Seismic Behavior of Top-Weld Bottom-Bolt Joints Between CFST Columns and H-Beams

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Abstract
In recent decades, connections between concrete-filled steel tubular columns (CFST) and H-steel beams have been well designed and implemented. However, owing to the detail of the weld, brittle failure often occurs at weld seams. In this study, an innovative joint was developed to connect CFST columns and H-steel beams using a top-weld bottom-bolt connection to minimize the effect of welding quality on the seismic resistance of joints. Six specimens were designed for cycle-reversed loading tests to discuss the seismic performance of this joint. Four configurations, including different connection methods, beam heights, column forms, and stiffener thicknesses, were considered in the test. The impacts of different configuration forms on the failure mode, strength, stiffness, ductility, and energy dissipation of the specimens were evaluated. The test results demonstrated that the columns with or without concrete had a significant effect on the deformation capacity. However, a smaller effect was observed on other indicators. The replacement of the through-diaphragm and an increase in the beam height adversely influenced the ductility of the joint. Moreover, changing the stiffener thickness and using a full-bolted connection affected the failure mode. The joint type analyzed in this study satisfies the strong column–weak beam design criterion and the related seismic provisions.

Keywords Seismic behavior · Concrete-filled steel tubular columns · Weld–bolt · Through-diaphragm connection · Outer rib annular plate connection

1 Introduction
Concrete-filled steel tubular (CFST) columns, which possess the advantages of steel and concrete, have been widely applied in frame structure systems and other modern building structures in the recent decades because of their high bearing capacity, excellent seismic performance, low cost, good fire resistance, and high construction efficiency [1, 2]. CFST components and structures have been studied extensively for several decades and are still being investigated and improved in terms of theoretical calculations, structural performance, and construction efficiency [3–5]. Connections are considered to be the most critical members in CFST composite frame structures. They have a significant impact on the overall load transfer mechanism and internal force distribution of structures. Under seismic action, connections are in a complex state in terms of the moment, shear, and axial compression, which can cause panel zone failure, resulting in the instability of an entire structure [6, 7]. Therefore, an important research direction is to study novel CFST column-beam joints with excellent seismic performance.

Several types of established CFST column-steel beam joints have been used extensively in many countries. Such joints include an internal diaphragm, outer annular plate, or a through-diaphragm [8–10]. Many studies on the performance of these connections have been reported owing to their practical and economic advantages. Among them, the studies on internal diaphragm connection have included the
experimental behavior and theoretical analysis of this type of joints by Qin et al. [11], Shim et al. [12], and Ricles et al. [13], Nie et al. [14]. These studies indicate that the internal diaphragm could promote the local stability of the joint zone. In addition, the internal diaphragm connection exhibits effective force transmission and high stiffness with good energy dissipation capability. However, difficulties may be entailed in welding the internal diaphragm during construction if the size of the column is too large. Additionally, the two welds at the top- and bottom-flange positions could be prone to stress concentration.

As another traditional connection method, the outer annular plate joint has also been studied by many scholars, including Shin et al. [15], Li et al. [16], and Dessouki et al. [17]. The joint has the advantages of high stiffness, clear force transmission, good ductility, and uniform stress distribution in the panel zone. For the convenience of construction and interior aesthetics, based on the outer annular plate, the outer rib annular plate connection was studied by Bai et al. [18] and Chen et al. [19]. However, the large steel usage limits the practicality of both outer ring plate and outer rib ring plate connections.

Unlike the above connections, which are all column continuous joints, the column in through-diaphragm joints is cut off at the panel zone, and the beams are connected to the column member using the through-diaphragm [20, 21]. Many studies have been conducted to evaluate the seismic behavior of such joint, including Wu et al. [22], Qin et al. [23, 24], Yu et al. [25], Matsui et al. [26], Kanatani et al. [27], Morino et al. [28], Nishiyama et al. [29, 30], Fujimoto et al. [31], Wang et al. [10], Chen et al. [32]. The results of previous studies have shown that this connection outperforms the internal diaphragm connection, showing more outstanding seismic performance and a higher degree of factory fabrication.

The traditional forms of CFST column-H steel beam connections use symmetrical connections, such as welded and full-bolted connections, for the top and bottom flanges. However, welded joints are prone to serious brittle failures, such as those observed in the Northridge earthquake in 1994 and Hanshin earthquake in 1995 [33]. The traditional full-bolted joints exhibit satisfactory performance compared to welded joints, thereby avoiding the problem of poor seismic performance of welded joints owing to welding. However, the cost and onsite construction accuracy requirements of full-bolted joints are high. Therefore, to overcome these limitations of traditional joints and to meet the requirements of construction accuracy, some scholars have studied bolt-welded joints, i.e., joints in which the top and bottom flanges are welded and bolted, respectively. However, only a few studies are available on the seismic performance of this connection form wherein the top and bottom flanges are asymmetrical [34, 35]. Accordingly, it is necessary to investigate the mechanical indicators and seismic performance of this asymmetric connection.

To minimize the defects of conventional connections and achieve satisfactory seismic behavior, an innovative joint between CFST columns and H-beams using an outer annular plate and through-diaphragm is introduced in this paper based on a bolt-welded connection. There is only one weld in this new TWBB joint, thus reducing the difficulty of welding on-site. In addition, the joints are more convenient for concrete pouring because there is only one diaphragm at the joint. More importantly, the bolted connection of the bottom flange avoids the risk of fracture at the bottom flange weld seam during seismic action.

Through-diaphragm connections and outer-annular-plate connections are known to exhibit good force transmission. Furthermore, through-diaphragm connections and outer-annular-plate connections are known to exhibit good force transmission. Therefore, the principal purpose of this study is to examine the effects of the connection method, connection type, column form, beam cross-sectional size, and stiffener thickness on the seismic performance of the proposed connection. To study seismic performance of H beams to CFST column joint with top-weld and bottom-bolt (TWBB) connection detailing, five specimens specifically were designed and tested under the cyclic loading. Additionally, as the seismic performance comparison, a traditional H beams to CFST column joint specimen with bolted connection were also tested.

2 Experimental Programs

2.1 Specimen Design

Five through-diaphragm and outer rib annular plate interior joints reinforced using vertical stiffeners to steel tubular columns and one outer rib annular plate interior joint reinforced using vertical stiffeners were subjected to cyclic loading tests to evaluate their seismic properties. The details and connection diagrams of the six test specimens are provided in Table 1 and Fig. 1, respectively. The height and length of all specimens were 2890 mm and 3850 mm, respectively. The heights of the upper and lower columns were 1595 mm and 1295 mm for connections JD1–JD3 and JD5–JD6, respectively. The CFST column had a section of $250 \times 250 \times 8$ mm. The dimensions of the beam section of the specimens were $300 \times 150 \times 6.5 \times 9$ mm, except for the H-beam section of the JD5 joint, which had dimensions of $350 \times 175 \times 7 \times 11$ mm. The H-beam web and connecting plate were connected using M20 and M24 friction-type bolts of grade 10.9. The high-strength bolts in the members were secured by the torque wrench. All mentioned specimens were full-scale models designed based on the Chinese...
Table 1 Summary of test specimens

| Specimen | Beam section (mm) | Vertical stiffener (mm) | Column form | Connection type | Connection method |
|----------|-------------------|-------------------------|-------------|-----------------|-------------------|
| JD1      | 300 × 150 × 6.5 × 9 | 570 × 100 × 10 | CFST        | I               | Bottom bolted     |
| JD2      | 300 × 150 × 6.5 × 9 | 570 × 100 × 10 | ST          | I               | Top welded        |
| JD3      | 300 × 150 × 6.5 × 9 | 570 × 100 × 15 | CFST        | I               | Bottom bolted     |
| JD4      | 300 × 150 × 6.5 × 9 | 570 × 100 × 10 | CFST        | II              | Bottom bolted     |
| JD5      | 350 × 175 × 7 × 11 | 570 × 100 × 10 | CFST        | I               | Bottom bolted     |
| JD6      | 300 × 150 × 6.5 × 9 | 570 × 100 × 10 | CFST        | bolted          |                   |

Type I refers to the through-diaphragm and outer rib annular plate interior connection, and Type II refers to the outer rib annular plate interior connection.

codes JGJ99-2015 and JGJ138-2016. Additionally, typical Chinese building structures were referenced while designing the models.

For specimens JD1 and JD5, the top and bottom flanges of the H-steel beam were connected to the outer rib annular plates and through-diaphragm by welding and bolting, respectively. The through-diaphragm, which penetrated the column, was welded to the tube. The H-beam web was bolted to a connection plate that was welded to the column. With a 570 × 100 × 10 mm section, the vertical stiffeners were welded to the outer rib annular plates. A circular chamfer with a radius of 50 mm was set at the connection. The through-diaphragm thickness was 1 mm larger than that of the beam flange, which was primarily used to improve the bending rigidity of the joint. To fill concrete, the center of each diaphragm reserved a 250-mm-diameter hole. As a reference specimen, JD6 was manufactured with a conventional full-bolted connection design.

The column of specimen JD2 was a hollow steel tube without concrete, as shown in Fig. 1b. As shown in Fig. 1c, Specimen JD3 was identical to the joint JD1, except that the vertical stiffeners of JD3 were 15 mm thick so as to examine the effect of the thickness of the vertical stiffener. Specimen JD4 had four outer rib annular plates and four vertical stiffener plates. The vertical stiffeners were groove-welded to the flange of the H-beam, as shown in Fig. 1d. A larger beam section size was chosen for specimen JD5 to evaluate the effect of the beam size on the new connection under seismic performance. Specimen JD6 was similar to joint JD1, except that the top flange of the H-beam of JD6 was bolted to the outer rib annular plates, as shown in Fig. 1f.

Table 3 Concrete material properties

| Strength grade | $f_{ck}$ (MPa) | $f_{tk}$ (MPa) | $E_c$ (MPa) |
|---------------|----------------|----------------|-------------|
| C45           | 31.2           | 2.8            | $3.40 \times 10^5$ |

2.2 The Fabrication of Specimens

All steel components were machined and fabricated in a specialized steel component factory, while the concrete pouring, curing, and assembly of the components were carried out in the fabrication factory. First, the steel tubes were poured using commercial concrete and cured under standard conditions for 28d. Then, the top flange of the beam and the outer rib annular plate were welded by butt bevel welding. Subsequently, high-strength bolts were used to connect the beam web to the connecting plate, the bottom flange to the diagram, and the top flange to the outer rib annular plate at the full-bolt joint. During the assembly of the specimens, it was found that the TWBB joints had the advantages of faster construction and lower installation accuracy requirements than the full-bolt joint. Also, it was more convenient to pour because only one diagram was set in the core area.

2.3 Material Properties

A single tensile test on steel was performed with four tensile samples cut from the steel columns, beams, and connections to measure the yield strength ($f_y$), ultimate tensile strength ($f_u$), and elastic modulus ($E_s$). For concrete, the axial compressive strength ($f_{ck}$), axial tensile strength ($f_{tk}$), and elastic modulus of concrete ($E_c$) were determined using the standard cylinder compression test after 28 days of standard curing. The average values of steel and concrete obtained from the material tests are listed in Tables 2 and 3, respectively.
2.4 Test Setup and Loading Procedure

The device used in the test (Fig. 2) primarily includes the loading device and reaction frame. The beam end had two 1000 kN push–pull actuators to apply low-cycle reversed loads. A hydraulic jack was placed on top of the column to apply an axial load of 770 kN and 480 kN to the joints with and without concrete-filled, respectively. The column of the
specimen was vertically installed on the ball joint base, and the bottom and top of the column were fixed using two horizontal rods and connected with the reaction wall to provide a horizontal reaction force. The bottom of the column used
2.5 Instrumentation

The load and displacement data for the beam ends were collected by the load sensors in the actuator. Beam-end load and displacement were automatically recorded by the load cell in the actuator. Before applying the vertical load to the beam end, an axial compression load should be applied slowly to the column top using a hydraulic jack (until 770 kN or 480 kN) while maintaining stability to ensure that the axial compression ratio is approximately 0.2. The load was controlled via the force before the specimen yielded. The load cycles for each grade were carried out once, with a 20 kN difference between grades. Displacement control was adopted after the specimens yielded, and the load cycles for each stage were carried out twice with 0.5 $\Delta_y$ ($\Delta_y$ refers to yield displacement) as the step difference, as illustrated in Fig. 3. The experiment was terminated when the specimen was damaged or the load dropped to 80% of the maximum load.

3 Test Results

3.1 General Observations

In specimen JD1, the top flanges of the left and right beams began to bulge as the specimen displacement reached 2.5 $\Delta_y$. At a specimen displacement of 3 $\Delta_y$, cracks appeared on the welds between the steel beam connecting plates and columns on both sides, and the top flange bulge on the left and right sides increased consistently (Fig. 5a). When the loading increased to 3.5 $\Delta_y$, cracks emerged on the weld between the right annular plate and column wall, and the weld between the right connecting plate and column wall continued to crack (Fig. 5b). Upon loading the specimen to a displacement of 4.5 $\Delta_y$, 3.5 mm and 0.85 mm-wide cracks appeared on the flange welds on the left and right beams, respectively. Simultaneously, the welds on the connection between the left connecting plates of the beam and column wall cracked. The cracks at the joint between the left connecting plate of the beam and the column continued to expand until a displacement of 5 $\Delta_y$. Furthermore, cracks emerged at the connection between the left top flange and web. Finally, as shown in Fig. 5c, the weld crack between the annular plate and left flange fractured, and the test was terminated.

For specimen JD2, the left beam top flange appeared to buckle after the displacement reached 3 $\Delta_y$. Additionally, the top flange of the left beam started to buckle and cracks occurred between the left and right connecting plates and column wall welds. When loading to 4 $\Delta_y$, the buckling at the left beam top flange increased (as shown in Fig. 6a), and the right beam top flange to buckle. The buckling of the right beam top flange continued to aggravate when the loading was increased to 4.5 $\Delta_y$. Furthermore, the cracks at the welding seam of the connecting plate and column wall continued to expand, and a large crack appeared on the top flange weld of the left beam. When the displacement reached 5.5 $\Delta_y$, the shear deformation was evident in the panel zone, as demonstrated in Fig. 6b. When the displacement increased to 6 $\Delta_y$, a weld fracture occurred on the right flange of the beam (Fig. 6c), and the test ended.

In the case of specimen JD3, the connection between the right and left sides of the diaphragm and the bottom flange of the steel beam slid by 3 mm and 6 mm successively when the displacement reached 3 $\Delta_y$. Moreover, as the displacement reached 3.5 $\Delta_y$, the top flanges of the beam buckled, whereas the weld between the stiffener and the column cracked (Fig. 7a). When the displacement reached 4 $\Delta_y$, the welding of the left beam top flange cracked, and the weld at the left diaphragm of the beam was completely broken (Fig. 7b). When the displacement increased to 4.5 $\Delta_y$, the welding crack at the top flange of the right beam (as shown in Fig. 7c) fractured, and the test was terminated.
When the specimen JD4 was loaded to $2.5 \Delta_y$, there was a 3 mm dislocation between the left beam bottom flange and the diaphragm, and the right beam top flange buckled slightly. This buckling continued to increase when the displacement...
reached $3 \Delta_y$ (Fig. 8). When the displacement reached $3.5 \Delta_y$ and $4 \Delta_y$, cracks emerged successively on the welds between the column wall and two sides of the connecting plate. The buckling of the right beam flange was severe. Eventually, the test was terminated because the weld crack on the right beam top flange fractured when the load reached $5 \Delta_y$.

For specimen JD5, a minor crack between the right annular plate of the beam and the column was noted at the weld while loading to $2.5 \Delta_y$, and small cracks emerged in the weld between the diaphragm and the lower column. When the load reached $3 \Delta_y$, the joint between the left side of the annular plate and top flange started to buckle and a weld crack was observed. When the load reached $3.5 \Delta_y$, a weld crack occurred between the right connecting plate and the column. Additionally, column buckling was also observed. When the loading reached $4 \Delta_y$, the top flange welds on the left and right sides broke completely (as shown in Fig. 9) and the specimen failed.

In contrast with the other specimens, no apparent phenomena were observed in the case of specimen JD6 before it was loaded to $4 \Delta_y$. When the displacement reached $4 \Delta_y$, the left beam bottom flange appeared to buckle slightly. When the displacement reached $6 \Delta_y$, the weld between the left side of the diaphragm and the column was partially broken, and the weld between the right side of the diaphragm and the lower column was fractured (Fig. 10a). Meanwhile, the left beam top flange also started to buckle. When the load continued to increase to $6.5 \Delta_y$, local buckling was observed at the variable section between the diaphragm and the bottom flange (Fig. 10b). The test was terminated when the load decreased to 80% of the maximum load.
Fig. 10 Failure mode of specimen JD6: a diaphragm fracture and b local buckling

Table 4 Characteristic values of each specimen

| Specimen | Direction | $P_y$ | $P_m$ | $\Delta_y$ | $\Delta_u$ | $\mu$ |
|----------|-----------|-------|-------|------------|------------|-------|
| JD1      | Positive  | 135.45| 151.85| 27.36      | 74.56      | 2.73  |
| JD1      | Negative  | 133.08| 151.60| 28.10      | 68.36      | 2.43  |
| JD2      | Positive  | 117.30| 135.10| 32.70      | 99.64      | 3.05  |
| JD2      | Negative  | 114.21| 134.85| 28.78      | 69.41      | 2.41  |
| JD3      | Negative  | 125.57| 142.65| 27.13      | 99.64      | 3.05  |
| JD3      | Negative  | 116.92| 145.70| 26.07      | 60.54      | 2.32  |
| JD4      | Positive  | 143.73| 161.05| 30.65      | 71.49      | 2.33  |
| JD4      | Negative  | 138.84| 163.40| 33.18      | 68.07      | 2.05  |
| JD5      | Positive  | 191.56| 215.75| 37.12      | 68.69      | 1.85  |
| JD5      | Negative  | 181.77| 204.10| 27.90      | 43.55      | 1.56  |
| JD6      | Negative  | 154.82| 175.95| 33.96      | 94.38      | 2.78  |
| JD6      | Negative  | 146.98| 168.60| 32.81      | 90.40      | 2.76  |

$P_y =$ yield load, $P_m =$ maximum load, $\Delta_y =$ yield displacement, $\Delta_u =$ failure displacement, and $\mu =$ ductility

3.2 Load–Displacement Response

Figure 11a–f demonstrates the hysteresis curves for specimens JD1–JD6. The six joint curves showed good linearity in the early phase, indicating that the joints were firmly connected before loading and the components were in the elastic stage. In the case of specimens JD1–JD5, the shape and change trends of the hysteresis curves were similar. The hysteretic loops became full gradually with increased displacement when the specimens reached the yielding load. The loops exhibited a spindle shape. The curves of JD1, JD2, JD3, and JD4 were stable and full. Compared with JD1, the hysteretic curve of JD2 was fuller and showed better ductility, but its bearing capacity was lower than JD1. As the difference between JD1 and JD2 depends on whether the column was filled with concrete, the filled concrete was an essential factor affecting the structural bearing capacity and energy consumption of this new connection. The beam-column connection types of specimens JD1 and JD4 were different, but their ultimate strength was essentially similar. Furthermore, the descending sections of the two curves were similar after reaching the peak load. As shown in Fig. 11e, a sudden drop in loading, which was caused by the top beam flange fracture, was noted in the curve of specimen JD5. In addition, the load capacity of the beam end of JD5 was significantly increased owing to the increased beam section size, resulting in a higher maximum load on its curve than the other specimens. Moreover, the pinching phenomenon was observed in the hysteresis response of the full-bolted specimen JD6 because of the large slip caused by the full-bolted connection, which made the curve anti-S-shaped. Because of the large displacement and bearing capacity of specimen JD6, the hysteretic curve encompasses a large area with high cumulative energy consumption (Fig. 17). Eventually, a more severe tear was observed at the diaphragm weld. The curve of JD6 was relatively symmetrical compared to other five specimens because the top and bottom flanges of JD6 were connected to the column using bolts.

The skeleton curves obtained from each hysteresis curve are presented in Fig. 12. The ascending, plateau, and descend-
Fig. 11 Load–displacement curves of specimens: a JD1, b JD2, c JD3, d JD4, e JD5, f JD6 and g load–displacement curves for all specimens.

The sections of the curves were clear. This reflected the elastic–elastoplastic–plastic process. The initial stiffness of each joint was essentially the same. However, owing to the effect of the component assembly and sensor errors, the slope...
of the skeleton curves was relatively small when loading in a positive direction. This effect improved as the load increased. As the load continued to increase, the turning point appeared in the curves, and the component entered the yielding stage. Subsequently, the load increment began to slow down and the displacement increased rapidly. After reaching the ultimate load, the load on the beam end of each joint dropped rapidly with the failure of the flange and diaphragm. By comparing the skeleton curve of each joint and referring to Table 4, we found that: (1) in JD1 and JD2, the bearing capacity of the joint increased by approximately 12% owing to the concrete filled in the steel tube. (2) Compared with JD1, the yield load and maximum load of JD3 were reduced. This indicated that increasing the stiffener thickness could not improve the bearing capacity of the beam end. (3) JD1 and JD4 have similar skeleton curves. Although the maximum load of JD4 was slightly larger than JD1, the two different connection types had a minor impact on the load capacity of the joint. (4) The yield load and maximum load of JD5 were significantly higher than those of JD1, indicating that increasing the beam section size could enhance the load capacity of the beam end. However, the load capacity decreased sharply after reaching the maximum load owing to the welding fracture on the top flange. (5) The ascending and descending sections in the curve of the full-bolted specimen JD6 were relatively flat. Compared with other components, the failure displacement of JD6 increased significantly and showed adequate deformation capacity and high bearing capacity.

4 Analysis and Discussion

4.1 Strength Degradation

Strength degradation is a phenomenon where the maximum load degrades with the increase in the number of cycles when the displacement is maintained constant under cyclic loading. This is a crucial factor in assessing the performance of the joints. The strength degradation of the specimen can be expressed as the ratio of the second cycle to the first cycle at the same displacement [36].

As shown in Fig. 13, the strength degradation curves of each specimen under both loading directions were symmetrical, and the strength degradation ratio at the beginning of the loading exceeded 0.95. Meanwhile, as shown in the load–displacement curves, the coincidence degree of two hysteresis loops was relatively high within the same load amplitude. This indicates that the strength of the component is stable without significant strength degradation at the beginning of the loading process. As the external load continued to increase, each specimen was damaged to varying degrees. Owing to the decrease in bearing capacity caused by the damage, the strength of the joints began to degrade significantly. However, the strength degradation ratio of each specimen remained stable before failure. This shows that the strength degradation of the TWBB joint and the full-bolt joint under the action of the earthquake is relatively small. Thus, the specimens can have a stable bearing capacity. It should be noted that the strength degradation ratios of JD5 in the positive loading direction and JD1 in the negative loading direction are less than 0.75 owing to the sudden fracture at the flange. This does not satisfy the requirements of the relevant standards [37].

4.2 Stiffness Degradation

The secant stiffness $K_i$ is used to analyze the stiffness degradation of the joint specimen, which is defined by Eq. (1):

$$K_i = \frac{+F_i + -F_i}{+\Delta_i + -\Delta_i}$$

where $i$ is the cycle stage, $K_i$ is the secant stiffness under the $i$th stage loading, $+F_i$ and $-F_i$ are the maximum loads of the $i$th both direction, $+\Delta_i$ and $-\Delta_i$ are the maximum displacement of the $i$th two-cycle direction.

As shown in Fig. 14, the overall stiffness degradation curves of all specimens are stable, and their degradation tendencies are almost identical. The stiffness degradation of the specimen JD1, JD3, and JD4 is very close. This indicates that the two connection types (through-diaphragm or outer rib annular plate interior) have no significant impact on the stiffness degradation behavior of the joints.

Specimen JD2 has the smallest secant stiffness of all specimens at each loading amplitude. This indicates that the concrete-filled in the rectangular steel tube has a greater effect on the stiffness of the specimen, which is conducive to enhancing the joint stiffness. However, the degradation curve of JD2 is flatter than that of other specimens. This is more conducive to the generation of plastic hinges on beams.
The initial stiffness of JD6 is lower than that of JD1, JD3, JD4, and JD5 because the bolted connection will produce bolt slippage. This indicates that the TWBB connection is more conducive to enhancing the stiffness of the joint. The initial stiffness of the specimen JD5 is higher than that of other specimens. This means that increasing the steel beam section can effectively enhance the initial stiffness of the joint.

4.3 Ductility

Ductility is the ability of the specimen to be subjected to inelastic deformation without a significant decrease in bearing capacity. Ductility is calculated using the ratio of the failure displacement $\Delta_u$ to the yield displacement $\Delta_y$ in the skeleton curve and is expressed as follows:

$$\mu = \frac{\Delta_u}{\Delta_y}$$

where $\Delta_y$ denotes the displacement corresponding to the yield strength, and $\Delta_u$ denotes the displacement at which the load after the peak declines to 80% of the ultimate value. The characteristics of all specimens are listed in Table 4, where the tangential stiffness method was used to obtain the yield load and yield displacement, as in Fig. 15.

Owing to the displacement error of the actuator, the ductility of all specimens under positive loading is greater than that under negative loading. The average ductility of specimen JD2 without concrete is 6% higher than that of JD1 with C45 concrete. This could be because the core region of JD2 is not restrained by concrete and thus has higher shear deformation and better plastic deformation capacities. Notably, there was a significant difference between the positive and negative loads of JD2. This is probably because of the early appearance of large cracks in the top flange of the left beam. This results in a lower ductility for negative loads because of the lower failure displacement. The ductility of JD3 is 8% less than that of JD1. This is probably because of the significant increase in the stiffness of the joint core region of JD3 after increasing the thickness of the stiff-
ener, which leads to premature damage at the weld seam. The average ductility of the outer rib annular plate joint in JD4 is 15% lower than that of the TWBB joint in JD1. This indicates that the connection type with upper outer rib annular plate and lower through-diaphragm has good ductility. For JD5 (larger beam cross-sectional size), the ductility is significantly lower than that of the other members. This is because the significant increase in the ultimate load value further damaged the defects in the members caused by welding defects and stress concentrations. The ductility of the full-bolt joint in JD6 is superior to other specimens. This may be because of the apparent sliding of the beam flange bolts in the full-bolt joint, thereby causing an increase in the loading displacement. Although the yield point appears later, the failure displacement is larger. Hence, specimen JD6 shows good ductility.

4.4 Energy Dissipation Capacity

The energy dissipation capacity reflects the deformation capacity of a connection, and it is crucial to minimize the energy transmitted to other structural members during cyclic loading. Energy dissipation capacity is expressed by the equivalent viscous damping ratio $h_e$ that is defined in Eq. (3) [38]. As shown in Fig. 16, $S_{ABCD}$ is the area enclosed by the hysteresis loop, and $S_{ODF+OEB}$ is the summation of the triangle areas ODF and OEB.

$$h_e = \frac{S_{ABCD}}{2\pi \cdot (S_{ODF} + S_{OEB})} \quad (3)$$

As can be observed from Fig. 17, the cumulative energy dissipation capacity of each specimen before yielding was small, but it increased rapidly after yielding. Moreover, the growth rate was primarily the same. When the specimens were damaged, JD2 and JD6 showed larger displacements owing to the absence of concrete and bolt slippage, thereby making their cumulative energy dissipation two times larger than that of JD3 and JD5. The curves of JD1 and JD4 were very close at the time of damage, and both specimens exhibited better energy dissipation capacities.

It can be observed from Fig. 18 that the equivalent viscous damping ratios of all specimens continuously increased with the increase in the number of cycles, except JD6. The equivalent viscous damping ratio of JD6 did not increase significantly in the early stage and decreased earlier than that of other specimens in the second-half stage. This indicates that the energy dissipation capacity of JD6 (full-bolt connection) was poor, and this can be attributed to the steel beam bottom flange of JD6 buckling prematurely because of bolt slippage. The equivalent viscous damping ratios of the through-diaphragm joint specimens JD1, JD2, JD3, and JD5 are higher than that of the outer rib annular plate joint JD4, indicating that the former specimens have better energy dissipation capacities. Therefore, compared with connection type II, connection type I (through-diaphragm and outer rib annular plate interior connection) shows a more considerable energy dissipation capacity. Owing to the large joint deformation of JD2, the maximum equivalent viscous damping ratio is large. However, considering the low bearing capacity of JD2, concrete of suitable strength grade should be poured to achieve better seismic performance in practical engineering applications. The equivalent viscous damping ratios of JD3 and JD5 are slightly higher than that of JD1. This demonstrates that JD3 and JD5 enhance the joint stiffness by increasing the thickness of the stiffener and cross-sectional size of the beam, thereby improving the energy dissipation abilities. JD6 exhibits a low equivalent damping ratio in Fig. 18 because of the bolt slip. The bottom flange of the steel beam buckled prematurely, so the equivalent damping ratio increased insignificantly in the early stage and decreased prematurely in the later stage. The displacement and load of JD6 are larger than that of other specimens. However, this resulted in lower values when we used Eq. (3) to calculate the equivalent damping ratio of the full-bolt joint.
4.5 Moment–Rotation Curves

The moment–rotation response is an essential method for determining the behavior of the joint between the concrete-filled steel tubular columns and H-beams. The moment–rotation hysteresis curve is denoted by the loops drawn with the bending moment and beam-column rotation angle as variables. The value of moment is taken from the load at the end of the beam multiplied by 1500 mm, which is the length of each side beam. The rotational angle is determined by the displacement at the loading point divided by the distance between the centerline of the top and bottom flanges of the beam. Panel zone deformation is not considered in this section. Therefore, the moment and rotation angle are calculated as follows:

\[ M = P \times L \]  \hspace{1cm} (4)

\[ \theta = \frac{(\delta_1 - \delta_2)}{h} \]  \hspace{1cm} (5)

where \( M \) is the bending moment subjected to the panel zone, \( P \) is the load at the end of the beam, \( L \) is the distance from the loading point to the column, \( \theta \) is the relative rotational angle of the beam and column, \( \delta_1 \) and \( \delta_2 \) are the deformation displacements of the top and bottom flanges of the steel beam with respect to the column, respectively, and \( h \) is the distance between the centerline of the top and bottom flange of the steel beam.

As shown in Fig. 19, the shuttle-shaped moment-rotation hysteresis curves of all specimens are stable and plump. The peak beam-column rotational angle of specimen JD2 is large, but the joint average maximum moment is 11.0% lower than that of JD1. This is because the JD2 steel tube does not contain concrete. Thus, the force in the panel zone was borne by the empty steel tube, which buckled earlier than the joint connecting the CFST column, thereby decreasing the joint bending moment. We found that filling concrete in the panel zone can improve the moment bearing capacity. The maximum moment and peak beam-column rotational angle of the specimen JD3 with increased stiffener thickness are similar to those of specimen JD1, indicating that increasing the stiffener thickness has little effect on the bearing and rotation capacities at the beam ends. The positive and negative maximum moments of specimen JD4 without the through-diaphragm are slightly higher (6.9%) than that of JD1. The peak beam-column rotational angles of these two specimens are almost identical. Specimen JD5 with increased beam cross-sectional size suffers from a premature fracture at the welds owing to the higher stress at the top flange welds at the end of the beam. This causes a significant increase in the bending moment. The average maximum moment of specimen JD6 reached 258.4 kN⋅m, which is 13.5% higher than 227.6 kN⋅m of specimen JD1. This shows that changing the top flange connection type has a considerable effect on the moment bearing capacity. Generally, the beam-column rotational angles of the weld–bolt joint are smaller than that of the full-bolt joint.

4.6 Moment Capacity Analysis

As the failure modes of all specimens involved damage near the weld seam, the extrusion failure of the plates and the shear damage of the bolts were not considered in the theoretical calculations. Therefore, the force on the joint specimens was simplified to be bending at the flange and shearing at the web while calculating the flexural bearing capacity. The maximum moment that the joint specimens can withstand is given using Eq. (6):

\[ M_u = f_u b t_1 (h - t_1) \]  \hspace{1cm} (6)

where \( b \) is the width of the flange, \( t_1 \) is the thickness of the flange, \( f_u \) is the ultimate tensile strength of the steel, and \( h \) is the height of the beam.

The strength of the steel was determined using the material properties presented in Table 2. According to Eq. (6), the maximum moment capacity of each specimen can be derived, as summarized in Table 5. The estimated values of yield moment \( M_y \) and maximum moment \( M_p \), according to the provisions in the code for design of composite structures JGJ 138-2016 [39] and the code for design of steel structures GB 50017-2017 [40], are also given in Table 5.

The difference in the degree of deviation between the theoretical and test results of the maximum moment capacity is probably due to the influence of different stiffener thicknesses and configuration forms. For the increased beam section in JD5, the top flange weld at the beam end was affected by the welding quality and stress concentration. The premature fracture of the weld seam occurred because of the significant increase in load at the beam end, which made the test value less than the theoretical value. Except for JD2 and JD5,
the ratios of tested to theoretical values of other specimens ranged from 1.06 to 1.19. Overall, the reliability of the theoretical calculations was demonstrated. Furthermore, the weld seam in the joint determined the ultimate strength of the joint since the maximum moment of the weld in each specimen was less than the maximum moment of the steel beam and the yielding moment of the column.

Eurocode 3 classified beam-column joints into three types: hinge, rigid, and semi-rigid, and gave the corresponding classification and calculation methods [41]. The rotational stiffness, $S_{\text{ini}}$, of each joint was calculated according to the regulations in Eurocode 3, as shown in Table 6. The results showed that each joint was a semi-rigid joint as the rotational stiffness in the range of $\text{EI}_b/2L_b$ and $25\text{EI}_b/L_b$. 

Fig. 19 Moment–rotation curves of all specimens: a JD1, b JD2, c JD3, d JD4, e JD5, and f JD6
Table 5  Moment capacity

| Specimen | $M_u$ (kN·m) | $M_{u_{test}}$ (kN·m) | $M_{u_{test}}/M_u$ | $M_{by}$ (kN·m) | $M_{bp}$ (kN·m) | $M_{cy}$ (kN·m) | $M_{cp}$ (kN·m) |
|----------|--------------|-----------------|------------------|-----------------|-----------------|-----------------|-----------------|
| JD1      | 204.28       | 227.59          | 1.11             | 180.28          | 261.32          | 433.67          | 518.61          |
| JD2      | 204.28       | 202.46          | 0.99             | 180.28          | 261.32          | 291.43          | 368.33          |
| JD3      | 204.28       | 216.26          | 1.06             | 180.28          | 261.32          | 433.67          | 518.61          |
| JD4      | 204.28       | 243.33          | 1.19             | 180.28          | 261.32          | 433.67          | 518.61          |
| JD5      | 348.48       | 314.89          | 0.90             | 280.32          | 420.48          | 433.67          | 518.61          |
| JD6      | 204.28       | 258.41          | 1.26             | 180.28          | 261.32          | 433.67          | 518.61          |

Table 6  Rotational stiffness of each specimen

| Specimen | $EI_b$ (N·mm²) | $L_b$ (mm) | $EI_b/2L_b$ (N·mm) | $25EI_b/L_b$ (N·mm) | $S_{j,ini}$ (N·mm) |
|----------|----------------|------------|--------------------|---------------------|-------------------|
| JD1      | $1.51 \times 10^{13}$ | 1375       | $5.50 \times 10^9$ | $2.75 \times 10^{11}$ | $3.71 \times 10^{10}$ |
| JD2      | $1.51 \times 10^{13}$ | 1375       | $5.50 \times 10^9$ | $2.75 \times 10^{11}$ | $4.89 \times 10^{10}$ |
| JD3      | $1.51 \times 10^{13}$ | 1375       | $5.50 \times 10^9$ | $2.75 \times 10^{11}$ | $3.52 \times 10^{10}$ |
| JD4      | $1.51 \times 10^{13}$ | 1375       | $5.50 \times 10^9$ | $2.75 \times 10^{11}$ | $3.67 \times 10^{10}$ |
| JD5      | $2.39 \times 10^{13}$ | 1375       | $8.69 \times 10^9$ | $4.34 \times 10^{11}$ | $4.44 \times 10^{10}$ |
| JD6      | $1.51 \times 10^{13}$ | 1375       | $5.50 \times 10^9$ | $2.75 \times 10^{11}$ | $4.48 \times 10^{10}$ |

4.7 Strain Distribution

Owing to the space limitation, only the outer rib annular plate and lower diaphragm were selected to discuss the strain distribution for ease in presenting. Figure 20 shows the strain curves at the key measuring points of the aforementioned two parts of specimens JD1, JD2, and JD6. Some data are abnormal in these curves because of the damaged strain gauge.

According to the material property test results, the yield strain of the outer rib annular plate was approximately 1664 $\mu$ε, indicating that the specimen enters the plastic stage when the strain reaches this value. From these curves, it can be observed that the data detected by the three measuring points, i.e., 16, 19, and 20, located near the weld exceeded the yield strain. This is consistent with the phenomenon of weld fracture observed during the test. Specimen JD6 has a smaller strain compared to JD1 and JD2. This is because the top flange of JD6 was connected using bolts, which share more stress, causing less deformation. By observing the data of individual strain measuring points from the beam length direction, the measurement points 16, 17, 19, and 20 near the annular plate-column welding position have larger strain values, which indicate that there is stress concentration at the weld. The value of each strain measuring point in the stress distribution direction increases gradually, indicating that the stress is mainly transmitted from the beam end to the column.

In the beam width direction, the strain value at the center of the annular plate was larger than that at the side. This concluded that the stress is mainly concentrated at the center of the outer annular plate.

The yield strain of the lower diaphragm measured using the material property test was 1664 $\mu$ε. As seen in Fig. 20d–f, the strain values of the measuring points 23 and 26 near the column increased and reached the yield strain. This shows that the strain at the diaphragm center near the column, where the stress concentration occurs, is relatively large. The strain values of the two measuring points, 25 and 28, at the far end are very small with small fluctuations. Additionally, the strain at these two measuring points is lower than the yield value. This indicates that the closer the positions are to the area where the diaphragm is welded to the column, the greater the contribution to the load capacity of the diaphragm and more significant the strain changes. The strain gauges 26, 27, and 28 near the diaphragm center have a significantly larger strain value than strain gauges 23, 24, and 25 near the outside. This indicates that the contribution of the position near the diaphragm center to the bearing capacity of the diaphragm is significantly greater than that of the position closer to the outside. Overall, the ultimate strain of the material ranged between 2600 and 2700 $\mu$ε, and the ultimate strain at each point of the test is more than 3000 $\mu$ε. This means that the full strength of the material can be exploited. Thus, these test data could provide some guidance on the practicality of the moment capacity design methods.

5 Conclusion

To study the seismic behavior of TWBB joints, five full-scale specimens with top-flange-welded-bottom-flange-bolted connection and one full-bolted connection specimen were tested under cyclic loading. Based on the test results, the following conclusions were summarized.
Before the specimens suffered any damage, except JD6, the top flange of the steel beam showed a local buckling phenomenon, and the beam end reached the ultimate state with the apparent plastic hinge. In the case of specimen JD6, the damage was caused by stress concentration at the through-diaphragm weld. Generally, there was no significant

Fig. 20 Strain distribution for specimens JD1, JD2, and JD6; a strain at outer rib annular plate of specimen JD1; b strain at the outer rib annular plate of specimen JD2; c strain at the outer rib annular plate of specimen JD6; d strain at the through-diaphragm of specimen JD1; e strain at the through-diaphragm of specimen JD2; and f strain at the through-diaphragm of specimen JD6.
deformation and damage in the panel zone and CFST column during the entire loading process. This reflected the criterion of strong column-weak beam.

Based on the obtained test results, the proposed TWBB joints demonstrated stable hysteresis response, high energy dissipation and rotation capacity, and considerable stiffness and strength. The novel joint proposed in this study exhibited moderate ductility according to ASCE/SEI. Moreover, the load bearing capacity was similar to that of conventional full-bolted joints. The difference value of the load bearing capacity was similar to that of conventional full-bolted joints. The difference value of the load bearing capacity was between $1.5$ and $4.5\%$. Compared with specimen JD6, the equivalent viscous damping ratio of other specimens using the hybrid connection was enhanced by at least $30\%$.

The effect of concrete on the performance of this new joint was also analyzed. We found that the steel tube column without concrete caused more extensive deformation to the diaphragm and annular plate, thereby lowering the load capacity of BWTT joints. However, this resulted in better rotational capacity. Generally, the proposed joint members exhibited good damage stability, ductility, and energy dissipation with or without concrete filling.

Compared with other specimens, the specimen JD4 with both top and bottom flanges connected to the outer rib annular plates had significantly low ductility and energy dissipation capacity. Although the increase in beam height (JD5) can improve the bearing capacity, the strength and stiffness degrade faster. When the specimen was damaged, the bearing capacity decreased abruptly, resulting in a small displacement and low ductility. Additionally, changing the stiffener thickness and using bolted connections have a greater effect on the failure mode of the joints. This could be owing to the unreasonable stiffness distribution due to structural design problems.

Adopting the TWBB joint to connect CFST columns and H-steel beams and adding concrete with suitable strength in the steel tube column can help in obtaining better seismic performance and provide a reference for the connection design.

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Data Availability The data used to support the findings of this study are included within the article.

Declarations

Conflict of interest The authors declare that they have no conflict of interest.

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