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Beam-to-column connection with SHN490 steel

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ABSTRACT

Seismic performance of beam-to-column connections used in the middle-low rise buildings was investigated through the cyclic loading test. Four connection specimens made of grade SHN490 steel were fabricated with test variables of beam flange-to-column and beam web-to-column connecting types (welded versus bolted). All of specimen size were used H -300×150×6.5×9. Loading sequence according to AISC seismic provision 2010 was applied to beam-to-column moment connections. 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.

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). However, there is still a lack of research on various steel moment connection details with high-strength steel. 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.

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2. EXPERIMENTS

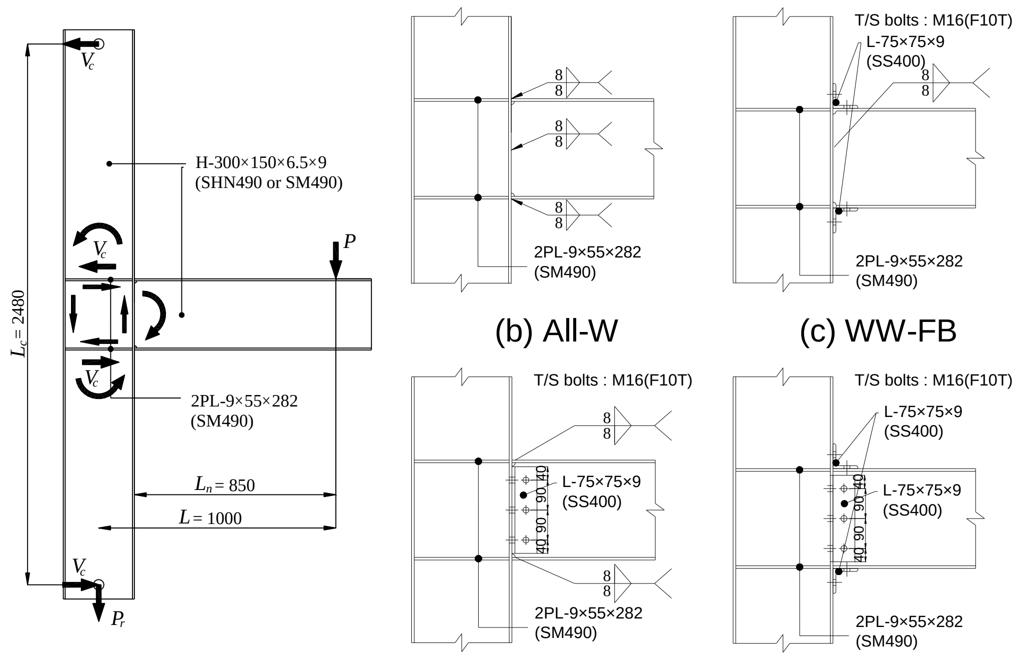
2.1. Details of Test Specimens

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

Table 1. Beam-to-column specimens

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

Fig. 1 displays the detail of four beam-to-column connections and the shape of the specimen. All other specimens were fabricated in the same shape. 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.



(a) Shape of specimen (d) WB-FW (e) All-B

Fig. 1 Shape of specimen and connection details

2.2. Test set-up and loading protocol

Fig. 2 shows the overall view of the specimen installation. Both ends of the connection of the specimen were linked to a fixed frame with a hinge, and an actuator of 2000 kN with a load cell exerted loading on the beam end. Fig. 3 shows the loading cycle and the failure point of each specimen during the test. The loading cycle was conducted by displacement control method to control for inter-story displacement angle (θ) in accordance with the cyclic loading protocol proposed by AISC (2010).

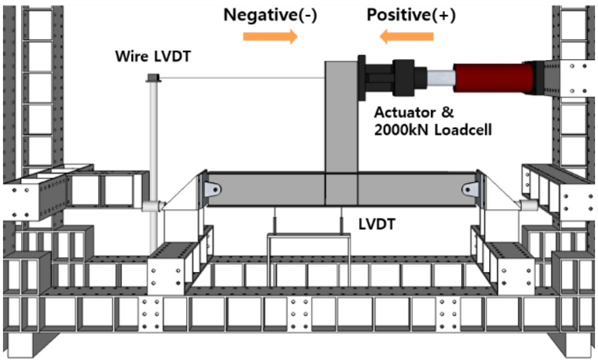


Fig. 1 Test set-up

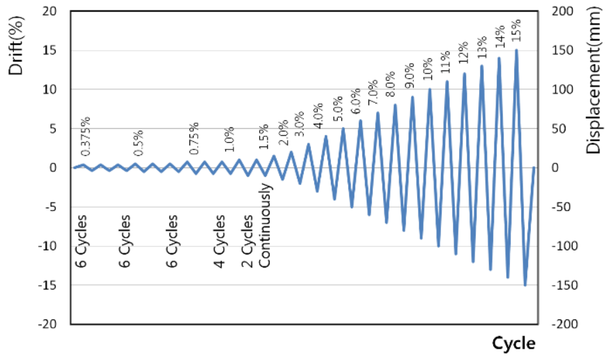


Fig. 2 Loading protocol

3. EXPERIMENTAL RESULTS

Table. 2 summarize the results of cyclic loading test on four beam-to-column specimens. 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 All-W, WW-FB, WB-FW, and All-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.

Table 2. Maximum moment and rotation

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

Fig. 4 shows applied load-rotation angle relationship and the envelope curve. The shear strength of the panel zone of All-W and WB-FW 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 WW-FB, All-W 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.

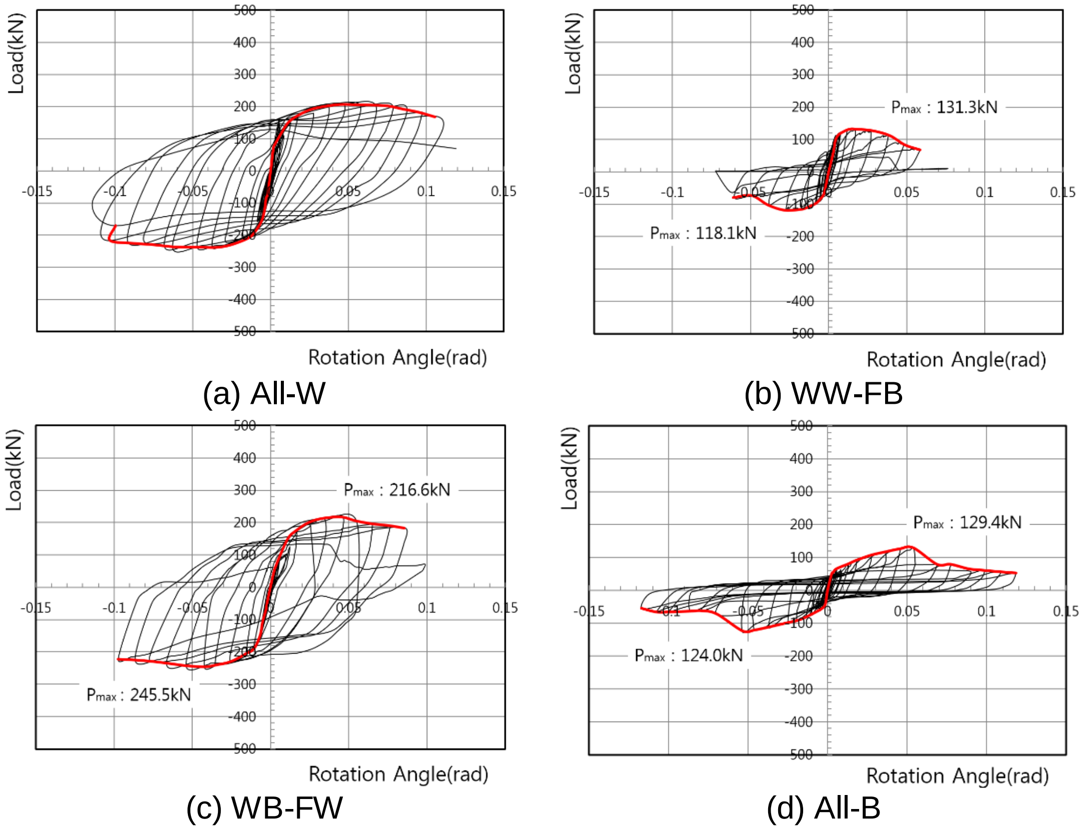


Fig. 4 Applied load vs. rotation relationship

4. CONCLUSIONS

The maximum resisting moment of All-W and WB-FW was governed by the panel zone strength, which less than that of full plastic moment of beam. In contrast, the shear strength of the panel zones of WW-FB, All-B specimens were smaller than the shear strength of the panel zone, and deformation of the panel zones was barely observed.

ACKNOWLEDGEMENT

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