Cyclic loading tests of Concrete-Filled Composite Beam-Column Connections with Hybrid Joint Details

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ABSTRACT

In the present study, a hybrid joint detail of welding and bar reinforcement for composite beam-column connections was proposed. Concrete-filled steel octagonal tube and U-section are used for the column and beam, respectively. In the beam-column joint, the top flange and web of the beam U-section are connected to the column tube plates by welding. However, to reduce stress concentration at the weld joints, the bottom flange of the beam is not welded to the column. Instead, to transfer the tension force of the beam flange, joint-reinforcing bars which is anchored directly to the column concrete is used. Four exterior connections with the hybrid and conventional joint details were tested under cyclic loading and their cyclic behaviors were investigated. The test results showed that the hybrid joints successfully transferred the beam moment to the column. The strength and ductility of the hybrid joints were comparable to the conventional weld joints with exterior diaphragms; however, the joint performance was significantly affected by the details of joint-reinforcing bars. On the basis of the test results, recommendations for the seismic design and detailing of the hybrid joint were given.

1. INTRODUCTION

Fig. 1 shows the composite moment frame structures using OCFT columns and TSC beams. As a horizontal member, the TSC beam consists of steel U-section and infilled concrete, placing concrete slab on top of the U-section. The steel U-section is fabricated by welding two L-plates (i.e. top flange and web) at both sides of the bottom flange plate. At the mid-span of the TSC beam where positive moment due to gravity load is large, the thickness of the bottom flange is increased. On the other hand, the
thickness of the bottom flange around the column face is decreased so that a moment transfer from the TSC beam to the column can be minimized. The TSC beam with different flange thicknesses at the mid-span and ends is spliced by using friction bolts.

The OCFT (Octagonal Concrete-Filled Tube) column consists of octagonal steel tube and infilled concrete. The octagonal tube is fabricated by welding two thin C-plates and two thick plates. If necessary, the moment strength of the OCFT column about the strong axis can be increased by using larger thickness of the front and rear thick plates. Further, when compared to the conventional rectangular tube section, the octagonal tube of the OCFT column has a less width-to-thickness ratio ($= \frac{b_{eff}}{t_2}$, see Fig. 1), and hence the resistance against web and flange local buckling can be enhanced. Thus, the OCFT column can be more effective in designing for compression load.

In the case of concrete-filled composite members such as the TSC beam and OCFT column, basically, the flexure resistance is relatively high because the steel plates are placed at the outside surface of the section (refer to Fig. 1). Further, the inelastic deformation capacity can be enhanced due to the steel sections confining the infilled concrete. However, since both the OCFT columns and TSC beams use thin plates, attention needs to be given to the beam-column joint transferring the beam moment to the column. In particular, if the beam flange and web are directly welded to the steel plates of the column, premature cracking and fracture at the weld joint can occur due to a stress concentration. To avoid the premature weld fracture, joint-reinforcing details such as exterior diaphragm and vertical and horizontal stiffeners can be used (Lee et al. 2008, Park et. al. 2003, Shin et al. 2004, Chen et al. 2004). According to the existing studies, diaphragms and stiffeners can decrease the stress concentration at the weld joint and as a result, the deformation capacity of the beam-column connection subjected to cyclic loading is significantly increased. However, in the cases of the conventional diaphragms and stiffeners, fabricate is difficult and welding cost is high.
In the present study, a hybrid joint detail of welding and bar reinforcement for concrete-filled composite beam-column connections was proposed. Cyclic loading tests of four exterior connections were performed to investigate the load-carrying capacity, deformation capacity, and failure mode. On the basis of the test results, recommendations for the seismic design and detailing of the hybrid joint were given.

2. HYBRID DETAILS FOR BEAM-COLUMN JOINT

Fig. 2(a) shows the conventional beam-column joint details for the moment frame structures using the OCFT column and TSC beam. At the conventional beam-column joint, basically, the beam flanges and webs are directly connected to the steel plate of the column by fillet welding without stiffener. To prevent premature out-of-plane displacement of the column plate due to the pull-out force of the beam flanges and webs, headed stud anchors are used at the column plate. Slab-reinforcing bars are also placed to increase the negative moment strength of the TSC beam at the column face. To secure the moment transfer from the beam to the column, the slab-reinforcing bars for negative moment must be well-anchored by 90° standard hooks within the infilled concrete of the OCFT column.

If the thickness of the column plate is not sufficient, exterior diaphragms can be used at the levels of the top and bottom flanges of the beam. In the beam-column joint
with exterior diaphragms, the diaphragms can restrain premature cracking and fracture at the weld joint, thereby increasing the stiffness and deformation capacity of the connection.

Fig. 2(b) shows the hybrid joint details for the moment connection of the OCFT column and TSC beam, proposed in this study. Basically, the top flanges and webs of the beam are connected directly to the column plate by welding; however, the bottom flange is not welded to the column plate. This is because the weld joint of the bottom flange, which is usually subjected to significant tension force under positive moment, is vulnerable to a premature weld joint failure due to stress concentration. Instead, to transfer the tension force of the bottom flange, joint-reinforcing bars (cross-sectional area \( A_{sb} \); yield strength \( f_{yb} \)) are used. To secure force transfer between the bottom flange and joint-reinforcing bars, the joint-reinforcing bars are welded to the bottom flange (Flare-Bevel welding). The weld size and length \( l_w \) should be sufficient to transfer the yield force of the joint-reinforcing bars \( (= f_{yb} A_{sb}) \). Further, the anchorage length and hook extension of the joint-reinforcing bars should not be less than the required \( l_{dh} (= 0.24 f_y d_b \geq \min (8 d_b, 150 \text{ mm})) \) and \( 12 d_b \) within the column concrete. (KCI 2012)

Under negative bending, the slab-reinforcing bars \( (A_{st}) \) provided within the concrete slab resist tension force, along with the top flange of the beam U-section (see Fig. 2(b)). In particular, a greater tensile stress occurs at the slab-reinforcing bars than at the top flange. Thus, the slab-reinforcing bars can relax stress concentration at the weld joint between the beam top flange and column plate. For this, first, the slab-reinforcing bars must be well-anchored within the infilled concrete of the column and second, shear connectors need to be provided enough to ensure a full composite action between the steel U-section and concrete slab.

Fig. 2 illustrates the plastic stress distributions to calculate the flexural strength of the TSC beam transferred to the OCFT column. In the case of conventional beam-column joint details (see Fig. 2(a)), for positive bending, the bottom flange and web of the U-section resist the tension force. On the other hand, in the case of the hybrid joint details proposed in the present study (see Fig. 2(b)), the bottom flange which is not welded to the column plate do not contribute to the positive flexural strength; instead, the joint-reinforcing bars and the web of the U-section resist the tension force under positive bending. Thus, the positive flexural strength of the TSC beam is calculated by using the plastic stress of the infilled concrete \( (0.85 f_{ck}) \) and the web and top flange of the U-section \( (F_y) \), neglecting the contribution of the slab concrete and reinforcements. According to AISC 360, AISC 358, and Eurocode 8, the slab concrete is vulnerable to premature crushing failure at the column face. Thus, the contribution of the slab concrete is not considered in the calculation of the positive flexural strength.

Under negative bending, the top flange and web of the U-section are subjected to tension force. Further, the slab-reinforcing bars anchored directly to the column concrete \( (f_{st}) \) can contribute to the flexural strength. Thus, the negative flexural strength of the TSC beam is calculated by using the plastic stress distribution of the infilled concrete, steel U-section, and slab-reinforcing bars. In the hybrid joint details (see Fig. 2(b)), it is assumed that the bottom flange exerts the compressive yield stress \( (F_y) \), though the bottom flange is not welded to the column plate. The compressive
resistance of the joint-reinforcing bars within the infilled concrete is neglected for conservative estimation.

3. TEST PROGRAM

3.1 Specimen details

Four exterior OCFT column-TSC beam connection specimens, J1 ~ J4, were prepared for the cyclic loading test. In this study, the beam-column joint details were considered as the primary test parameter. Thus, conventional welded beam-column joint details were used for J1 and J2 (see Fig. 2(a)), while the hybrid joint details of welding and bar reinforcement were used for J3 and J4 (see Fig. 2(b)). Basically, the geometric properties and reinforcement details of the OCFT column, TSC beam, and concrete slab were the same. However, the beam-column joint details were different in the specimens as follows.

In J1 (see Fig. 3(a)), the U-section of the beam was connected to the thick plate of the column ($t_1 = 14$ mm). Exterior diaphragms (thickness 6 mm) were used to prevent premature tension fracture from occurring at the weld joint between the beam flange and column plate. In J2 (see Fig. 3(b)), twelve $\phi 19$ headed stud anchors were used at the column plate to transfer the tension force of the beam flange directly to the column concrete.

Fig. 3(c) and (d) show the beam-column joint details of J3 and J4 with the hybrid joint details of welding and bar reinforcement. The top flange and web of the beam were welded to the column plate; however, the bottom flange was connected by using the joint-reinforcing bars ($A_{sb} = 1548$ mm$^2$; $f_{yb} = 421$ MPa) without welding with the column plate. The total cross-sectional areas of the joint-reinforcing bars ($A_{sb} = 1548$ mm$^2$) were determined such that the yield force of the bottom flange ($= F_{yb}b_{bf}t_{bf} = 715$ kN) was similar to that of the joint-reinforcing bars ($= f_{yb}A_{sb} = 651$ kN). In J3 (see Fig. 3(c)), the joint-reinforcing bars were anchored by 90° standard hooks both in the column and beam. Further, a clear spacing of 28 mm between the bottom flange and joint-reinforcing bars was secured for concrete placement. On the other hand, In J4 (see Fig. 3(d)), the joint-reinforcing bars were welded to the bottom flange of the beam for direct force transfer. The length of the weld joint ($l_s = 250$ mm) was determined so that the weld joint strength exceeded the yield force of the joint-reinforcing bars. By welding the joint-reinforcing bars at a distance of 350 mm from the column face, a plastic deformation region for the joint-reinforcing bars was secured.
3.2 Material strengths

The compressive strengths of the concrete used for the TSC beam and OCFT column were $f_{ck} = 21.3$ MPa. The yield strengths of 6 mm and 14 mm plates were $F_y = 426$ MPa and 372 MPa, respectively. The yield strength of the slab-reinforcing bars for
negative moment (D25) was $f_{yt} = 673\text{MPa}$. The yield strength of the joint-reinforcing bars (D22) used in J3 and J4 was $f_{yb} = 421\text{MPa}$.

3.3 Testing method

Fig. 4 shows the set-up for the cyclic loading test. The OCFT column was horizontally placed with pin supports at both ends, while the TSC beam with concrete slab was placed vertically. An actuator for cyclic loading was installed horizontally at the top of the beam. The shear span from the column face to the loading point was $l_s = 3000\text{mm}$ and the column span between two pin supports was also $l_c = 3000\text{mm}$. The loading protocol was planned in accordance with AISC 341 (AISC 2005). Linear variable differential transformer (LVDTs) were used to measure displacements at the loading point and pin supports, and deformations of the column, beam, and beam-column joint.

4. TEST RESULT

4.1 Lateral load-drift ratio relationships and failure modes

Fig. 5 shows the lateral load-drift ratio ($P \cdot \delta$) relationships. The lateral drift ratio $\delta$ was calculated by dividing the lateral displacement at the loading point by the beam shear span ($l_s = 3000\text{mm}$). In the figure, the positive and negative lateral loads ($P$) cause positive and negative beam moments, respectively. The peak strengths of the positive and negative directions, $P_u^+$ and $P_u^-$, were marked as the circles. The nominal strengths $P_n^+$ and $P_n^-$ of the specimens calculated by using the actual material strengths
were also marked as the horizontal dotted lines. The nominal strengths $P_{n}^+$ and $P_{n}^-$ were calculated by dividing the flexural strengths of the TSC beam (see Section 2) by the shear span $l_s$. In J1, the effective shear span ($= l_s - 130$ mm, see Fig. 3(a)) rather than $l_s$ was used considering the dimension of the exterior diaphragms. The primary failure modes of specimens at each cyclic loading step were indicated in Fig. 6.

In J1 with exterior diaphragms (see Fig. 5(a)), stiffness degradation occurred at $\delta = 1\%$ and a ductile behavior without significant strength-degradation was maintained during cyclic loading at $\delta \geq 2\%$ and $\delta \leq -2\%$. A tensile fracture occurred first at the weld joint between the bottom flange and column plate at $\delta = 2\%$ under positive moment. Then, the weld joint fracture was transmitted at $\delta = +3\%$ to the weld joint between the bottom flange and diaphragms (see Fig. 5(a)). At $\delta = -4\%$ where the negative peak strength ($P_{u}^- = 293$ kN) was reached, a local buckling occurred at the exterior diaphragms due to compressive force under negative moment. After the peak strength, the load-carrying capacity was significantly decreased as the bottom flange of the beam was separated completely from the column plate. However, the weld joint failure of the top flange did not occur throughout the test because stress concentration at the weld joint was relaxed to some extent due to the concrete slab.

In J2 (see Fig. 5(b)), stiffness degradation began to occur at $\delta = 1\%$. A premature weld joint fracture occurred at $\delta = +2\%$. After $\delta = +3\%$, the load-carrying capacity was decreased due to the weld joint fracture of the bottom flange (see Fig. 6(d))

Fig. 5 Lateral load-drift ration relationship
and (f)). However, the weld joint fracture did not occur at the top flange. The peak strengths of J2 were greater than those of J1 with exterior diaphragms, though the deformation capacity was relatively less.

Fig. 5(c) and 5(d) show the test results of J3 and J4 with hybrid beam-column joint details, respectively. Flexural yielding occurred at $\delta = 1\%$ and the peak strengths $P_u^+$ and $P_u^-$ were reached at $\delta = 2\%$. In the case of J3, the peak strengths were 10% less than those of J4 and thus the energy dissipation capacity was significantly decreased. Such poor behaviors were attributed to slip deformation of the joint-reinforcing bars, which was discussed in detail in Section 4.4.

In J3 with the joint-reinforcing bars, the weld joint fracture initiating at the bottom of the web occurred at $\delta = +2\%$ (Note that the bottom flange was not welded to the column plate). After that, the weld joint fracture propagated further and finally the bottom flange of the beam was completely separated from the column (see Fig. 6(g).
Concrete crushing occurred at the bottom of the beam, though the concrete were confined with the U-section. As a result, the load-carrying capacity under positive loading was significantly decreased after $\delta = +2\%$ (see Fig. 5(c)). In J4, the weld joint fracture occurred at $\delta = +2\%$ at the bottom of the web (see Fig. 6(j)). Although the bottom flange was completely separated from the column plate at $\delta = +4\%$ (see Fig. 6(k)), the overall load-carrying capacity of J4 was not significantly decreased because the joint-reinforcing bars directly anchored to the column concrete transferred the tension force of the beam flange to the column. It should be noted that, under negative loading at $\delta = 4\%$, a tensile fracture occurred at the weld joint between the top flange and column plate and as a result, the load-carrying capacity was significantly decreased during the ensuing behavior (see Fig. 6(l) and 6(d)).

4.2 Concrete failure and slab reinforcing bar strains

Fig. 7(a) shows the cracking and failure aspects of the slab concrete. In all specimens, the cracking and failure modes of the slab concrete were almost the same. For negative loading in which the slab is subjected to tension force, flexural cracks occurred in the transverse direction to the slab-reinforcing bars. For positive loading, on the other hand, splitting cracking began to occur at $\delta = 1\%$ along the longitudinal direction. Then, concrete spalling significantly occurred around the column face at $\delta = 2\%$ and as a result, the slab-reinforcing bars were exposed without cover. In addition,
slip deformation at the interface between the slab concrete and column plate occurred at the periphery of the column and ultimately, the concrete of the slab end was completely spalled out.

Fig. 7(b) shows the strains of the slab-reinforcing bars used for J1 and J3. The horizontal and vertical axes of the graph are the lateral drift ratio and slab-reinforcing bar strains, respectively. Five strain gauges (SR1 ~SR5) were installed at a distance of 50 mm from the column face. In the cases of the slab-reinforcing bars (SR3 ~SR5) with headed bar anchorage at the slab ends, the strains were significantly less than the yield strain (= 0.0021). In the cases of the slab-reinforcing bars (SR1 and SR2) that were anchored directly to the column concrete, in contrast, the strains were significantly greater than the yield strain and large strain reversals occurred during the load cycles. This indicates that, although high-strength reinforcing bars of Grade 600 MPa were used and the anchorage length of the 90° hooks was less than the required \( \frac{l_{dh}}{l_{dh,req}} \equiv 0.65 \), the slab-reinforcing bars yielded completely experiencing large plastic strains. In particular, the slab-reinforcing bars underwent tensile strains even under positive loading where the slab is subjected to compression. This indicates that, due to the residual strain, the slab-reinforcing bars were subjected to significant compressive stress at large tensile strains. Thus, the buckling of the slab-reinforcing bars occurred during the cyclic loading, and severe cover spalling followed after the bar buckling (see Fig. 7(a)).

Regarding the load transfer mechanism at the composite beam-column joint with concrete slab, the following conclusions can be inferred from the failure mode of the concrete slab and the strains of the slab bars shown in Fig. 7.

1) When the beam is subjected to positive moment, the concrete of the slab did not contribute to the flexure of the TSC beam. This is because concrete crushing of the slab occurred early at the column face. Premature concrete crushing failure of the slab concrete at the column interface was reported in AISC 358 Chapter 6.

2) When the beam is subjected to negative moment, the slab-reinforcing bars anchored directly to the column concrete fully contributed to the flexure resistance of the beam. However, the slab bars which were not anchored directly to the column concrete did not contribute to the flexural resistance of the TSC beam.

Fig. 8 Steel strains measured from U-section flanges

\[ \frac{320}{490} = 0.65 \]
4.3 Strains of joint-reinforcing bars

Fig. 8(a) and (b) show the strains measured from the joint-reinforcing bars in J3 and j4, respectively. The horizontal and vertical axes denote the bar strains and lateral load, respectively. In J4 with the joint-reinforcing bars welded to the beam bottom flange, the joint-reinforcing bars underwent significant plastic strains while the beam bottom flange remained almost in elastic strains. This indicates that the tension force of the bottom flange under positive loading was successfully transferred to the joint-reinforcing bars through the weld joint. On the other hand, in J3 where the joint-reinforcing bars were not welded to the beam bottom flange, force transfer between the joint-reinforcing bars and the bottom flange should be carried out through bond stress of the surrounding concrete. However, since the surface of the bottom flange was very smooth, the bond resistance of the concrete was not sufficient and thus slip deformation occurred significantly. In other words, the force transfer between the joint-reinforcing bars and the bottom flange were insignificant. Consequently, as shown in Fig. 8(a), the strains of the joint-reinforcing bars in J3 were considerably less than those of J4. Further, the load-carrying capacity of J3 was 10 % less than that of J4 and the hysteretic energy dissipation were significantly decreased due to the pinching in the cyclic behavior.

5. CONCLUSION

In the present study, a hybrid beam-column joint detail of plate welding and bar reinforcement for TSC beam-OCFT column moment connection was proposed. Cyclic loading tests were performed to investigate the seismic performance of the hybrid joint details. The major findings of the present study can be summarized as follows:

1) In the TSC beam-OCFT column joint of all specimens, weld joint fracture occurred early at a comparatively small drift ratio of $\delta = 2\%$. In J1 with exterior diaphragms and J4 with hybrid joint details, the strength-degradation was relatively insignificant even after the weld joint fracture occurred; thus the deformation capacity was comparable to $\delta = 4\%$. In J1 where the weld joint was not reinforced, however, the brittle failure occurred at $\delta = 3\%$ after the weld joint fracture.

2) In J3 and J4 with hybrid joint details, the joint-reinforcing bars successfully transferred the flexural moment of the beam to column. The tension force of the beam flange was directly transferred to the column concrete through the joint-reinforcing bars and thus the ductile behavior occurred even after the weld joint fracture of the beam flange. In J4 where the joint-reinforcing bars were welded to the beam flange, the joint-reinforcing bars underwent significant plastic strains and primarily contributed to the energy dissipation capacity during cyclic loading. On the other hand, in J3 where the joint-reinforcing bars were not welded to the beam flange, bond-slip occurred significantly after $\delta = 2\%$, and consequently the load-carrying capacity and energy dissipation capacity were significantly reduced.

3) When the concrete slab was subjected to compression under positive loading, cracking and crushing of concrete occurred significantly at the column face after $\delta = 1\%$. This indicates that the concrete and the reinforcing bars of the slab did not
contribute to the positive moment strength of the beam. In contrast, the slab-reinforcing bars anchored directly to the column concrete underwent significant plastic strains during cyclic loading and thus contributed to the negative moment strength of the beam. However, the reinforcing bars anchored at the slab end did not contribute to the flexure resistance of the beam.

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