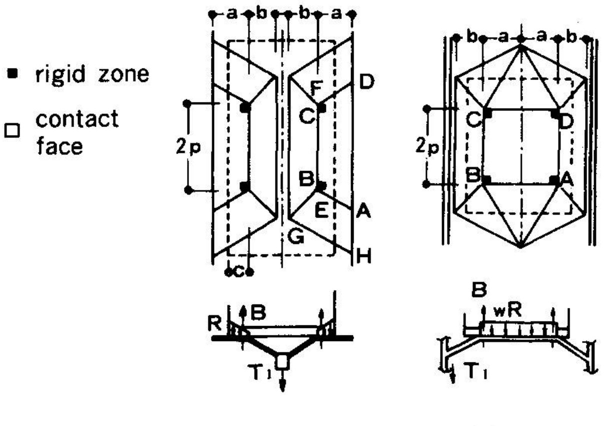
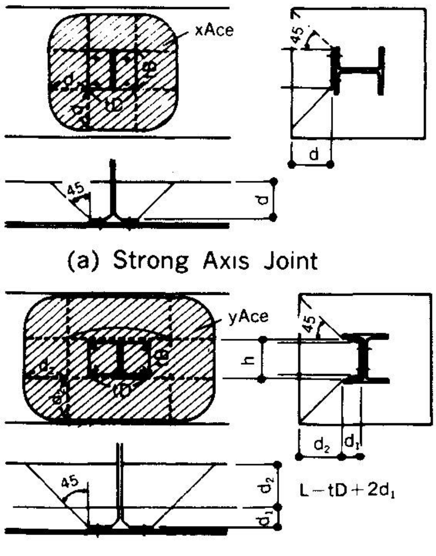


Fig. 2 Test Specimen



(a) Flange (b) Web

Fig. 3 Collapse Mechanism of Steel Column



(b) Weak Axis Joint

Fig. 4 Projected Area

$$_{j}M_{u} = T_{y} \cdot j_{b} \tag{1}$$

$$T_{y} = T_{s} + T_{c} \tag{2}$$

in which $_{j}M_{u}$ is the ultimate flexural strength of the connection, T_{y} is the pull-out strength of the beam flange-to-column connection being covered by concrete, T_{s} is the yield strength of the connection between the steel column and steel beam, and T_{c} is the pull-out strength of the covering concrete. T_{s} and T_{c} are given by following equations, respectively (see Figs.3 and 4).

For strong-axis specimens,

$$\left.\begin{aligned} {}_{x}T_{s} &= \frac{4}{b}\,(2p + \sqrt{16a\cdot b + 7\,b^{2}}\,)\,m_{p} \\ {}_{x}T_{c} &= 32.7\ {}_{x}A_{ce}\,\sqrt{f_{c}'},\quad {}_{x}A_{ce} = d\,[2\,({}_{t}D + {}_{t}B) + d\,] \end{aligned}\right\} \tag{3}$$

and for week-axis specimen,

$$\left.\begin{aligned} {}_{y}T_{s} &= \frac{8}{b}\,(p + \sqrt{4a.b + 3\,b^{2}}\,)\,m_{p} \\ {}_{y}T_{c} &= 32.7\,{}_{y}A_{ce}\,\sqrt{f_{c}'} \\ {}_{y}A_{ce} &= d_{2}\,[2\,(L + h) + d_{2}] + L\cdot h - {}_{t}D\cdot{}_{t}B,\ L = {}_{t}D + 2\,d_{1} \end{aligned}\right\} \tag{4}$$

in which m_{p} is the full plastic moment per unit width of steel column flange, and A_{ce} is effective projected area on concrete surface when cone failure is presumed to occur in the concrete part of the column.

Details of beam-to-column connections are shown in Fig.5 and outlines of HISPLIT used are shown in Fig.6. The mechanical properties of the steel, reinforcements and concrete are shown in Table 2. The high-strength bolts are of F10T grade and are 22 mm in diameter specified by Japanese Industrial Standards. These were tightened to a specified pretension of 221.5 kN. In the specimens with composite beam, CX-0,CX-P, and CY-0, concrete slab of 2 m in width was attached to the steel beam along its entire length with stud connectors. Two rows of studs of 19 mm in diameter were welded to the beam flange with a space of 230 mm along the length of beam.

Constant axial load of 0.3 N_{y}, where N_{y} is the axial yield load, was applied to the column. And a couple of reverse transverse load P was repeatedly applied at beam ends.

3.2 Test results

3.2.1 Load-deflection curves of subassemblage

The relationship between load on the beam ends P and deflection at the loading point δ is shown in Fig.7. Predicted values of the ultimate flexural strength of the connection calculated using Eqs. 1, 2, 3 and 4 are shown in terms of transverse load at beam ends P_{u}' and by chain lines in the figure. Each beam in the three strong-axis specimens did not yield up to its maximum load. Accordingly it can be seen that the maximum strength of these specimens was governed by the strength of the connection as predicted. However, these maximum loads were 30~40 % greater than the predicted values P_{u}'. The reason for this will be explained that the concrete of the beam end will restrain the rotation of the beam.

The load-deflection curves of CX-0 and CX-P with composite beams were quite similar; they showed stable softening-type up to the maximum loads, but after that they gradually changed into hardening-type and load carrying capacity tended to decrease. Specimen SX-0, with steel-only beam, exhibited about 40 % less elastic rigidity and maximum strength almost 25 % less than the above two specimens. However, it maintained a hysteresis curves of softening-type throughout testing.

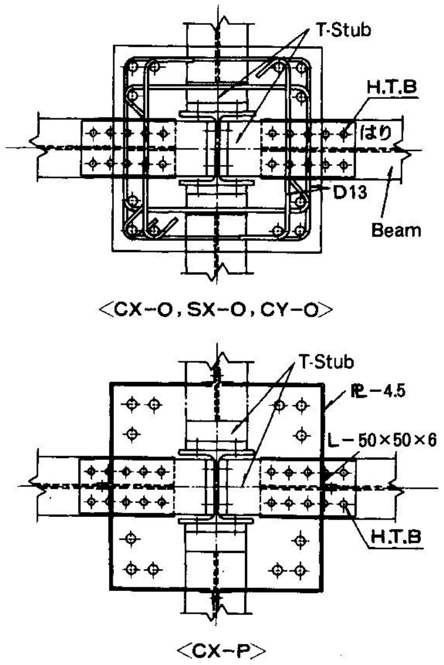


Fig. 5 Details of Beam-to-Column Connections

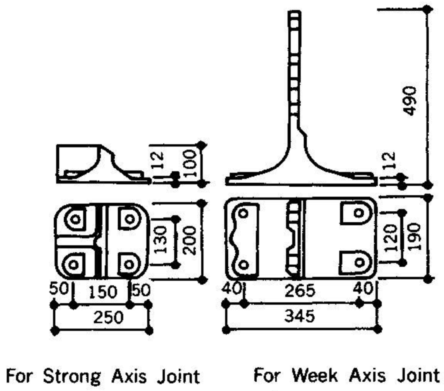


Fig. 6 HISPLIT

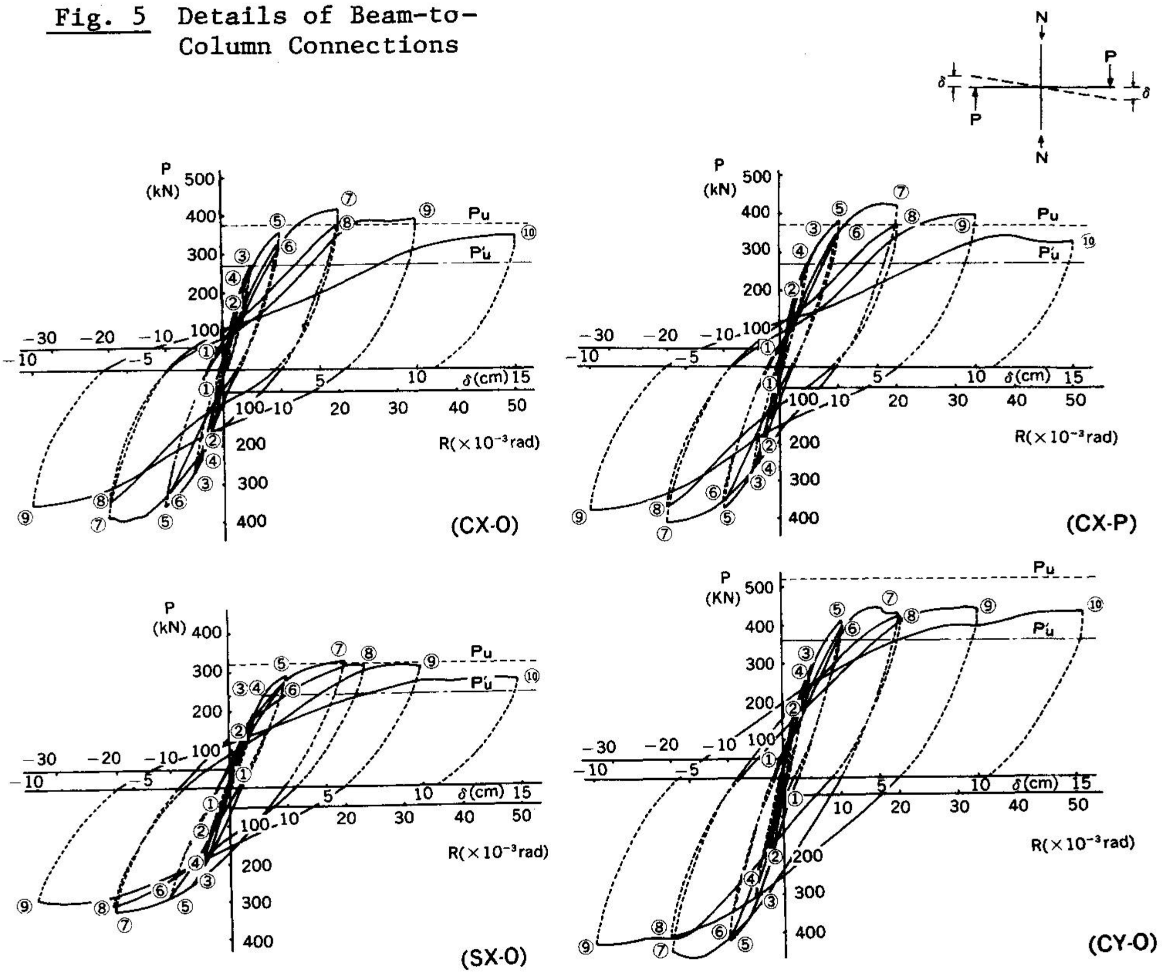


Fig. 7 P-δ Relationship of Subassemblages

In weak-axis specimen CY-0, the local buckling occured in the beam flanges at about $\delta = \pm 5$ cm. However, the buckling load was about 25 % greater than P'_u of this specimen, and the hysteresis curve depicted spindle-type even after buckling.

3.2.2 Pull-out deformation of beam flange-to-column connection

The relationship between load P and pull-out deformation Δ_j of the beam flange-to-column connection is shown in Fig.8. The Δ_j of the three strong-axis specimens were very small at 1 mm or less up to about half of each maximum load. However, at each maximum load, Δ_j increased to 3.2~5.4 mm, which occupied 40~45 % of the deflection at loading point δ , and is thus the most important factor in δ . Furthermore, Δ_j tends to increase rapidly after maximum load. In the light of these observations, it can be seen that all of beam-to-column connections of the three strong-axis specimens yielded at each maximum load.

On the other hand, in the specimen CY-0 the beam end was embedded more deeply in the column than CX-0, so that Δ_j of CY-0 was as a whole smaller; and CY-0 was less damaged around the connection throughout testing.

3.2.3 Behaviour of joint panel

The relationship between load P and shear deflection angle γ of the joint panel is shown in Fig.9. In both specimens SX-0 and CX-0, whose joint panels are reinforced with square hoops and tie-bars, shear yield of steel web panels was observed, but yield of hoops and tie-bars were not observed at each maximum load. Therefore, the concrete panels of these specimens seem to have some reserve strength at each maximum load. In specimen CX-P, whose joint panel is encased in plates instead of the square hoops and tie-bars, the shear yield load of steel web panel and rigidity were greater than that of specimen CX-0 and SX-0. Therefore, it can be said that shear properties of the joint panel encased by plates are superior or equivalent to that of the joint panel reinforced with square hoops and tie-bars. As regards specimen CY-0, its joint panels showed stable behaviour up to its maximum load.

4. ANALYSIS OF THE ULTIMATE STRENGTH OF THE CONNECTION

From the test results described in the previous chapter, it was found that the maximum loads in the strong-axis specimens were governed by the flexural strength of the connection, but the maximum loads in these specimens far exceeded that calculated on the basis of pull-out strength of beam flange-to-column connection. So, the ultimate flextural strength of the beam-to-column connection is analysed theoretically using following assumptions.

- Ultimate flexural strength of the connection can be predicted by summing the resistant moment on the basis of the pulling-out strength of beam flange-to-column connection given by Eq. 2 and the additional that on the basis of the bearing strength of the concrete encasing beam end.
- The bearing stress acts evenly along the whole embedded length of beam flange under compression which is embedded in the column ; and the force acts at the center of the embedded length as a concentrated load.

A moment resisting model of beam- to-column connection based on above assumptions is shown in Fig.10. Considering the moment equilibrium at the column steel surface of the connection, Eq. 5 is introduced.

$$T_y \cdot j_b = M_b + \frac{1}{2} Q_b (D_c - {}_sD_c) - \frac{1}{4} P_n (D_c - {}_sD_c) \qquad (5)$$

in which M_b is the bending moment of the beam at the column surface ($=P \cdot l_b$), l_b is the length of beam, Q_b is shear force in beam (= P), and P_n is total bearing force which is expressed by Eq. 6.

Materials	Thickness or diameter (mm)	Yield stress (MPa)	Tensile strength (MPa)	Conpressive strength (MPa)
Steel Plates	4.5	276	431	
	9	281	473	
	11	344	518	
	14	265	482	
	17	294	463	
Reinforcements	13ϕ	364	520	
	25ϕ	357	533	
Concrete				25

Table 2 Mechanical Properties of Materials

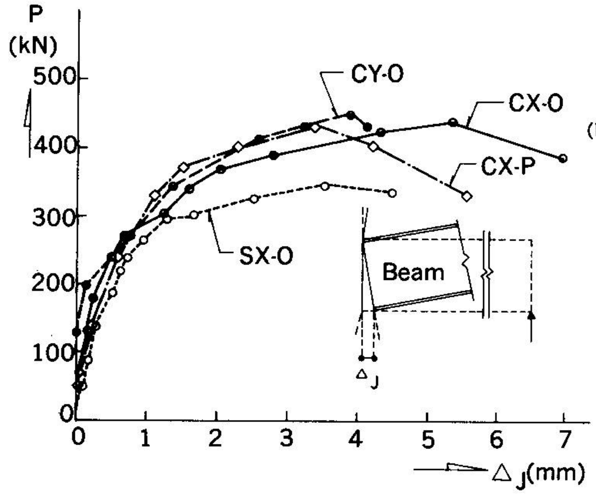


Fig. 8 P-Δ_J Relationship of Beam Flange-to-Column Connection

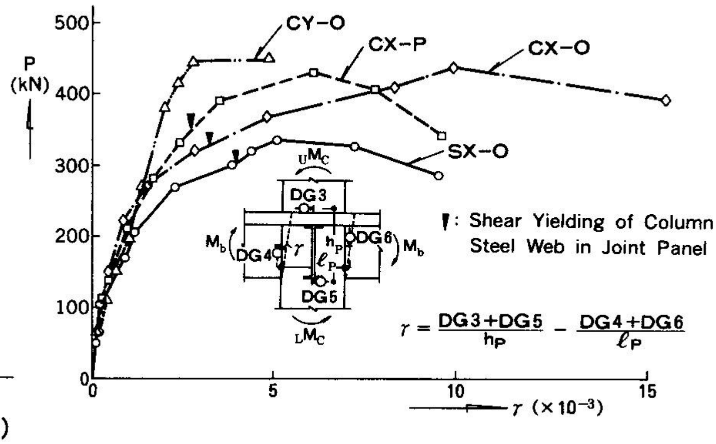


Fig. 9 P-γ Relationship of Joint Panel

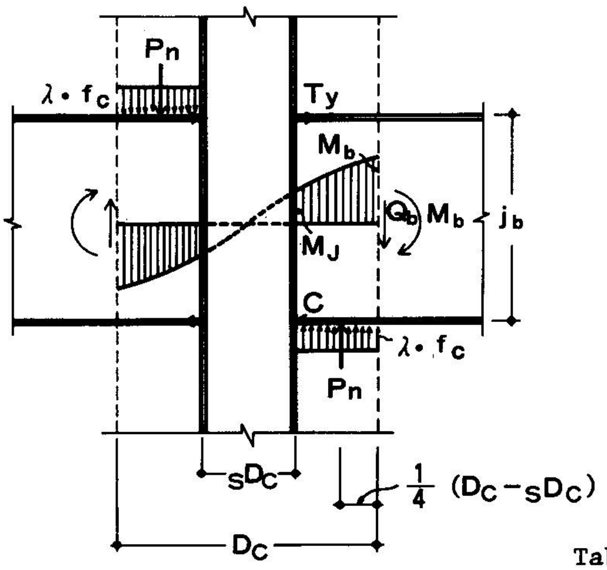


Fig. 10 Moment Resisting Model of Beam-to-Column Connections

Specimen No.	ePmax (kN)	Pu (kN)	Pu/ePmax (%)
CX-O	409.1	366.9	89.7
SX-O	328.6	321.8	97.9
CX-P	418.9	366.9	87.6
CY-O	446.9	515.5	115.4

$_eP_{max}$ = experimental maximum load

Table 3 Comparison of Ultimate Strength between Predictions and Test Results

$$P_n = \frac{1}{2}(D_c - {}_sD_c) \cdot b \cdot \lambda \cdot f_c' \tag{6}$$

in which f_c' is compressive strength of concrete, λ =ratio of the bearing stress to F_c. λ varies by the amount of reinforcements in the concrete and by the extent of the area acting bearing force, etc.. However, in this case λ was fixed at 2.0, a value chosen on the basis of Ref. [2]. Therefore, substituting $M_b = P \cdot l_b$, $Q_b = P$ and P_n in Eq. 6 at $\lambda = 2.0$ in Eq. 5, and solving Eq. 5 for the transverse load at the beam ends P, the ultimate load of the connection P_u is given by Eq. 7.

$$P_u = \frac{2}{D_c - {}_sD_c + 2\,l_b}\left[T_y \cdot j_b + \frac{1}{4}(D_c - {}_sD_c)^2 \cdot b \cdot f_c\right] \tag{7}$$

in which T_y is given by Eqs. 2, 3, and 4.

The prediction values of the ultimate flexural strength of beam-to-column connection of the specimens are calculated by using Eq. 7. These predictions are compared with test results in Table 3 and are shown by broken lines in Fig. 7. They have a good agreement with test results in strong-axis specimens. However, they can not compare with test results in weak-axis specimen, because its maximum load was governed by buckling strength of beam.

5. CONCLUSIONS

The main conclusions obrained by this investigation are as follows:

—— The ultimate flexural strength of beam-to-column connections in strong-axis specimens can be predicted accuratelly by Eq. 7.

—— Beam-to-column connections in this composite structure have a stable restoring characteristics as a moment resisting rigid connection before yielding of them.

—— The rigidity and flexural strength of the composite beam-to-column connection are greater than that of the steel beam-to-column connection.

—— The structural behaviour of the joint panel encased with steel plates is evaluated as the same as that of the joint panel reinforced with square hoops and tie-bars.

From above results, in order to design a beam-to-column connection as a mement resisting rigid connection using KM system, it is desirable that the ultimate flexural strength of the connection as calculated by Eq. 7 exceeds the full-plastic strength of beam. Also it is necessary that the depth of the concrete covering around the connection should be of sufficient depth and that the confining effect of the concrete should be increased by hoops or others.

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