Cyclic Loading Tests of Composite Moment Frames Using Octagonal CFTs

J J Lim¹ and T S Eom²

¹ Ph.D. candidate, Dept. of Architectural Engineering, Dankook Univ., 152 Jukjeon-ro, Gyeonggi-do, Korea
² Associate Prof., Dept. of Architectural Engineering, Dankook Univ., 152 Jukjeon-ro, Gyeonggi-do, Korea
E-mail: doublej17@naver.com

Abstract. This research investigates the seismic performance of two interior beam-to-column connections with floor slabs. Concrete-filled octagonal steel tube column, H-section steel beam, and U-section steel beam were used for composite moment frames, respectively. To release the stress concentration at the column face, two design methods were proposed. First, H-section steel beam passed through the column, directly. Stopper plates were also used at the column faces to decrease the slip deformation. Second, top and bottom flanges of U-section beam were not welded to the column face. Instead, to transfer the tensile force of the beam flange, reinforcement bars passing through the column were welded to the beam flanges. The test results showed that the ductility of moment connections was sufficient in the case of through details; However, the load-carrying capacity was significantly affected by the details of beam-to-column connections.

1. Introduction
Concrete filled tube (CFT) has a good flexural capacity, because the steel plates which resist the tension or compression are placed along the perimeter of the column section. Also, the steel tube confines the concrete core; at the same time the concrete inhibits the buckling of steel tube, which increases the stiffness and ductility. Therefore, CFT columns are used for many high rise buildings in highly seismic region. Recently, to maximize the composite effect, the octagonal steel tube was fabricated by bending a thin steel plate of thickness of 6 mm. The octagonal steel tube is fabricated by welding by two C-shaped plates and two planer plates (see Fig. 1). The rib plates at the corner increase the buckling resistance of steel tube. Also, octagonal section similar to circular section has a better concrete confinement effect than that of rectangular section. These guidelines, written in the style of a submission to IOP Conference Series: Materials Science and Engineering, show the best layout for your paper using Microsoft Word. If you don’t wish to use the Word template provided, please use the following page setup measurements.

In the connection where the beam frames into the column, particular attention should be paid to the connection details to secure the seismic performance required by AISC 341 [1]. This is due to the stress concentration followed by weld joint fracture at the beam-column connections. Such weld joint fracture can be easily more occur in the case of octagonal CFT column with thin steel plates.

Previous studies on the seismic performance of steel beam-CFT column moment connection have focused on reducing stress concentration at the weld joint and transferring beam flange force to column. Special details at the steel beam-CFT column connection can be divided into three categories. First, additional stiffener, cover plate, and diaphragm are welded at the connection [2, 3, 4, 5, 6]. Such
details increased the stiffness and ductility of connections, effectively. However, in some cases where the connection details of the additional plates and diaphragms were inappropriate, the brittle rupture at the weld joint occurred. Second, end-plate and T-stub welded to the beam flanges were bolted to the flange of CFT column [7, 8]. In the beam-column moment connection with such details, the tensile force of beam flange transferred to the column plate without weld joint fracture. However, in the case where the stiffness of column flange was not sufficient, the load-carrying capacity and the deformation capacity decreased due to the out-of-plane deformation of column plate. Third, additional steel plate, reinforcement bar, and diaphragm were passed completely through the column [9, 10, 11, 12, 13]. In this case, the beam-column connections showed superior ductility over the other comparable connections without a through detail.

Figure 1. Composite member sections

In the present study, seismic performance of composite moment connections using octagonal concrete filled tubes (OCFT) was investigated as follows.

1) Two full-scale beam-column connection specimens were tested, including with OCFT columns, with H-section beams, and with U-section composite beams.
2) H-section beams were directly passed through the OCFT column as the beam plates welded to the column flange. In the U-section beam-OCFT column connection, beam was not passed through the column and welded to the column plate. The beam flanges were not welded to the column to reduce a stress concentration. Instead, reinforcement bars were passed through the column and welded to the flanges of through beam.
3) Concrete slab was installed at the top of beam. Sufficient studs were also installed for full composite action between the concrete slab and the steel beam.
4) To evaluate the compressive resistance of slab, effective width of slab was designed to have sufficient transverse reinforcement bars. A transverse beam which passes through the column was also installed.
5) For each type of specimen, deformation capacity, load carrying capacity, energy dissipation capacity and failure mode were evaluated.

2. Moment connection details
Fig. 2 (a) shows the moment connection detail using conventional H-shaped steel beam and OCFT column. To transfer the beam force to the concrete infill of OCFT without a severe out-of-plane deformation of column plate, the entire section of steel beam is passed through the column with welding. Additional steel plates are also welded to cover the holes made by through beam. To increase the negative moment strength at the connection, holes are drilled in the column flange, and slab reinforcement bars pass through the column with sufficient anchorage length. Transverse beam passes through the column and longitudinal beam, respectively, with the area loss of the web of longitudinal beam.

Fig. 2 (b) shows the hybrid moment connection of welding and through reinforcement bars for an interior beam-column connection. The U-section composite beam was not passed through the column. Instead, to transfer the beam flange force to the column and the opposite beam, reinforcement bars are passed through the column and welded to the top and bottom flanges of U-section beams. To secure the yield strength of through reinforcement bars, the weld size and length should be sufficient. Note
that, to secure a plastic region without welding, the welding between the beam flanges and through reinforcement bars is performed far away from the column face. Holes are also drilled at the column flange for through reinforcement bars in slabs and U-section beams.

![Diagram of composite beam-OCFT column connections](image)

**Figure 2.** Details of interior composite beam-OCFT column connections

### 3. Test program

#### 3.1 Tests parameters and section details

In the present study, as indicated in Fig. 3, two full-scale interior beam-column connections, J1 and J2, were fabricated. The dimension symbols in cross section of the OCFT column, H-section beam, and U-section beam were indicated in Fig. 1. The sectional dimension of OCFT column (b1 b2) was 650 mm × 650 mm. The thickness of the planer plate and C-shaped plate of OCFT column was 6 mm. The specimens were designed for strong column-weak beam condition as the bending moment strength of OCFT column was $M_{nc} = 2898$ kN. The clear height of OCFT column was 3030 mm and the shear span between the column face and the loading point was 3055 mm. The entire length, width, and thickness of concrete slab were 7400 mm, 3000 mm, and 180 mm, respectively. The dimensions of the column, beam, and slab were same in both specimens. However, details of the beam-column connections differed in each specimen as follows.

In J1 (see Fig. 3(a)), H-section steel beam ($h = 482$ mm, $b = 300$ mm, $t_w = 11$ mm, $t_f = 15$ mm) passed through the OCFT column with welding at the column face. For a transverse beam, H-section steel beam ($h = 400$ mm, $b = 200$ mm, $t_w = 9$ mm, $t_f = 11$ mm) was used. Considering a required beam moment strength, beam splices with inclined end-plates were installed at a distance of 1200 mm. The cover plates were welded for continuity of column flanges. The continuity plate was designed to have the same thickness as the column flange. The concrete slab was installed at top of the steel beams. For full composite action between the steel beam and concrete slab, 2-Φ22 shear studs were placed at a spacing of 140 mm. Twenty SD400 D13 bars were used for longitudinal slab reinforcement bars, while four SD400 D22 bars passed through the column were used. The transverse slab reinforcement bars (D10 SD400) were installed closely to prevent the buckling of through slabs-reinforcement bars in the vicinity of the connection. To decrease the slip deformation between the beam flange and the concrete infill of OCFT column, stopper plates were used at the column face. The stopper plates were welded only to the beam bottom flanges. The welding details were designed to resist the yield strength of beam bottom flanges ($= b_f F_y$).

In J2 (see Fig. 3(b)), the slab details were the same as those of J1. U-section composite beam was used instead of H-section steel beam. The beam flanges were not welded to the column plates to avoid stress concentration. Instead, through reinforcement bars welded to the beam flanges were used. The welding between the through reinforcement bars and beam flanges was provided at a distance of 250...
mm for a plastic hinge region without welding. The yield strength of bottom through reinforcement bars ($f_y A_{sj1} = 699$ kN) was about 24% less than that of bottom flange ($F_{ybf} = 916$ kN).

Figure 3. Connection details of specimens

The material strengths of the steel plates, reinforcement bars, and concrete were represented in Table 1. The actual yield strengths of 6 mm- and 15 mm-thick plates used for OCFT column, U-section beam and H-section beam were $F_y = 509$ and 424 MPa, respectively. The yield strengths of slab reinforcement bars were $f_y = 535$ MPa for D10 and $f_y = 476$ MPa for D13. For the through reinforcement bars located in the concrete slab and the U-section beam, the yield strength of D22 bar was $f_y = 460$ MPa. The compressive strength of the concrete was $f_{ck} = 38$ MPa. Compression tests of three concrete cylinders were performed on the first day of cyclic tests.

3.2 Loading and measurement plan

Fig. 4 shows the test set-up for cyclic loading test of interior beam-column connection. The OCFT column was erected vertically and pinned at the bottom, while the beam with concrete slab was horizontally placed with pin supports at both ends. The transverse beam was not supported as the column and beam was supported laterally to prevent the transverse displacement. Displacement history for cyclic loading was planned in accordance with the SAC protocol in AISC 341-02 [1]. Note that an axial load was not applied to the OCFT column because it was not feasible in the laboratory to apply a meaningful axial force to the full-scale column. Axial load is beneficial in decreasing diagonal shear cracking in the beam-column connection, but it can accelerate the buckling of the steel tube.

Figure 4. Test set-up and measurement
4. Test results

4.1 Load-displacement relationships and failure modes

Fig. 5 shows the lateral load-drift ratio ($P-\delta$) relationships of the connections under cyclic loading. The lateral drift ratio $\delta$ was calculated by dividing the lateral displacement at the loading point by the clear height of the OCFT column (= 3060 mm). Initial slip deformation at the beam end with pin connection occurred in both specimens. Cyclic loading plan was modified by considering the initial slip deformation of 6.5 mm (see Fig. 5).

In J1 with through H-section beam and stopper plates (see Fig. 5(a)), flexural cracks transverse to the beam span occurred at $\delta= \pm 0.75\%$ and the concrete crushing at the column face occurred at $\delta= \pm 3.0\% \sim 4.0\%$. The stiffness of connection decreased severely at $\delta= \pm 2.0\%$ as the beam flanges yielded in tension. Also, the weld joint fracture at the column face occurred at $\delta= \pm 3.0\%$ (see Fig. 6(a)). However, the load-carrying capacity maintained even after the weld joint fracture. This indicates that the stopper plates can effectively prevent the slip deformation between the beam flange and the concrete infill after weld joint fracture. The buckling of beam bottom flanges and weld joint fracture between the cover plates and column plates occurred at $\delta= \pm 7.0\%$ (see Fig. 6(a)), which resulted in significant strength degradation.

In J2 with U-section beam (see Fig. 5(b)), the peak strengths were $P_u^+ = 741$ and $P_u^- = 704$ kN at $\delta= \pm 4.0\%$, respectively. After the peak strengths, the load-carrying capacity decreased gradually during repeated load cycles. The maximum lateral load at $\delta= \pm 6.0\%$ was 598 kN greater than 80% of peak strength $P_u^+ (= 741$ kN). Weld joint fracture between the beam webs and column plates also occurred about $\delta= \pm 4.0\%$. This result indicates that the through reinforcement bars welded to the beam flanges successfully transferred the flexural moment of the beam to column.

![Figure 5. Test set-up and measurement](image)

![Figure 6. Failure modes of beam-to-column connections under positive moment](image)

4.2 Design flexural strength and load-carrying capacity

Fig. 7 illustrates the plastic stress distributions to calculate the flexural strengths of the H-section and U-section beams with concrete slabs. The compressive resistance of concrete in slab is limited to the column width ($b_1$). This is because the concrete crushing occurred only at the column face (see Fig. 8).
The slab reinforcement bars passing through the column develop their yield strength $f_y$ in compression and tension, respectively.

Figure 7. Test set-up and measurement

In the case that H-section beam passes through the column with welding and stopper plates, the beam web plates welded to the tube wall do not develop their full yield stress, since the thin steel plates were used for OCFT column. In addition, since the web plate should carry shear force of the beam to the column, its contribution to the flexural strength can decrease. Therefore, the design plastic stress of the beam is reduced to 0.5 $F_y$. In the case of a hybrid moment connection using U-section beam and through reinforcement bars, the top and bottom flanges developed their yield strength $F_y$ only in compression.

As indicated in Fig. 5 and Fig. 7, the lateral strengths calculated by plastic stress distribution. In J1, the peak strengths were similar to the design lateral strengths ($P_u/P_n = 1.03$). In J2, the peak strengths $P_u$ were also similar to the design lateral strengths $P_n$ ($P_u/P_n = 1.04$).

4.3 Strain measurement results

Fig. 9 illustrates the peak strain distributions at each load cycles. In J1 with the through H-section beam reinforced with stopper plates, the strain of bottom flange was significantly greater than the yield strain (≈0.002) and large strain reversals occurred during load cycles. However, the strain of bottom web was nearly constant after yielding. This indicates that the slip deformation of bottom flange was limited due to the stopper plates. The neutral axis calculated from the stress distributions was located above due to composite action between the steel beam and the concrete slab. The strains of through reinforcement bars in slab were significantly greater than the yield strain (≈0.002). This indicates that the through slab reinforcement bars yielded experiencing large yield strains, thereby contributing to the negative moment strength of TSC beam at the column face. In J2, since the top and bottom flanges were not welded to the column plate, the tensile strains were nearly zero. The strain of bottom web welded to the column plate was constant after yielding due to the out-of-plane deformation of column plate and weld joint fracture.
5. Summary and concrete

In the present study, full-scale interior beam-column connections using OCFT column, H-section beam and U-section were fabricated. Cyclic loading tests were performed to investigate the load-carrying capacity, deformation capacity and failure modes. The major findings of the present study can be summarized as follows:

1) In J1, although the through H-section beam was used with welding, the out-of-plane deformation of steel tube occurred at the weld joint. However, the stopper plates successfully decreased the slip deformation between the beam bottom flanges and the OCFT column. Therefore, the load carrying capacity was maintained even after the weld joint fracture at the column face and the deformation capacity reached $\delta = \pm 7.0\%$

2) In J2 with hybrid connection details, the through reinforcement bars successfully the flexural moment of the beam to column. The tensile force of the beam flange was directly transferred to
the column concrete and the opposite beam flange through the reinforcement bars. Therefore, ductile behavior occurred without welding between the beam flange and column plate.

3) When the concrete slab was subjected to compression under positive bending, the compressive resistance of concrete was limited to the column width. Through reinforcement bars in concrete slab underwent significant plastic strains during cyclic loading and thus contributed to the positive and negative moment strength of the beam. However, other slab bars bypassing the column underwent relatively small elastic strains. To evaluate the compressive resistance of slab, effective width of slab was designed to have sufficient transverse reinforcement bars. A transverse beam which passes through the column was also installed.

6. References

[1] ANSI/AISC 341-02 2010 Seismic Provisions for Structural Steel Buildings (Chicago)
[2] Morino S, Kawaguchi J, Yasuzaki C and Kanazawa S 1993 In Composite Construction in Steel and Concrete II 726
[3] Schneider S P and Alostaz Y M 1998 J. of Constructional Steel Research 45 726
[4] Ricles J M, Peng S M and Lu L W 2004 J. of Structural Engineering 130 223
[5] Qin Y, Chen Z and Wang X 2014 J. of Constructional Steel Research 98 35
[6] Shin K J, Kim Y J, Oh Y S and Moon T S 2004 J. of Engineering Structure 26 1877
[7] Wu L Y, Chung L L, Tsai S F, Shen T J and Huang G L 2005 J. of Constructional Steel Research 61 1387
[8] Azizinamini A and Schneider S P 2004 J. of Constructional Engineering 130 213
[9] Mirghaderi S R, Torabian S and Keshavarzi F 2010 J. of Engineering Structure 32 2034
[10] Jeddi M Z, Ramli Sulong N H and Arabnejad Khanouki M M 2017 J. Engineering Structure 131 477
[11] Beutel J, Thambiratnam D and Perera N 2002 J. of Engineering Structure 24 29
[12] Korol R M, Ghobarah A and Mourad S 1993 J. of Structural Engineering 119 3463.

Acknowledgments

The present research was conducted by the research fund of Transportation and Maritime Affairs of Korea (Code 17CTAP-C129746-01, Infrastructure and transportation technology promotion research Program). The authors are grateful to the authorities for their support.