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Seismic response of high-strength steel moment connections used in special moment frames

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ABSTRACT

This study elucidates experimentally the cyclic behaviour of two moment connection subassemblages using high-strength steel. Due to the high yield ratio of the high-strength steel, the moment connections were designed to characterize widened beam flanges at the column-to-beam interface. The test results indicated that specimens formed a plastic hinge in the beam section away from the column face. Extensive yielding and plastification were observed in the beam section at the intended plastic hinge location. Local buckling of the beam flange and web occurred finally and caused the strength deterioration. Both specimens achieved an interstory drift angle of 5% rad and the plastic rotation of more than 3% rad. Using the high-strength steel, specimens with widened beam flange moment connection can develop satisfactory strength and ductile behaviour.

Keywords: *High-strength steel; moment connection; plastic hinge.*

1 INTRODUCTION

In recent decades, high-strength steel has been intensively developed and studied. The use of the high-strength steel can be in the bridges, high-rise buildings and large span structures [1-5]. Compared to normal-strength steel, high-strength steel characterizes higher yield and tensile strengths. The use of the high-strength steel benefits the high-rise buildings, especially the columns at the lower stories. The frequent use of the high-strength steel is SM570, with specified ultimate strength of 570 N/mm^2. The specified yield strengths of SM570 steel range from 430 to 450 N/mm^2 and the tensile strengths range from 570 to 720 N/mm^2. Further, the high-strength steel characterizes a higher yield ratio which is defined as the ratio of the yield strength to the tensile strength. The yield ratio significantly influences the plastification of the steel plate. With a lower yield ratio, the steel plate can develop large plastic zone and achieve a ductile behaviour.

Special moment frames are widely used in high seismic area because the frames can resist the earthquake exerted forces by developing plastic hinges to dissipate energy. The most common moment connections used in special moment frames is the type of welded flange and bolted web, in which the beam flange is welded to the column via complete joint penetration groove weld and the beam web is bolted to a shear tab which welded to the column. However, these moment connections were severely damaged during the 1994 Northridge and 1995 Kobe earthquakes. Many moment connections showed a brittle failure before they achieved a ductile manner [6-9]. These brittle failures include the cracks at weldment, buckling or fracturing of the beam flange and web, buckling or fracturing of the column flange, fracturing of the shear tab, and buckling of the panel zone.

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After these earthquakes, the moment connections have been extensively studied to improve their seismic behaviours. Two improvements are most used that are the strengthening and weakening design strategies. In the strengthening strategy, the beam is strengthened using the cover plate, widened plate, rib plate, or side plate to reinforce the beam close to the column [10-14]. The weakening strategy is to reduce beam section by trimming the beam flange or drilling holes on the beam flange [15-17]. Both strategies are intended to move away the plastic hinge from the column face. This study experimentally investigates the cyclic behaviour of the strengthened moment connections using high-strength steel. Two specimens representing an exterior moment connection were fabricated using SM570 steel.

2 EXPERIMENTAL INVESTIGATION

Two specimens were tested to explore the cyclic behaviour of moment connections. Hysteretic behaviour, moment capacity, story drift, and plastic rotation were presented.

2.1 Test specimens

A widened beam flange connection was proposed to force the plastic hinge formed on the beam away from the column [18]. The beam flanges are enlarged at the beam ends near the column. The widened beam flange functions as a reinforcement for the beam-to-column connection, and the reinforcement is intended to provide large flexural capacity of the beam at the beam-to-column interface, as shown in Fig. 1. Due to the widened beam flange at the joint, the plastic hinge is expected to form on the beam section beyond the widened flange.

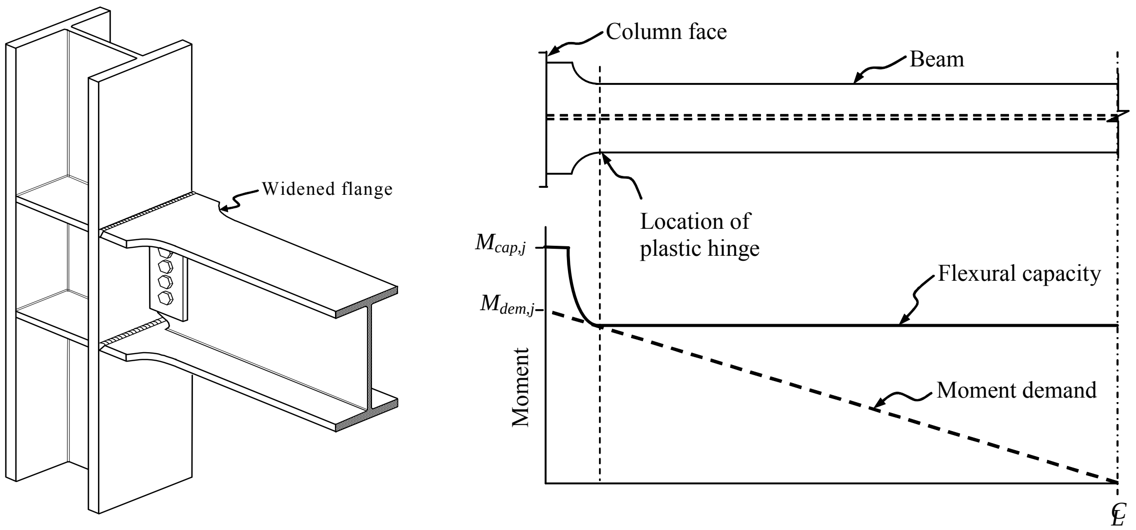

Figure 1 – Widened flange connection and moment gradient.

Two large-scale specimens were designed to represent an exterior moment connection subassemblage. The subassemblage includes a column between two inflection points at two stories and a beam of a half span. The connection details are illustrated in Fig. 2. Both specimens have H-shaped beam of H550x250x13x21 (beam depth, beam width, web thickness, and flange thickness in mm) and the column of H600x420x21x30. The beam flanges were welded to the column flange via completely joint penetration weld. To match the high-strength steel, electrode of E81 flux-coated wire was used to perform the completely joint penetration weld, and the CO_2 was used as the shielding gas. Shear tabs were used to transmit the shear force at the beam-to-column joint. The use of the high-strength steel in the beam section results in the large plastic moment capacity, and subsequently the large shear force. Thus, twelve M20 F14T super-high-strength bolts were utilized to transfer the shear force. F14T bolts confirm to a tensile strength of 1400 to 1490 N/mm^2.

As shown in Fig. 2, specimen WF1 and WF2 have different details at the widened beam flanges. Specimen WF1 and WF2 were designed to form a plastic hinge on the beam at a distance of 447 and 315 mm from the column face, respectively. Once the plastic hinge formed away from the column, the demand of the beam at the column face is lower. To avoid weak panel zone, doubler plate of 13 mm thick was welded to the panel zone to satisfy the code requirement for the panel zone. As shown in Fig. 3, both specimens were fabricated to have the weld access hole with the shape of a quarter circular arc which has a radius of 35 mm but with a small radius of 10 mm at the joint with the beam flange to attenuate stress concentration. This configuration of the weld access hole is frequently used in practical engineering because the weld access hole can be machined automatically.

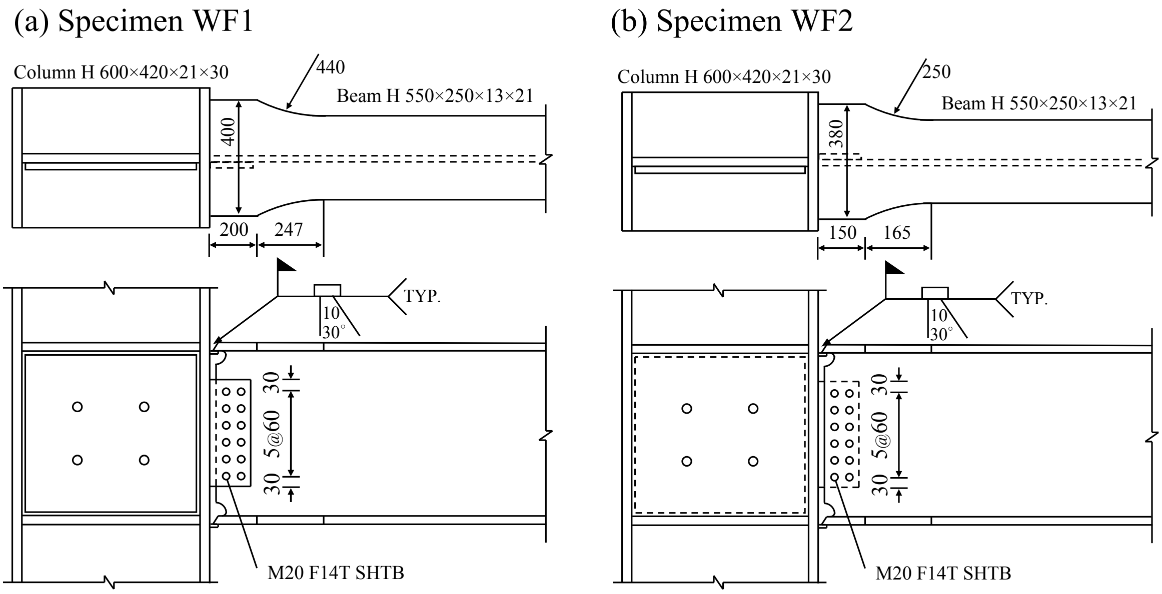

Figure 2 – Connection details of the specimens.

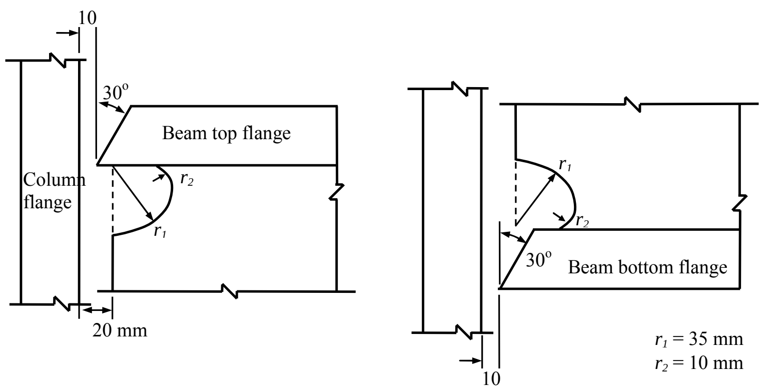

Figure 3 – Details of weld access hole.

2.2 Test setup and loading history

Because the specimens represent a subassemblage of an exterior moment connection, the test was setup to provide hinges at the column. Fig. 4 shows the test setup. To easy simplify the setup, the column was placed horizontally and the beam was set upright. Thus, a horizontal hydraulic actuator applied a lateral force to the beam tip. To prevent the out of plane deformation of the specimens, lateral brace was installed.

A predetermined cyclic displacement history was adopted to apply a lateral force to the beam tip. The story drift angle sequences specified in the AISC seismic provisions [19] was followed. Fig. 5 shows the story drift angle sequence. The sequence starts with six successive cycles at the story drift angles of 0.375, 0.5 and 0.75% rad. Four cycles of 1% rad were followed. Afterward, two cycles of 1.5, 2, 3, 4 and 5% rad were succeeded until the specimens failed.

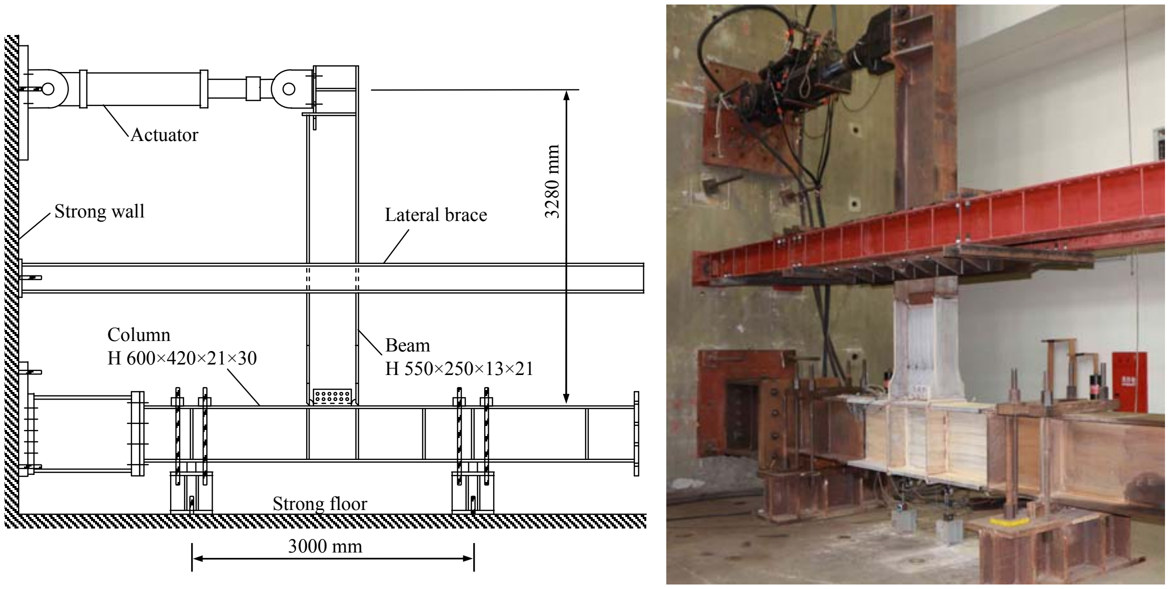

Figure 4 – Test setup.

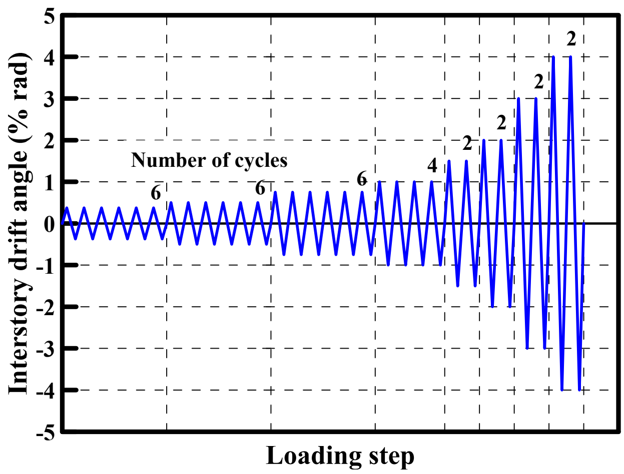

Figure 5 – Loading history.

3 EXPERIMENTAL RESULTS

The cyclic behaviours of the specimens are described. Hysteretic response and connection moment capacity are also discussed. Tensile coupon tests were conducted to obtain the mechanical properties of the high-strength steel used for the specimens. Table 1 tabulates the yield and tensile strengths of the steel plates used in the specimens. Notably, the yield ratios of the beam flange and web are 0.86 and 0.84, respectively, which are higher than those of the normal-strength steel.

Table 1: Mechanical properties of the high-strength steel

Coupon location	Steel plate (thickness in mm)	Yield strength (N/mm^2)	Tensile strength (N/mm^2)	Yield ratio
Beam flange Column web	21	491	572	0.86
Beam web	13	486	581	0.84
Column flange	30	520	638	0.81

3.1 Cyclic behaviour

Two specimens showed similar behaviour such as yielding and plastic hinge formation. The flaking of the whitewash initially occurred on the beam flange beyond the widened beam flange. Sign of the yielding of the beam web was noticed during the cycles of 2% rad story drift angle. The yielding of the beam flange and web spread with the increase of the story drift angle. Slightly local buckling of the beam flange and web was noticed after the cycles of 4% rad story drift angle. As shown in Fig. 6(a) and Fig. 7(a), extensive yielding of the beam section demonstrated the formation of the plastic hinge. Both specimens showed no sign of cracks at the test conclusion. Fig. 6(b) and Fig. 7(b) present excessive buckling of the beam section in the cycles of 5% rad story drift angle. The tests were terminated because of the stroke limitation of the actuator.

(a) 4% rad story drift angle (b) 5% rad story drift angle

Figure 6 – Yielding and local buckling of the WF1 beam section at 4 and 5% rad story drift angle.

(a) 4% rad story drift angle (b) 5% rad story drift angle

Figure 7 – Yielding and local buckling of the WF2 beam section at 4 and 5% rad story drift angle.

3.2 Hysteretic response

Cyclic behaviour of the specimens is presented by the hysteretic curves of the beam tip load and beam tip displacement relations. Further, the hysteretic curves are presented in relations between the moment and total plastic rotation. The moment was calculated by multiplying the beam tip load by the length to the column face. The total plastic rotation can be calculated by subtracting the elastic rotation from the total angle of rotation. Fig. 8 shows the hysteretic curves of load versus displacement at the beam tip, and Fig. 9 presents the hysteretic curves of moment versus total plastic rotation. Both specimens showed that the strength began to deteriorate after the cycles of 4% rad story drift angle because of the local buckling of the beam section. The strength degradation of specimen WF2 was severe due to additional lateral torsional buckling. As presented in Fig. 8 and Fig. 9, both specimens reached a story drift angle of more than 4% rad and developed a plastic rotation of over 3% rad. The total plastic rotations of specimen WF1 and WF2 were +3.6, -3.7% rad and +3.7, -3.4% rad, respectively. The plastic rotation of the subassemblage was entirely contributed from the beam plastic hinge while the column and panel zone remained elastic.

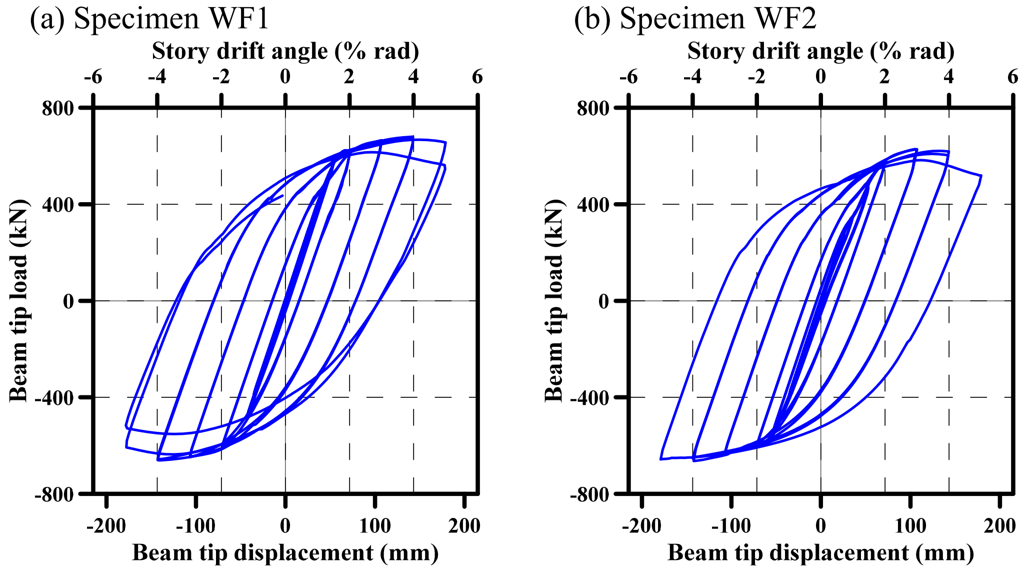

Figure 8 – Hysteretic curves of load versus displacement at the beam tip.

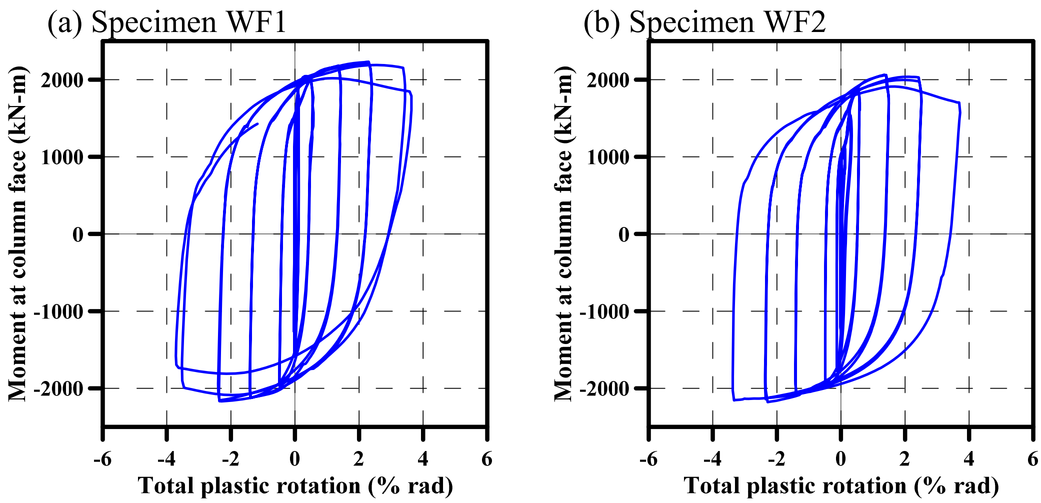

Figure 9 – Hysteretic curves of moment versus total plastic rotation.

3.3 Connection moment capacity

Table 2 tabulates the maximum test moments and plastic flexural strengths for both specimens. The $M_{j,test}$ and $M_{ph,test}$ are the maximum test moments calculated at the column face and plastic hinge location, respectively. The M_{pj} and M_p are the calculated plastic flexural strength based on the material strengths at the column face and plastic hinge location, respectively. When the ratio of $M_{j,test}$ to M_{pj} is less than unity, the beam capacity is higher than moment demand at the column face. However, the ratio of the maximum test moment to plastic flexural strength at the plastic hinge, $M_{ph,test}/M_p$, are greater than unity, it implies that plastic hinge developed with strain-hardening. The ratios of the maximum test moment to flexural strength shown in Table 2 indicate the effectiveness of the widened beam flange.

Table 2: Test moment and flexural strength summary

Specimen	Maximum test moment		Plastic flexural strength		Ratio of max test moment to flexural strength	
	At column face $M_{j,test}$ (kN-m)	At plastic hinge $M_{ph,test}$ (kN-m)	At column face M_{pj} (kN-m)	At plastic hinge M_p (kN-m)	At column face $\frac{M_{j,test}}{M_{pj}}$	At plastic hinge $\frac{M_{ph,test}}{M_p}$
WF1	+2231	+1927	2605	1786	0.86	1.08
	-2169	-1873	2605	1786	0.83	1.05
WF2	+2063	+1865	2496	1786	0.83	1.04
	-2178	-1969	2496	1786	0.87	1.10

4 CONCLUSION

Using high-strength steel, two large-scale exterior moment connection subassemblages were tested cyclically. The moment connections feature a widened beam flange at the beam-to-column interface. Test results demonstrated both specimens developed extensive yielding on the beam flange and web at a predetermined zone. Due to the reinforcement of the widened beam flange, no damage in the groove weld and the weld access hole region was observed. Therefore, plastic hinge formed on the beam section at a distance away from the column. Both specimens developed a satisfactory rotation of story drift angle for the moment connection used in special moment frames.

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