Research Article

Hysteretic Behavior of Beam-to-Column Joints with Cast Steel Connectors

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This study proposed a beam-to-column joint equipped with a new type of cast steel connector. The cast steel connector concentrated the primary portion of the deformation and energy dissipation of the joint and was installed with full bolted connections, rendering it a replaceable energy dissipation component and facilitating the rapid repair of the joint after an earthquake. Three full-scale specimens were fabricated and tested to investigate the hysteretic behaviors of the proposed joints under cyclic loadings. The results showed that the proposed cast steel connector exhibited reliable ductility and energy dissipation capacity. The beam-to-column joints with cast steel connectors under appropriate configuration can limit the deformation to the cast steel connector and protect the remaining joint components from plastic deformation. A more detailed finite element analysis was performed to investigate the hysteretic behavior of the joint further. The FEM results illustrated that the thickness of the vertical leg of the cast steel connector can significantly influence the stiffness and bearing capacity of the joint. Meantime, it would improve the hysteretic behavior effectively. The proposed beam-to-column joints with cast steel connectors can achieve the requirement of stiffness and load-bearing capacity and can be widely applicable in practical engineering.

1. Introduction

Studies on beam-to-column joints of steel moment-resisting frames are of great importance in steel structure research. Traditional rigid joints perform unsatisfactorily and suffer varying degrees of damage under seismic excitation [1–3]. The primary causes of this poor performance are the complex configuration of the joints, welding defects, brittleness [4], and stress concentrations [5, 6]. To overcome these drawbacks, a number of new types of joints were proposed. One type of the joints was designed to be stiffer ones relative to the beam by increasing the size of the cover plate of the beam end and weakening the end section of the beam, such that the plastic hinge can be moved away from the joint zone [7–11]. As a result, the strength of the joints could be guaranteed. Other type of joints was relatively more flexible compared with the beam such that the joint connection sustains most of the deformation, and hence, the energy can be dissipated. These joints need not to be welded, so there are no associated welding defects; thus, the seismic performance was improved

[12–16]. Among these joints, the ones with top-and-seat angle connection with high-strength bolts have drawn more attention by many researchers. To further improve the seismic performance of the joints, the top-and-seat angle connection was strengthened by welding rib stiffeners with the angle legs. Research showed that the stiffened angle steel exhibits higher strength, initial rotational stiffness, and energy absorption capacity, where the major improvement was contributed by the added rib stiffener [17]. However, the stiffened angle steel has two major drawbacks that are needed to be addressed. The first one is that the angle steel was made of rolled steel, which restricted the shape and size of the connector. Another one is the welding defects between the rib stiffener and angle legs. Consequently, the strength and stiffness of these connectors could not be fully explored and the application in practical engineering was restricted.

Casting technique provides the possibility to overcome the aforementioned drawbacks of the stiffened angle steel. In fact, cast steel components can be manufactured by one-step forming technique, which not only allows appropriate

structure configurations, but also avoids the possibility of welding defects. Moreover, the cast-steel components can be installed with full bolt connection, and the material strength can be fully utilized. In theory, the casting technique can provide components with arbitrary shapes for engineering applications. The cast steel components can be installed into any joint location, with advantages including installation flexibility and high practicability. Although cast steel components are more expensive than traditional elements, mass production can reduce the costs.

Studies on cast steel structure components have begun prior to this work. Researchers had attempted to apply cast steel components to engineering structures and investigated related properties as early as the 1960s. The cast steel joints were used most widely in situations characterized by complex stress distributions and shapes. Webster et al. reported the casting process and advantages of the cast steel joints widely used in offshore structures in Britain [18]. Marston summarized recent developments of cast steel joints and elaborated their advantages in terms of material properties, mechanical performance, and manufacturing costs [19]. Broughton et al. examined the advantages of cast steel joints in terms of casting cost and fatigue performance based on an offshore oil platform structure [20]. Ma et al. experimentally studied fatigue performance of cast steel joints [21] and focused primarily on the stress concentration, material $S-N$ curves, and initial defects of the joints, and ultimately proposed design fatigue curves of the cast steel joints. de Oliveira et al. from the Cast ConneX Corporation tested the cast steel joints used in the tube brace end [22]. Static and pseudostatic tests indicated that these types of joints could be applied in seismic designs. Recently, a new cast steel Yielding Brace System (YBS) equipped with a specially designed cast steel connector was proposed. Through the flexural yielding of the triangular fingers of the cast steel connector, the seismic energy of the system was dissipated [23]. Wang et al. proposed an H-beam to tubular column cast modular joint, which was an integral assembly of the joint zone, beam linking zone, and column linking zone. The seismic performance of the joint was experimentally studied, and comments on available design expressions were developed [24].

In recent years, cast steel connectors were studied in seismic steel moment-resisting frames [25]. The Kaiser bolted bracket consisting of proprietary cast high-strength steel brackets was proposed [26]. The idea of the Kaiser connection was to create a more rigid cast piece such that all deformations could be restricted to the beam and panel zone. Fukumoto reported concrete-filled steel tube columns and I-shaped section beams with cast steel connectors [27]. The influence of all factors, including different connector types, was experimentally analyzed. It was seen that cast steel connectors can improve the joint strength. In addition, the required joint performance can be obtained by adjusting the connector shape.

Fleischman et al. experimentally and theoretically analyzed the beam-to-column joints of integrally casted steel moment-resisting frames [28–31]. It was found that the fragility of the welding joints under seismic excitation

promoted the development of these types of joints. Cast steel joints are not specifically restricted regarding shape. They can actively avoid stress concentrations, and the material properties can be chosen according to practical requirements, which are not available for traditional joint types. Experimental study and finite element analysis (FEA) indicated that these types of joints exhibit remarkable durability and enhanced energy dissipation characteristics. At the same time, a new type of beam-to-column joint equipped with a cast steel connector was proposed. It avoided the bolt prying force and provided the structure with sufficient durability by the large deformation of the joint connector. Tong et al. analyzed the performance of the cast steel connector and proposed a simplified type of connector which uses full bolted connections rather than welding [32–34]. Through testing and numerical simulation, the preliminary design was proposed and the satisfactory energy dissipation capability was verified.

In summary, the stiffened angle steel can be employed to avoid the drawbacks of traditional technology and improve the seismic performance of traditional beam-to-column joints. However, its application was hampered by the shape and size limitation of the rolled steel and welding defects between the rib stiffener and angle legs. Casting technique paves the way to overcome these drawbacks, owing to the great freedom in geometry shaping and weld-free feature, as verified by the afore-reviewed researches.

Based on the above observations, a new type of cast steel connector was proposed by combining the stiffened angle steel and steel casting technique. The proposed cast steel connector was designed to be a relative flexible component with respect to the beam such that the connector concentrates the major part of the deformation and energy dissipation of the joint. As a result, the beam and column of the joint can be protected from excessive deformation. Besides, the proposed cast steel connector can be installed with full bolts, facilitating the construction and avoiding the welding defects. The geometric shape of the cast steel connector was similar to the stiffened angle steel, comprising three perpendicular components, a vertical leg, horizontal leg, and rib stiffener, as shown in Figure 1. It should be noted that the design philosophy of the proposed cast steel connector is different from the Kaiser bracket connection, although they have a similar shape and both were fabricated with casting technique. The proposed cast steel connector is relative flexible such that it suffers the most deformation, whereas the Kaiser bracket is more rigid such that all deformation is developed in the beam and panel zone. Intuitively, a full two-sided symmetric shape seemed better than the single-sided shape of the proposed connector. However, it should be noted that the single-sided shape connector is easy to install in engineering practice (due to the space limitation of the beam and column flange) and has a larger deformation capacity in plastic stage, although the stiffness and strength are slightly smaller. Considering that the design intent of the cast steel connector is to provide a flexible connection such that the connection suffers the most portion of the deformation, it is reasonable to adopt the single-sided shape rather than a full two-sided symmetric shape.

FIGURE 1: Proposed cast steel connector.

The cast connector was placed on both the top and bottom flanges of beam and connected to the beam and column with bolts. The proposed beam-to-column joints with cast steel connectors were then complete, as shown in Figure 2. Three full-scale tests were performed to investigate the hysteretic performance of the proposed joints. A finite element (FE) model was constructed and verified based on the test results by using ANSYS software. In addition, a total of 13 models with various vertical leg, horizontal leg, and rib stiffener thicknesses were analyzed by ANSYS to perform a parameter analysis. By comparing the hysteretic curves and equivalent viscous damping ratios of the specimens, the influence of the cast steel connector dimensions on the hysteretic performance of the joints was analyzed.

2. Experimental Program

To investigate the hysteretic behavior and assess the seismic performance of the proposed beam-to-column joints with cast steel connectors, three full-scale test specimens, DT1–DT3, were fabricated. The thickness of the vertical legs and rib stiffeners of the connectors were varied to assess the influence of connector dimensions on the hysteretic behavior of the joints.

2.1. Test Specimens. The test specimens were T-shaped models comprising a column and a beam. The column was a hot-rolled section of HM300×200 with dimensions H294 mm×200 mm×8 mm×12 mm. The beam was a welded section with dimensions H250 mm×200 mm×6 mm×8 mm. Horizontal stiffeners with a thickness of 10 mm were welded to the column web at locations corresponding to the beam flanges. The column height was 1.94 m, and the beam length was 1.06 m. The steel used for the column and beam was Q235-B (Chinese Code, the yield strength is 235 N/mm²) [35]. The three joints were assembled with full bolt connection by the proposed cast steel connectors. For each one of the joints, four cast steel connectors are employed and placed on both the top and bottom flanges of the beam, as shown in Figure 2.

The design of the cast steel connector and the bolt gage is based on the following considerations. First, the tensile capacity of the bolts connected to the column should be greater than the shear capacity of the bolts connected to the beam, and they should be both greater than the tensile capacity of the cast steel connector. Then, the possible deformation and

FIGURE 2: Assembly position of cast steel connectors.

failure occur to the cast steel connector. And the bolts should be in elastic stage during the loading process. Besides, the design should satisfy the configuration requirements of edge distance and center distance of the bolt holes.

Specifically, the cast steel connector has a similar shape with the stiffened angle steel as shown in Figure 1. The geometric configuration of the cast steel connector was designed on the basis of the stiffened angle steel and according to EC3 [36] and Chinese Code GB50017-2017 [35]. The dimensions of DT1 were designed such that it is flexible relative to the beam and could be used as energy dissipation component. Dimensions of DT2 and DT3 are the same as those of DT1, except that the thickness of the vertical leg of DT2 and the rib stiffener of DT3 was increased to 18 mm and 8 mm, respectively.

The cast steel connector was manufactured by precision casting with the material of nonwelding steel Z230 (Chinese Code, the yield strength is 230 N/mm²) [37], in which the heat treatment of normalizing and tempering at 800°C was adopted. In order to avoid stress concentrations, fillets with a 12 mm radius were adopted along each of the three intersecting lines between the legs and rib stiffener of the cast steel connector. The casting process design, casting mold orientation, QA/QC and NDT process, and requirements for creating these cast prototypes are selected and performed by the cast steel manufacturer according to related codes [38–40].

Details of the cast steel connectors corresponding to the three specimens are shown in Figure 3 and presented in Table 1, respectively. A total of sixteen M20 grade 10.9 high-strength bolts were used to connect the proposed connectors to the beam and column for each of the specimens. The bolts were designed according to Chinese Code GB50017-2017 [35]. Each bolt was installed using the calibrated wrench method to achieve a pretightening force of 155 kN and torque of 445 N·m, respectively, according to Chinese Code JGJ 82-2011 [41].

2.2. Test Setup and Loading Procedure. The test was designed to simulate the full-scale beam-to-column joint subassembly in a steel moment-resisting frame subjected to lateral loading. The schematic drawing and on-site photo of the test setup are shown in Figures 4 and 5, respectively. Before testing, an axial load (corresponding to 20% of the yield load of column) was applied to the column to ensure that the

FIGURE 3: Details of the cast steel connector (unit: mm).

TABLE 1: Primary parameters of cast steel connectors (unit: mm).

Specimen	Horizontal leg thickness	Vertical leg thickness	Rib stiffener thickness
DT1	6	12	4
DT2	6	18	4
DT3	6	18	8

FIGURE 4: Test setup and measurement devices (unit: mm, ① to ⑥ denote displacement transducers 1 to 6, respectively.).

whole section of the column is under compression. Vertical loads were then applied to the beam tip.

The loads in the test were applied under hybrid force and displacement control [42], as shown in Figure 6. Force-controlled loads were first applied with an initial value of 25 kN and an increment of 12.5 kN. The initial force was much less than the plastic load, and the increment was half of the first load step. The force-controlled loading was cycled

until visible separation between the vertical leg of the cast steel connector and column flange was observed. Then, the loading mode was switched to displacement-controlled loading.

The tests were terminated when any one of the following phenomena was observed:

- (1) Component fracture (of the cast steel connector or the bolts)

FIGURE 5: Test setup. (a) Global view. (b) Connection.

FIGURE 6: Loading procedure.

- (2) Out-of-plane buckling of beam
- (3) Applied load can no longer be sustained by the specimen
- (4) Rotation between beam and column exceeds 0.1 rad

2.3. Instrumentation. Instrumentation on the specimens is comprised of six displacement transducers and two load transducers, as shown in Figure 4. Displacement

transducers 1 and 2 measured vertical displacements of the beam end, 3 and 4 measured the horizontal displacement of the column at the joint zone, and 5 and 6 measured the panel zone deformation at the joint zone. The six measured displacements were denoted by $\Delta_1 - \Delta_6$ (elongation is taken as positive and shortening as negative). Load transducers 1 and 2 were used to measure the vertical force applied to the beam tip and axial load of the column, respectively.

2.4. Calculation of the Moment and Rotation of Joints.

Based on the measured data, the moment M and rotation θ_j of the beam end were obtained. The moment is calculated by $M = FL$, where F is the vertical load provided by the hydraulic jack and L is the distance from the loading point at the beam tip to the column face. The joint rotation θ_j includes primarily the shear deformation of the joint zone and deformation of the joint connection. However, it is difficult to measure θ_j directly. Therefore, θ_j is derived by using the measured displacements of the beam and column as shown in Figure 6 and is calculated as follows:

$$\theta_j = \theta_{1,2} - \theta_{3,4} + \theta_{5or6}, \quad (1)$$

where $\theta_{1,2}$ is the rotation of the beam end, comprising the contribution of the deformation of the connection (including the connector, bolts, column flange, and beam flange), deformation of the joint zone (primarily the column web), bending deformation, and rigid rotation of the column; $\theta_{3,4}$ is the bending deformation of the column (governed by the line stiffness of column), rigid rotation of the column, and shear deformation of the joint zone; and θ_{5or6} is the shear rotation of the joint zone (primarily the column web) (Figure 7).

The rotation of the beam end, $\theta_{1,2}$, is calculated as follows:

$$\theta_{1,2} = \arctan \frac{|\Delta_1 - \Delta_2|}{l_{1,2}}, \quad (2)$$

where $l_{1,2}$ is the distance between displacement transducers 1 and 2.

Rotation $\theta_{3,4}$ can be calculated as follows:

$$\theta_{3,4} = \frac{|\Delta_3 - \Delta_4|}{a}, \quad (3)$$

where a is the length of the vertical edge of the measured joint zone.

The shear rotation of the column web, θ_{5or6} , is calculated as follows:

$$\theta_{5or6} = \arccos \left(\frac{2c\Delta_{5or6} - \Delta_{5or6}^2}{2ab} \right), \quad (4)$$

where Δ_{5or6} is the displacement measured by displacement transducer 5 or 6 (elongation is taken as positive and shortening as negative), and b and c are the lengths of the horizontal edge and diagonal line of the measured joint zone, respectively.

3. Materials

Tensile tests were conducted on coupons to obtain the material mechanical properties of the steel components. The tested coupons were cut directly from all three parts of the components along the longitudinal orientation according to Chinese Codes GB/T 228.1-2010 [43] and GB/T 2975-1998 [44]. The tensile test results are presented in Table 2.

4. Test Results and Discussions

4.1. Test Results and Observation. During the test, the three specimens were subjected to two primary stages of experimental observation, corresponding to the loading under force and displacement control, and are discussed in the following sections.

4.1.1. Specimen DT1. First stage: the specimen before loading is shown in Figure 8(a). Cyclic forces, with an initial value of 25 kN and increments of 12.5 kN, were imposed on the beam tip according to the loading procedure in Figure 6. The specimen was elastic, and no visible change occurred in this stage. The vertical leg of the cast steel connector and column flange will separate when the load increased to 50 kN (53 kN·m and 3.52 mrad for joint), and displacement at the beam tip is 8 mm.

Second stage: cyclic displacements with both initial and incremental values of 8 mm were applied to the beam tip in accordance with the loading procedure in Figure 6. When the applied displacement approached 16 mm, the reaction force at the beam tip was 90 kN. During this process, a loud noise was heard from the specimen when the downward load on the beam tip reached 67.5 kN (71.55 kN·m and 7.88 mrad for joint), and the bolts started to slip at that time, meaning that the friction force of the high-strength bolt connection had been exceeded. The bolts continued to slip until the applied displacement reached 32 mm. During this process, deformation of the cast steel connector gradually increased, whereas the reaction force at the beam tip did not increase and remained less than 90 kN. When the displacement at the beam tip increased to 82.6 mm, the horizontal leg of the cast steel connector at the bottom flange of the beam fractured because of fatigue, as shown in Figure 8(b). The corresponding reaction force was 105.9 kN (112.25 kN·m and 35.01 mrad for joint). The load-bearing capacity of the beam-to-column joint thus decreased significantly. The loading was maintained and cycled at approximately 80 mm. The reaction force was less than 90 kN. The maximum distance between the vertical leg of the cast steel connector and the column flange reached 15 mm, as shown in Figure 8(c). The loading stopped, as the load-bearing capacity could no longer be increased by continuing cycling the loading. The vertical leg of the cast steel connector in the top flange of the beam buckled and warped significantly, implying that the resulting prying force of the bolt was significantly higher, as shown in Figure 8(d). The residual plastic deformation of the joint after unloading is shown in Figure 8(e). The beam deformed linearly throughout the whole test, indicating that no plastic hinge appeared, as shown in Figure 8(f).

4.1.2. Specimen DT2. First stage: the test observations of DT2 were similar to those of DT1, except that the applied load was 75 kN (79.5 kN·m and 6.40 mrad for joint) before obvious separation between the vertical leg of the cast steel connector and column flange, and significant deformation of the vertical leg and rib stiffener of the cast steel connector

FIGURE 7: Calculation diagram of θ_j .

TABLE 2: Results of tensile coupon tests of steel components.

Component	Elastic modulus, E (10^5 N/mm ²)	Yield strength, f_y (N/mm ²)	Ultimate strength, f_u (N/mm ²)	Elongation (%)
Cast steel connector	1.92	363	602	15.5
Beam flange	2.08	355	524	32.5
Beam web	2.10	392	519	32.5
Column flange	2.07	315	505	30.7
Column web	2.04	330	527	32.7
Bolt	2.02	946	1044	—

FIGURE 8: Continued.

FIGURE 8: Test observation of specimen DT1. (a) Specimen DT1 before testing. (b) Fracture of the horizontal leg of the cast steel connector. (c) Separation between the vertical leg of the cast steel connector and column flange. (d) Warping of the vertical leg end of the cast steel connector. (e) Buckling of the rib stiffener of the cast steel connector. (f) Deformation of DT1.

was observed. The corresponding displacement at the beam tip was 8 mm.

Second stage: the loading procedure of DT2 in the second stage was identical to that of DT1. Similar to DT1, a loud noise was heard from DT2 and the bolts started to slip when displacement at the beam tip reached 16 mm. At the same time, the vertical leg of the cast steel connector separated from the column flange, and buckling deformation of the column flange was clearly observed. The load-displacement curve of DT2 exhibited an inflection point, indicating the stiffness degradation of DT2. When the applied displacement reached 24 mm, the reaction force at the beam tip was 100 kN (106.0 kN·m and 12.43 mrad for joint). A loud noise was then heard continuously from DT2 and the bolts continued slipping. At the end of the loading, the horizontal leg of the cast steel connector at the top flange of the beam fractured because of fatigue, as shown in Figure 9(a). The displacement at the beam tip and the width of the gap between the vertical leg of the cast steel connector and column flange were -58 mm and 10 mm, respectively, as shown in Figure 9(b). The test was terminated because of the significant decrease in the load-bearing capacity. The buckling deformation of the rib stiffener of the cast steel connector and the residual deformation of DT2 after unloading are shown in Figures 9(c) and 9(d), respectively.

4.1.3. Specimen DT3. First stage: the test observations of DT3 in this stage were identical to that of DT2. The load was also 75 kN (79.5 kN·m and 3.44 mrad for joint) when separation was clearly observed between the vertical leg of the cast steel connector and the column flange. However, the corresponding displacement at the beam tip was 10 mm, which was marginally greater than those of DT1 and DT2.

Second stage: the initial displacement and subsequent displacement increment were both 10 mm. A loud noise was heard from DT3 when displacement at the beam tip reached -20 mm for the first time, indicating that the bolts had started to slip. In the second cycle, with a displacement of 20 mm, the vertical leg of the cast steel connector

separated significantly from the column flange, and clear buckling deformation of the column flange was observed. At a displacement of 30 mm, the reaction force at the beam tip was approximately 120 kN (127.2 kN·m and 16.68 mrad for joint). The bottom flange of beam buckled significantly and then restored to plane under cyclic loading. Visible deformation of the rib stiffener of the cast steel connector was observed, and a loud noise was heard continuously from the specimen. At a displacement of 40 mm, the load did not increase significantly, and the peak load was 136 kN (144.4 kN·m and 72.50 mrad for joint). Apparent deformation occurred on the rib stiffener and horizontal leg of the cast steel connector. With the gradual increase in the displacement, deformation of the column flange, gap width between the vertical leg of the cast steel connector and column flange, and deformation of the cast steel connector increased. When the applied displacement reached 100 mm, the horizontal leg of the cast steel connector at the bottom flange of beam fractured at the location of the bolts because of fatigue, as shown in Figure 10(a). The test was terminated because of excessive deformation. The beam web and flange buckled significantly, as shown in Figure 10(b). Large residual deformation of the column flange and the cast steel connector was observed when the specimen was uninstalled, as shown in Figures 10(c) and 10(d). Deformation of the column flange contributed partially to the overall connection rotation, which is difficult to repair.

4.2. Discussion of Test Results. Based on the material strength and dimensions, the nominal flexural moment resistance of the beam can be calculated as 169.94 kN·m. And the nominal flexural moment resistance of the cast steel connection (DT1) is 316.5 kN·m if that tensile yielding is assumed to occur over the full section. It seems that the connection has a larger strength than the beam. However, test results showed that the cast steel connector exhibits flexural failure mode, indicating that the actual strength of the connection is smaller than that of the beam. Therefore, it is inappropriate to treat the cast steel connector as a tensile component when calculating the flexural moment

FIGURE 9: Test observation of specimen DT2. (a) Fracture of the horizontal leg of the cast steel connector. (b) Deformation of the vertical leg of the cast steel connector and column flange. (c) Buckling of the rib stiffener of the cast steel connector. (d) Residual deformation.

FIGURE 10: Test observation of specimen DT3. (a) Failure of the cast steel connector. (b) Failure of the joint. (c) Deformation of the column flange. (d) Deformation of the cast steel connector.

resistance, since the connector exhibits rather complicated deformation and failure mode. Based on the test results, all of the three tested specimens can be classified as partial strength connections according to Eurocode 3 [36]. Concerning the stiffness, the specimens can be considered as semirigid connections when employed in all frames, if the beam length is at least 20 times larger than the height of the beam section [45].

Table 3 presents the failure characteristics of the three specimens based on the test results. Fracture of the horizontal leg of one of the cast steel connectors can be observed for all of the specimens. It can be seen that increasing the thickness of both the vertical leg and rib stiffener significantly improved the strength of the cast steel connector, amplified the deformation of the connected beam and column, and changed the failure mode. The increased thickness also helped to make full use of the energy dissipation capacity of the joint components but restricted the energy dissipation capability of the cast steel connector itself.

The hysteric curves of the joints in the tests are shown in Figure 11. It can be seen that the initial stiffness and flexural capacity increased with the thickness of the vertical leg and rib stiffener. When the load-bearing capacity approached the elastic limit, the moment-rotation curves exhibited pinching behavior because of the slipping of the cast steel connector, which affected the hysteric performance of the joint. However, it can also be observed that the load-bearing capacity was not significantly decreased by the failure of one of the cast steel connectors, indicating that this type of joint has a significant load-bearing capacity and rotation capacity margin.

The hysteric performance of DT1 was primarily provided by the cast steel connector. However, the load-bearing capacity was relatively low, the cast steel connector failed earlier, and the rotation when the test stopped was small, because of the thinner vertical leg and rib stiffener.

The hysteric curve of DT2 exhibited a plumper shape. The load-bearing capacity was apparently greater than that of DT1 because of the significant improvement in strength and stiffness of the cast steel connector. More importantly, it showed that there was full exploitation of the hysteric capabilities of the cast steel connector and column flange and web because of the appropriate configuration of the cast steel connector and column. The final displacement at the beam tip was relatively small because of the earlier failure of the cast steel connector.

Both the stiffness and the load-bearing capacity of DT3 were relatively high. The hysteric curve indicated that DT3 had the greatest rotation capacity. However, because of the small deformation of the cast steel connector, the energy dissipation capacity was provided primarily by the plastic deformation of the beam and column. Therefore, DT3 exhibited the characteristics of a strong joint.

In summary, the different connectors exhibited different static and dynamic characteristics. The beam-to-column joints with cast steel connectors can meet the required stiffness and bearing capacity. The requirements of the joint

design can be achieved by choosing different sizes of cast steel connectors. Cast steel connectors exhibited reliable ductility and energy dissipation capabilities, do not require welding (and thus are not exposed to the risk of welding defects), and make full use of the material, with better overall performance.

5. Finite Element Analysis

A nonlinear FEA was performed to further investigate the hysteretic behavior of the joints by using ANSYS. The FE model of the specimen was established to simulate the three hysteretic tests, based on which the FE model was validated. A total of 13 models divided into 3 groups with different thicknesses of the vertical leg, horizontal leg, and rib stiffener of the cast steel connector were then adopted to perform a parameter analysis based on the established FE model.

5.1. FE Model. The FE models were built with solid elements that can simulate three-dimensional spatial deformation and buckling. Specifically, the steel components were modeled by SOLID45 elements, which can be used to simulate the mechanical behavior of steel. The cast steel connector was modeled by SOLID92 elements to adapt the refined mesh and elements at the chamfering part and to better simulate the properties of cast steel. The contact interactions related to the cast steel connector were simulated by pair-based contact elements TARGE170 and CONTA174, while the remaining contact interactions were simulated by TARGE170 and CONTA173. Pretension of the bolts was modeled by the PRETS179 element. The tangential friction with a coefficient of 0.3 as recommended by Chinese Code GB50017-2017 [35] was assumed for the contact surfaces between the connector and beam flange and column flange. The FE model was meshed with manually specified meshing sizes. The core areas of the joint were subdivided into elements with a size of less than 5 mm, as shown in Figure 12.

Bilinear kinematic hardening stress-strain relationships for the steel materials of the connector, beam, column, and bolt were adopted. The specific stress-strain relationship was expressed as follows:

$$\begin{aligned}\sigma &= E\varepsilon, & \varepsilon \leq \varepsilon_0, \\ \sigma &= \sigma_0 + E_t \left(\varepsilon - \frac{\sigma_0}{E} \right), & \varepsilon > \varepsilon_0,\end{aligned}\tag{5}$$

where σ and ε are the stress and strain, respectively; E is the elastic modulus; E_t is the strain hardening modulus, $E_t = 10\% \cdot E$; and σ_0 and ε_0 are the yield stress and strain, respectively. The material characteristics, such as the elastic modulus and yield stress, were the same as in the physical test. The von Mises yield criteria and the flowing rules were adopted. The bilinear stress-strain curve and stress-strain curve of the cast steel are presented in Figure 13. It is a commonly used model to simulate the stress-strain relationship of steel.

TABLE 3: Failure characteristics of the specimens in the hysteretic tests.

Specimen	Failure characteristics			
	Cast steel connector deformation	Column flange deformation	Beam deformation	Displacement at the beam tip
DT1	Large	Unnoticeable	Unnoticeable	Large
DT2	Moderate	Moderate	Unnoticeable	Moderate
DT3	Small	Moderate	Buckling of the flange and web	Small

FIGURE 11: Hysteretic curves in the tests.

5.2. Comparison with Test Results. The three hysteretic tests were simulated by the established FE model. Each loading level was cycled only once in the FEA to reduce computation costs. The nonlinear geometry and true stress/strain were considered in the FE model. The 3D solid element can simulate the local buckling and large strain, as shown in Figure 14. Comparisons of the deformations after loading (with the same load level) between FEA and test results are shown in Figure 14. It can be found that the deformation mode and failure characteristics of the joints in the tension zone of FEA were consistent with that of test results. Therefore, the FE model can

be used to simulate the local buckling and large strain and further analyze the proposed joint in this paper.

Comparisons of the moment at each loading level between the FEA and the tests are presented in Table 4, and comparisons of the moment-rotation hysteretic curves between the FEA and the tests are shown in Figure 15.

It can be seen from Table 4 and Figure 15 that the FEA results were in good agreement with the test results: the initial and unloading stiffness correlated well with the test results, as did aspects such as the overall hysteric performance and the developing trend. This showed that the

FIGURE 12: Finite element model. (a) Global view. (b) Joint mesh.

FIGURE 13: Stress-strain curve comparison.

FIGURE 14: Continued.

FIGURE 14: Comparison of the deformation between FEA and test results. (a) Deformation of test DT1. (b) Deformation of test DT2. (c) Deformation of test DT3. (d) Deformation and stress of FEA DT1. (e) Deformation and stress of FEA DT2. (f) Deformation and stress of FEA DT3.

TABLE 4: Comparison of moment at various loading levels between FEA and test results.

Specimen Load cycle (rotation)	DT1 (kN·m)			DT2 (kN·m)			DT3 (kN·m)		
	FEA	Test	Error (%)	FEA	Test	Error (%)	FEA	Test	Error (%)
1 (15 mrad)	105	105	0	127	121	5	145	129	12
2 (25 mrad)	110	95	16	141	128	10	154	135	14
3 (35 mrad)	109	98	11	140	133	5	152	141	8
4 (45 mrad)	109	103	6	139	139	0	152	145	5

FIGURE 15: Continued.

FIGURE 15: Comparison of moment-rotation hysteretic curves between FEA and test results. (a) DT1. (b) DT2. (c) DT3.

FIGURE 16: Loading procedure of the cycle loading in FEA (mm).

TABLE 5: Primary parameters of cast steel connectors for FEA (unit: mm).

Model	A1	A2	A3	A4	A5	B1	B2	B3	B4	B5	C1	C2	C3	C4	C5
Vertical leg	8	12	16	20	24	16	16	16	16	16	16	16	16	16	16
Horizontal leg	8	8	8	8	8	4	6	8	10	12	8	8	8	8	8
Rib stiffener	8	8	8	8	8	8	8	8	8	8	4	6	8	10	12

established FE model can be used to further analyze the hysteretic behavior of the joints.

5.3. Parameter Analysis. Based on the established FE model, a total of 13 models with various cast steel connector dimensions were analyzed to perform a parameter analysis. For calculation convenience, the load was imposed under displacement control, as shown in Figure 16. The 13 models were divided into three groups: A, B, and C. Each group comprised five models, named as A1–A5, B1–B5, and C1–C5 for groups A, B, and C, respectively. The dimensions of the models were the same as for the test, except that the

vertical leg thickness in group A, the horizontal leg in group B, and the rib stiffener in group C of the cast steel connectors were changed, as shown in Table 5. Models A3, B3, and C3 had the same dimensions; therefore, the actual number of models used for the parameter analysis was 13.

The moment-rotation hysteretic curves of groups A, B, and C are presented in Figures 17–19, respectively. It can be observed that hysteretic curves of all the specimens, except for specimen B1, exhibit a plumper shape. The hysteretic curves of B1 exhibited a remarkable pinch behavior, which was caused by the earlier yield of the model because of the relatively thin horizontal leg of the cast steel connector. Stress analyses indicated that stress in the key part of the

FIGURE 17: Moment-rotation hysteretic curves of model group A.

FIGURE 18: Moment-rotation hysteretic curves of model group B.

FIGURE 19: Moment-rotation hysteretic curves of model group C.

FIGURE 20: Equivalent viscous damping ratio of model joints. (a) Group A. (b) Group B. (c) Group C.

joints did not exceed $1.3f_y$ when the joint rotation reached 30 mrad, which indicated reliable rotation capacity. The thickness of the cast steel connector legs had a significant influence on the hysteretic performance of the joints. With the increases in thickness, the yield load, initial rotation stiffness, and energy-dissipation capability of the joints were improved.

5.4. Hysteretic Behavior of the Joint. An equivalent viscous damping (EVD) ratio [46] based on the hysteretic curves was adopted to quantitatively investigate the energy dissipation capability of the joints. The EVD ratio h_e was calculated as follows:

$$h_e = \frac{1}{2\pi} \cdot \frac{A_h}{F_m \Delta_m}, \quad (6)$$

where A_h is the area within one complete cycle of a hysteresis curve and F_m and Δ_m are the maximum force and corresponding displacement, respectively.

For the convenience of analysis, the EVD ratio-joint rotation curves are shown in Figure 20. It can be seen that the vertical leg of the cast steel connector has a significant influence on the hysteretic behavior of the joints; however, the horizontal leg (group B) and rib stiffener (group C) does not. The h_e values of A3, A4, and A5 were essentially identical, meaning that thicker vertical legs do not necessarily improve the energy dissipation capability of the joint. Similar phenomena can be observed for groups B and C, indicating that the energy dissipation capability is provided primarily by the connector deformation. A thicker vertical leg, rib stiffener, and horizontal leg do not improve the energy dissipation capability.

The h_e of the joints increased rapidly up to 20 mrad. When the rotation was greater than 20 mrad, h_e increased slowly and tended to a constant greater than 0.22, indicating good hysteretic behavior, and the portion of plastic energy dissipation did not increase significantly when the joints were in the plastic stage.

6. Conclusions

A novel beam-to-column joint with cast steel connectors was proposed. Three full-scale specimens were tested under cyclic loading to investigate their hysteretic behavior. Finite element simulations of the hysteretic tests and FE model validation were then performed using ANSYS. Based on the established and validated FE model, 13 specimens were analyzed under cyclic loading for parameter analysis. Combining the tests and FEA results, the following conclusions can be drawn:

- (1) The thickness of the vertical leg and rib stiffener had a significant influence on the stiffness and bearing capacity of the joint, of which the influence of the vertical leg was greater.
- (2) Failure of the single cast steel connector had no clear influence on the hysteretic behavior of the joint. Deformation of the cast steel connector provided ductility to the joint. The joint had good rotational capacity and seismic performance. The requirements of joint design can be met by choosing appropriate sizes of the cast steel connectors.
- (3) This type of joint exhibited good hysteretic behavior, which can be effectively improved by varying the thickness of the vertical leg of the cast steel connector. The thickness of the rib stiffener and horizontal leg had little influence on the EVD ratio.

Data Availability

All data included in this study are available upon request by contact with the corresponding author.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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