Research Article

Cyclic Testing of Bolt-Weld Joints Reinforced by Sleeves Connecting Circular CFST Columns to Steel Beams

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This study examined the design of joints reinforced by sleeves for connecting circular concrete-filled steel tube columns to steel beams. Six half-scale specimens, including four bolt-weld joints reinforced by sleeves and two bolt and stiffened end-plate joints, were designed and tested under cyclic loading to evaluate the seismic behavior of these joints. The joint construction and beam-column stiffness ratio were taken as the main parameters in the tests. The seismic behaviors, including the failure modes, hysteretic curves, ductility, strength and stiffness degradation, and energy dissipation, were investigated. The experimental results showed that no obvious bolt loosening, fracture, or widespread weld cracking appeared in the joints reinforced by sleeves. Furthermore, the joint strength and stiffness were markedly increased by the sleeves in the joint core area. Overall, most specimens exhibited full hysteretic loops and excellent ductilities, the equivalent viscous damping coefficients were 0.263–0.532, and the ductility coefficients were 1.77–3.42. The interstory drift ratios satisfied the requirements specified by technical regulations. The connections of these types exhibit favorable energy dissipations and can be effectively utilized for building construction in earthquake-prone areas. This research should contribute to the future engineering applications of concrete-filled steel tube to composite structure.

1. Introduction

Composite structures such as concrete-filled steel tube (CFST) have been widely used in civil engineering due to their excellent performance and have been extensively studied in recent years. Sevim et al. [1] studied the structural response of full-scaled rectangular columns under both vertical and lateral loads using numerical methods. The results demonstrated that the nonlinear models revealed more accurate results than those of linear models and that composite columns provided more safety and ductility compared to reinforced concrete columns. Essopjee and Dundu [2] conducted tests on 32 concrete-filled double-skin circular tube (CFDSCT) columns. The results showed CFDSCTs of 1 m lengths failed by yielding of the steel tubes and the other CFDSCTs failed by overall buckling due to their large slenderness. In addition, new formulas were developed to predict the results of the strengths of the CFDSCTs and the predicted result agrees well. Robinson and Melby [3] investigated the performance of short-span concrete-filled rectangular glass fiber-reinforced polymer (GFRP) tubes with different levels of bonding between the concrete core and GFRP tube by experimental tests and finite element analysis method. Comparing the performance of each configuration showed a two fold increase in stiffness and strength as a result of bonding between the concrete core and GFRP tube. Furthermore, it was concluded that bonding of the flanges was most critical with web bonding providing only a slight increase in performance.

As the key part of the CFST structure, joints directly influence the strength, stiffness, and seismic performance of the whole structural system [4]. Therefore, research on CFST column-beam joints is important for engineering theory and practical applications. Prinz et al. [5] investigated the strengthening of bolted beam-column connections, having no column web stiffeners, using more than one bolt on either side of the column web through the test of six full-scale bolted beam-column connections. The results indicated that
closer inner-bolt spacing relative to the column web increased connection moment capacity but decreased rotation capacity. Zeng et al. [6] conducted experiments on five half-scale interior joints to investigate the internal mechanisms and seismic performance of high-strength concrete-encased composite structure joints. The results showed high-strength concrete increased the joint strength and had relatively little effect on the stiffness and ductility. Liao et al. [7] studied the effects of RC slabs on the seismic behavior of composite joints with concrete-encased CFST columns under cyclic loading. The results revealed that the presence of an RC slab could significantly enhance the beam strength and thus shift the joint failure mode from beam failure to joint and column failure. Zhang and Zhang [8] proposed a new type of joint of the CFDST column to steel beam and investigated seismic failure. The results indicated that the joint had the high bearing performances of the new type of joint by experimental tests.

A circular steel tube to confine internal concrete is stronger than a square steel tube, which increases the vertical bearing capacity of circular CFST columns. However, it also has a more complex connection with a steel beam than a square CFST column; this has limited the use of circular CFST columns; thus, they have relatively fewer joint forms. Seismic behavior tests of the core bolt and stiffened end-plate joints of square CFST columns and steel beams [9–12] have shown that these joints have excellent overall seismic performance, but the core bolts easily loosen under seismic loads and are prone to brittle failure when suddenly overloaded. Other studies [13–18] have shown that, because of the complex stress state of the bolt and steel tube when a similar joint is used for circular CFST columns and steel beams in seismic tests, the bolt breakage, pull-out, buckling, and tearing of the column wall easily occur under repeated seismic loads; this affects the restraint of the steel tube on concrete in the joint core area. Therefore, there are potential safety hazards for this type of CFST column-beam joint.

This study examined two kinds of beam-column connections reinforced by sleeves to solve the existing problems for bolt and stiffened end-plate joints connecting circular CFST columns to steel beams. The first kind is a blind bolt and weld joint reinforced by a circular sleeve: arc end plates are set around the core area of a joint and welded to each other with vertical fillet welds and then to the column wall with horizontal fillet welds. The second kind is a core bolt and weld joint reinforced by a square sleeve: square end plates are set around the core area of the CFST column and welded to each other to form a square sleeve; upper and lower cover plates are welded to the square sleeve and then welded to the column wall; high-strength concrete is filled in the space between the column and square sleeve. Structural optimization of the joints reinforced by sleeves showed that the circular or square sleeves formed by welding the outer end plates not only enhanced the stiffness of the joint core area but also made the welds and bolts in the joint core area bear the bending moment and shear force together, which enhanced the strength of the joint.

In this study, four bolt-weld joint specimens reinforced by sleeves and two bolt and stiffened end-plate joint specimens were designed and tested under cyclic loads at the top end of the column. The specific purpose of the study was to investigate the effect of the sleeves on seismic performance, including the failure modes, hysteresis curves, bearing capacity, ductility, energy dissipation, and strength and stiffness degradation. Based on these results, some useful conclusions and suggestions are proposed for the seismic design of steel-beam-to-CFST-column joints based on the results, which have significance for the application of CFST columns in composite structures.

2. Experimental Program

2.1. Design of Specimens. Six half-scale specimens (four bolt-weld joints reinforced by sleeves and two bolt and stiffened end-plate joints) were designed and tested under cyclic loading to evaluate the seismic behaviors. The dimensions of the joints are shown in Figure 1.

Table 1 presents the parameters of the specimens, which included one blind bolt and arc stiffened end-plate joint (J-1), two blind bolt and weld joints reinforced by a circular sleeve (J-2, J-3), one core bolt and square stiffened end-plate joint reinforced by a square sleeve (J-4), and two core bolt and weld joints reinforced by a square sleeve (J-5, J-6). The section of each column comprised a circular seamless steel tube filled with high-strength concrete. The steel beams of joints J-1, J-2, J-3, and J-5 had H-shaped sections with dimensions of 250 mm (height) × 125 mm (flange width) × 6 mm (web thickness) × 9 mm (flange thickness). The steel beams of specimens J-3 and J-6 were box sections where an H-shaped section was strengthened with longitudinal outward ribbed plates; the dimensions of the H-shaped section were 244 mm (height) × 175 mm (flange width) × 7 mm (web thickness) × 11 mm (flange thickness). All steel components were made from Q345B grade hot-rolled section steel. The arc end plates or square end plates of the specimen were connected to the CFST column by 10.9 grade strength and M20 size bolts. A pretension torque (410 N·m) was applied to the connecting bolts of the specimens before the test. The specific geometric sizes and construction of specimens J-1–J-6 are shown in Figures 2(a)–2(h).

2.2. Material Properties. According to GB/T 228.1-2010 [19], the mechanical properties of all steel materials were measured and are presented in Table 2. The average compressive strength of the concrete was $f_{cu} = 55$ MPa.

2.3. Cyclic Testing Apparatus. For the experiments to simulate the mechanical properties of beam-column joints under an earthquake and consider the actual strength (P)-displacement ($\Delta$) effect, the column end loading mode under a vertical load was adopted in the test. The column root was fixed to a rigid support with one-way hinges, and the top of the column was subjected to vertical load through a hydraulic jack with spherical hinges that could slide horizontally along the steel frame beam. An electrohydraulic
Table 1: Details of joint specimens.

| Specimen | Beam-to-column stiffness ratio | Axial compression ratio | End plate thickness (mm) | Weld size between end plates (mm) | Weld size between an end plate and tube (mm) |
|----------|-------------------------------|------------------------|-------------------------|-------------------------------|----------------------------------|
| J-1      | 0.48                          | 0.3                    | 16                      | —                             | —                                |
| J-2      | 0.48                          | 0.3                    | 16                      | 8                             | 4                                |
| J-3      | 0.94                          | 0.3                    | 16                      | 8                             | 4                                |
| J-4      | 0.48                          | 0.3                    | 16                      | —                             | —                                |
| J-5      | 0.48                          | 0.3                    | 16                      | 8                             | 4                                |
| J-6      | 0.94                          | 0.3                    | 16                      | 8                             | 4                                |

Figure 1: Joints (unit: mm).

Figure 2: Continued.
A servo actuator was used to load the top of the column along the horizontal direction. Two rigid links with load cells were established at the free ends of the beams, and the two ends of the links were articulated with the beam end and the foundation, respectively. The steel beams were supported on both sides by out-of-plane braces to prevent the specimens' instability during testing. The test site is shown in Figure 3.

### 2.4. Loading Procedure

First, the top of the column exerted an invariable vertical load according to the axial compression ratio. Then, under the control of displacement, a lateral cyclic loading was gradually imposed on the top side of the column. Figure 4 shows the loading history of the test specimens, i.e., step (n)-displacement (Δ) relational curve. Each lateral displacement (±6, ±12, and ±18) was executed for only one cycle at first. Subsequently, as the lateral...
displacements progressively increased (±24, ±36, ±48, ...), they were performed for three cycles until the failure of the specimen was serious or the load decreased to 85% of the ultimate strength of the specimen [20].

2.5. Layout of Instruments. As shown in Figure 5, seventeen linear variable displacement transformers (LVDT) were installed to measure deformations of the columns and beams, relative rotation of the columns and beams, and shear deformation of the joint core area. As shown in Figure 6, strain gauges were pasted on the end plates, column walls, flange, and web of the steel beams to obtain the strain distributions in the core area of the joint.

3. Results and Discussion

3.1. Failure Modes

3.1.1. Specimen Characteristics. The specimen J-1 was a blind bolt and arc stiffened end-plate joint, and the beam had lower stiffness. As shown in Figure 7, both the core zone and the steel beams of the joint were damaged. The flanges of the steel beam bent slightly on both sides (Figure 7(a)), and the bolts loosened and even fractured under cyclic loads (Figure 7(b)). There were large gaps between the arc end plates and column wall.

The specimen J-2 was a blind bolt and weld joint reinforced by a circular sleeve, and the beam had lower stiffness. As shown in Figure 8, the steel beams of the joint were damaged. The beam flanges and webs buckled, and plastic hinge failure occurred on both sides. In addition, the left flange of the beam fractured. The column and arc end plates were not damaged. No bolt shank fracture or weld cracking was observed in the joint.

The specimen J-3 was a blind bolt and weld joint reinforced by a circular sleeve, and the beam had greater stiffness. As shown in Figure 9, the column of the joint was damaged. The beam flanges and webs buckled, and plastic hinge failure occurred on both sides. The core bolts loosened under cyclic loads, and there was a large gap between the square end-plate and column wall. The square end-plates also showed bending. The column and square sleeve were not obviously damaged, and no weld cracking was observed at the joint.

The specimen J-4 was a blind bolt and square stiffened end-plate joint reinforced by a square sleeve, and the beam had lower stiffness. As shown in Figure 10, both the core zone and the steel beams of the joint were damaged. The beam flanges and webs buckled, and plastic hinge failure occurred on both sides. The core bolts loosened under cyclic loads, and there was a large gap between the square end-plate and column wall. The square end-plates were not obviously damaged, and no weld cracking was observed at the joint.

The specimen J-5 was a core bolt and weld joint reinforced by a square sleeve, and the beam had lower stiffness. As shown in Figure 11, the steel beams of the joint were damaged. The beam flanges and webs buckled, and plastic hinge failure occurred on both sides. The column, square sleeve, and square end-plates were not obviously damaged. In addition, no bolt shank fracture or weld cracking was observed at the joint.

The specimen J-6 was a core bolt and weld joint reinforced by a square sleeve, and the beam had greater stiffness. As shown in Figure 12, the column of the joint was damaged. The column showed bending (Figure 12(a)), and the steel tube buckled and fractured at the upper and lower ends of the joint core area (Figures 12(b) and 12(c)). The square sleeve and square end-plates were not obviously damaged. In addition, no bolt shank fracture or weld cracking was observed in the joint.

3.1.2. Failure Characteristics. Table 3 lists the failed components of the test joints. The bolt and stiffened end-plate joints (J-1 and J-4) failed via beam-type failure mode, and...
the bolts loosened and fractured (the bolt stress of arc end-plate joints is more unfavorable) under repeated loads. However, the bolt-weld joints reinforced by sleeves showed beam-type or column-type failure in the test, and almost no damage was observed in the core area of the joints. This can be mainly attributed to the following: (1) the circular or square sleeve formed by welding the outer end plates enhanced the stiffness of the joint core area; (2) the combination of welds and bolts in the core area of the joints greatly improved the connection between beams and columns, which made the welds and bolts in the joint core area bear the bending moment and shear force together, thus enhancing the bearing capacity of the joint.

Joints J-3 and J-6 were designed to have larger beam-column stiffness ratios and have column-type failure modes to verify the bearing capacity of the joint core area. It should be noted that in the structural design, the beam-column stiffness ratio should be controlled to avoid column end failure of the joints. The horizontal welds of J-3 and J-6 at the end plates and column or cover plate were slightly damaged, which was because the welds were small in size or below the
standard quality; this criterion should be considered in design and construction.

3.2. Hysteresis Curves. The hysteretic curve reflects the seismic performance of the structure during the process of repeated loading, such as bearing capacity, deformation characteristics, stiffness degradation, and energy consumption [21]. Figure 13 shows the load-displacement hysteresis curves of the specimens in the test. Accordingly, the following conclusions can be obtained:

(1) The hysteretic curves of the specimens were shuttle-shaped or bow-shaped and indicated good energy dissipation capabilities based on their sufficiently large envelope areas.

(2) The hysteretic curves of specimens J-1 and J-4 were all pinched in the middle of the loading period. However, the pinching degree of J-1 was small, and its hysteretic loop varied from a shuttle shape to a bow shape. The pinching degree of J-4 was large, and its hysteretic loop changed from a shuttle shape to a Z shape. The main reason was the relaxation and slip of blind bolts and core bolts in the two specimens under vertical and horizontal repeated loads; their degrees of slip differed because of the differing joint construction in the core areas.

(3) The hysteretic loops of specimens J-2, J-3, J-5, and J-6 were all fully shuttle-shaped throughout the loading process. This is because the sleeves formed by welding the arc or square end-plates enhanced the stiffness of the connection between the beam and column. Meanwhile, the bolts of these joints did not slip and relax, and the welds in the joint core area did not crack. Thus, the hysteretic curves of these joints did not shrink.

(4) The hysteretic curves of specimen J-2 and J-5 were similar. Because the two joints had the same cross sections for their columns and beams and their beam-column stiffness ratio was relatively low, they failed by plastic hinge failure at the beam ends. The hysteretic behavior of the joints was mainly affected by failure of the steel beams, and other factors had a relatively small influence.

(5) The hysteretic loops of specimens J-3 and J-6 were not full. This is mainly because they had greater beam-column stiffness ratios than the other four specimens, and the maximum load corresponding to the same loading displacement was larger than that to the others. The hysteretic curves of J-3 and J-6 had relatively poor plumpness, smaller straight sections, and almost no descending sections, but they showed greater rigidity and bearing capacity.

3.3. Skeleton Curves. The skeleton curve is an envelope curve that is obtained by successively connecting the extreme points of loads in the same direction (tension or compression) on the hysteretic curve. The skeleton curve represents the locus of the maximum horizontal force peak value achieved by each cyclic loading, which reflects the different stages and characteristics (e.g., strength, stiffness,
ductility, energy dissipation, and collapse resistance) of the component under stress and deformation. The skeleton curve of each specimen in the test is shown in Figure 14. The following conclusions were obtained:

1. The joints all include three deformation stages: elasticity, elastic-plasticity, and damage. The skeleton curves of joints J-1, J-2, J-4, and J-5 were similar. This is mainly because these four joints had the same cross sections for their columns and beams, and their beam-column stiffness ratios were relatively low, so their failure modes were mainly of the plastic hinge type at the beam end.

Table 3: Failure components of the joints.

| Component | J-1 | J-2 | J-3 | J-4 | J-5 | J-6 |
|-----------|-----|-----|-----|-----|-----|-----|
| Column    | No  | No  | Failure | No  | No  | Failure |
| Beam      | Failure | Failure | No  | Failure | Failure | No  |
| End plate | No  | No  | Failure | No  | No  | Failure |
| Bolt      | Failure | No  | No  | Failure | No  | No  |
| Weld      | —   | No  | Failure | —   | No  | No  |

Figure 12: Failure modes of J-6: (a) column bending; (b) steel tube buckling; (c) steel tube fracture.

Figure 13: Load-displacement hysteretic curves of the joints: (a) J-1; (b) J-2; (c) J-3; (d) J-4; (e) J-5; (f) J-6.
(2) The ultimate strength of specimens J-3 and J-6 was almost twice that of specimens J-1 and J-4. This shows that the sleeves formed by welding the arc end-plates or square end-plates made the welds and bolts in the joint core area bear the bending moment and shear force together, which enhanced the joint strength.

(3) Specimen J-6 had a greater ultimate strength than specimen J-3. This indicates that the core bolt and square end-plate joint reinforced by the square sleeve had a greater strengthening effect than the blind bolt and arc end-plate joint reinforced by the circular sleeve.

3.4. Strength and Ductility. The key points of the $P$-$\Delta$ skeleton curves were obtained according to JGJ/T 101-2015 [20], as shown in Figure 15, where $P_y$, $\Delta_y$, $P_m$, $\Delta_m$, $P_u$, and $\Delta_u$ represent the yield strength and displacement, ultimate strength and displacement, and damage strength and displacement, respectively, of the joint. The specific values of the key points of the test joints are listed in Table 4. Although specimens J-1, J-2, J-4, and J-5 had different joint constructions in the core area, they were all designed to have strong columns and weak beams, and their beam sections were the same in form and size. Their failure modes were all plastic hinge failure at the beam end, so the yield strength and displacement, ultimate strength and displacement, and damage strength and displacement were very similar. Except for the damage displacement of J-3, which was relatively small, the other strengths and corresponding displacements of specimens J-3 and J-6 were much larger than those of the other specimens. This reflects the difference in construction of the joints and their failure modes.

The interstory drift ratio ($\theta$) is defined as $\theta = \arctan(\Delta_H/H)$, where $H$ is the frame height and $\Delta_H$ is the lateral displacement of the column end. The ductility coefficient is defined as $\mu_\theta = \theta_u/\theta_y$, where $\theta_y$ and $\theta_u$ represent the drift ratios of the joints at yielding and failure, respectively. The detailed results for the yield drift ratio $\theta_y$, failure drift ratio $\theta_u$, and ductility coefficient $\mu_\theta$ of the specimens are presented in Table 4. In accordance with CECS 159:2004 [22], for mid-rise and high-rise CFST frames, the drift ratio limit value $[\theta_y]$ is 1/250 for the elastic stage and $[\theta_u]$ is 1/50 for the elastic-plastic stage. In this test, the joints obtained were $\theta_y = (3.15 - 6.09)[\theta_y]$ and $\theta_u = (1.8 - 2.4)[\theta_u]$. In addition, according to the Chinese standard [23], the ductility coefficients of all joints except J-3 met or exceeded the limit value of 2.0 for a steel reinforced concrete composite structure.

According to Table 4, specimens J-2 and J-5 had slightly lower ductilities than specimens J-1 and J-4. Although the four joints failed by plastic hinge failure at the beam end, the ultimate strengths of J-1 and J-4 decreased relatively slowly because of bolt loosening, which enhanced the deformation ability. Although the bolt-weld joints J-3 and J-6 both failed by column bending, the weld cracking in the core area of joint J-3 led to lower constraints when damaged. Moreover, the two joints were constructed differently, which led to a large difference in ductility.

3.5. Stiffness Degradation. The average loop stiffness $K_j$ is calculated according to the following formula to evaluate the stiffness degradation of the joints under horizontal cyclic load:

$$K_j = \frac{\sum_{i=1}^{n} p_j^i}{\sum_{i=1}^{n} \Delta_j^i},$$

where $p_j^i$ and $\Delta_j^i$ are the peak load and corresponding displacement of the $i$th cycle when the displacement control reaches $j$ and $n$ is the number of cycles.

As shown in Figure 16, all specimens showed similar trends for the stiffness degradation. Before yielding, the stiffness degradation was faster; after yielding, it was gentler, and there was no large or sudden change in stiffness. Specimens J-3 and J-6 had higher stiffness in each stage than
the other four specimens because their beams had high stiffness.

3.6. Strength Degradation. The total strength degradation coefficient $\lambda_i$ is used to evaluate the strength degradation of the joints under horizontal cyclic load, and it is estimated as the ratio of the maximum bearing capacity at each step loading displacement to the maximum bearing capacity during the entire loading process of the joints. As shown in Figure 17, the strengths of all joints increased steadily before reaching peak strength, and the trends were very similar. This indicates that the specimens had good elastic-plastic properties. After the peak strength was reached, the strengths of J-1, J-2, J-4, and J-5 decreased slowly, and they showed obvious plastic failure characteristics. Because of their sudden destruction, the strengths of J-3 and J-6 did not decline, and the specimens showed obvious brittle failure characteristics.

3.7. Energy Dissipation. The equivalent damping coefficient ($h_e$) is defined as follows:

$$h_e = \frac{S_{BEF} + S_{DEF}}{2\pi(S_{AOB} + S_{COD})}. \quad (2)$$

In Figure 18, $S_{BEF}$ and $S_{DEF}$ are the areas of the hysteresis loops of BEF and DEF, respectively, and $S_{AOB}$ and $S_{COD}$ are the triangle areas of AOB and COD, respectively.

As shown in Table 4, the $h_e$ values of joints ranged from 0.263 to 0.532, which are similar to the value for a steel-reinforced concrete joint (about 0.3) [24]. Therefore, the proposed joint types in this study are reasonable for construction because of their favorable energy dissipations.

Table 4: Test results.

| Specimen | $P_y$ (kN) | $\Delta_y$ (mm) | $P_m$ (kN) | $\Delta_m$ (mm) | $P_u$ (kN) | $\Delta_u$ (mm) | $\theta_y$ (rad) | $\theta_u$ (rad) | $\mu$ | $h_e$ |
|----------|------------|----------------|------------|----------------|------------|----------------|----------------|----------------|------|------|
| J-1 (+)  | 226.95     | 22.08          | 283.76     | 48.01          | 241.20     | 89.44          | 0.0110         | 0.0447         | 4.06 | 0.334|
| J-1 (−)  | 219.10     | 30.17          | 257.67     | 77.48          | 237.40     | 70.33          | 0.0151         | 0.0480         | 3.18 | 0.532|
| J-2 (+)  | 234.76     | 22.18          | 286.44     | 54.17          | 243.47     | 72.05          | 0.0111         | 0.0379         | 3.42 | 0.264|
| J-2 (−)  | 257.83     | 28.52          | 295.03     | 58.83          | 250.78     | 77.48          | 0.0143         | 0.0387         | 2.71 | 0.319|
| J-3 (+)  | 375.15     | 27.70          | 474.72     | 60.02          | 462.19     | 72.05          | 0.0138         | 0.0360         | 2.61 | 0.517|
| J-3 (−)  | 371.29     | 40.70          | 420.84     | 60.01          | 419.92     | 72.03          | 0.0203         | 0.0360         | 1.77 | 0.263|
| J-4 (+)  | 196.94     | 20.95          | 264.39     | 60.02          | 221.48     | 83.96          | 0.0105         | 0.0420         | 4.00 | 0.319|
| J-4 (−)  | 233.31     | 30.13          | 281.35     | 60.02          | 241.38     | 84.02          | 0.0151         | 0.0420         | 2.78 | 0.319|
| J-5 (+)  | 238.71     | 23.25          | 289.72     | 59.99          | 246.26     | 78.59          | 0.0116         | 0.0393         | 3.39 | 0.263|
| J-5 (−)  | 257.88     | 29.90          | 304.22     | 60.01          | 258.81     | 79.44          | 0.0149         | 0.0397         | 2.66 | 0.319|
| J-6 (+)  | 390.95     | 29.59          | 509.72     | 72.03          | 497.21     | 96.01          | 0.0148         | 0.0480         | 3.24 | 0.263|
| J-6 (−)  | 386.57     | 38.69          | 503.53     | 83.99          | 476.16     | 96.02          | 0.0193         | 0.0480         | 2.49 | 0.263|
stiffened end-plate joints. This is because the energy dissipation capacity of bolt joints J-1 and J-4 decreased with the bolt loosening and fractured under cyclic loads of joints in the failure stage. However, the bolt and weld joints reinforced by sleeves exhibited excellent seismic performance for energy dissipation.

4. Conclusions

Experiments were performed to examine the seismic behavior of bolt-weld joints reinforced by sleeves connecting CFST columns to beams. The conclusions are summarized as follows:

(1) The failure mode of the bolt and weld joints reinforced by sleeves is mainly determined by the beam-column stiffness ratio. The failure mode is plastic hinge failure at the beam end when the ratio is small and is bending failure at the column end when it is large; no bolt loosening or fracture and widespread weld cracking were observed in the joint core area. This indicates that the bolt-weld joints reinforced by sleeves are a reasonable construction and meets the seismic design requirement of a strong joint and weak components.

(2) The sleeves in the joint core area enhance the joint strength and stiffness of the connection between the beam and column. They prevent weld cracking and bulging of the column wall in the core area as the welded joint becomes damaged and prevent the defects of bolt relaxation and even brittle fracture when the bolt and stiffened end-plate joints are damaged.

(3) Compared with the bolt and stiffened end-plate joints, the strength and stiffness of the bolt-weld joints reinforced by sleeves were markedly increased by the sleeves in the joint core area. The equivalent viscous damping coefficients of the new types of joints were 0.263–0.532, and the ductility coefficients were 1.77–3.42. Overall, most specimens exhibited full hysteresis loops and excellent ductilities. The interstory drift ratios satisfied the requirements specified by technical regulations. These types of connections exhibit favorable energy dissipations and can be effectively utilized for building construction in earthquake-prone areas.

Therefore, the research results of this paper not only contribute to the design of new joints but also to reinforcement of existing bolt and stiffened end-plate joints. In addition, owing to the limited number of test specimens, the influence of different parameters on seismic performance of these new types of joint needs further study. Therefore, the authors plan to study the mechanical properties of these joints by further finite element analyses.

Data Availability

The figures presenting the test data analysis were all drawn in Origin 8.0. The raw data are presented in the figures, including the hysteresis curves, skeleton curves, and stiffness degradation. Some data used in the calculation process are based on standards listed in the references, which should be simple and clear for readers with a civil engineering background.

Conflicts of Interest

The authors declare that there are no conflicts of interest related to this study.

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