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Seismic Evaluation of Beam-Column Weak-Axis Connections in Small-Size Steel Structures

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***Abstract*— In this study, cyclic loading tests for beam-to-column weak-axis connections were performed to investigate the seismic performance. The connections were developed to improve the constructability based on the investigation of the existing small-size steel structures. The primary test parameter is the number of high-tension bolts which are used to connect steel beam and column using exterior and interior flange plates. Test results showed that the number of high-tension bolts had a significant effect on the cyclic behavior of weak-axis connections. When more than four bolts were used in the connections, the stiffness and strength of the connections were satisfied with the requirements for semi-rigid connection presented in current design codes. All of the specimens showed the bearing failure around bolt holes and fracture at the beam flange. However, when the web height and the flange width is relatively small, the number of the bolts used in the connections might be limited.**

Index Terms— **Seismic Evaluation, Beam-column connections, Weak-axis, Small-size steel structures, Stiffness.**

I. INTRODUCTION

Small-size steel structures which are defined as the building with less than three floors and with smaller gross area than 500 m^2 may have a high possibility of severe damage during the earthquake because those buildings are constructed without consideration of appropriate seismic design criteria (KSEA, 2010). In particular, most of beam-column connections of small-size structures have various details that are unsuitable for the design criteria. According to the investigation of existing small-size steel structures, some beam-column weak-axis connections were constructed as pinned connections. Thus, the development of beam-column weak axis connections is needed to ensure proper seismic performance. In this study, the connections were developed to improve the constructability and flexural performance based on the investigation of existing small-size steel structures. Through the cyclic loading tests, the seismic performance of improved connections is evaluated.

II. TEST PROGRAM

A. Characteristics of the test specimens

Cyclic loading tests were performed to evaluate the seismic performance of for beam-column weak-axis connections. A total of three test specimens was tested. The primary test parameter is the number of high-tension bolts used to connect steel beam and column using exterior and interior flange plates. Fig. 1 shows the details of the specimen. In this study, the test specimens were adopted the H-shaped beam (H-400×200×8×13 in mm) and column (H-300×300×10×15 in mm) sections given in the small building structure guidelines (KSEA, 2010). The BCW1, BCW2, and BCW3 specimen had a number of four, eight, and twelve high-tension bolts, respectively. For manufacturing the beam-column connection, H-shaped beam and column with a section of H-400x200x8x13 and H-300x300x10x15 were used, respectively. Overall height of the column and a length of the beam is identical (= 2000 mm). The distance between the beam end and a loading point is 1850 mm.

The plastic moment ($M_{pb} = Z_bF_{yb}$) of the beam section in the direction of strong-axis using the nominal strength of the steel is equal to 384.4 kN·m, and the value ($M_{pc} = Z_cF_{yc}$) of the weak-axis column section is 160.7 kN·m. Here, Z_b (= 1,330 × 10^3 mm^3) and Z_c (= 684 × 10^3 mm^3) are the plastic section modulus of the beam and column section, and F_{yb} and F_{yc} are the nominal yield strength (= 235 MPa) of the beam and column section, respectively. Thus, the ratio (M_{pc}/M_{pb}) of the plastic moment between the column and beam section is 0.34. This does not satisfy the strong column-weak beam condition presented in the current steel structure criterion. In general, small-size buildings are designed only for gravity loads, since the effect of lateral forces is relatively small. In addition, the columns in

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small-size steel structure is subjected to a relatively low compressive force, so the column depth is smaller than the beam depth. In this case, since the earthquake regulations stipulated in the current standards are not enforced in small-size steel structures, it is possible to design the weak column-strong beam joint.

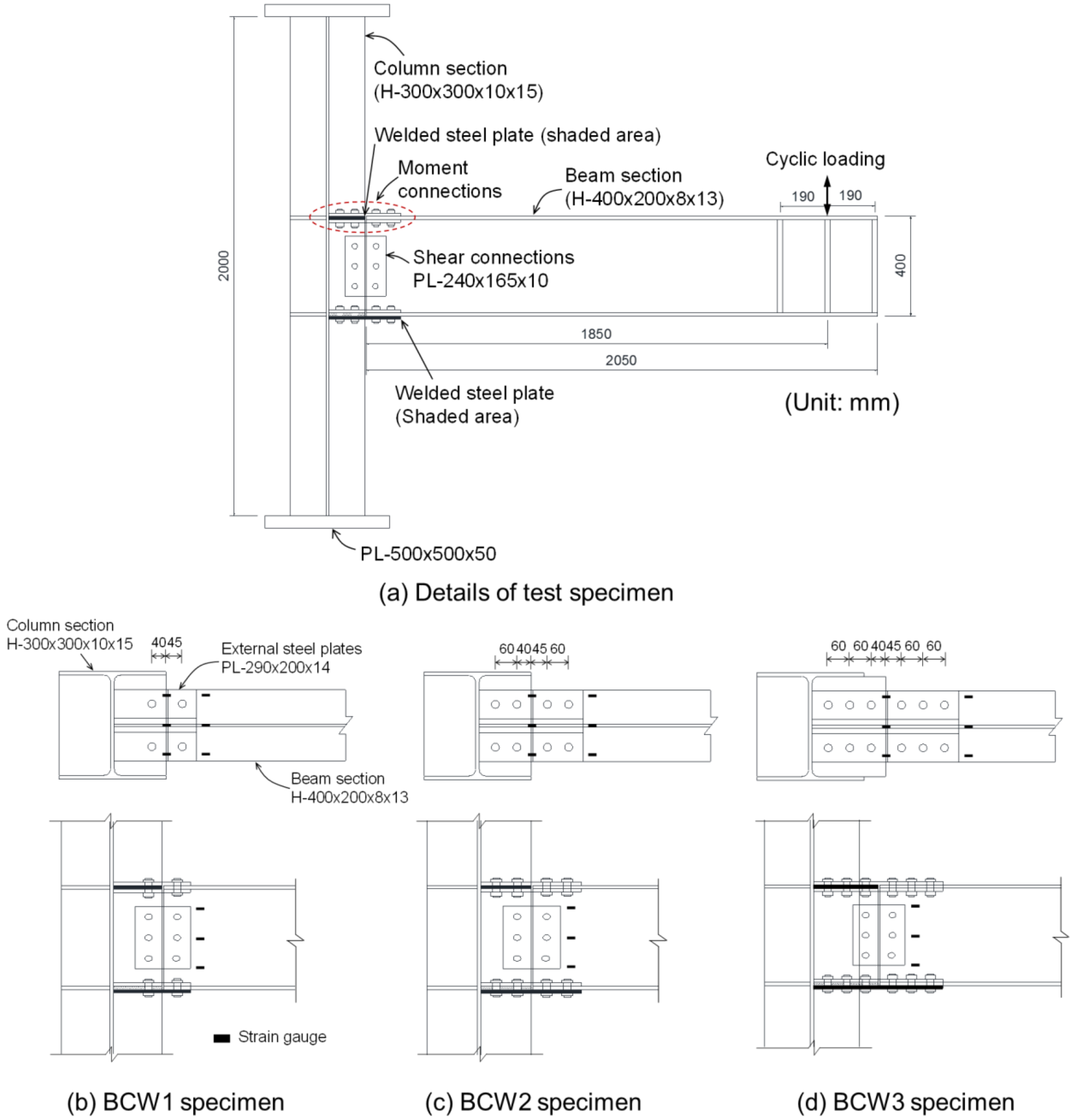

Fig. 1: Details of the test specimens

For connecting the beam and column, exterior and interior flange steel plates were placed to connect the extended steel plate at the joint and beam flange. Here, the extended steel plate was installed on the web of the column in the joint. In addition, to use the shear connections vertical stiffener with three bolt holes was installed in the joint. After assemblage of flange connections, the vertical stiffener is connected to the web of the beam using shear tab

B. Test Setup and Measurements

Fig. 2 shows the test setup and loading history curve. The test specimens were subjected to cyclic loading at the end of the beam using a 2000 kN dynamic actuator [see Fig. 2(a)] in displacement control. Fig. 2(b) shows the loading history curve. For lateral cyclic loading, loading history in ANSI/AISC 341-10 (2010) was used.

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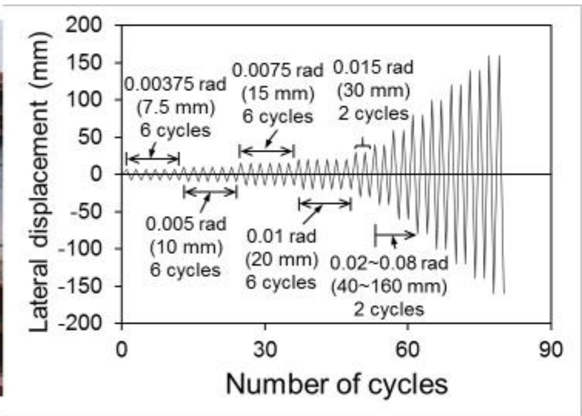
A strain gauge was attached to measure the strain of the beam web, beam flange, flange connecting plate. The installation position of the strain gage and the installation position of the displacement gauge (LVDT) are shown in Fig. 1. Three strain gauges were installed 100 mm apart from the web connection plate to measure deformation of the beam web. To measure the flexural deformation of the beam flange, six strain gauges were attached, three on the flange and 100 mm away from the edge of the flange connection plate. A total of 6 strain gauges was attached, three at the top and bottom of the flange connection plate. The lateral displacement of the beam was measured by a wire gauge installed at the point of application.


(a) Test setup



(b) Loading history

Fig. 2: Test setup and loading history

C. *Material Properties*

In order to determine the material properties of the materials used in the test specimens, uniaxial tensile tests were performed according to ASTM E8/E8M (2016). Table 1 shows the material test results. All the steels used in the test specimens were SS400 grade steel. Yield strength, tensile strength and elastic modulus were measured by material test.

Table 1: Material properties

Components		Yield strength (MPa)	Tensile strength (MPa)	Modulus of elasticity (GPa)
Steel beam (SS400)	Web (8 mm)	377.3	443.0	200.0
	Flange (13 mm)	289.0	444.5	197.2
Steel column (SS400)	Web (10 mm)	351.9	498.2	206.2
	Flange (15 mm)	329.0	476.4	203.8
Shear connection (SS400) (10 mm)		324.0	490.7	204.4
Flange plate (SS400) (14 mm)		297.8	468.3	194.6

III. TEST RESULTS

A. *Failure modes of the test specimens*

Fig. 3 shows the failure modes for each specimen at the end of the tests. For the BCW1 specimen, as shown in Fig. 3(a), the flange of the beam was fractured after the flange connection plate yielded. The bottom connecting plate yielded first at 0.02 rad, and then the top connecting plate yielded at 0.0268 rad. However, the web of the beam did not yield until the end of the experiment. When the rotation angle was 0.06 rad, fracture of the lower flange of the beam occurred.

For BCW2 and BCW3 specimens, the fracture failure of the beam flange occurred after the connection plate yielded, and finally the web of the beam was also yielded. For the BCW2 specimens, the beam and the lower connecting plate yielded at a rotation of about 2%. Then, the lower and upper flange of the beam yielded at -0.032 and -0.034 rad, respectively. After the bottom flange of the beam yielded, the web of the beam yielded at -0.045

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rad. As shown in Fig. 3(b), when the rotation angle was 0.06 rad, the fracture occurred in the lower flange of the beam. For the WS3 specimens, as shown in Fig. 3(c), the upper and lower flanges of the beam were yielded at -0.024 rad and -0.025 rad, respectively. The web of the beam web yielded at 0.036 rad.

(a) BCW1 (b) BCW2 (b) BCW3

Fig. 3: Failure modes of the test specimens

B. Moment-rotation relationship

Fig. 4 shows the moment-rotation angle relationship for each specimen. The yield point was defined by the equivalent energy method, as shown in Fig. 5, the moments when the areas enclosed by the enveloped curves and the idealized elastomeric curves are equal are defined as the yield moment (M_y), and the rotation angle corresponding to the yield moment is defined as the rotation angle at yielding. The stiffness (k_y) was defined as the slope of the line between the origin and the point at which the plastic state begins over the elastic state in the envelope curve. In addition, Fig. 4(d) shows the envelope curves that means the moment-rotation angle relationship obtained using the first cycle of target displacement.

Test results showed that the peak moment of the BCW1 specimen did not reach the plastic moment as shown in Fig. 4(a). On the other hand, in case of the BCW2 and BCW3 specimen, the peak moment exceeded the plastic moment [see Fig. 4(b) and (c)]. The ratio between the peak moment and the plastic moment strength of the BCW1, BCW2, and BCW3 specimen for positive (+) and negative (-) loading were 0.79 (+) and 0.85 (-), 1.25 (+) and 1.25 (-), and 1.52 (+) and 1.55 (-), respectively. These results indicate that as the length of the connecting plate increases, the bearing failure at the bolt hole was delayed as much as possible.

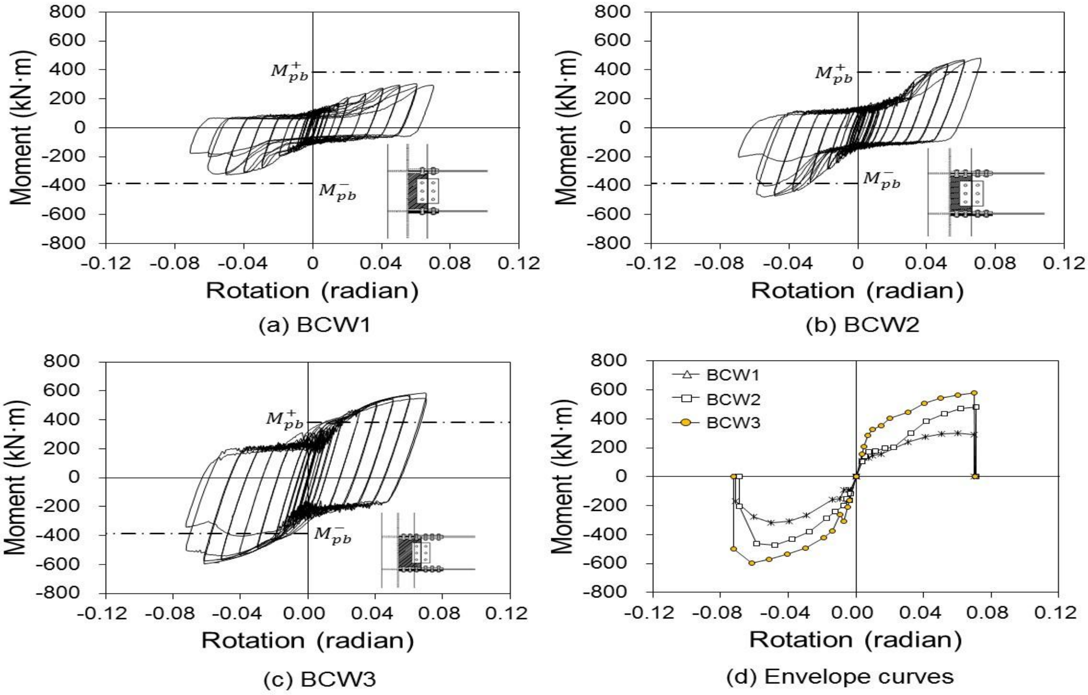

Fig. 4: Moment-curvature relationship of the test specimens

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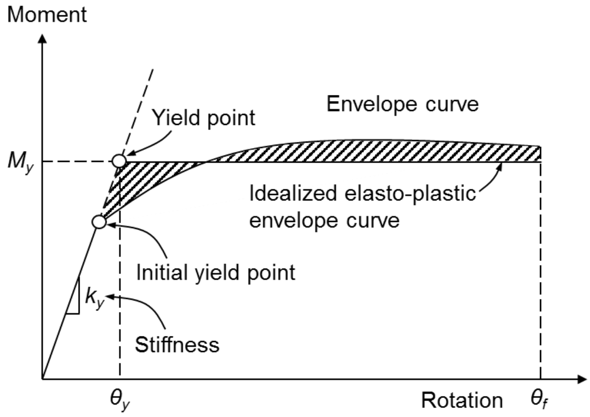

Fig. 5: Definition of yield point and initial stiffness

IV. SEISMIC EVALUATION

A. Stiffness

The stiffness of the weak-axis connection was compared with the AISC 360-10. For comparisons, the stiffness of the test specimen was determined the slope of the first cycle at the target rotation angle of 0.00375 rad. Table 2 summarizes the stiffness ratio between the test results and ANSI/AISC 360-10 (2010). The comparison of stiffness by ANSI/AISC 360-10 (2010) shows that only BCW3 specimens can be considered as partially restrained steel connection. However, the stiffness of the test specimens was about 5 to 11% of the stiffness when the beam-column joint is assumed to be the fixed end, regardless of test parameter. Therefore, BCW1 and BCW2 specimens can be regarded as the simple connection, but the BCW3 specimen as partially restrained or semi-rigid connection.

Table 2: Stiffness ratio of the test specimens

Specimens	AISC 360-10 (k_y / k_{AISC})			
	Positive loading		Negative loading	
	Pinned	Fixed	Pinned	Fixed
BCW1	0.74	0.07	0.69	0.07
BCW2	0.68	0.07	0.94	0.09
BCW3	1.05	0.10	1.10	0.11

B. Moment Strength

According to ANSI/AISC 341-10 (2010), the ordinary, intermediate, and special moment frame are required a rotation angles of 0.02, 0.03, and 0.04 rad, respectively. In addition, the intermediate requires that the strength should be maintained by 80% of the peak strength at 0.02 rad. Also, in the case of the special moment frame, the strength at 0.04 rad should be greater than 80% of the peak strength. As shown in Fig. 4, experimental results show that all specimens meet the displacement requirements for special moment frames. In all specimens, no strength deterioration occurs even if the rotation angle of the joint exceeds 0.04 rad. Table 3 shows the ratio of the moment strength at 0.02 rad and 0.04 rad. As a result of the comparison, the BCW3 specimen satisfied the requirements of the special moment frame proposed in ANSI/AISC 341-10 (2010). On the other hand, the BCW1 and BCW2 specimens did not satisfy all the requirements of the intermediate moment frame and the special moment frame.

Table 3: The ratio between the moment and plastic moment of the beam section

Specimens		At 0.02 rad		At 0.04 rad	
		$M_{@0.02\ rad}$ (kN. m)	$M_{@0.02\ rad}$ / M_{pb}	$M_{@0.04\ rad}$ (kN. m)	$M_{@0.04\ rad}$ / M_{pb}
BCW1	Positive loading	197.3	0.51	273.0	0.71
	Negative loading	-195.6	0.51	-309.1	0.80

BCW2	Positive loading	200.5	0.52	385.2	1.00
	Negative loading	-287.8	0.75	-430.4	1.12
BCW3	Positive loading	402.1	1.05	507.8	1.32
	Negative loading	-421.2	1.10	-533.2	1.39

V. CONCLUSION

In this study, the details of the beam-column weak-axis connections which improve the constructability for the small-size steel construction were developed and seismic performance was also evaluated through the cyclic loading test on the proposed weak-axis connections. The conclusions of the experimental study are as follows:

(1) The flexural moment strength of the BCW2 and BCW3 specimens with eight high-tension bolts and twelve bolts at the beam-column connection exceeded the plastic moment strength. On the other hand, the strength of the BCW1 specimen with four bolts was about 80% of the strength of the plastic moment. These results indicate that most of the moment strength is concentrated around the bolt hole of the beam flange due to the relatively short flange connection plate length.

(2) In all of the specimens, local fracture and bearing failure occurred around the bolt holes of the beam flange. However, the flange connecting plate did not exhibit remarkable failure modes. For the BCW2 and BCW3 specimens, the ultimate failure occurred around the last bolt hole in the direction of the flange connecting plate's length. This result is due to the fact that the displacement of the connection is larger at the bolt holes at the edges of the connecting plate than the bolt holes in the panel zone.

(3) The comparison of the stiffness between test results and the values obtained from AISC 360-10 showed that the BCW1 and BCW2 specimens can be considered as the simple connection and BCW3 specimens as partially restrained connection.

(4) The BCW3 specimen with twelve high-tension bolts satisfied the requirements of the special moment frame proposed in ANSI/AISC 341-10 (2010). Consequently, the number of the high-tension bolts to be needed for connecting the beam and column section was significantly affected to the seismic behavior of the steel beam-column weak-axis connections.

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