

Research Article

Internal Force Transfer Mechanism and Bearing Capacity of Vertical Stiffener Joints in CFDST Structures

Yufen Zhang ^(b),¹ Dongfang Zhang ^(b),² and Krushar Demoha³

¹School of Civil and Transportation Engineering, Hebei University of Technology, Tianjin 300401, China
 ²School of Civil Engineering, Chang'an University, Xi'an 710061, Shaanxi Province, China
 ³Department of Mechanical Engineering, University of Auckland, 1142 Auckland, New Zealand

Correspondence should be addressed to Yufen Zhang; yufenzh@gmail.com

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This paper firstly studied the internal force transfer mechanism of vertical stiffener joints in concrete-filled double steel tubular (CFDST) frame structures on the basis of finite element modeling (FEM). Analytical models of shear force and bending moment were established through the appropriate material constitutive equations and equilibrium theory. Then, the proposed models were used to predict and evaluate the shear and bending resistance of the vertical stiffener joint. Six joint specimens were tested to verify the rationality of the theoretical models, and the design suggestions for construction were subsequently discussed. The analysis indicated that the vertical stiffener together with the anchorage web played a dominated role in the internal force transfer mechanism. The computed bending resistance obtained by the tension model agreed well with the measured experimental data, and the shear resistance in the panel zone was sufficient to guarantee the ductile failure in the test. The vertical stiffener determined the plastic hinge so as to ensure the strong connection between the CFDST column and the steel beam. The ribbed anchorage web was an effective way of increasing the shear and bending resistance.

1. Introduction

Concrete-filled steel tubular (CFST) column combines the advantages of steel and concrete and has been actively researched and widely implemented [1–5]. In these several years, concrete-filled double steel tubular (CFDST) column has been applied as the section consisting of an outer steel tube and an inner steel tube, both filled with concrete. According to the reported research [6–10], the CFDST column exhibits notable constructional and mechanical benefits than the traditional CFST column, such as higher strength, higher ductility, better fireproof property, and stronger collapse resistance. It was firstly applied in the construction of the city hall in Wuppertal, Germany.

The beam-column joint is the key element transferring internal force and undertaking integrity in the CFDST frame structure. Among the several joint types for CFDST columns, which were developed based on the traditional joints in the CFST structure [11–13], vertical stiffener joints have

the simple construction details and the reasonable loadtransferring mechanism. Vertical stiffener joints between square CFST columns and steel beams were firstly studied by Chen and Miao [14] and Miao [15]. Recently, vertical stiffener joints for L-shaped CFST columns [16] and connection with ring stiffeners for gangue concrete-filled steel tube columns [17] were conducted under the low cyclic loading test. The results showed that stiffeners were the main force-transferring member to fulfill the beam-column transmission system. Since the CFDST column has double steel tubes, the anchorage web between them is embedded to improve the behaviors of the vertical stiffener joint. Therefore, the vertical stiffener joint in the CFDST structure has the different configurations, which will provide different internal force-transferring mechanisms.

In these several years, some experimental research and numerical analysis have studied the force transfer mechanism to analyze the behavior of beam-column connections or other relational structures [18–23]. Joint strength calculation mainly includes bending strength and shear strength. And the mechanical characteristics and transfer mechanism need to be analyzed to meet the seismic requirements of "strong connection" philosophy. The joint assemblies should be designed to avoid the use of continuity plates and supplementary web plates, and with welded stiffeners, the global structural response can be significantly modified because of the stiffness improvement of the structural system [24]. Thus, the main objective of this research is to explore the internal force transfer mechanism and evaluate the bearing capacity of the vertical stiffener joints between CFDST columns and steel beams. On the basis of stress distribution in finite element modeling (FEM), analytical models of shear force and bending moment are presented to effectively assess the internal force transfer mechanism, and then the shear resistance of the cracked vertical stiffener joint and the ultimate bending resistance are formulated with consideration of appropriate material constitutive equations and equilibrium theory. The interest of the experiment investigation is focused on the validation of the proposed model and the solution approach. All the research results can provide researchers and engineers with a better knowledge concerning the mechanical performance of the vertical stiffener joint in the CFDST structure.

2. Description of Vertical Stiffener Joints

Figure 1 shows the details of vertical stiffener joints between a CFDST column and two adjacent H-shaped steel beams. The anchorage web welded on the inner circular tube connects with the beam web using junction plates and bolts, as shown in the ichnography. The vertical stiffener cohered with the outer square tube in the joint core, and its overhangs were groove welded to the horizontal end plate at the right angle. The arc section is used to reduce the width of the other part of the end plate, and then it connects with the beam flange by butt weld, as shown in Figure 1(a). Hence, the embedded anchorage web in the concrete between double tubes can improve the continuity and integrality of the joint and cooperatively transfer the internal force with the vertical stiffener. Since the embedded part between double steel tubes of the anchorage web is higher than the beam, ribbed joints can be designed, as shown in Figure 1(c). The presence of ribs may influence the stress distribution in the beam-column connection, the length of the internal force lever arm, and the load-carrying capacity of the joint [25].

3. Finite Element Model

ABAQUS was utilized to simulate the vertical stiffener joint in a CFDST frame structure. The material geometry and properties of the joints' components in the below-mentioned experiment were applied in the FEM. The solid elements C3D8R and structured adaptive grid method were used to model steel and concrete. A reasonable failure criterion was selected for the steel tube, steel beam, and infilled concrete, respectively. Between the steel tube and infilled concrete, hard contact was used in the normal direction, while the Mohr–Coulomb friction model with the friction coefficient 0.25 was used in the tangential direction. The FE model was established, as shown in Figure 2. The fixed axial force acted on the column top, and the cyclic loading at both the ends of the adjacent beams was synchronized in the opposite direction (pulling and pushing). Firstly, the results of the FEM were validated by the experimental results. In terms of the force-transferring path, the joint bears both bending moment and shear force. The FEM was utilized to highlight the stress distribution or force flows in the whole loading process, especially in terms of how the main connectors (vertical stiffener and anchorage web) take functions to affect joints' bearing capacity. These analyses could help us better comprehend the internal force transfer mechanisms in accordance with the visual failure deformation under the ultimate load.

4. Theoretical Analysis

4.1. Shear Force-Transferring Model and Shear Resistance in Panel Zone. Shear force in the panel zone is transmitted by the interaction of the outer steel tube wall, vertical stiffener, anchorage web, and core concrete, as shown in the shear stress distribution in Figure 3. As can be seen in Figures 3(a) and 3(b), the vertical stiffener is in the state of pure shear stress during the whole loading process. The vertical stiffeners are directly welded to the steel tube webs, which can effectively share the shear bearing capacity on the basis of protecting the steel tube and horizontal end plate. In contrast with the unribbed joint, the rib of the anchorage web has the stress concentration to reduce the magnitude of concrete's shear stress, as shown in Figure 3(d). Beyond that, the shear stress distribution of external concrete between the double steel tubes is basically similar for both the unribbed joint and the ribbed joint. The steel tubes also have the same principle of shear stress distribution as concrete. Besides, the material strength at the peak load has not reached the yielding shear stress yet, so the failure mechanism of the vertical stiffener joint should not occur in the panel zone. After this investigation, we can get the possible exact horizontal shear force in the panel zone supported by the main components of the vertical stiffener joint.

The calculation formula in the CFST structure proposed by the Architectural Institute of Japan [26] considers the shear strength of both the steel tube and infilled concrete. Thus, the shear resistance of the vertical stiffener joint in the CFDST structure contains the contribution from double steel tubes, force-transferring connectors, and core concrete. Among the contributions, concrete provides the shear bearing capacity in the diagonal strut model under the theory of plane shear state. Shear resistance of each component takes into account every stress state, and it can be computed by analyzing different deformation mechanisms; then, the ultimate shear capacity of the joint will be obtained based on the principle of limit equilibrium superposition. The validity and practicability of the strength superposition method for estimating shear strength were verified explicitly by many researchers [27–30]. Therefore, the shear resistance of each component is analyzed as follows.

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FIGURE 1: Details of vertical stiffener joints. (a) Ichnography. (b) Elevation for the unribbed joint. (c) Elevation for the ribbed joint.



FIGURE 2: Finite element model.

4.1.1. Steel Tubes. Since the confining effect of the concretefilled square steel tube is complex, the square tube is firstly equivalent to the circular steel tube using the method of the equal area [7]. Under the axial compressive force on the column, the circumferential tensile stress will decrease the shear capacity of the steel tube, so the circumferential stress $\sigma_{s\theta}$ of the steel tube is considered in the stress state, as shown in Figure 4, where σ_{sz} stands for the compressive stress under axial compression and τ_s is the shear stress of the steel tube. They can be expressed as $\sigma_x = \sigma_{s\theta}$, $\sigma_y = \sigma_{sz}$, and $\tau_{xy} = \tau_s$, and then the principal stresses are described as

$$\sigma_{1} = \frac{\sigma_{oz} + \sigma_{o\theta}}{2} + \sqrt{\left(\frac{\sigma_{oz} - \sigma_{o\theta}}{2}\right)^{2} + \tau_{o}^{2}},$$

$$\sigma_{2} = 0,$$

$$\sigma_{3} = \frac{\sigma_{oz} + \sigma_{o\theta}}{2} - \sqrt{\left(\frac{\sigma_{oz} - \sigma_{o\theta}}{2}\right)^{2} + \tau_{o}^{2}}.$$
(1)

Yield stress f_{sy} under the von Mises yield criterion of the steel tube web is

$$f_{sy} = \sqrt{\sigma_{sz}^2 + \sigma_{s\theta}^2 - \sigma_{sz}\sigma_{s\theta} + 3\tau_s^2}.$$
 (2)

So the shear stress τ_s of the steel tube can be deduced as

$$\tau_s = \frac{1}{\sqrt{3}} \sqrt{f_{sy}^2 - \sigma_{sz}^2 - \sigma_{s\theta}^2 + \sigma_{sz}\sigma_{s\theta}}.$$
 (3)

Then at the ultimate state of the joint, the ultimate shear resistance V_s of steel tube webs can be obtained as follows based on the trilinear model of shear [31, 32], as shown in Figure 5:

$$V_s = \frac{A_s}{\sqrt{3}} \sqrt{f_{su}^2 - \sigma_{sz}^2 - \sigma_{s\theta}^2 + \sigma_{sz}\sigma_{s\theta}},\tag{4}$$

where A_s represents the cross-sectional area of the steel tube web and f_{su} represents the ultimate shear strength of the steel tube.



FIGURE 3: Shear stress distribution at the maximum load. (a) Unribbed joint. (b) Ribbed joint. (c) External concrete in the unribbed joint. (d) External concrete in the ribbed joint.

4.1.2. Vertical Stiffeners. The pure shear stress state of the vertical stiffener is $\sigma_x = 0$, $\sigma_y = 0$, and $\tau_{xy} = -\tau_v$, and then the principal stresses are $\sigma_1 = \tau_v$, $\sigma_2 = 0$, and $\sigma_3 = -\tau_v$. Yield stress f_{vy} under the von Mises yield criterion of the vertical stiffener is $f_{vy} = \sqrt{3}\tau_{vy}$. Similarly, ultimate shear resistance V_v of the vertical stiffener is

$$V_{\nu} = \frac{f_{\nu u} A_{\nu}}{\sqrt{3}},\tag{5}$$

where A_v denotes the horizontal cross-sectional area of the continuous vertical stiffener and f_{vu} is the ultimate strength of the vertical stiffener.

4.1.3. Anchorage Web. Similar to the vertical stiffener, the anchored web is also in the pure shear state, and the ultimate shear resistance V_a is calculated by

$$V_a = \frac{f_{au}A_a}{\sqrt{3}},\tag{6}$$

where A_a is the shear area of the anchorage web. The horizontal cross-sectional area between the double tubes is taken for the unribbed anchorage web, while the whole horizontal cross-sectional area is taken for the ribbed one.



FIGURE 4: Stress state of the steel tube.



FIGURE 5: Trilinear shear force-deformation model.

4.1.4. Core Concrete. The external core concrete and internal core concrete are confined by steel tubes, so the shear resistance of concrete can be taken as a constant value after reaching the maximum shear strength, as shown in Figure 6, where γ_p and γ_p represent the peak shear strain and stress, respectively, and $\tau_p = 0.42 f_{cu}^{0.55}$ [33], in which f_{cu} is concrete's ultimate compressive strength. The shear resistance V_c of core concrete is calculated by

$$V_c = \tau_p \left(A_{co} + A_{ci} \right), \tag{7}$$

where A_{co} and A_{ci} , respectively, represent the external and internal concrete cross-sectional area.

4.2. Moment-Transferring Model and Bending Resistance. To get a better view, corresponding contours of the vertical stiffener and anchorage web are highlighted. The maximum principal stress distribution of the vertical stiffener in the yielding stage is shown in Figure 7. It indicates that the upper overhang on the left is under tension basically, while the lower one is under compression. In terms of the forcetransferring path, the tensile force exerted on the steel beam flanges is firstly transmitted from the end plate to the overhang of the vertical stiffener, then to the web of the outer steel tube, and eventually to the core region of the joint via the anchorage plate. Accordingly, due to the fact that bending moment can be transformed into a couple at the steel beam end, the upper and lower beam flanges are acted upon by tensile force and compressive force, respectively. The Mises stress distribution of the anchorage web at the



FIGURE 6: Shear stress-strain curve of the core concrete.



FIGURE 7: Maximum principal stress distribution of the vertical stiffener.

maximum load is specified, as shown in Figure 8. The anchorage web like a small cantilever slab transmits the shear force and bears the tension passing from the beam flange. Then, the anchorage web is capable of passing the tension exerted on itself to the inner tube, making the joint core zone an integrated part. As can be seen in Figure 8, the stress of the anchorage web embedded between double steel tubes is small, and the stress value at the connection with the upper and lower flanges of the steel beam is large. The advantage of the ribbed anchorage web is that the stiffening rib can firstly concentrate the internal force on itself and then transfer force to the double steel tubes. As the overall bearing capacity of the ribbed joint is higher than that of the unribbed one, the stress value of the ribbed anchorage web is also larger.

FE analytical models demonstrate that the vertical stiffener together with the anchorage web can transmit axial internal force of the beam flange through the horizontal end plate. Since bending moment is transmitted from the beam to the column in the form of tensile and compressive beam flanges, the connection performance and working capability of the beam flange, especially the one under tension, which will directly determine the entire bearing capacity of the joint, are of vital importance to the mechanical properties in



FIGURE 8: Mises stress contours at the maximum load. (a) Unribbed anchorage web. (b) Ribbed anchorage web.

the joint zone [34]. If the tensile flange can transmit the axial force successfully, the transmission of axial force in the compressive flange is also satisfied, so the tension model, as shown in Figure 9, is adopted to discuss the problem of transfer mechanism and bearing capacity of the joint. Direction 1 in Figure 9 stands for the tension F_1 of the flange transmitted by the vertical stiffener, and Direction 2 stands for the horizontal resultant force F_2 assumed by the column flange and anchorage web. In this new type of joint, the force on the flanges is directly passed on from the horizontal end plate to the vertical stiffener. On the premise of a guaranteed weld and according to the static equilibrium as well as the ultimate strength of the steel, the calculation of the ultimate tension assumed by the vertical stiffener F_1 can be expressed as follows:

$$F_1 = f_{\nu u} t_s h_s, \tag{8}$$

where t_s is the thickness of the vertical stiffener and h_s is the vertical height of the overhang of the vertical stiffener. We ignore the contribution of the concrete for its relatively poor tensile strength and the contribution of the steel tube web for its out-of-plane stress, and then the ultimate tension F_2 from the ribbed anchorage web can be calculated by

$$F_2 = f_{au} t_a h_a, \tag{9}$$

where t_a is the rib thickness and h_a is the rib height of the anchorage web. The transmission of the unribbed anchorage web is supposed to be halved. Finally, on the premise of the equivalent tension and compressive force, the ultimate bending moment can be calculated by

$$M_{uC} = FH_b = (2F_1 + F_2)H_b, \tag{10}$$

where H_b denotes the beam height.

FIGURE 9: Tension model.

5. Validation through Experimental and FEM Investigations

5.1. Test Specimens. Experimental investigations on six vertical stiffener joints between CFDST columns and steel beams were performed under cyclic loads. The parameters of these test specimens are illustrated in Table 1, where l_0 is the overhang length of the stiffener and n is the axial compression ratio. According to the principle of same width as the steel beam web, the rib is 50 mm in height and 200 mm in length. The section $\Box B \times t$ of the outer steel tube is shown in Table 1, and the inner steel tube is of circular section type of $O133 \times 6 \text{ mm}^2$. The steel properties of the specimens are shown in Table 2. Three coupons of the steel tubes and sheets were tested under tension to obtain the average width (t), yield and ultimate strength (f_y , f_u), and modulus of elasticity (E_s) . The anchorage web and vertical stiffener have the same material properties as the square steel tube. f_{cu} is the compressive strength of the concrete of 59.5 MPa. The steel

TABLE 1: Test specimens and calculation results.

Specimens	$\Box B \times t$ (mm ²)	l _O (mm)	п	Anchorage web	<i>l_p</i> (mm)	M_{uE} (kN · m)	M_{uC} (kN · m)	V _{uE} (kN)	V _{uC} (kN)	M_{uC}/M_{uE}	V_{uC}/V_{uE}
SBJ1-1	250×8	80	0.23	Unribbed	900	214.5	208.0	1703.9	4070.9	0.97	2.28
SBJ1-2	250×8	120	0.23	Unribbed	860	226.0	208.0	1774.2	4544.6	0.92	2.45
SBJ2-1	250×8	80	0.23	Ribbed	900	246.3	240.9	1959.5	4841.8	0.98	2.37
SBJ2-2	250×8	120	0.23	Ribbed	860	263.0	240.9	2126.1	5315.5	0.92	2.41
SBJ3-1	250×8	80	0.40	Unribbed	900	210.1	208.0	1669.5	4070.9	0.99	2.32
SBJ3-2	250×5	80	0.23	Ribbed	900	250.9	240.9	1856.6	4544.2	0.96	2.60

TABLE 2: Steel properties.

Steel	<i>t</i> (mm)	f_y (N/mm ²)	$f_u (\text{N/mm}^2)$	E_s (N/mm ²)
Outer steel tube	7.6	338.12	481.70	2.26×10^{5}
Inner steel tube	6.4	323.08	491.39	2.15×10^{5}
End plate	11.5	272.61	445.86	2.13×10^{5}
Beam flange	10.6	272.41	447.40	2.21×10^{5}
Beam web	6.6	291.00	457.23	2.16×10^{5}

beam section was $H244 \times 175 \times 7 \times 11 \text{ mm}^4$, and the thickness of the anchorage web was taken as the same beam web thickness of 7 mm. Vertical stiffener joints were tested by quasistatic cyclic loading similar to the FE modeling. The test was conducted in a self-equilibrating reaction frame, as shown in Figure 10.

5.2. Results and Validation. According to the experimental phenomena, the yielding of the joint all started in the arc section of the horizontal end plate. In the end, serious damage occurred at the horizontal end plate where the overhang of the vertical stiffener terminated, and then the cracks propagated towards the CFDST column. Figures 11 and 12 show the failure modes in the test and simulation. The plastic hinges were observed on the beam flange close to the horizontal end plate. Apart from some stress concentration phenomenon of the ribbed anchorage web, vertical stiffeners and steel tubes in the panel zone are basically in the elastic state. As can be seen in Figures 11 and 12, the accumulated plastic deformation, local buckling, and joint core stress distribution are well simulated. As predicted in a relatively strong panel zone, limited inelastic deformation could develop before significant flexural yielding developed in the beam section, and the joint specimens failed at the plastic hinges in the vicinity of the arc section of the horizontal end plates. Therefore, the design of the vertical stiffener joint is in line with the seismic principle of "strong shear and weak bending."

According to the theoretical and experimental data of six joint specimens, the ultimate shear capacity in the panel zone and the ultimate bending moment of the vertical stiffener joints are listed in Table 1. The ultimate shear capacity of the joint was computed by the principle of limit equilibrium superposition. Based on the aforementioned shear capacity equations of the joint components, the shear load capacity V_{uC} is obtained by equations (4)–(7) to assess the cracked vertical stiffener joint with shear failure in the panel zone. M_{uC} is the computed bending resistance

obtained from equation (10). V_{uE} represents the ultimate shear force in the panel zone at the ultimate load in the test. M_{uE} represents the ultimate bending moment at the beam plastic hinge corresponding to the average peak value of east and west load in the test.

When the beam-column assemblies are subjected to an axial force N on the column and a quasistatic cyclic loading P at the beam end, the internal force diagram in the panel zone is shown in Figure 13, where V_j represents the horizontal shear force in the panel zone, L_b is the pure beam length on one side, and H is the calculation height of the whole column. According to the equilibrium condition, the horizontal shear force in the test can be calculated by the method in [35] as follows:

$$V_{j} = \left(\frac{M_{l}}{H_{b} - t_{b}} + \frac{M_{r}}{H_{b} - t_{b}}\right) - V_{o} = \frac{2PL_{b}}{H_{b} - t_{b}} - \frac{PL}{H}, \quad (11)$$

where V_o represents the reactive force at the horizontal roller; t_b represents the thickness of the beam flange; and M_l and M_r are, respectively, the bending moment at left and right beam ends. Then, V_{uE} is the shear force corresponding to the average maximum quasistatic cyclic load P_{max} at both east and west beam ends. The bending moment at the plastic hinge can be calculated by

$$M_{uE} = P_{\max} l_p, \tag{12}$$

where l_p stands for the distance from the beam end to the overhang of the vertical stiffener.

As can be seen in Table 1, the calculation results of ultimate shear resistance V_{uC} are significantly larger than those obtained from the test data V_{uE} . The reason is that V_{uC} is the computed shear resistance corresponding to the assumed shear failure, which was computed by the conceptual model for assessing the cracked panel zone under shear failure. But both the test and the FEM verified that the failure mode of the joint is bending failure, so V_{uE} is the shear force corresponding to experimental bending failure. Though the theoretical shear resistance was not checked via joints of full

FIGURE 10: Photographs of the specimen test. (a) Overhead view. (b) Upward view.

FIGURE 11: Failure mode of the joint without ribs. (a) Test. (b) Simulation.

FIGURE 12: Failure mode of the joint with ribs. (a) Test. (b) Simulation.

FIGURE 13: Force diagram of the testing joint.

shear strength because the bending resistance limited the maximum internal shear force in the panel zone, it indicates that the joint has sufficient shear storage capacity to avoid the brittle failure, and the mean value of V_{uC}/V_{uE} is 2.41. Therefore, shear resistance in the panel zone provides a large amount of ductility and extra storage capacity to improve seismic performance. On the contrary, the ultimate bending resistance between calculations and experimental results is very close, and the mean value of M_{uC}/M_{uE} is 0.96. Therefore, the tension model for moment transfer is featured with a favorable accuracy, and it can be applied in predicting and evaluating the bending resistance of vertical stiffener joints.

6. Discussions

Based on the numerical results, general failure modes and transferring mechanisms of internal forces were closely investigated herein, and the bearing capacity computed by a conceptual model was compared with the test results. Next, in order to guarantee the ductile failure, the structural elements should be designed in the future engineering application according to the different contribution to the shear capacity and bending resistance.

In Table 2, by comparison of the specimens of overhang length 120 mm and 80 mm, it has been found that increasing the overhang length of the vertical stiffener does not improve the bending resistance effectively, but the plastic hinge mostly depends on the position of the vertical stiffener, so it can protect the joint core to ensure the strong beam-column connection. Meanwhile, lengthening the vertical stiffener's overhang increases the joint's shear resistance to have more storage capacity to avoid the brittle shear failure in the panel zone. Hence, the vertical stiffener determines the plastic hinge to ensure a higher bearing capacity and stiffness because the joint core zone is further effectively protected.

FIGURE 14: Brittle fracture and bending of the rib.

In the calculation of shear resistance of the vertical stiffener joint, axial compression ratio was taken into consideration for only the steel tubes, while it was not considered in the calculation of bending resistance. For the specimen SBJ3-1, the axial compression ratio is 0.4. The shear force and bending resistance obtained in the test have no evident changes from those of the specimen SBJ1-1. Therefore, the calculation method of vertical stiffener joints under different axial load levels in this study sounds reasonable, and axial compression ratio has a little influence on the joint's shear and bending resistance.

Concerning the behavior of the ribbed joints SBJ2-1 and SBJ2-2, the presence of the ribbed anchorage web enlarges the internal lever arm, thus increasing the depth of the panel and reducing the actual shear stress. The ribbed anchorage web is an effective way of increasing the bending resistance, and it also improves the computed shear resistance corresponding to the assumed shear failure. But when the overhang is short as 80 mm for SBJ2-1, tearing developed on the connected steel tube under tension and local buckling developed on the relatively thin rib under compression, as shown in Figure 14.

In comparison with the joint specimens SBJ2-1 and SBJ3-2, it was found that different thickness of outer steel tubes influenced the shear resistance of vertical stiffener joints because the tube web was the main shear component in the joint core. And under the condition of a strong panel zone, the tube's thickness has no effect on the bending resistance, while it deteriorated the foregoing tearing of the steel tube connected with the rib, so improving local thickness of the steel tube in the joint core is appropriate to improving the joint performance.

7. Conclusions

(1) Analytical models of shear force and bending moment were established based on the appropriate material constitutive equation and equilibrium theory to analyze the force transfer mechanism including shear force and bending moment. Shear force in the panel zone is transmitted by the interaction of the outer steel tube wall, vertical stiffener, anchorage web, and core concrete. Bending moment is transmitted from the beam to the column in the form of tensile and compressive beam flanges, and the vertical stiffener together with the anchorage web can transmit the internal force of the beam flange through the horizontal end plate. The failure mode investigated by the test and simulation is the plastic hinges in the vicinity of the arc section of the horizontal end plate. So, the mechanical characteristics and transfer mechanism of the vertical stiffener joint can satisfy the seismic requirements of the strong panel zone and "strong column-weak beam."

- (2) Six joint specimens were tested to verify the rationality of the theoretical models. The results computed by analytical models and the available experimental results were compared. The computed bending resistance obtained by the tension model agreed very well with the measured data from the quasistatic loading experiment. Although the theoretical shear resistance cannot be checked via test results due to limiting the maximum shear by the bending failure, the joint is verified to have enough shear storage capacity to guarantee the seismic design principle of "strong shear and weak bending."
- (3) The vertical stiffener determines the plastic hinge and shear capacity of the joint to guarantee its ductile failure. Axial compression ratio has a little influence on the joint's shear and bending resistance, while the ribbed anchorage web is an effective way of increasing the bearing capacity. Improving local thickness of the steel tube in the joint core is appropriate to improving the joint performance.

Notations

- A_a : Shearing area of the anchorage web
- Cross-sectional area of inner concrete A_{ci} :
- Cross-sectional area of outer concrete A_{co} :
- A_s : Shearing area of the steel tube
- *B*: Width of the outer tube
- Ultimate strength of the anchorage web f_{au}:
- Ultimate compressive strength of concrete f_{cu} :
- Yield strength of the steel tube f_{sv} :
- Ultimate strength of the steel tube f_{su} :
- Yield strength of the vertical stiffener f_{vy} :
- Ultimate strength of the vertical stiffener f_{vu} :
- Calculation height of the whole column H:
- H_h : Height of the steel beam
- H_c : Height of the column
- h_a : Rib height of the anchorage web
- h_{s} : Height of the vertical stiffener
- L_h : Distance from the beam end to the column surface
- l_0 : Overhang length of the vertical stiffener

- l_p : Distance between the beam end and the overhang of the vertical stiffener
- $M_{\mu F}$: Experimental ultimate bending moment
- $M_{\mu C}$: Computed ultimate bending moment
- Axial load level $(n = N_0/N_u)$ n:
- P_{\max} : Maximum quasistatic cyclic load at the beam end
- *R*: Radius of the inner tube
- Thickness of the beam flange t_b :
- Thickness of the anchorage web t_a :
- Thickness of the vertical stiffener t_s :
- $V_j: V_a:$ Horizontal shear force in the panel zone
- Shear resistance of the anchorage web
- V_C : Shear resistance of concrete
- V_o : Horizontal reactive force
- V_s : Shear resistance of the steel tube
- V_{uC} : Computed shear resistance
- V_{uE} : Experimental shear force at the ultimate load
- V_{ν} : Shear resistance of the vertical stiffener
- τ_s : Shear stress of the steel tube
- Peak shear stress of concrete τ_p :
- Peak shear strain of concrete γ_p :
- σ_{sz} : Compressive stress
- Circumferential stress. $\sigma_{s\theta}$:

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Disclosure

Yufen Zhang and Dongfang Zhang are the co-first authors.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this article.

Authors' Contributions

Yufen Zhang and Dongfang Zhang contributed equally to this work.

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