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# Investigation on the Statical Characteristics of Beam-to-Column Connections

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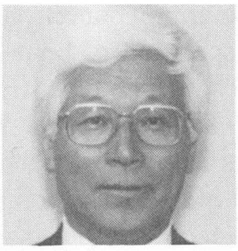

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## Summary

The statical characteristics of WBFW (web bolted and flange welded) type beam-to-column connections were experimentally investigated. The specimens with H-shaped columns or RHS columns were tested. The main test parameters are the disposition of bolts at the bolted web connection and the section of column members. From this test, the basic statical characteristics, such as ultimate bending strength eMu, plastic deformation ability, failure mode, are obtained. The results are summarized that $\alpha$ value (=eMu／Mp Mp:full plastic moment of the connected beam) and deformability of specimens are influenced by the bending strength of bolted web connection and by wide-to-thickness ratio of column flange.

## 1. Introduction

In Japan beam-to-column connections of small or medium size steel building frames are fabricated at shop by welding. On the other hand WBFW type beam-to-column connections, which are executed at the building site, are commonly used at high rise building frames. Up to now, at the design of beam connections it is customly assumed that the flange connected parts carry the whole bending moment and the web connected part carries the whole shear force. But the depth of beams are getting large, web connected parts should be carry some bending moment in addition to shear force. But the stress transfer capability of the bolted web connections is not so large because such bolted connections are commonly single shear connections. In addition the section of columns are usually square hollow section, and in such case the bending stress transfer at the web connection is considerably restricted. From such view points a series of experimental study on the WBFW type beam-to-column connections were executed in order to investigate the effect of the bending strength of the bolted web connection on the overall performance of the beam-to-column connection.

## 2. Test Specimens and Test Setup

The objective of this test was mainly the investigation of the influence of the bending strength of the bolted web connections to the maximum bending strength and the deformation capacity of the whole beam-to-column connections. The main experimental parameters are cross sections of column and disposition of bolts at the web connected part. The specimens using H-shaped columns were cantilever type (H series). Horizontal cyclic loads were applied to the tip of the beam. The specimens using RHS columns were T-Shaped type (B series). The test setup of this test series is shown schematically in Fig.1. Horizontal cyclic loads were applied to the top of the RHS column. Those specimens were cyclically loaded under displacement ±2cδp, ±4cδp, ±6cδp and ±8cδp, where cδp is the calculated elastic displacement corresponding to the plastic moment at the end of the beam. A total of fifteen nearly 1/2 scale model specimens with different type of column, different cross-sections of beam, and different bolt disposition at the bolted web connection were prepared, as summarized in Table 1. The character p, g, e, n and m in Table.1 are corresponding to the length and so on shown in Fig.2. Bolts were F10T class high strength bolts and they were tightened by nut lotation method. The connection details of B series specimens are illustrated in Fig.3.

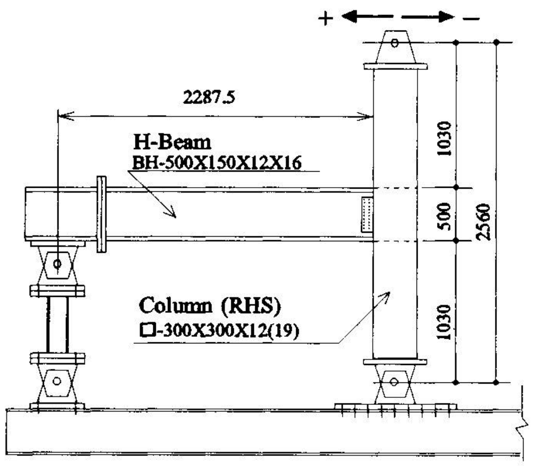


*Fig.1 Test Setup (RHS Column)*

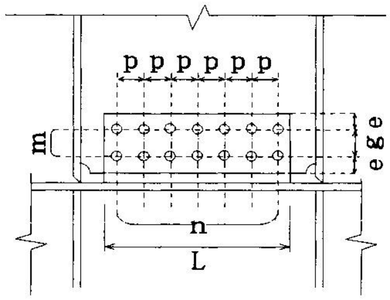


*Fig.2 bolted web connection details*

*Table 1 Test Specimens*

| Specimen | Column | Beam | Gasette Plate tg (mm) | L (mm) | H.T.B (F10T) | p (mm) | g (mm) | e (mm) | n | m |
|---|---|---|---|---|---|---|---|---|---|---|
| H-1-1 | BH-400×350×16×16 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 1 |
| H-1-2 | BH-400×350×16×16 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 2 |
| H-1-3 | BH-400×350×16×16 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 3 |
| H-2-1 | BH-400×350×16×16 | BH-500×100×16×16 | 22 | 380 | M16 | 50 | 50 | 40 | 7 | 1 |
| H-2-2 | BH-400×350×16×16 | BH-500×100×16×16 | 22 | 380 | M16 | 50 | 50 | 40 | 7 | 2 |
| H-2-3 | BH-400×350×16×16 | BH-500×100×16×16 | 22 | 380 | M16 | 50 | 50 | 40 | 7 | 3 |
| H-3-1 | BH-400×350×16×16 | BH-500×100×12×12 | 19 | 360 | M20 | 50 | 50 | 30 | 7 | 1 |
| H-3-2 | BH-400×350×16×16 | BH-500×100×12×12 | 19 | 360 | M20 | 50 | 50 | 30 | 7 | 2 |
| B12-1-1 | □-300×300×12 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 1 |
| B12-1-2 | □-300×300×12 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 2 |
| B19-1-1 | □-300×300×19 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 1 |
| B19-1-2 | □-300×300×19 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10 | 2 |
| B19-1-S1 | □-300×300×19 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 10x1+4x1 | |
| B19-1-S2 | □-300×300×19 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | 4 | 2 |
| B19-1-0 | □-300×300×19 | BH-500×150×12×16 | 19 | 375 | M12 | 35 | 35 | 30 | — | — |

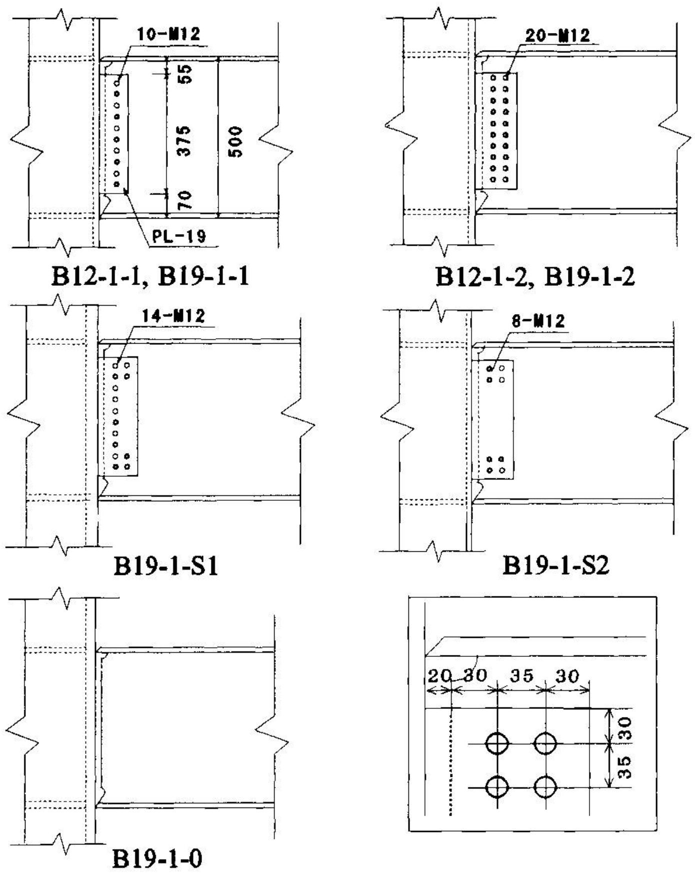


*Fig.3 Connection Details (B series)*

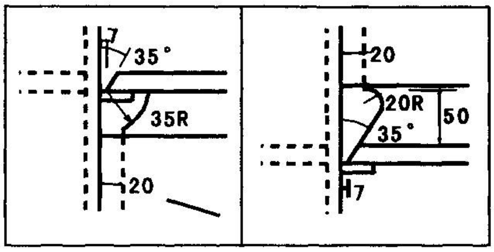


*Fig.4 Weld Access Hole Details*

*Table 2 Mechanical Properties*

| Specimen | | σy (t/cm²) | σu (t/cm²) | EL (%) |
|---|---|---|---|---|
| H series | PL-12 | 2.97 | 4.38 | 22 |
| | PL-16 | 2.67 | 4.21 | 22 |
| | PL-19 | 2.61 | 4.18 | 30 |
| | PL-22 | 2.56 | 4.14 | 34 |
| B series | PL-12 | 2.87 | 4.25 | 33 |
| | PL-16 | 2.82 | 4.17 | 31 |
| | CO-12 | 4.05 | 4.75 | 21 |
| | CO-19 | 4.50 | 5.10 | 19 |

σy : Yeild Strength σu : Tensile Strength
EL : Elongation

*Table 3 Test Results*

| Specimen | eMu (tcm) | Mp (tcm) | α | Failure mode | η | ηs |
|---|---|---|---|---|---|---|
| H-1-1 | 7121 | 5063 | 1.41 | F.M.1 | 27.4 | 15.2 |
| H-1-2 | 7426 | 5063 | 1.47 | F.M.1 | 32.6 | 17.1 |
| H-1-3 | 7661 | 5063 | 1.51 | F.M.1 | 49.3 | 28.8 |
| H-2-1 | 5217 | 4384 | 1.19 | F.M.2 | 8.0 | 5.3 |
| H-2-2 | 5664 | 4384 | 1.29 | F.M.2 | 11.9 | 7.6 |
| H-2-3 | 6181 | 4384 | 1.41 | F.M.2 | 13.0 | 8.1 |
| H-3-1 | 4230 | 3809 | 1.11 | F.M.2 | 12.9 | 6.7 |
| H-3-2 | 4606 | 3809 | 1.21 | F.M.2 | 14.9 | 7.7 |
| B12-1-1 | 6485 | 5162 | 1.26 | F.M.1 | 12.6 | 7.1 |
| B12-1-2 | 6673 | 5162 | 1.31 | F.M.1 | 13.5 | 8.2 |
| B19-1-1 | 6701 | 5162 | 1.30 | F.M.1 | 15.3 | 8.8 |
| B19-1-2 | 6989 | 5162 | 1.35 | F.M.1 | 18.3 | 10.4 |
| B19-1-0 | 6269 | 5162 | 1.21 | F.M.2 | 5.8 | 3.2 |
| B19-1-S1 | 7110 | 5162 | 1.38 | F.M.3 | 20.1 | 10.4 |
| B19-1-S2 | 6365 | 5162 | 1.23 | F.M.3 | 12.8 | 6.8 |

eMu : maximum bending strength
Mp : plastic bending moment (caliculated value)
α : maximum strength ratio of bending strength ( = eMu/Mp )
η : cumulative inelastic deformation ratio (based on M − θ curve)
ηs : cumulative inelastic deformation ratio (based on skeleton M − θ curve)

As for specimen B19-1-0, only the flanges were connected without web connection. Complete penetration groove welds were used to connect the beam flanges to the column flanges in all specimens. The detail of the weld access hole are shown in Fig.4. The material of all beams and H-shaped column were SS400 steel and material of all RHS columns were STKR 400 steel. The mechnical properties of the material of the members used in the specimens are summarized in Table 2.

## 3. Experimental Results

Each specimen was subjected to cyclically increasing displacements. Cyclic loading was continued until failure occurred at the connection. When all specimens reached the inelastic range, small cracks occurred at the flange of the beam near the weld access hole. At the ultimate stage the fracture of the beam flange, which started from the center of the flange, occurred and developed to overall the flange. Ultimately the local buckling was also observed at the compression side beam flange. The overall fracture of the beam flange at the specimens using the beam with usual cross-section, H-1-1, H-1-2, H-1-3, B12-1-1, B12-1-2, B19-1-1, B19-1-2, B19-1-S1 and B19-1-S2, occurred after sufficient plastic deformation (F.M.1). At the specime B19-1-S1 and B19-1-S2 web connecting bolts broke off simultaneously. At the H-2 type and H-3 type specimens brittle fracture occurred at the overall flange (F.M.2).

In Table 3 the maximum bending strength at the end of the beam eMu, which corresponds to the horizontal maximum load, the plastic bending moment Mp, maximum strength ratio $\alpha$ = eMu/Mp, failure mode, cumulative inelastic deformation ratio $\eta$ which were computed from the normalized beam end moment (M/Mp) versus the normalized beam end lotation ($\theta$/c$\theta$p) curves, cumulative inelastic deformation ratio $\eta$s from the skeleton curves of M/Mp-$\theta$/c$\theta$p relationships are summarized. The plastic lotation at the beam end c$\theta$p were calculated based on plastic displacement c$\delta$p. The skeleton curves of M/Mp-$\theta$/c$\theta$p relationship are shown in Fig.6.

Fig.7 shows the maximum strength ratio $\alpha$ versus the number of web bolt lines m relationship of the specimens using beams with same cross section. As for the specimen without web connction, B19-1-0, the $\alpha$ value was plotted at the number of bolt line m=0.

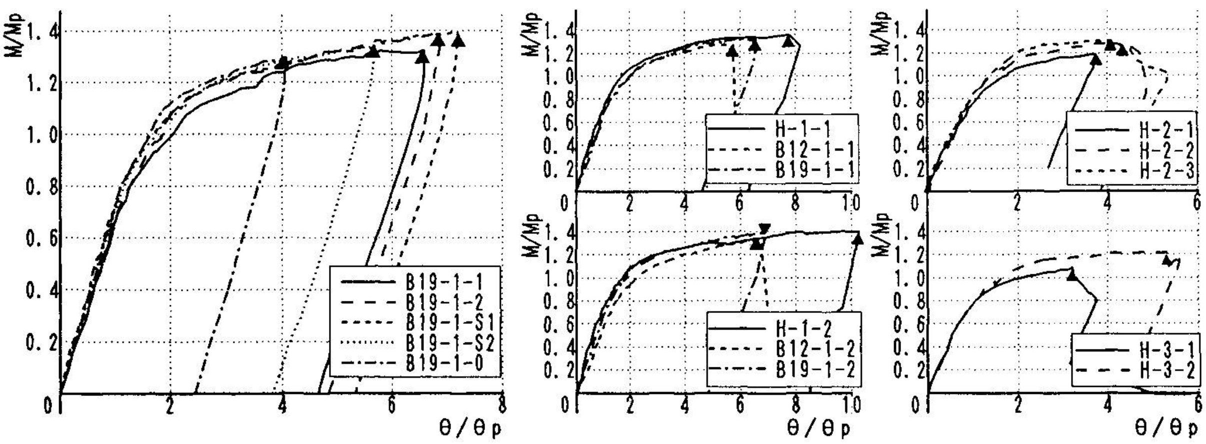


*Fig.6 M/Mp - θ/θp*

The maximum bending strength of the beam end connection increases with increasing of bolt line at the web connected part. The maximum strength ratio α versus the width-to-thickness ratio of the column flange D/t relationships are shown in Fig.8, where D and t are the width and the thickness of the column. As for the specimens using H-shaped columns the maximum strength ratio were plotted against D/t=0. It is observed that the maximum bending strength decreases with increasing of D/t.
The maximun strength ratio α versus the cumulative inelastic deformation ratio ηs relationship about H series specimens are shown in Fig.9. As above mentioned, the fracture of beam flanges of the H-2 and H-3 type specimens occurred in brittle manner, because the cross-section of the beam of those specimens were so special that the section module of the flange is extremely small in comparison with usual beam section. Therefore the value of ηs are very small, nevertheless number of bolt line at the web connected part increases. When the beams with usual cross-section such as that of the H-1 type specimen are used, ηs values are larger than 15 and it increases with increasing of bolt line. Fig.10 shows the α - ηs relationship of B series specimens. It is clear that ηs values of B series specimens were considerably small in comparison with those of H series specimens. The main reason of this fact is considered to be the difference of the cross section shape of the column. When the columns are RHS, web connected parts of the beam end connections do not work effectively to transfer the bending stress from web of the beam to column, because the out of plane stiffness of the web connected parts of the columns is very small. The tendency of this phenomenon appears more clearly, when D/t of the RHS column becomes large. From Fig.10 it becomes clear that the plastic deformation ability of the connected beams increases with incresing of the maximum bending strength of the beam end connections and that the maximum bending strength of the beam end connections increases with increasing of the maximum bending strength of the web connected parts. The bolts located near the flange work better than those located center of the web. But it is necessary to connect center part of the web at least one line to get better performance of the beam end connection.

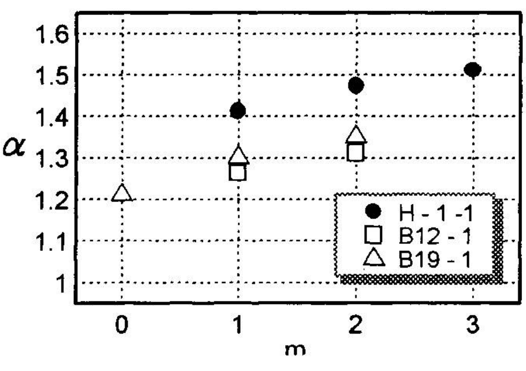


*Fig.7 α - m Relationships*

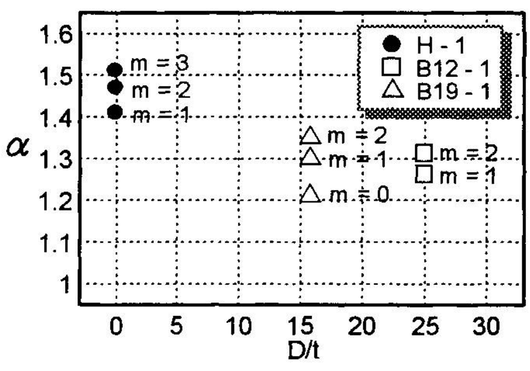


*Fig.8 α - D/t Relationships*

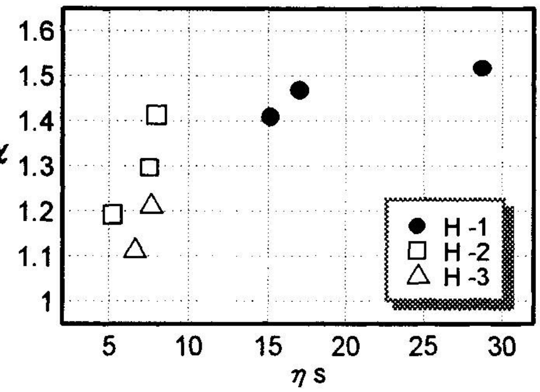


*Fig.9 α - ηs Relationships (H - series)*

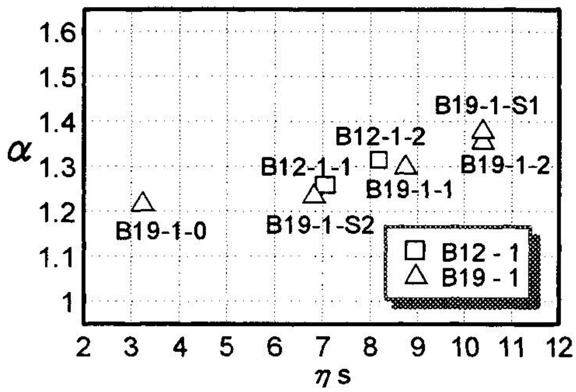


*Fig.10 α - ηs Relationships (B - series)*

## 4. Conclusion

In order to investigate the statical characteristics of WBFW type beam-to-column connections, experimental study was carrid out. Main items obtained from this study are summarized as follows.

1) The shape of the section of column member has large influence upon the statical characteristics of the beam end connections. In case column members are RHS, the maximum bending strength of the beam end connections are smaller than the case of using H-shaped column.
2) The plastic deformation ability increases with increasing of the maximum bending strength of the beam end connections.
3) The maximun bending strength of the beam end connections increases with increasing of the bending strength of the bolted web connection.
4) At the bolted web connection the bolts located near the flange work better than those located center part of the web. But it is necessary to connect center part of the web.

## 5. References

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