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CYCLIC TEST RESULTS OF BEAM-TO-COLUMN CONNECTION USING SHN490 STEEL

SO-YEONG KIM[1], KYUNG JAE SHIN[1], SWOO-HEON LEE[2], SUNG-YOUNG PARK[1], and DA-SOM CHU[1]

[1]*Dept of Architecture, Kyungpook National University, Daegu, Korea*
[2]*Dept of Civil Engineering, The University of Texas at Arlington, USA*

This experimental study was conducted to evaluate the seismic performance of beam-to-column connection of mid/low-rise building. Four specimens with beam-to-column connections of SHN490 steel were prepared depending on joint types: (1) web welded & flange welded; (2) web welded & flange bolted; (3) web bolted & flange welded; and (4) web bolted & flange bolted. All of beam-to-column specimen size were used H-300 ×150×6.5×9. Relatively less stiff connections were made by using seat or web angles. Cyclic loading was applied at the tip of beam following AISC 360 load protocol. Applied load vs. rotation relationship for different connection are shown. Two connection with flange welded were governed by the panel zone strength but two connection with flange bolted were governed by the failure of seat angle with large rotation.

Keywords: Steel moment connection, Cyclic loading, seismic performance, SHN490.

1 INTRODUCTION

Moment resisting frame had adequate ductility to manifest satisfactory resistance against strong earthquake before 1994 Northridge earthquake and 1995 Kobe earthquake. After then, steel moment connections types has been changed and developed by many researchers (Roeder and Foutch 1996, Roeder 2002); Roeder and Foutch (1996) examined that the panel zone yielding reduces flexural ductility of the beam and welding type (size and process) affects the capacity of the beam through literature reviews and statistical analyses. The three new connections of welded-flange-welded-web (WUF-W), reduced-beam section (RBS), and bolted-flange-plate (BFP) connections were introduced in Roeder (2002). SHN490 steel, newly developed steel for seismic structure, had been widely applied to earthquake resistance structure owing to its small variation of mechanical properties. Thus, fundamental data for efficient earthquake resistance design is to be provided through the investigation of the performance of SHN490 steel.

2 EXPERIMENTAL PROGRAMS

2.1 Material Test

The coupons for tensile test were prepared according to KS D0801 by cutting SHN steel material in rolling direction for the flange and web. UTM of 600kN capacity was used for the tensile test, and the average of three test results are shown in Table 1. The tensile test indicated that both yield strength and tensile strength satisfied the specified standard (Fy=325MPa, Fu=490MPa).

Table 1. Mechanical properties of steel.

Coupons	**Steel**	F_y **(MPa)**	F_u **(MPa)**	F_y / F_u
Flange	SHN490	448.4	570.9	0.79
Web	SHN490	477.5	597.2	0.80

2.2 Details of Test Specimens

The four specimens were classified by the joint type of web and flange of the beam at the beam-to-column joint: (1) the web and flange of the beam are all welded (SHN-W-W); (2) web of the beam is welded and the flange is connected using seat angle and fastened with high-strength bolt (SHN-W-B); (3) web of the beam is connected using angle and fastened with high-strength bolt and the flange is welded (SHN-B-W); and (4) the web and flange are all fastened with high-strength bolt using angle (SHN-B-B). Table 2 shows the specification of the four specimens.

Table 2. Beam-to-column specimens.

No.	**Specimen**	**Connection Type**		**Section Steel (mm)**	
		Web	**Flange**	**Beam and Column**	**Seat or Web Angle**
①	SHN-W-W	Welded	Welded	H-300×150×6.5×9 (SHN490)	L-75×75×9 (SS400)
②	SHN-W-B	Welded	Bolted		
③	SHN-B-W	Bolted	Welded		
④	SHN-B-B	Bolted	Bolted		

Figure 1 displays the detail of four beam-to-column connections and the shape of the specimen. SHN-W-W specimen had the beam-column connection all welded by size 8mm fillet welding. The panel zone of column was stiffened at the level of beam flange by continuity plate of 9mm thickness as thick as the beam flange thickness. Others were fabricated in the same shape and were classified into three types in accordance with the type of beam-to-column connection. Figures 1(c)~(e) show the detail of connection fastened with T/S (twisted-off type) high-strength bolt of M16.

Figure 1(c) portrays the detail of the SHN-W-B connection. L shape angle section was used to fasten the flange of the beam and column with a bolt, and the web was fillet-welded at 8mm thickness. Figure 1(d) shows the detail of the SHN-B-W connection. The web of the beam and column were bolted using L shape angle section, and the flange of the beam was fillet-welded at 8mm thickness. Figure 1(e) illustrates the detail of the connection of SHN-B-B specimen. Both flange and web of the beam were bolted using L shape angle section.

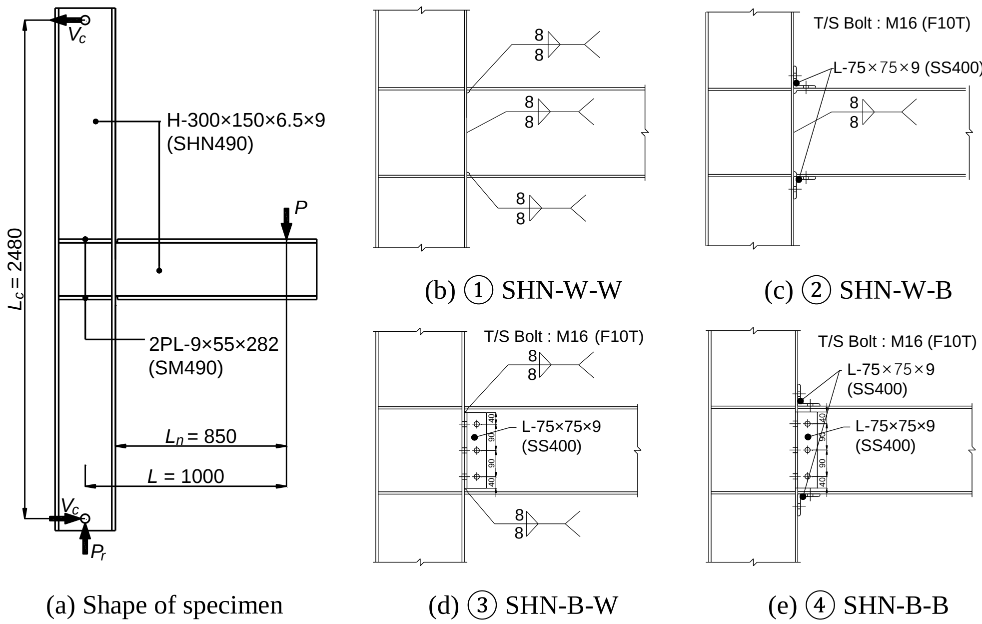


Figure 1. Shape of specimen and connection details.

2.3 Loading and Measurement

All connections were subjected to cyclic loading on the top flange of beam end using 2000kN actuator. A wire gauge was located at the bottom flange of beam end to measure deflection as shown Figure 1(a).

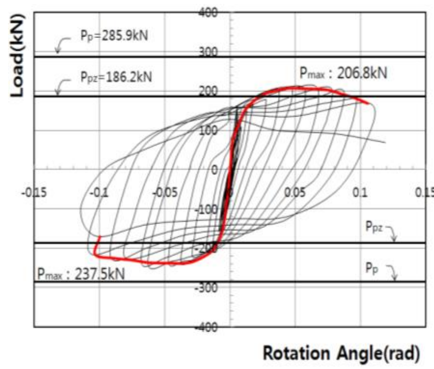
3 RESULTS AND DISCUSION

Table 3 summarizes the results of cyclic loading test on four beam-to-column specimens. The load is multiplied to the distance from the center of the column to the loading point to compute the flexural moment of the beam. Inter-story drift angle (θ) is computed by dividing the deflection of the beam end by the distance from the measuring point. Maximum moment capacities of SHN-W-W, SHN-W-B, SHN-B-W, and SHN-B-B specimens were 237.50, 131.32, 245.49, and 129.36kN·m when their rotation angles were 0.0501, 0.0150, 0.0479, and 0.0570 (rad), respectively.

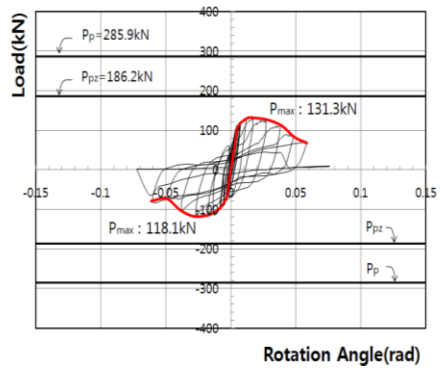
Figure 2 shows applied load-rotation angle relationship and the envelope curve. The shear strength of the panel zone of SHN-W-W and SHN-B-W specimens were 711.89kN and 735.94kN, respectively, to be much greater than the design shear strength and exhibited deformation due to yielding of the panel zone. In contrast, the shear strength of the panel zones of SHN-W-B, SHN-B-B specimens were 325.99kN, 387.80kN, respectively, to be smaller than the shear strength of the panel zone, and deformation of the panel zones was barely observed.

Table 3. Maximum moment and rotation.

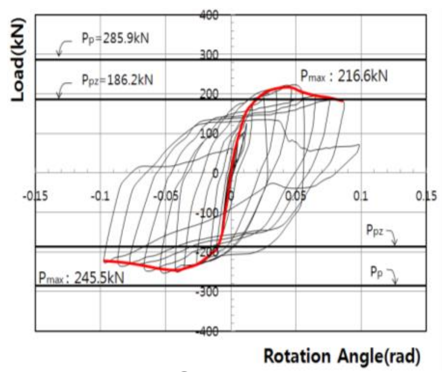
Specimen	**Moment (kN·m)**		**Rotation Angle (rad)**	
	Negative (-)	**Positive (+)**	**Negative (-)**	**Positive (+)**
① SHN-W-W	237.5	206.8	0.0501	0.0501
② SHN-W-B	118.1	131.3	0.0300	0.0150
③ SHN-B-W	245.5	216.6	0.0479	0.0510
④ SHN-B-B	124.0	129.4	0.0565	0.0570



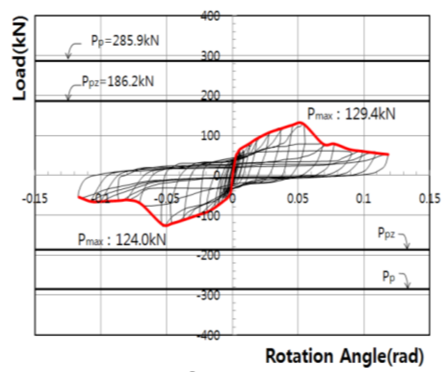
(a) ① SHN-W-W



(b) ② SHN-W-B



(c) ③ SHN-B-W



(d) ④ SHN-B-B

Figures 2. Applied load vs. rotation relationship.

P_p is the applied load when a plastic hinge is formed at rigid connection and can be 285.9kN by Eq. (1)

$$P_p = \frac{M_p}{L_n} = \frac{Z_x F_{yf}}{L_n} \tag{1}$$

where,
P_p : Applied load when a plastic hinge is formed at rigid connection.
M_p : Plastic moment of beam member.

Z_x : Plastic section modulus of beam member.
F_{yf} : Yield strength of the flange in accordance with tensile test.
L_n : Distance from loading point to the column face.
P_{pz} indicates the maximum applied load for panel zone nominal shear strength (Rv) and is computed 186.23kN by Eq. (2).

$$P_{pz} = \frac{(R_v + V_c)(d_b - t_f)}{L} \tag{2}$$

$$V_c = PL / L_c \tag{3}$$

where,
R_v : Shear bearing force of the panel zone (R_v=0.6$F_{yw}d_ct_w$)
F_{yw} : Yield stress of the web(from material test)
d_c : Depth of column
t_w : Thickness of column web
V_c : Shear force exerted on the column by P.
d_b : Depth of beam
t_f : Thickness of beam flange
L : Distance from the loading point to the center of the column
L_c : Total length of column between reaction points

Although the maximum moment of SHN-W-W specimen with both web and flange of the beam welded at the connection with the column was expected to be the largest, the maximum moment of SHN-B-W specimen was 3.3% greater than that of SHN-W-W based on the test measurement. However, since both SHN-W-W and SHN-B-W specimens exhibited greater shear force on the panel zone than the shear bearing force of the panel zone to manifest the failure mode due to the failure of the panel zone, the measured test strengths were deemed to be within the experimental error range. The Pmax value of remaining specimens of SHN-W-B, SHN-B-B did not reach the value of Ppz, indicating that the joint (connection) part yielded before the yielding of the panel zone.

4 CONCLUSIONS

The test result of beam-to-column connections using SHN490 steel material can be summarized as follows.

- The maximum resisting moment of SHN-W-W and SHN-B-W was governed by the panel zone strength, which less than that of full plastic moment of beam. The moment capacity of SHN-W-B and SHN-B-B was governed by the failure of seat angle, which is around the half of full plastic moment capacity of beam.
- The design shear strength of the panel zone was found to be 558.70kN. The shear strength of the panel zone of SHN-W-W and SHN-B-W specimens were 711.89kN and 735.94kN, respectively, to be much greater than the design shear strength and exhibited deformation due to yielding of the panel zone. In contrast,

the shear strength of the panel zones of SHN-W-B, SHN-B-B specimens were 325.99kN, 387.80kN, respectively, to be smaller than the shear strength of the panel zone, and deformation of the panel zones was barely observed.

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