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EXPERIMENTAL INVESTIGATION OF FLANGE PLATE CONNECTIONS BETWEEN CONCRETE-FILLED TUBE COLUMN AND STEEL BEAM

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SUMMARY

This study investigates the cyclic behavior of flange plate connections between steel beam and rectangular CFT column, in which the flange plates were penetrated through the CFT column. The steel tube of flange plate connections tore at the tips of the weld joining the flange plate to the steel tube. Specimens with adequate shear capacity in the panel zone formed the plastic hinge in the beam, while specimens with weak panel zone demonstrated the shear cracking in the concrete core, and fracturing of the steel tube. The specimen with connection of whole beam through the column possessed stable hysteresis curve.

INTRODUCTION

Composite structures have become popular in recent decades. Concrete-filled tube (CFT) columns are commonly used in the composite structures for buildings. There are many medium-rise and high-rise buildings adopting the CFT columns in the U.S., Japan and other countries. The use of the CFT columns in the composite buildings possesses several advantages over other types of columns. Steel tube can serve as a form for casting concrete and provide continuous confinement to the concrete core, resulting in the increase of the compressive strength and strain of the concrete. Moreover, the filled concrete delays the local bucking of the steel tube. In general, the use of the CFT column leads to an economic construction.

The CFT columns are usually designed to join the steel beams in the composite buildings. Hence, the connections between the CFT column and the steel beam need to be detailed. Furthermore, their force transferred mechanism and seismic performance have to be established and understood. The connections between CFT columns and steel beams have been studied extensively. Several connection details have been developed and tested. Morino et al. [1] conducted the test for connections with through diaphragm. Prion and McLellan [2], and Ricles et al. [3] tested for the beam connecting to CFT column using through bolts. They found that adequate details of bolting had well seismic performance. Ricles et al. [4] studied the CFT connections including interior diaphragm plate and shear stud. Their test results showed that a CFT panel zone possesses exceptional ductility, and a smaller ratio of width-to-thickness of the steel tube provides higher shear strength. Kang et al. [5] tested the connections that were reinforced externally with

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T-shaped stiffeners, and with a reinforcing bar or bent plate penetrated into the column. They found that the reinforcement can increase the strength of the connections and improve the hysteresis behavior.

Chen and Lo [6] tested four connections with flange plates penetrated through the circular CFT column. Test results indicated that connections with adequate shear strength in the panel zone exhibited stable hysteresis behavior, whereas the connections with weak panel zone demonstrated shear yielding in the concrete inside the steel tube. This study presents the experiment evaluation for the connections using the flange plates penetrated through the rectangular CFT columns. The experimental results are used to elucidate the ultimate flexural strength of the connections and the inelastic rotation of the specimens under cyclic loading.

EXPERIMENTAL PROGRAM

As one of the multiphase research programs for a full-scale composite braced frame test, this research conducted an experiment to investigate the behavior for one of the connection subassemblages. The full-scale frame is a three-story three-bay frame with CFT composite columns and buckling restrained braces (BRB) [7]. Figure 1 presents the configuration of the CFT/BRB frame. The connection used in this research was a subassemblage including the connection between rectangular CFT column and steel beam in the first floor.

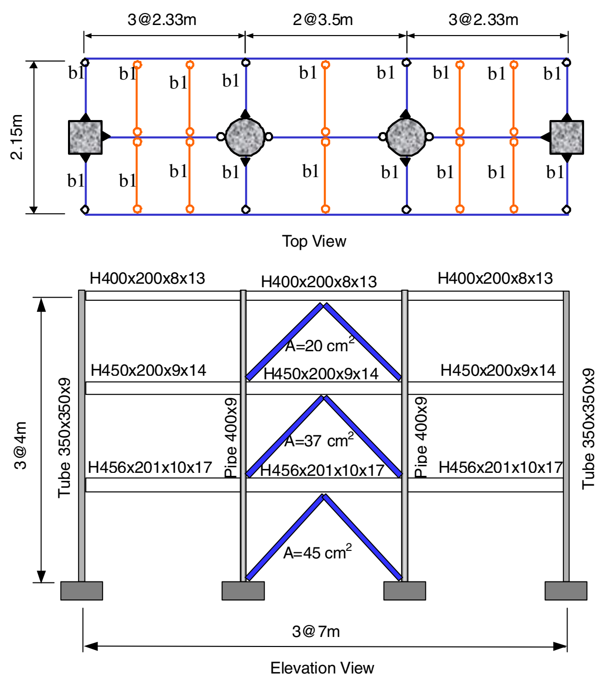


Fig. 1 Full-scale CFT/BRB composite braced frame

Test specimens

A total of six specimens were constructed and tested to failure. Table 1 presents the details of the test specimens. Four specimens represent an exterior T-shaped subassemblage, which are identified with first two letters H4, while two specimens are cruciform. All six full-scale specimens were designed to have the beams and the CFT columns proportioned to comply with the strong-column weak-beam seismic design criteria. The design of the H4 series specimens were intended to have strong panel zone to concentrate the inelastic deformation in the steel beam. The other two cruciform specimens, having weak panel zone capacity, were designed for the purpose of developing the shear deformation in the panel zone area. The estimated shear strength of the CFT column in the panel zone was assumed to be the capacity superposed from both the contributions of the steel tube and the concrete core.

Table 1 Details of test specimens

Specimens	Steel Tube (mm)	Beam (mm)	Joint type	Weld to tube	Trans. weld in flange plate
H4GL	350×350×9	H450x200x9x14	Exterior	Groove	No
H4GT	350×350×9	H450x200x9x14	Exterior	Groove	Yes
H4FT	350×350×9	H450x200x9x14	Exterior	Fillet	Yes
H4BT	350×350×9	H450x200x9x14	Exterior	Groove	--
H2GT	350×350×9	H280x180x8x18	Interior	Groove	Yes
H3GT	350×350×9	H300x200x9x20	Interior	Groove	Yes

All CFT columns were constructed using the steel tube of 350 mm square, with tube wall of 9 mm thick. The steel tube confirmed the ASTM A572 Gr. 50 steel. The columns were designed to fill the concrete with specific compressive strength of 34.5 MPa (5,000 psi). However, the average concrete strength measured during the test of the specimens was 46.0 MPa, except that specimen H4BT had the concrete strength of 25.6 MPa. The sizes of the steel beam are H450×200×9×14 (mm), H280×180×8×18, and H300×200×9×20. The later two steel beams were not a normal size because the shallow section of the beam was designed to achieve large shear forces in the panel zone. All the steel beams confirmed to the ASTM A572 Gr. 50 steel.

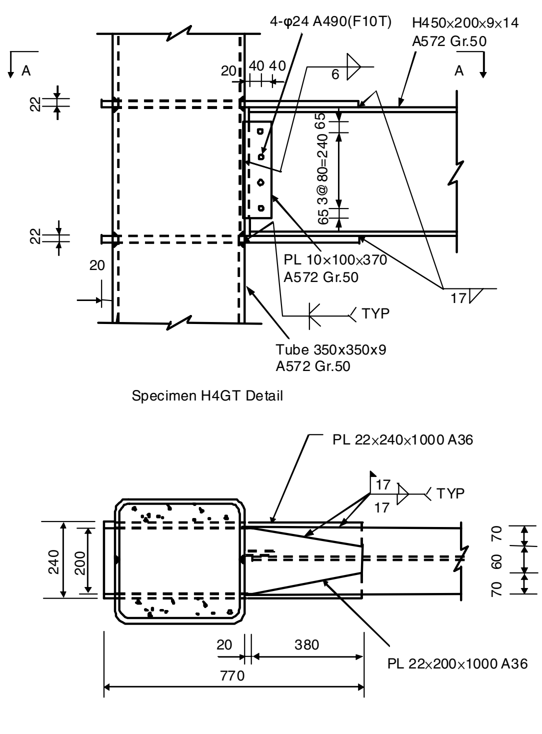


Fig. 2 Connection details of specimen H4GT

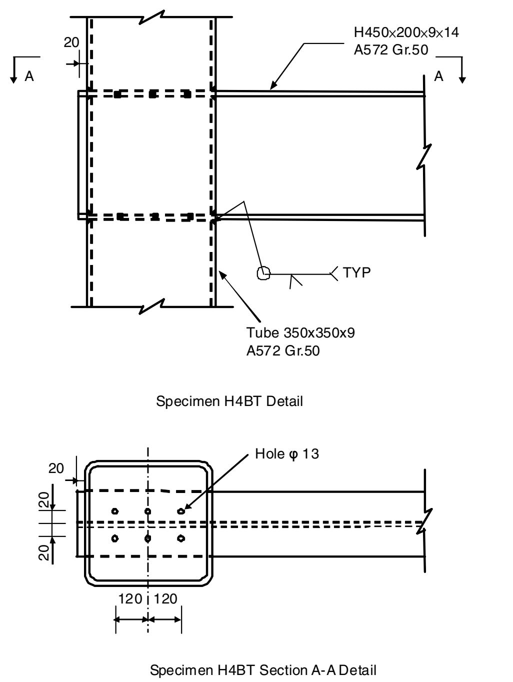


Fig. 3 Connection details of specimen H4BT

Figures 2-4 present the connection details of the specimens. In order to construct the flange plate connection, slots were cut on the steel tube at the desired position corresponding to the beam flange. Two flange plates were penetrated through the steel tube before the columns were cast with concrete. The flange plate used above the top flange of the beam was the shape of trapezoid, to easily perform the fillet weld that joined the flange plate to the beam flange. Nevertheless, a rectangular flange plate that was wider than the width of the beam flange was adopted underneath the bottom flange of the beam, to undertake the welding that avoided the overhead position.

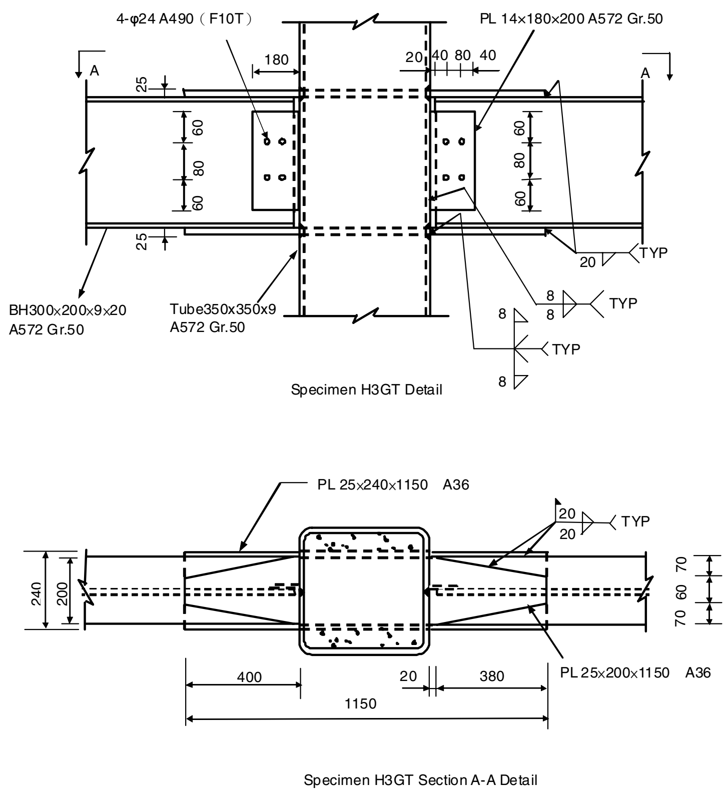


Fig. 4 Connection details of specimen H3GT

Test setup and procedure

Figure 5 illustrates the schematic of the test setup that was designed to simulate the loading and boundary conditions for a connection subassemblage in a frame subjected to the seismic load. A concentrated axial load exerted by a hydraulic jack was applied continuously to the CFT column to simulate the gravity force. The axial force on the column was approximately 10% of the squash load of the CFT column, which was calculated from the frame analysis. Two servo-hydraulic actuators applied cyclic lateral force to the tips of the cantilever beams. The cyclic force was simulated by a predetermined displacement history, as shown in Fig. 6.

EXPERIMENTAL RESULTS

Behavior and modes of failure

Specimens H4GL, H4GT and H4FT exhibited the similar behavior and the failure mode. The whitewash flaked on the beam flange beyond the flange plate at a story drift of 1.5%. Minor tearing of the steel tube occurred at the both tips of the groove weld or fillet weld that joined the flange plate to the steel tube at a story drift of 2.0%. The tearing became severe when the interstory drift was increased. Extensive yielding continuously occurred in the beam flange and the web. Meanwhile, the steel tube within the panel zone exhibited minor shear yielding revealed by the diagonal flaking of the whitewash. At the cycles of 6%

story drift, the beam flange and the web beyond the flange plate displayed local bucking. Significant local bucking occurred in the beam section as well as the serious tearing of the steel tube at the weld joining the steel tube and flange plate caused the drop of the connection strength at a 7% story drift. The test was terminated owing to the considerable degradation of the strength. Figure 7 shows the appearance of specimen H4FT at the test conclusion.

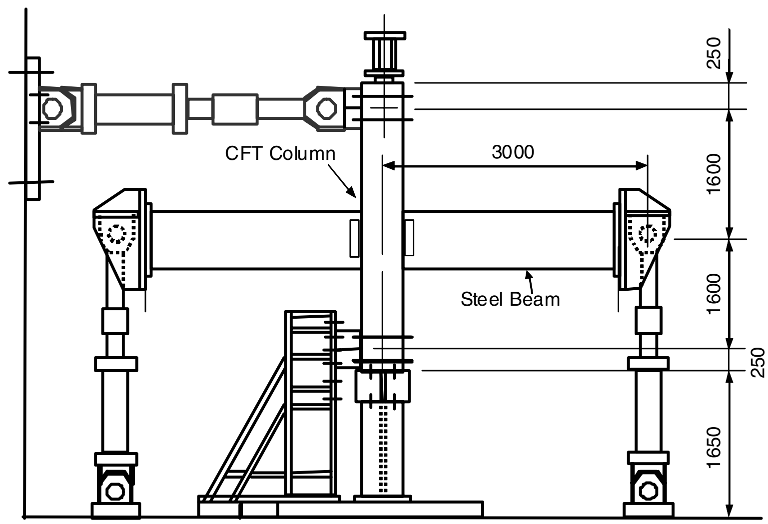


Fig. 5 Test setup

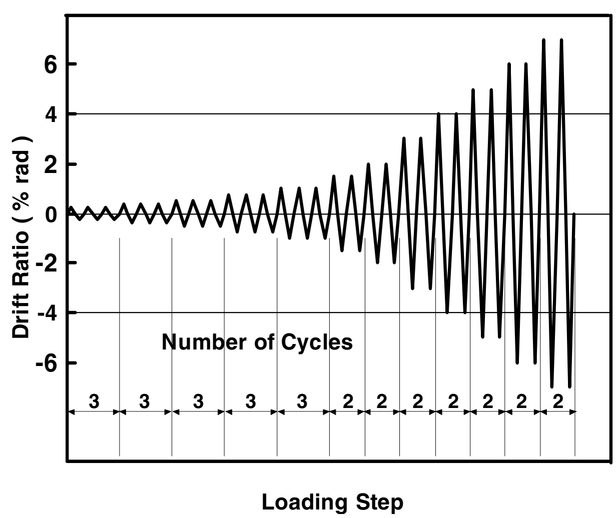


Fig. 6 Loading history

The skin of the steel tube within the panel zone was removed to examine the failure of the concrete after the test was done. No diagonal cracking on the concrete was found; however, cracking formed beside of the flange plates, which indicates that the flange plates might slip during the test.

Specimen H4BT, whose whole beam section was inserted to the steel tube, behaved different from the previous three specimens. The beam flange near the column started to yield at a story drift of 1.5%. The beam web consequently yielded at the following cycles. Extensive yielding occurred in the beam section, but no other damage could be found. Until the cycles of 5% story drift, the beam flange and the web buckled locally. The test was ended after the cycles of 7% story drift owing to the significant strength drop caused by the local bucking of the beam.

Fig. 7 Appearance of specimen H4GT at a story drift of 7%

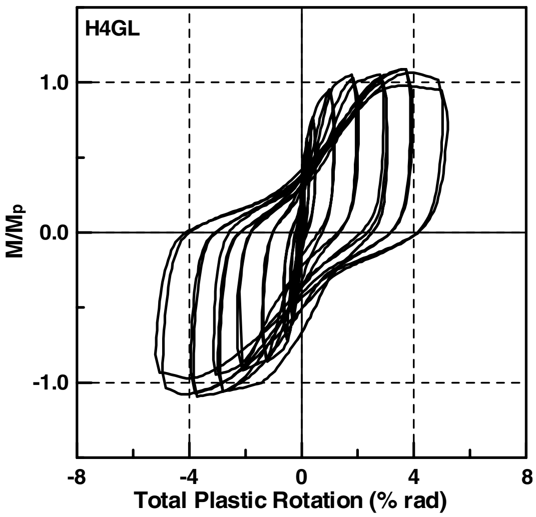
Specimens H2GT and H3GT showed identical behavior and failure mode. The steel tube tore at the groove weld joining the steel tube and the flange plates at the story drift of 2%. Minor shear yielding of the steel tube occurred during the cycles of 4% story drift. The tests were terminated after the cycles of 7% story drift because the steel tube fractured completely at the groove weld. After the skin of the steel tube was removed, it was found that the shear cracking occurred on the concrete core inside the steel tube within the panel zone area, especially on the specimen H3GT.

Hysteretic behavior

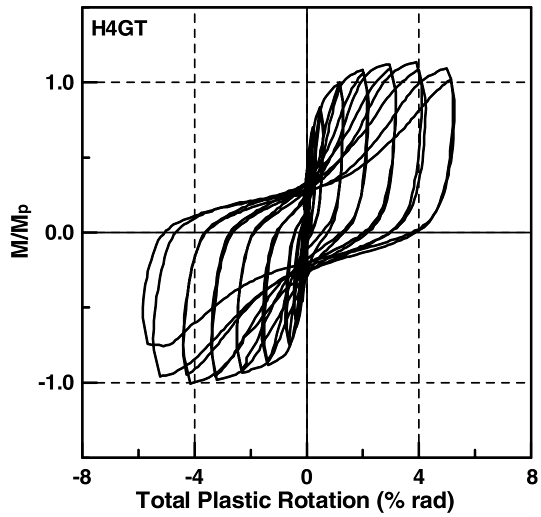
Figure 8 plots the hysteresis curves of beam moment versus total plastic rotation of four exterior connection specimens. The moment was normalized to the plastic flexural strength of the beam. The hysteresis curves of specimens H4GL, H4GT and H4FT exhibited minor pinching phenomenon, which should be attributed to the tearing of the steel tube at the groove weld connecting to the flange plate, and the slippage of the flange plates inside the concrete. However, these three specimens developed flexural strengths greater than their plastic flexural strengths of the beam. Furthermore, specimen H4BT displayed a stable hysteresis curve with the degradation in the strength due to the local buckling of the beam section.

Figure 9 presents the beam moment versus total plastic rotation response of specimens H2GT and H3GT. These hysteresis curves demonstrated the similar behavior as that of specimen H4GT. However, both specimens did not develop their plastic flexural strengths. Pinching of the curves was contributed from the slippage of the flange plates, diagonal shear cracking of the concrete core and the fracture of the steel tube.

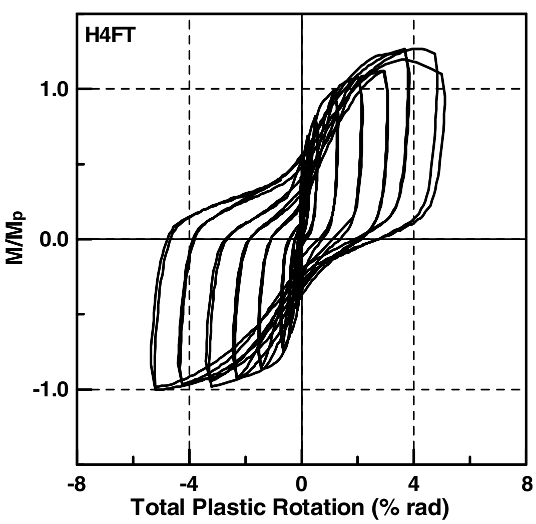
No noticed effect of the groove or fillet weld, which joined the flange plate to the steel tube, on the hysteretic behavior was observed from these tests. The behavior of the specimen without transverse weld between the flange plate and the beam flange was not different from that of specimens with transverse weld.



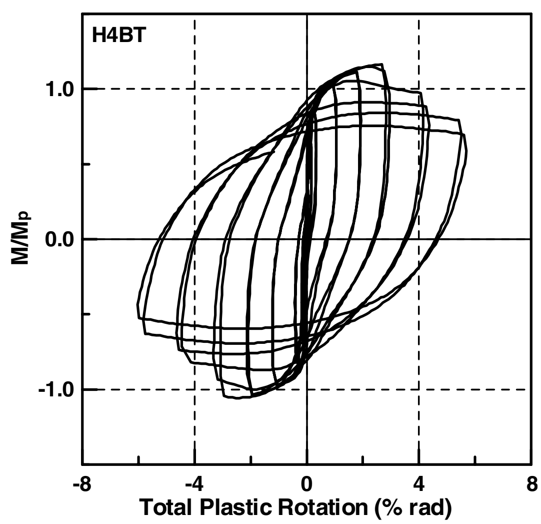
(a) Specimen H4GL



(b) Specimen H4GT



(c) Specimen H4FT

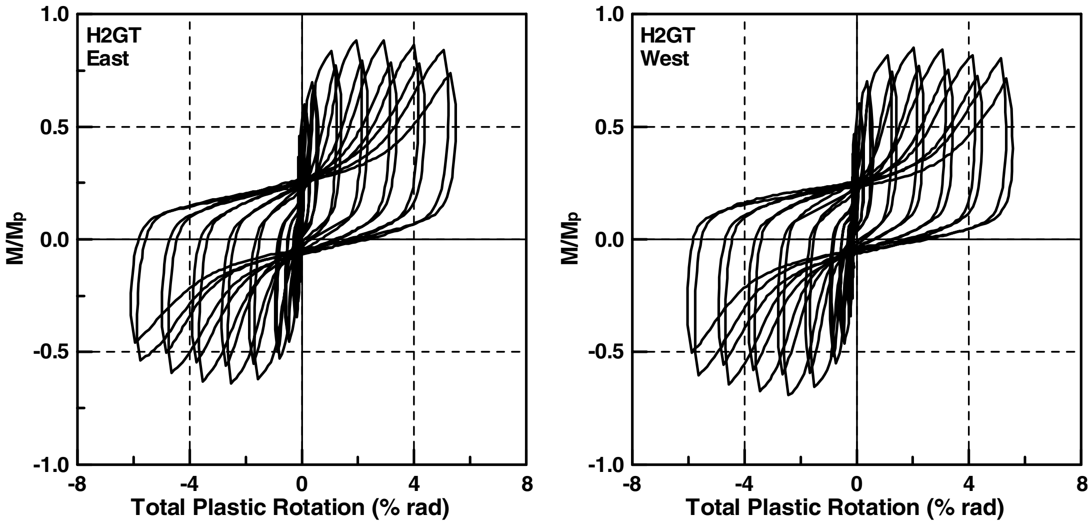


(d) Specimen H4BT

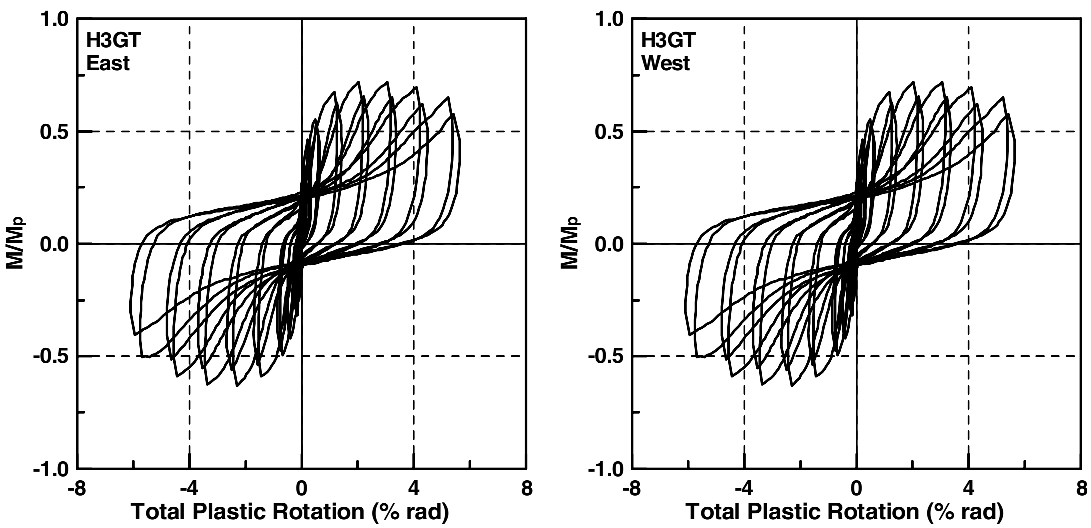
Fig. 8 Moment-plastic rotation response of exterior connections

Flexural strength

Table 2 lists the flexural strength summary of the specimens. The calculated plastic flexural strength of the beam was computed based on the material strength measured in the coupon test. The maximum test flexural moment was computed to multiply the force at the beam tip by the distance from the beam tip to the plastic hinge, which located at the one-fourth of the beam depth beyond the flange plate or the column. The ratios of maximum test moment to calculated strength for four specimens with exterior connection are greater than one that implies the beams were stressed beyond their plastic flexural strengths at the plastic hinge. The ratios, less than one, for two specimens with interior connection indicate that the beams did not reach their plastic flexural strength because of the premature failure of the tearing of the steel tube.



(a) Specimen H2GT



(b) Specimen H3GT

Fig. 9 Moment-plastic rotation response of interior connections

Table 2 Flexural strength summary

Specimens		Calc. plastic flexural strength M_P (kN-m)	Max. test flexural moment M_{test} (kN-m)		Ratio of M_{test}/M_P	
H4GL		687	+750	-752	1.09	1.09
H4GT		687	+779	-692	1.13	1.01
H4FT		687	+861	-692	1.25	1.01
H4BT		687	+797	-715	1.16	1.04
H2GT	E	351	+296	-190	0.84	0.54
	W	351	+284	-214	0.81	0.61
H3GT	E	545	+350	-241	0.64	0.44
	W	545	+357	-323	0.66	0.59

CONCLUSIONS

This research was conducted experimentally to investigate the behavior of flange plate connections between the steel beam and the rectangular CFT column. Specimens were designed to simulate exterior and interior beam-to-column connections. The test results of three specimens with exterior, flange plate connection show that the tearing of the steel tube initiated at the both tips of the weld joining the flange plate to the steel tube because of the possible localized stress concentration. However, the plastic hinge formed in the beam beyond the flange plate. The essentially identical hysteresis curves show the characteristics of the pinching effect. The specimen with the beam through the CFT column did not tear at the steel tube and formed the plastic hinge in the beam led to a stable hysteresis curve. Two interior connection specimens with weak panel zone failed due to the premature fracture of the steel tube and the shear cracking in the concrete core.

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REFERENCES

1. Morino S, Kawaguchi J, Yasuzaki C, Kanazawa S. “Behavior of concrete-filled steel tubular three-dimensional subassemblages.” Proceedings of Composite Construction of Steel and Concrete II, 1991:726-41.
2. Prion HG, McLellan AB. “Connecting steel beams to concrete-filled steel columns.” Proceedings of ASCE Structures Congress on Composite Compression Members, San Antonio, TX, 1992: 918-21.
3. Ricles JM, Lu LW, Peng SW. “Experimental behavior of seismic moment resisting connections for CFT column composite frame.” Proceedings of Six U.S. National Conference on Earthquake Engineering, 1998.
4. Ricles JM, Lu LW, Sooi TK, Vermaas G. “Seismic performance of CFT column-to-WF beam moment connections.” Proceedings of the Third International Workshop of Connections in Steel Structures III: Behavior, Strength, and Design, Italy, 1995:99-114.
5. Kang CH, Shin KJ, Oh YS, Moon TS. “Hysteresis behavior of CFT column to H-beam connections with external T-stiffeners and penetrated elements.” Engineering Structures, 23, 2001:1194-201.
6. Chen CC, Lo SH. “Behavior of steel beam to circular CFT column connections.” Proceeding of the 7th International Conference on Steel Concrete Composite Structures, Sydney, Australia, 2003.
7. Tsai KC, Weng YT, Lin ML, Chen CH, Lai JW, Hsiao PC. “Pseudo dynamic tests of a full-scale CFT/BRB composite frame: displacement based seismic design and response evaluations.” Proceedings of the International Workshop on Steel and Concrete Composite Construction, Taipei, Taiwan, 2003:165-76.