Cyclic Testing of the Column-tree Type and the WUF-B Weak-axis Steel Moment Connections

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15 WCEE
LISBOA 2012

SUMMARY:
The cyclic testing of two full-scale specimens investigated the seismic performance of the column-tree type and the welded unreinforced flange-bolted web (WUF-B) type weak-axis steel moment connections, which were manufactured according to existing and widely used major-axis steel moment connections. Through the test results, the column-tree type connection specimen was shown to have more than a 5% story drift capacity as compared to the WUF-B type connection specimen's 4%. These specimens were also shown to have higher strength capacities than the nominal design strength. The column-tree type connection specimen showed a slow degradation of strength after reaching the maximum strength. However, the WUF-B type connection specimen showed a rapid degradation of strength with brittle behavior.

Keywords: moment resisting frame, weak-axis steel moment connection, cyclic testing, seismic performance

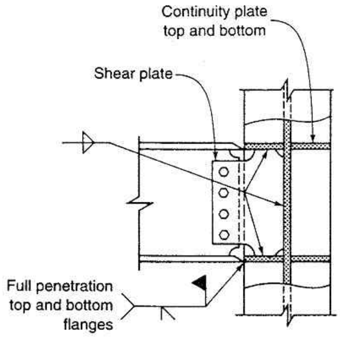
1. GENERAL

The steel moment frames have been extensively used in areas where earthquakes are a frequent occurrence. This is because the steel moment frames have excellent seismic performance. But after the Northridge earthquake that occurred in 1994 the beam-to-column connections of moment frames proved to display unsatisfactory seismic performance. So, the necessity to develop new connection details that have excellent seismic performance was brought up. Thus the prequalified connections were developed through the research of seismic performance of major-axis connections (FEMA-350, 2000) in US.
Recently, the possibility of earthquakes in Korea has increased. Hence there are local researchers who are concerned about evaluating the seismic performance of the beam-to-column connections of moment resisting frames, and who as a result have compiled significant research results from such endeavors. However, these research results have largely been concentrated on the strong-axis moment connections. In the United States the weak-axis moment connections hardly used for steel moment frame structures. Thus researches for the weak-axis connections can hardly be found. In Korea on the other hand, the weak-axis moment connections are as commonly used for steel moment frame structures as are the strong-axis moment connections. Nevertheless, the seismic performance of weak-axis moment connections can't be ensured in the absence of the related literatures. So it's of great urgency to evaluate the seismic performance of weak-axis connections.
For this purpose, an experimental program was designed and performed with two types of weak-axis steel moment connection types— the column-tree type connection and the welded unreinforced flange-bolted web(WUF-B) type connection. These two types of connections were selected upon consulting various existing field data and literatures, which showed that these two types of connection are most common within the weak-axis steel moment connections used in Korea.

2. LITERATURE REVIEW

Based on the investigation of construction sites and design documents, Fig.1 shows the details of the column-tree type and WUF-B type weak-axis steel moment connections, which are extensively used for actual moment frame structures.

(a) Column-tree type



(b) WUP-B type

Figure 1. The details of weak-axis connections

Kim et al.(2004) and Park et al.(2007) evaluated the structure performance of the weak-axis connections, but the research conclusions were obtained under monotonic gravity load. The evaluations of the seismic performance of the column-tree type and the WUP-B type connections under cycle load are limited. Also, even in the Seismic Provisions for Structural Steel Buildings approved by the AISC, there are no criteria regarding the weak-axis connections because these connections are not used for steel moment frames in USA.

Therefore, the application of the column-tree type and the WUF-B type weak-axis steel moment connections is difficult under the given circumstances, where that the seismic performance of weak-axis connections cannot be determined. Nevertheless, this issue needs to be resolved because these types of weak-axis connections are being widely used for moment frame structures.

Hence, this paper briefly introduces the design method of weak-axis connection details. Then the paper evaluates the seismic performance of the column-tree type and the WUF-B type weak-axis steel moment connections through the cyclic testing of two full-scale specimens.

3. FULL-SCALE TEST PROGRAM

This research program tested two full-scale weak-axis connection specimens that were manufactured according to related literatures and the prequalified connections shown in FEMA-350 using a predetermined cyclic deformation history. The two specimens were the CT-W specimen and the WUF-B specimen. The specimen configurations are summarized in Table 3.1.

Table 3.1. Weak-axis Connection Specimens

Specimen	Type	Beam	Column
CT-W	Column-Tree Type	H-600×200×11×17	H-400×400×13×21
WUF-B	Welded Unreinforced Flange-Bolted Web	H-600×200×11×17	H-400×400×13×21

3.1. SPECIMEN DESIGN

Specimens were fabricated using the rolled H-shape. The beam size was H-600×200×11×17 and the column size was H-400×400×13×21. High-strength bolts used the M22 T/S. Generally, in order to prevent the soft-story damage, the strong column-weak beam system should be the structure system. The following relationship shall be satisfied at beam-column connections,

$$\frac{\Sigma M^{*}_{pc}}{\Sigma M^{*}_{pb}} > 1.0 \tag{3.1}$$

where, ΣM^{*}_{pc} is the sum of the moments in the column above and below the joint at the intersection of the beam and column centerline. ΣM^{*}_{pb} is the sum of the moments in the beam at the intersection of the beam and column centerline.
The specimens were designed to meet the criteria as defined in Appendix S in AISC Provisions.

3.1.1. Column-tree type specimen
In the column-tree type specimen, the short stubs of beam were welded to the weak-axis of the column in the shop and then the middle portion of the beam spans were bolted to the column-tree in the field. Thus, the system consisted of a field bolted-shop welded structural system. The column-tree connection details are shown in Fig. 2.

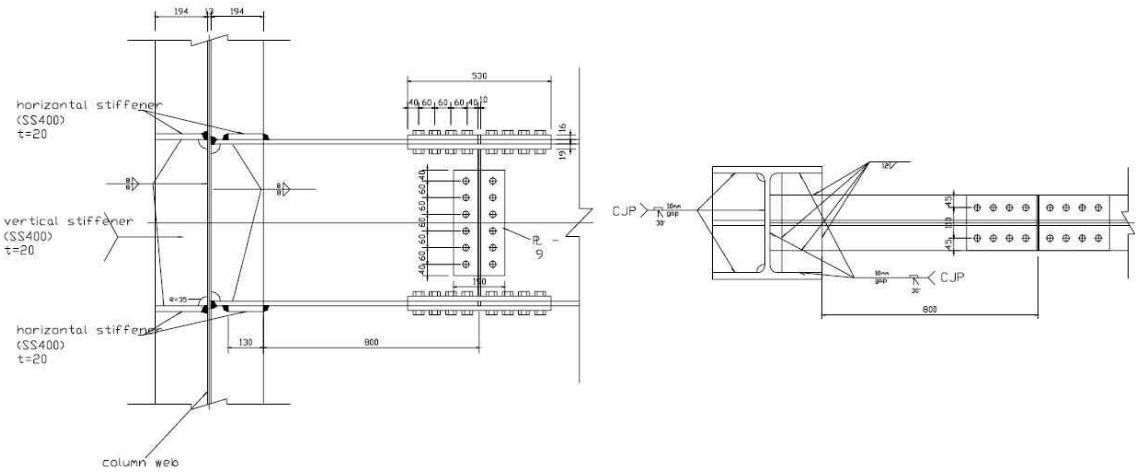


Figure 2. CT-W type specimen

The horizontal stiffeners and the short stubs of beam were welded using complete joint penetration. Welding procedures were covered by AWS D1.1 (Structural Welding Code-Steel, American Welding Society). Bolted connections used high-strength bolts and this ensured that the moments of the beam could completely transmit to the column tree. The nominal strengths of bolted connections are given in Table 3.2.

Table 3.2. Nominal Strength of Bolted Connections

	Slip strength	Bearing strength	Bolt breaking	Block shear
Strength(kN)	7356.8	359	172.9	841.76

3.1.2. WUF-B type specimen

The WUF-B type specimen was fabricated in accordance to the pre-Northridge WUF-B beam-to-column connections. The top and bottom flanges were welded to the horizontal stiffeners using complete joint penetration (CJP), meeting the requirements of the AWS D1.1. But the weld backing bars were not removed from the flange weld joint. The WUF-B type connection details are shown in Fig. 3.

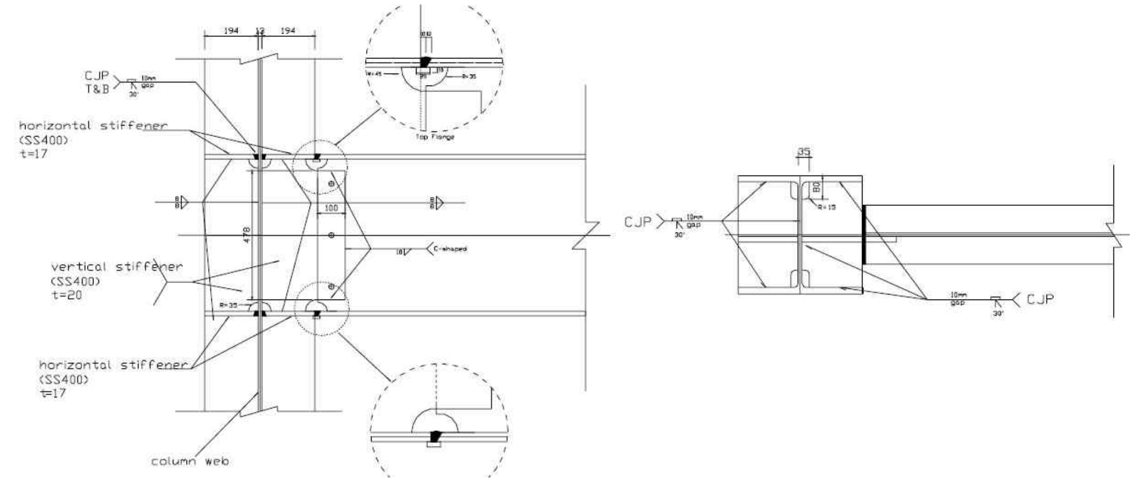


Figure 3. WUF-B type specimen

3.2. MEASUREMENT AND LOADING HISTORY

The test specimens were subjected to cyclic loads according to the requirements prescribed in Section S6.2 for beam-to-column connections of Seismic Provisions for Structural Steel Buildings (2005). The loading sequence is shown in Fig. 4.

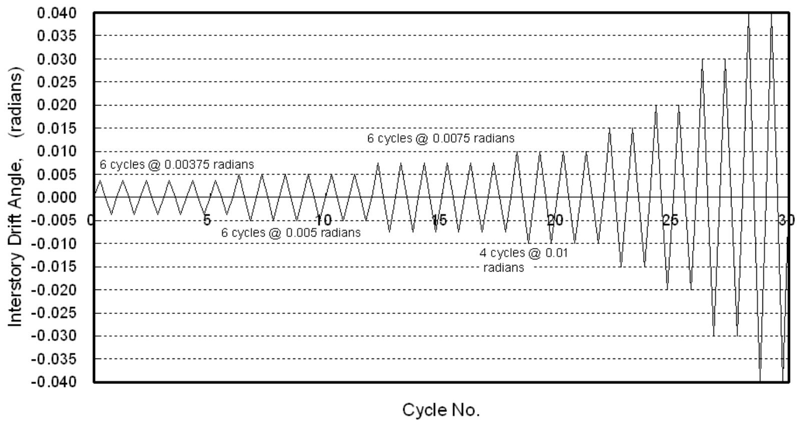


Figure 4. Loading sequence

3.3 TEST SETUPS

Each specimen was tested in the horizontal plane, as shown in Fig. 5. A 500kN actuator was used to impose the predetermined cyclic deformation history. A guide frame was also placed in order to prevent any out-of-plane instability of the beam.

Linear varying displacement transducers (LVDTs) were attached to the specimens to identify the displacement of structures. Also, strain gauges were attached to the backside and front of the beam web as well as the top and bottom of the beam flange to identify local strains. This set up is shown in Fig. 5 and Fig. 6.

After strain gauges and LVDTs were attached, the specimens were whitewashed in order to better observe the deformations of the specimens.

The test was conducted by controlling the level of axial or rotational deformation imposed on the specimens. The test was stopped promptly if a specimen broke, or if the maximum allowable deformation of the actuator was reached, or when the measured strength of the specimen dropped to the maximum strength of 80% after the maximum strength.

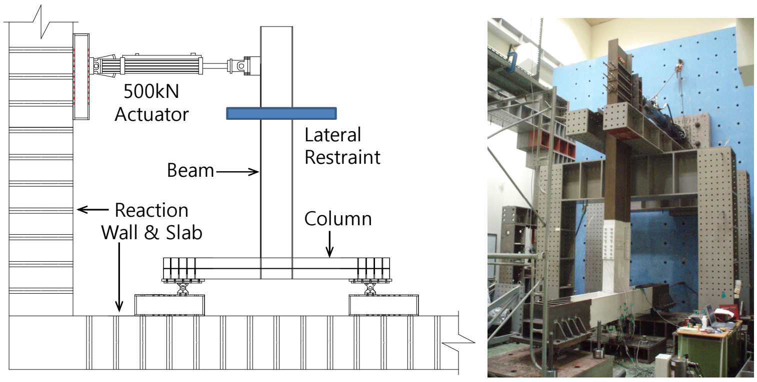


Figure 5. Test setup

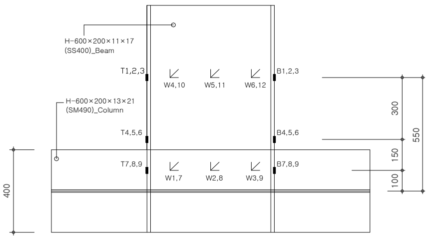


Figure 6. Gauges Layout

4. TEST RESULT

4.1 MATERIAL TESTS

In order to understand the mechanical properties of the H-shape steels used for these specimens, the research collected twelve coupons from the beam and column of each specimen after the test ended. Then the research included the testing of each of the coupon tensile properties according to the test requirement prescribed in KS B 0801. The tensile coupon test results are summarized in Table 4.1. Note that actual yield strength of each coupon exceeded the nominal yield more than 1.2 times.

Table 4.1. Properties of the coupons

Shape	Location	Class	Actual yield strength/Nominal yield strength (MPa)	Tensile strength (MPa)	Yield ratio
H-400×400×13×12	Web	SM490	398.4/325	559.7	0.71
	Flange	SM490	407.9/315	571.4	0.71
H-600×200×13×12	Web	SS400	336.3/245	470.5	0.71
	Flange	SS400	292.3/235	455.8	0.64

4.2 LOAD-DISPLACEMENT CURVES AND MODE OF FAILURE

The load-displacement curve of CT-W and WUF-B specimens are shown in Fig. 7 and 8, respectively.

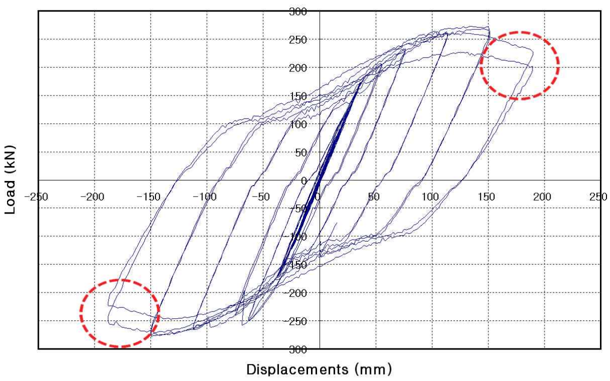


Figure 7. Load-displacement Curve for CT-W

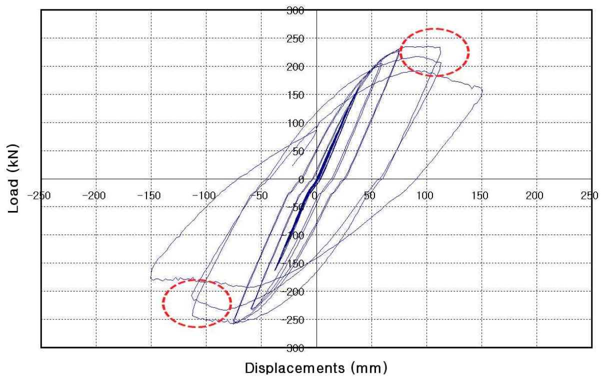
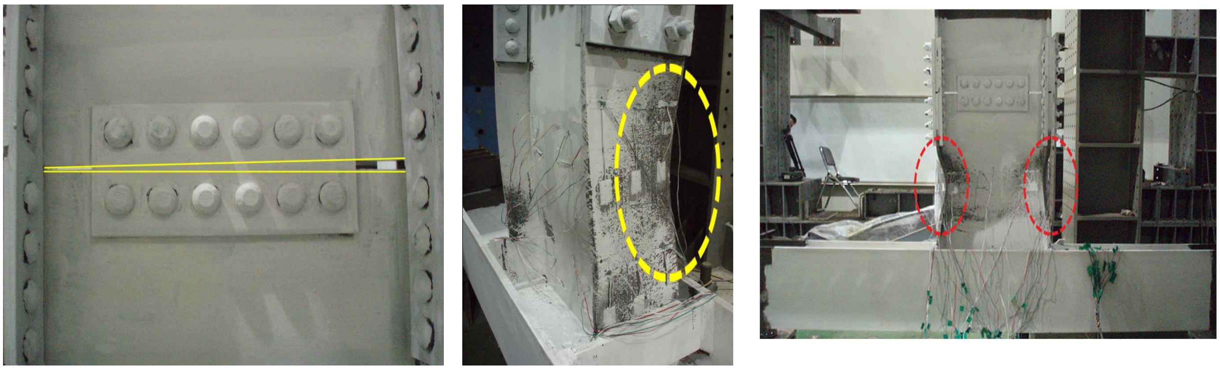


Figure 8. Load-displacement Curve for WUF-B

4.2.1. CT-W specimen

Slip of bolts was detected at a control displacement of 75.7mm (story drift ratio δ_r=2%), while the gap between the column tree and the beam unsymmetrically expanded, as shown in Fig. 9(a). The local buckling of the beam top flange occurred, at a control displacement of 151.4mm (story drift ratio δ_t=4%), and this is shown in Fig. 9(b). Meanwhile, the gap between the column tree and the beam slightly decreased. The maximum resistance strength was detected at the control displacement of 151.4mm (story drift ratio δ_t=4%) and this is depicted in Fig. 7. Finally, local buckling occurred at the beam top flange at the control displacement of 189.25mm (story drift ratio δ_t=5%). The resistance strength began to fall and the test was stopped at the maximum resistance strength of 80% as shown in Fig. 9(c).

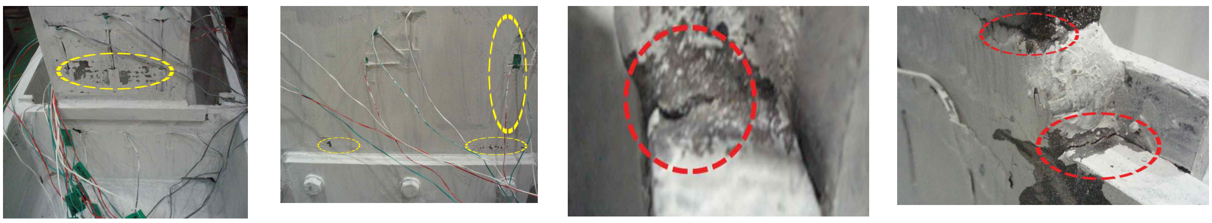


(a) Slip of bolts(δ_t=2%)   (b) Buckling of bottom flange(δ_t=4%)   (c) Buckling of top and bottom flange

Figure 9. Failure shapes of CT-W specimen

4.2.2. WUF-B specimen

The yielding of the beam bottom flange was detected at the control displacement of 59.7mm (story drift ratio δ_t=1.5%), and this is shown in Fig. 10(a). The yielding of the beam's top flange occurred at the control displacement of 75.7mm(story drift ratio δ_t=2%). Also, the yielding of the beam web occurred at the control displacement of 113mm (story drift ratio δ_t=3%), and this is shown in Fig 10(b). Preceding these occurrences, the weld joint of the beam bottom flange split and the resistance strength began to fall, as shown in Fig 10(c). The maximum resistance strength was detected at the control displacement of 113mm (story drift ratio δ_t=3%) as shown in Fig. 8. When the fracture of the weld joint at the beam top flange expanded, the resistance strength rapidly decreased at the control displacement of 151.4mm (story drift ratio δ_t=4%), as shown in Fig 10(d). The test was stopped at the maximum resistance strength of 80%.



(a) Bottom flange yielding   (b) Web yielding   (c) Weld joint crack   (d) Weld joint crack

Figure 10. Failure shapes of WUF-B specimen

5. ANALYSIS OF TEST RESULT AND INVESTIGATION

In this chapter, the paper will try to obtain hysteretic curves of story drift ratio and moment plastic-rotation for each of these specimens through analyzing the cyclic test results and comparing the inelastic behavior of each specimen by these hysteretic curves.

A story drift ratio(δ_t), which can be used to get a story drift angle, is expressed as the quotient of a displacement of the beam end divided by a distance from the center of the column to the load point, expressed as,

$$\delta_t = \frac{\Delta}{L_{cb}} \times 100(\%) \tag{5.1}$$

where, Δ is the displacement of the beam end and L_{cb} is the distance from the column center to the load point.

And the moment plastic-rotation (θ_P), which can evaluate plastic deformable ability of a connection, is given as follows,

$$\theta_p = \frac{\Delta - P/K_i}{L_{cb}} \times 100(rad) \tag{5.2}$$

where, P is the applied load and K_i is the initial stiffness of the beam-to-column connection.

Hysteretic curves of story drift ratio and moment plastic-rotation for each of these specimens are shown in Fig. 11 and Fig. 12, respectively. These figures plot δ_t versus M_f/M_p, where M_f is the beam moment at the column face, and M_p is the beam plastic moment calculated from the specimen test results. M_f/M_p is greater than 1 because of the over-strength that results from the strain hardening of the connection components.

5.1 SEISMIC PERFORMANCE EVALUATION

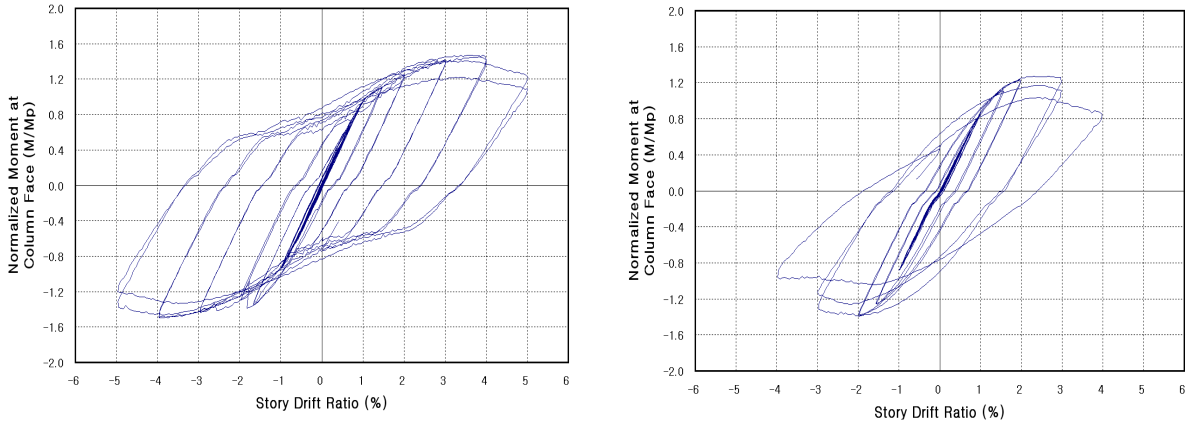
According to the requirement of Seismic Provisions for Structural Steel Buildings(2005) approved by the AISC, for beam-to-column connections used in Special moment frames(SMF) the intermediate moment frames(IMF) and ordinary moment frames(OMF) shall be capable of sustaining a moment plastic- rotation of at least 0.03, 0.02 and 0.01radian, respectively.

However, the requirement of SAC(2000) is stricter, which states that for beam-to-column connections, the SMF and IMF both shall be capable of a total rotation angle of at least 0.04 radian, where a total rotation angle is used instead of a plastic rotation angle in consideration of a elastic rotation angle of 0.01 radian. Furthermore, the SAC (2000) requires that the beam-to-column connections of SMF bear a load cycle of a total rotation angle of 0.04 radians at least, and that the beam moment strength be more than the beam nominal plastic moment strength of 80%.

In this test, the CR-T specimen that was used as a column-tree type connection had sustained a story drift ratio of 0.04 radian for one cycle at least through observing Fig. 11. Therefore, the column-tree type connections can be used for SMF. Although the WUF-B beam-to-column connections are banned from using for IMF and SMF, this type of connections is used as widely as column-tree type connections. In this test, the maximum story drift ratio of the WUF-B specimen was 0.03 radians, and this can be attributed to the fracture in the weld joint at the beam top flange. So, the WUF-B type beam-to-column connections can't be used for SMF since this type of frame requires a story drift ratio of 0.03. Yet, the WUF-B type connection does indeed satisfy the requirement of IMF, which requires a story drift ratio of 0.02.

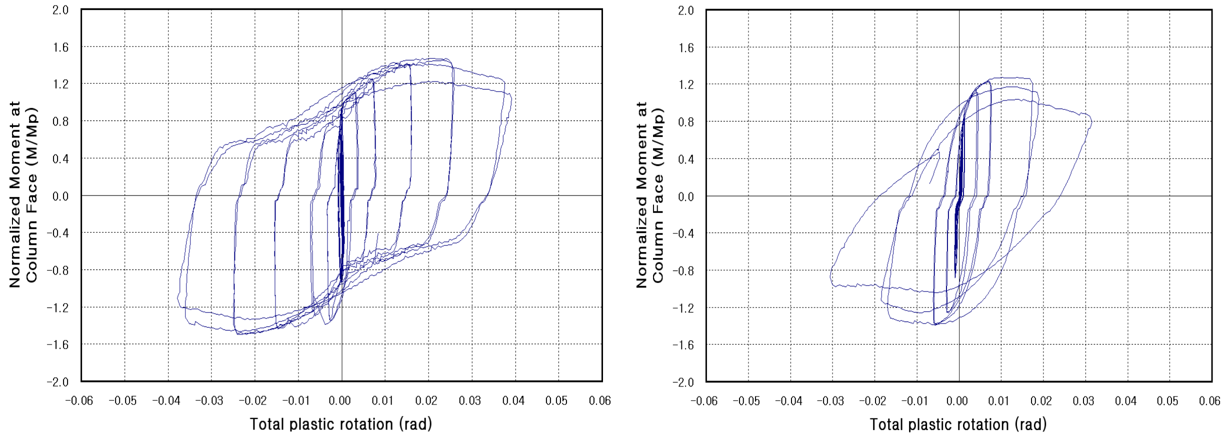
5.2 COMPARISON OF STORY DRIFT RATIO

This chapter compared the story drift ratio of each specimen, which was obtained under the same test conditions. As shown in Fig. 13, the story drift ratio of CR-T specimen as a column-tree type connection was 25 percent more than the story drift ratio of WUF-B specimen as a WUF-B type connection. Regarding the downward trend of strength following the maximum strength, the CR-T specimen was slower than WUF-B specimen. The reason for this is that the separation of the column tree and the beam from each other at the joint could have absorbed some story drift. This mechanism is very similar to the effect of fuse element, but different from that of the RBS (reduced beam section), where a portion of the beam near the column was weakened in the reduced beam section scheme so that plastic hinging would occur at the designated location. In contrast, the WUF-B specimen as a pre-Northridge WUF-B type beam-to-column connection had little plastic rotation capacity because the early fracture appeared at the weld joint of the beam flange.



(a) Story drift ratio of CR-W (b) Story drift ratio of WUF-B

Figure 11. Hysteretic curves of story drift ratio



(a) Moment-plastic rotation of CR-W (b) Moment-plastic rotation of WUF-B

Figure 12. Hysteretic curves of moment-plastic rotation

6. CONCLUSIONS

The column-tree type and the WUF-B type weak-axis steel moment connection specimens were tested cyclically to study the seismic performance of the two different types of connections. The following conclusions can be made for the specimens:

1. For the CR-T specimen as a column-tree type connection, local buckling of the beam flange appeared at a story drift ratio of 0.05 radians. Then, the resistance strength of the specimen began to slowly fall. But the specimen could sustain the beam nominal plastic moment strength of 80% at the story drift ratio of 0.04 radians. Therefore, the column-tree type connections can be used for SMF. However, a very important point to note is that the displacement characteristics due to the slip of bolts need more research through further experiments and analyses.
2. For the WUF-B specimen as a WUF-B type connection, the fracture of the beam flange weld joint

occurred at the story drift ratio of 0.03 radians. Then, the resistance strength of the specimen began to fall rapidly. Even so, the WUF-B type connections can satisfy the requirements of story drift ratio of 0.03 radians to be applied to IMF. The resistance strength of the specimen rapidly decreased when the fracture of the weld joint occurred. Hence, further study is necessary to fully understand the effects of the fracture of the weld joint.

AKCNOWLEDGEMENT

This research project was funded by the NATIONAL RESEARCH FOUNDATION of KOREA (KRF-2006-003 -D00628). Participants in the research were sponsored by the BK21 Project.

REFERENCES

Sang Seup Kim, Do Hyung Lee, Jeong Tae Ham and Kyu Suk Kim.(2004).Development and strength evaluation of beam-to-column connection details in weak axis of H-shape column.*Journal of korean society of steel construction***16:1**,169-180.

Sang Seup Kim, Do Hyung Lee and Jeong Tae Ham.(2004).The structural behavior of strong axis connections by type of weak axis connection -in case of loading gravity load.*Journal of korean society of steel construction***16:2**,275-284.

Jong Won Park, Jae Hoon Kim, Sang Woo Jeon and Yong Jun Oh.(2007).A study on the economy of weak-axis beam-to-column connections.*Journal of korean society of steel construction***19:6**,663-670.

Cheol Ho Lee, Seoung Min Kang and Jin Ho Kim.(2006).A balanced panel zone strength criterion for reduced beam section steel moment connections.*Journal of korean society of steel construction***18:1**,59-69.

Cheol Ho Lee and Jong Won Parkd .(1998).Cyclic seismic testing of full-scale column-tree type steel moment connections.*Journal of korean society of steel construction***10:4**,629-639.

Chad, S. Gilton.and Chia-Ming. Uang (2002).Cyclic response and design recommendations of weak-axis reduced beam section moment connections.*Journal of Structural Engineering, ASCE***128:4**,452-463.

Rentschler, G. P., Chen, W. F, and Driscoll, G. C (1980).Tests of Beam-to-Column Web Moment Connections.*Journal of the Structural Divisio, ASCE***106:ST5**,1005-1022.

Uang, C. M., Bondad, D., and Lee, C. H. (1998).Cycle performance of haunch repaired steel moment connections: experimental testing and analytical modeling, *Engineering Structures*, **20:4-6,**552-561.

AISC.(2005). Seismic Provisions for Structural Steel Buildings, Chicago, IL, U.S.A

ANSI/AWS D1.1-00.(2006).Structural Welding Code-Steel, American Welding Society, Miami, FL, U.S.A.

FEMA. (2000).Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings: FEMA-350, SAC Joint Venture, Richmond, Calif, U.S.A.

RCSC. (2004).Specification for Structural Joints Using ASTM A325 or A490 Bolts: LRFD, AISC.