STATICAL CHARACTERISTICS OF THE IMPROVED RIGID BEAM-TO-COLUMN CONNECTIONS OF STEEL STRUCTURE

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SUMMARY

The statical characteristics of shop fabricated type, WBFW (web bolted and flange welded) type, and improved type beam-to-column connections were experimentally investigated. The specimens with WF-beam and SHS columns were loaded cyclically. The main test objectives are the effect of bending strength of the web connection of the usual beam-to-column connections and the effect of spread flange in improved beam-to-column connections. From this test, the basic statical characteristics, such as ultimate bending strength, plastic deformation capacity, failure mode are obtained. The results are summarized that maximum strength and deformation capacity are influenced by the bending strength of web connection. The improved beam end connections with spread flanges performed better than usual beam end connections.

INTRODUCTION

In Japan most of beam-to-column connections of multi-story steel buildings are designed as moment resisting rigid connections. 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. At the Great Hanshin-Awaji Earthquake many those two type of beam-to-column connections fractured in brittle manner. After the Earthquake many types of improved beam-to-column connections were proposed and investigated experimentally.

In this experimental study, first of all the fundamental statical characteristic of the usual beam-to-column connections were investigated. Here, "usual" means without any reinforcement at beam end. In the first stage shop fabricated type and WBFW type usual beam-to-column connections were tested. In addition beam-to-column connection without web joints was tested in order to make clear the effect of bending moment carried by web jointed part at beam end connections. From this study it becomes clear that the bending moment transfer ability of the web jointed parts have evident influence upon the statical characteristics of the beam-to-column connections. But the bending moment transfer ability of the web jointed parts is nearly 15% of the total bending capacity of the beam end connection at most. In order to obtain more effective performance at beam end connections, it is clear that some reinforcement is required at beam flange. From this viewpoint the improved beam-to-column connections is proposed. In the second stage the improved type connections were tested. The proposed beam-to-column connections have short length horizontally spread flanges, which are fabricated at fabricator shops and connected with beams by high strength bolts at construction site. The wide spread flange parts of the beam end are fabricated occasionally as outer type of diaphragms at beam-to-column connections. In this case fabrication of this type connections is very simple and cost saving, because it is not necessary to cut and re-weld column members. In this paper the results of an experimental study on the statical characteristics of improved beam-to-column connections are also presented.

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TEST SPECIMENS AND TEST SETUP

The shape of specimens using SHS columns and WF-beams was upside-down T type (Fig.1). The column members were □-400x400x19 and the beam members were H-500x200x10x16. The material of all beams was SS400 steel and material of all SHS columns was BCR295 steel. The mechanical properties of the material of the members used in the specimens are summarized in Table 1. The details of beam end connection were illustrated schematically in Fig.2. Full penetration groove welds were used to connect the beam flanges to the column flanges in all specimens. The specimen C21-B had WBFW connection. The bolted web connection was designed to carry full plastic moment of the web of the beam. The specimens C21-W1 had usual shop fabricated type connection where web of beam was fillet welded with fillet size of 0.7tw (tw: thickness of web). As for specimen C21-0, only the flanges were connected and web was not connected. The specimen C21-W2 and C21-W3 were designed as the improved beam-to-connections with spread flanges. Spread flange and diaphragm was made from one steel plate in a body. In C21-W2 the through type diaphragm was used and in C21-W3 the outer type diaphragm was used. All the bolts were F10T class high strength bolts.

Horizontal cyclic loads were applied to the tip of the beam. Those specimens were cyclically loaded under displacement of 2cδp, 4cδp, 6cδp and 8cδp, where cδp is the calculated elastic displacement at the tip of beam corresponding to the full plastic moment at the end of the beam.

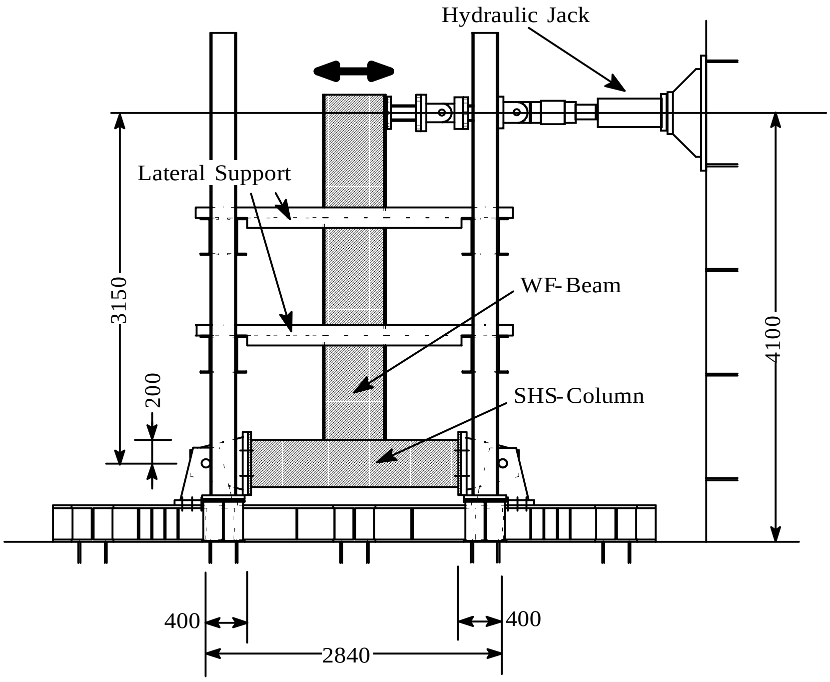

Fig.1 Test Setup

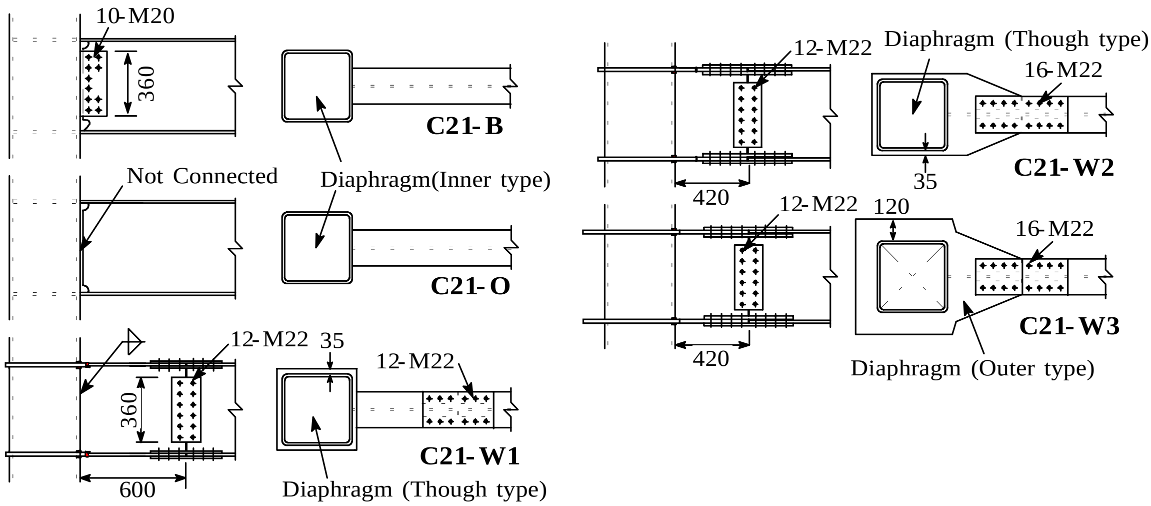

Fig.2 Beam End Connection Details

Table 1 Mechanical Properties of Steel

Name	Grade	Yeild Point (t/cm^2)	Tensile Strength (t/cm^2)	Elongation (%)	Yeild Ratio (%)	Used Member
PL-10	SS400	3.45	4.47	26.0	77	Beam Web
PL-16	SS400	3.15	4.38	28.8	72	Beam Flange
GPL-12	SS400	2.78	4.46	30.1	62	Gasette Plate
SPL-9	SS400	3.04	4.31	28.8	71	Splice Plate
SPL-16	SS400	2.72	4.27	32.9	64	Splice Plate
CPL-19	BCR295	4.41	5.36	19.5	82	Column
DPL-19	SS400	2.13	4.27	32.6	50	Diaphragm

EXPERIMENTAL RESULTS

Each specimen was subjected to cyclically increasing displacements. Cyclic loading was continued until failure occurred at the connection. But about C21-W2, C21-W3 any clear failure did not occur until ultimate loading, which was limited by the ability of loading apparatus. Load-displacement relationships are shown in Fig.3, and test results are summarized in Table2. In the specimens C21-0 (without web connection) and C21-B (WBFW type), the small cracks occurred at the flange of the beam near the weld access hole, when the beam end connection reached inelastic range. At the ultimate stage the brittle fracture of the beam flange, which started from the center of the flange at the weld access hole, occurred and developed to overall the flange. In the specimen C21-W1 (shop fabricated type) local buckling occurred at the compression side flange of the beam at the beam end, but no fracture occurred at the tension side flange. The specimen C21-W2 and C21-W3 (improved type) showed the similar behaviors. The small local buckling occurred at the plastic range after sufficient plastic deformation, but the load didn't decrease clearly until the final loading. At the specimen C21-W1, C21-W2 and C21-W3 slip of high strength bolted connection occurred at the beam joint after the load reached full plastic moment of the beams. In Table2, the horizontal maximum load at the tip of the beam Pmax, the maximum bending strength at the end of the beam Mmax, which corresponds to the load Pmax, the full plastic moment of the beam Mp, maximum strength ratio α (= Mmax/Mp), cumulative inelastic deformation ratio η which were computed from the normalized beam end moment (M/Mp) versus the normalized beam end lotation (θ/cθp) curves of each loading cycle, cumulative inelastic deformation ratio ηs and ηs' from the skeleton curves of M/Mp- θ/cθp relationships are summarized. The value ηs was cumulated until final cycle, and ηs' was cumulated until rotation angle of the beam is 1/25. In order to compare the plastic deformation ability from equal loading condition, ηs' is seems to be better parameter than ηs, because the slip was included in ηs of the specimen with the beam joint. The rotation at the beam end cθp were calculated based on displacement cδp. The skeleton curves of M/Mp-θ/cθp relationship are shown in Fig.4. The maximum strength ratio α versus the cumulative inelastic deformation ratio ηs’ relationship are shown in Fig.5.

Table 2 Test Results

Specimen	Pmax	Mmax	Mp	α	ƒ Å	ƒ sÅ	ƒ sÅ'	Failure mode
C21-O	23.50	7403	6867	1.08	11.3	2.76	2.02	Brittle fracture at flange
C21-B	25.90	8159	6867	1.19	25.1	4.97	2.11	Brittle fracture at flange
C21-W1	26.80	8442	6867	1.23	29.3	3.91	2.20	Local backling at flange
C21-W2	35.05	11041	6867	1.61	39.4	6.10	2.76	No failure
C21-W3	-31.50	9923	6867	1.44	34.7	5.65	3.03	No failure

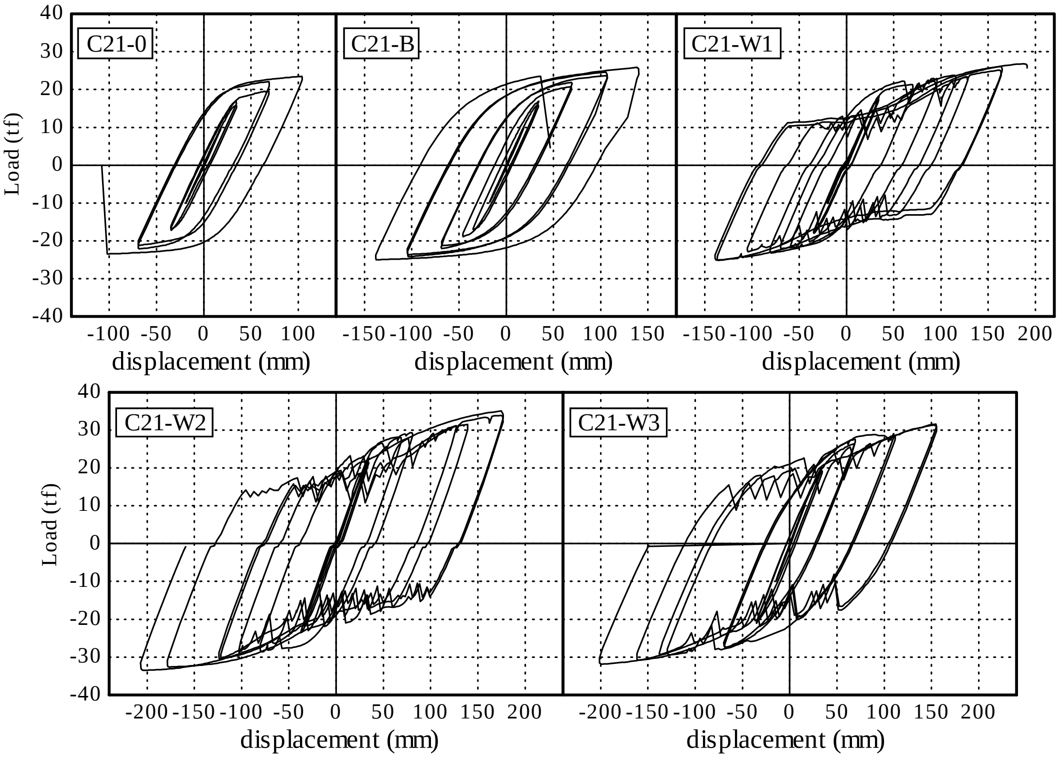

Fig.3 Load Displacement Relationships

INVESTIGATION

Judging from the test results of C21-B, C21-W1, it becomes clear that the statical characteristics of WBFW type connection, such as ultimate strength, plastic deformation capacity, were almost similar to those of the shop fabricated type connection, though their failure modes were different. The performance of C21-0 without web connection is worse than usual shop fabricated connections or WBFW connections. This result means that the web connection contribute to transfer bending moment of the beam not only shear force of the beam. From the test results of C21-W2 and C21-W3 with the improved beam end connections, it becomes clear that the values of α and ηs’ are considerably higher than those value of the other specimen. The reason of this fact is based on the effectiveness of spread flanges of the beam. The difference of the behaviors is not clear in both improved connections with the thorough type diaphragm and the outer type diaphragm.

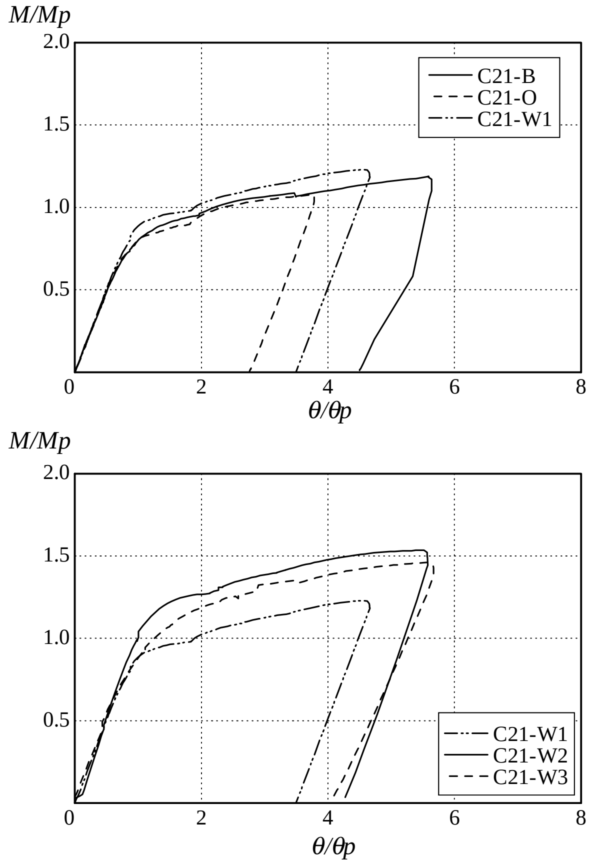

Fig.4 Skeleton Curves

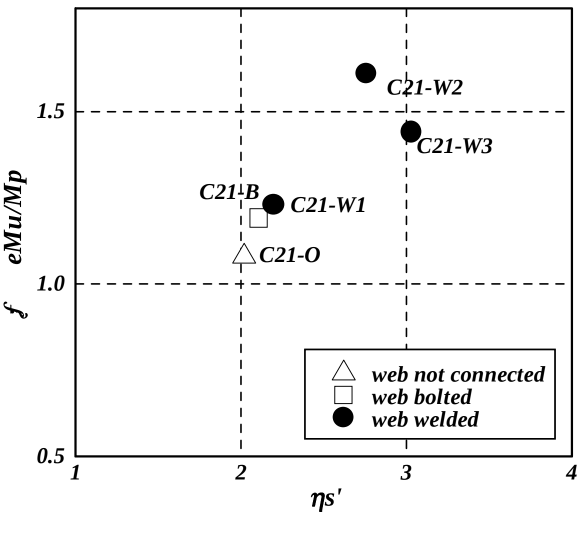

Fig.5 α−ηs'Relationships

Fig.6 shows the joint efficiency of web connection rw versus ηs for WBFW type beam-to- column connections. In the plots of the figure the previous data [1)-3)] were included with the data of this test. rw was the ratio of the calculated maximum bending strength at the web connection (jMu) [4)] to the full plastic moment of web part of the beam (Mwp). The data of rw=0 means the specimen without web connection. ηs' increases with increasing of rw. When the columns are SHS, 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 SHS column becomes large. As mentioned above, the bending strength of the bolted web connection was designed to carry the full plastic moment of the web of the beam in the specimen C21-B, but the value of r_W was nearly 0.5. The reason of this reduction of r_W was based on the effect of out of plane stiffness of column. From Fig.6 it is considered that web connection with appropriate efficiency must be expected for sufficient deformation capacity of the beam.

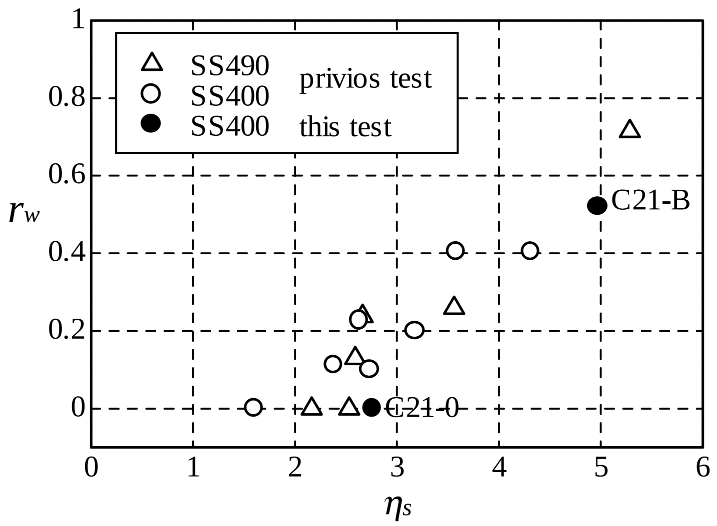

Fig.6 r_W–η_S Relationships

CONCLUSION

From this experimental study following items become clear. The statical characteristics of WBFW type usual connections were not different from those of the shop fabricated type usual connections, when web jointed parts were appropriately designed to carry certain bending moment. The moment carrying capacity of the web jointed parts is very important to design beam-to-column moment connections with good structural performance. But in order to get better performance at beam end connection, some reinforcement of the flange of the beam is required. The statical characteristics of the improved beam-to-column connections are much better than the usual beam-to-column connections that means the spread flange at the beam end works very effectively.

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