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Preface

These proceedings contain the papers presented at the TENTH INTERNATIONAL CONFERENCE ON ADVANCES IN STEEL STRUCTURES (ICASS 2020) held in Chengdu, China, from 21 to 23 August 2022. The international conference series on Advances in Steel Structures was initiated in 1996 under the support of The Hong Kong Polytechnic University, which remains very active in fostering its continuation—joined a few years later by the Hong Kong Institute of Steel Construction.

These proceedings bring together most recent findings in numerical, theoretical and experimental research, as well as its practical implementation in design practice in the areas of Assembled Structure, Bridge, Cold-formed Steel, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue, Fire, High-Strength Steel, Impact and Protection, Intelligent Construction, New Material, Seismic Resistance, Stability, Stainless Steel, Structure Systems, Testing & Monitoring. The papers presented in these proceedings come from a wide range of countries/regions and will be a great reference source.

Specially, the subject matter has been categorized under the broad heading of:

**Volume I:** Keynotes Lectures, Assembled Structure, Bridge, Cold-Formed, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue


Each of the papers was subjected to stringent review by a panel of experts in the respective area. This peer review began with an assessment of the submitted abstracts and following this, authors were invited to submit their full manuscripts. Each manuscript was then carefully reviewed by relevant experts, and their recommendations on accepting, rejecting or modifying the submissions were strictly adhered to, before inclusion in the conference proceedings.
EXPERIMENTAL STUDY AND NUMERICAL ANALYSIS ON SEISMIC BEHAVIOR OF ASSEMBLED BEAM-COLUMN JOINTS WITH C-SHAPED CANTILEVER SECTION (ID NUMBER: 197)

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Abstract: A kind of assembled steel beam-column joint with C-shaped cantilever section was proposed. The influences of the lengths of cantilever sections and cover plates on seismic performance of the joints were discussed through low-cycle reciprocating loading tests and numerical simulations. Then the sensitivity analysis of key parameters such as thickness and width of flange plate, bolt number and cover plate’s length were carried out. The results show that the joint consumed energy through warping deformations of end plate and the friction slippages between flange of beam, C-shaped cantilever section and cover plate. By reasonably increasing the lengths of C-shaped cantilevers section and cover plates, it can ensure that the joints have high bearing capacities, while significantly improving energy dissipation capacities of the joints. Parameter analysis showed that increasing thickness of the flange plate can effectively improve the stress concentration at root of the cantilever section. Reducing width of flange plate has a great impact on bearing capacity and initial stiffness of the joint with the maximum drop amplitude of 13.1% and 18.9%, respectively.

Keywords: Assembled joints; Full bolted connection; Connection stiffness; Ductility; Failure mode; Stress concentration

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1 INTRODUCTION

At present, fabricated beam-column joints are mainly connected by H-shaped steel columns and H-shaped steel beams. Owing to the differences of the stiffness and stability of H-shaped steel column in two principal axis directions are great, it is not suitable for multi-rise and high-rise steel structures. On the contrary, the square steel column has good integrity and the stiffness is the same in two main axis directions. A large number of researches have been carried out on the connection between square steel column and steel beam. Liu et al. [1-2] proposed a new-type bolted truss-to-column joint and carried out experimental researches and the finite element numerical simulation, the results showed that the joint have good seismic performance and good static bearing capacity after the welding seams fracture. Some scholars at home and abroad have carried out a lot of experimental researches and finite element numerical simulation the connection of square steel column and H-shaped steel beam with the blind bolt [3-4], which indicated that the connections have a good hysteretic property but low bearing capacities. Through the experiment and the finite element simulation research, doung et al. [5] found that setting the internal diaphragm in the square steel column can effectively improve the connection
performance of the beam-column joint. Cao et al. [6] developed a new type of outer sleeve assembly between steel beam and square steel column, which has good energy dissipation and ductility through the numerical analysis. When the outer sleeve and the split bolt were combined to complete the connection of the square steel column and the H-shaped steel beam, the joint had a large rotation capacity due to the split effect of the split bolts while the energy consumption capacity decreased rapidly [7]. In addition, Málaga-Chuquitaype et al. [8] proposed a new type of bolted joint connected with combined channel/angle steel components. The connection had the advantages of simple construction and convenient installation, but the stiffness of its key connecting components is small, resulting in low bearing capacity. Jiang et al. [9-10] carried out low cyclic loading tests and finite element analysis on a new type of connection with a cantilever section and cover plates, it is found that the joints mainly dissipated energy through the slip of bolts and the plastic deformation of cover plates, showing better seismic performance. Sabbagh et al. [11] designed an I-beam-to-CHS column steel moment joint which can realize various energy dissipation mechanisms through flange cover plates and eliminate the problems about column web deformation and the outer diaphragm strain concentration. Sadeghi et al. [12-13] independently developed a new type of connection component for beam-column joints in modularized steel structures. It can be seen from experimental investigation and the numerical study that the connection was a fully resistant and semi-rigid connection, which was applicable to moment frames. Naserabad [14-15] designed and numerically study three support shapes on behavior of new bolted connection BBCC. The results showed that single beams support can meet all the AISC seismic provision requirements for special moment frames with and without a continuity plate. Song et al [16] conducted static and cyclic loading tests to investigated the mechanical performance of double-angle bolted steel I-beam to square column connections and evaluated the damage level of connections.

In order to realize the quick assembly of square steel columns and steel beams at site, ensure the joints have simple connection structure with good seismic performance, this paper proposed a new type of assembled H-shaped beam-square steel column joint with a C-shaped cantilever section. The welding work of the novel joint was completed in the factory, and the C-shaped cantilever section welded on the square steel column can be used as the installation support so as to improve the construction progress and reduce the construction difficulty, the cyclic loading tests were conducted to investigated the influences of the length of the cantilever section and the cover plate on mechanical properties of joint, and the failure mode, bearing capacity and sliding energy dissipation capacity of the joint were given. Combined with the results of the finite element analysis, the seismic performance of the joint is evaluated comprehensively.

2 EXPERIMENTAL PROGRAM

2.1 Composition of the novel joint

The new type of fabricated joint consists of a square steel column with a C-shaped cantilever section, a H-shaped steel beam welded with an end plate, one lower flange cover plate and high strength friction type bolt groups as shown in Figure 1. The production and installation process of the joint is as follows: each component of the connection is made in the factory, in which the C-shaped cantilever section and the square steel column are welded together by groove welding; the end plate is welded at the cutting part of the beam web and lower flange. At the installation site, the upper flange of the H-shaped steel beam is placed on the upper flange plate of the C-shaped cantilever section and the two are connected with high strength bolts. At the same time, the end plate is connected with the vertical plate of the C-shaped cantilever section by high-strength bolts, and the beam lower flange, the cantilever lower flange plate and the flange cover plate are finally connected by high-strength bolts.
2.2 Specimen design and material test

According to relevant stipulations of Code for Design of Steel Structure (GB50017-2017) [17] and Code for Seismic Design of Buildings (GB50011-2010) [18], three full-scale specimens were designed, recorded as EBJD-1, EBJD-2 and EBJD-3 respectively. All specimens met the seismic design requirements of “strong column and weak beam”, and the length of beam and column were 1600mm and 2000mm, respectively. The geometric dimensions of specimen EBJD-2 was shown in Figure 2. The lengths of the C-shaped cantilever sections and the lower flange cover plates were set as control parameters to assessed the influence on mechanical performance of the joints, and specific parameters of each test specimens are shown in Table 1. The welded square steel columns were adopted as steel columns of specimens, the cross-section dimensions were 350 × 16 mm. The section dimensions of hot-rolled H-shaped steel beams were 400 × 200 × 8 × 13 mm, and the thickness of the cantilever section web and the cover plate were 16mm and 20mm, respectively. The steel strength grade of all components were Q235B, and grade 10.9 M20 friction type high-strength bolts were adopted. A torque wrench was used to apply pretension about 155kN to high-strength bolts according to stipulations in Technical Regulations on High-Strength Structural Steel Bolted Connections (JGJ 82-2011) [19].
The standard specimens for material property tests were cut from the corresponding positions of the base metal according to relevant requirements of Steel and steel products—Location and preparation of test pieces for mechanical testing (GB/T2975-2018) [20], and the tensile tests were implemented according to the methods specified in Metallic materials—Tensile testing at ambient temperature part 1: Method of test at room temperature (GB/T228-2010)[21]. The values of specific material properties were shown in Table 2.

### Table 1: Basic parameters of specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column section /mm</th>
<th>Beam section /mm</th>
<th>Cantilever length /mm</th>
<th>Length of cover plate /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>EBJD-1</td>
<td>□ 350x16</td>
<td>H400x200x8</td>
<td>420</td>
<td>400</td>
</tr>
<tr>
<td>EBJD-2</td>
<td>□ 350x16</td>
<td>H400x200x8</td>
<td>320</td>
<td>400</td>
</tr>
<tr>
<td>EBJD-3</td>
<td>□ 350x16</td>
<td>H400x200x8</td>
<td>320</td>
<td>580</td>
</tr>
</tbody>
</table>

### Table 2: Test results of material properties of specimens.

<table>
<thead>
<tr>
<th>Thickness /mm</th>
<th>fy /MPa</th>
<th>fu /MPa</th>
<th>E /GPa</th>
<th>δ /%</th>
<th>fy/fu</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>261.54</td>
<td>461.12</td>
<td>203.32</td>
<td>28.11</td>
<td>0.57</td>
</tr>
<tr>
<td>13</td>
<td>278.93</td>
<td>440.59</td>
<td>204.56</td>
<td>29.78</td>
<td>0.63</td>
</tr>
<tr>
<td>16</td>
<td>259.40</td>
<td>402.83</td>
<td>208.71</td>
<td>36.56</td>
<td>0.64</td>
</tr>
<tr>
<td>20</td>
<td>252.80</td>
<td>440.50</td>
<td>206.16</td>
<td>33.85</td>
<td>0.57</td>
</tr>
</tbody>
</table>

### 2.3 Test loading scheme and loading device

The horizontal arrangement was adopted in this test. The left and right sides of the square steel column were hinge constrained by the fixing devices, and the axial pressure ratio of the column end was controlled at 0.2 by the hydraulic jack. The cyclic load at the beam end was applied by a 100t MTS actuator. Lateral supports were arranged on both sides of the actuator to prevent the out of plane instability of the test specimens caused by the eccentric loads at the beam end. The loading devices were shown in Figure 3.

![Figure 3: Loading device.](image)

The loading system was conducted through the seismic code of the AISC [22], the horizontal reciprocating load is applied, and the lateral displacement angle between layers was taken as the control parameter for loading. The loading system was shown in Figure 4. When one of the following phenomena occurred during the test, the loading ended: (1) obvious fracture occurred...
in the key parts of the specimen; (2) The bearing capacity of the beam end was reduced to 85% of the peak load.

2.4 Arrangement of displacement meter

The arrangement of thirteen displacement meters were shown in Figure 5. Displacement meters Dt1, DT2 and DT3 were selected to measure horizontal displacement at different positions of the beam flange. Displacement meters DT4~DT7 were respectively used to measure the vertical deformation of the column web, which were respectively arranged at the corresponding positions of the stiffeners set in the column. Displacement meters DT8 and DT9 were arranged at the intersection of the connection core area to measure the shear deformation of the relevant area. Displacement meters DT10~DT13 were respectively arranged at both ends of the end plate to measure the warpage deformation of the end plate.

3 EXPERIMENTAL PHENOMENA

3.1 Specimen EBJD-1

At the initial stage of loading, the specimen was in the elastic stage, and there was no other phenomenon except some noise on the beam web. When the angle displacement was -0.01rad, the upper flange of the beam began to yield, and few of extrusion deformation was occurred at the of the end plate. When the angle displacement reached -0.02rad, the end plate and the vertical plate of the C-shaped cantilever segment were separated, and a gap about 3mm can be observed obviously. The size of the gap increased continuously in the subsequent reciprocating
loading process with a maximum of about 10 mm, as shown in Figure 6(a). The plastic region of the beam upper flange extended along the beam length direction, as shown in Figure 6(c) when the connection rotation reached 0.03rad. When the load amplitude was 0.04rad, a loud noise came from the test specimen, and the connection weld between the lower flange plate of the C-shaped cantilever section and the column was torn through, as shown in Figure 6(b). At this time, the bearing capacity of the specimen EBJD-1 dropped sharply.

3.2 Specimen EBJD-2

The phenomena at the initial loading process of specimen EBJD-2 were basically the same as that of specimen EBJD-1, as shown in Figure 7. When the rotation displacement was -0.03rad, the web close to the end plate presented obvious yield area. During the process of 0.04 rad displacement loading, tear-out failure was generated at the weld between the column flange plate and the column web was torn, and the crack increased continuously in the subsequent loading process. When the rotation displacement of EBJD-2 was -0.0575rad, the width of the crack reached about 10 mm, the test specimen was considered to be damaged and the loading was stopped.

3.3 Specimen EBJD-3

The test phenomena of EBJD-3 were roughly the same as specimen EBJD-2, as shown in Figure 8. The primary difference between the two was that the warpage deformation of the end plate in specimen EBJD-3 was smaller than that of specimen EBJD-2. Moreover, the weld between the upper flange plate of the cantilever section and the square steel column occurred a crack when the rotation displacement was -0.03rad, and the crack was close to through flange plate when the angle displacement was -0.0475rad.

![Figure 6: Failure modes of specimen EBJD-1.](image)

![Figure 7: Failure modes of specimen EBJD-2.](image)
Figure 8: Failure modes of specimen EBJD-3.

4 ANALYSIS OF EXPERIMENTAL RESULTS

4.1 Hysteresis curves

The moment(M)-rotation(θ) relationship curves of specimens EBDJ-1, EBJD-2 and EBJD-3 were shown in Figure 9, respectively. It can be seen from before the failure of the joints, the hysteresis curves were relatively plump with a certain pinch at the zero point, almost shuttle shaped, which showed that although the novel joints had a certain friction and sliding, they had good energy dissipation capacity and bending bearing capacity as a whole. It can be seen from Figures 9(a) and (c) that the butt welds of specimen EBJD-1 and EBJD-3 broke through at the later stage of loading, resulting in obvious declining segments in the curves. The hysteresis curve of specimens EBJD-2 was shown in Figure 9(b), which indicated that there was obvious pinching phenomenon in specimen EBJD-2. It can be concluded that the hysteresis curves of EBDJ-1 and EBJD-3 were plumper than that of EBJD-2 by comparing the hysteresis curves of three specimens, indicating that the energy consumption capacity of the joints can be improved by properly increasing the lengths of C-shaped cantilever section and cover plate. Moreover, the positive and negative sizes of the hysteresis curves of three specimens were not completely symmetrical. The reason was that the connection stiffness difference between the upper and lower flanges of the joints, as well as the asymmetry of splicing, caused the plastic regions of the upper and lower flanges on steel beams asymmetrically.

Figure 9: Load-displacement curve of specimens.
4.2 Skeleton curves

Figure 10 shows the skeleton curves of each test specimens. It can be known that the trend of the three skeleton curves were approximately coincident. Take specimen EBJD-2 as an example, the specimen was in the elastic stage at the initial stage of loading, and the skeleton curve was basically linear. When the angle displacement of the joints was 0.01 rad, the skeleton curve began to enter nonlinear segment due to the elastic-plastic deformation of the end plate and beam flange plates. In the meantime, the slippage between the beam flanges, the C-shaped cantilever section and the lower flange cover plate occurred. The bearing capacity of the joint increased slowly, and the rigidity of the specimen gradually decreased. The skeleton curve showed a significant nonlinear relationship as the progress of the cyclic loading. There was no obvious separation of three groups of curves before the relevant welds presented the tear-out failure, indicating that the stiffness of each specimen was relatively close. Besides, the increase of the lengths of C-shaped cantilever section and the lower flange cover plate had little effect on the bearing capacity and the stiffness of the joint.

4.3 Stiffness degradation curves

The secant stiffness can effectively reflect the stiffness of the joint after entering the yield stage, which is an important index to measure the seismic performance of the joint. Figure 11 is the stiffness degradation curves of three specimens, of which abscissa was the rotation displacement of the joint, and ordinate was the secant stiffness of the test specimen, calculated according to the following equation [23]:

$$ K_i = \frac{|F_i^+| + |F_i^-|}{|\delta_i^+| + |\delta_i^-|} $$

where $F_i^+$ and $F_i^-$ are the peak loads in the positive (push) and negative (pull) direction of the i-th level circle respectively, $\delta_i^+$ and $\delta_i^-$ are the corresponding rotation angular of $F_i^+$ and $F_i^-$. 
It can be known from the curves that the stiffness degradation laws of three specimens were basically the same, which were consisted of straight-line decline segments and curve decline segments. Each specimen was in the elastic stage at the initial stage of loading, and the stiffness degradation was slow. When the rotation displacement was 0.01 rad, the secant stiffness of three specimens were in the straight-line decline segment owing to the plastic deformation of end plates and beam flanges. When the angular displacement was 0.015rad, the secant stiffness entered into curve decline segments, and finally tended to be horizontal, indicating that this kind of joint had good stiffness and bearing reserve, and it can ensure that the structure will not have excessive lateral displacement under the action of horizontal earthquake. In addition, it can be observed that the stiffness degradation curve of specimen EBJD-1 was generally higher than that of the other two specimens, which showed that increasing the length of the C-shaped cantilever can effectively improve the damage resistance of the joint and ensure the continuous energy consumption of the joint.

![Figure 11: Stiffness degradation curves.](image)

4.4 Displacement ductility coefficient

The "equivalent energy method" was used to determine the yield load and yield angle displacement of each test specimen, as shown in Figure 10. Table 4 exhibits comparison of bearing capacities and rotation angles obtained through tests of three specimens in different stages. The ultimate bending moment \(M_u\) in the table was 85% of the peak bending moment. However, \(M_u\) was taken as the ultimate bending moment \(M_{max}\) if there was no ideal decline segment in the skeleton curve, and the displacement ductility coefficient was calculated through the following equation:

\[
\mu = \frac{\theta_u}{\theta_y}
\]  

(2)

Table 4 shows that the displacement ductility coefficients of the three specimens were all greater than 3.0, indicating that the specimens had better plastic deformation capacity and meet the seismic design requirements. Moreover, it can be observed that the ductility coefficient of EBJD-2 was the largest, reaching 4.05. The analysis reasons were as follows: (1) during the whole loading process, the butt groove weld at the root of C-shaped cantilever section in specimen EBJD-2 did not present the brittle fracture, so that the bearing capacity and rotational capacity of this kind of joint were greatly exerted. (2) A large difference in the stiffness of the upper and lower flange of the joint was generated due to excessive increase of the cover plate’s length in specimen EBJD-3, which was easier to reduce the plastic deformation ability of the specimen.
Figure 12: Sketch of equivalent energy method.

Table 4: Ductility coefficient of specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading direction</th>
<th>Yield stage</th>
<th>Peak stage</th>
<th>Ultimate stage</th>
<th>Average of μ</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>My / (kN·m)</td>
<td>θy / 10-2rad</td>
<td>Mmax / (kN·m)</td>
<td>θmax / 10-2rad</td>
</tr>
<tr>
<td>EBJD-1</td>
<td>positive</td>
<td>314.85</td>
<td>0.79</td>
<td>428.49</td>
<td>2.72</td>
</tr>
<tr>
<td></td>
<td>negative</td>
<td>-375.15</td>
<td>-0.89</td>
<td>-489.41</td>
<td>-2.70</td>
</tr>
<tr>
<td>EBJD-2</td>
<td>positive</td>
<td>321.85</td>
<td>0.77</td>
<td>460.59</td>
<td>3.04</td>
</tr>
<tr>
<td></td>
<td>negative</td>
<td>-380.25</td>
<td>-1.13</td>
<td>-562.78</td>
<td>-4.69</td>
</tr>
<tr>
<td>EBJD-3</td>
<td>positive</td>
<td>327.88</td>
<td>0.83</td>
<td>448.95</td>
<td>2.89</td>
</tr>
<tr>
<td></td>
<td>negative</td>
<td>-368.18</td>
<td>-1.01</td>
<td>-507.34</td>
<td>-2.79</td>
</tr>
</tbody>
</table>

4.5 Energy-dissipating capacity

The equivalent viscous damping coefficient was used to measure the energy-dissipating capacity of three specimens, as shown in Figure 13, indicating that the energy-dissipating capacities of specimens EBJD-1 and EBJD-3 were higher than that of specimen EBJD-2, and the equivalent viscous damping coefficient of each test specimen was greater than 0.3. It can be concluded that increasing the lengths of the cover plate and the cantilever section can effectively improve the the energy-dissipating capacity of the joint, with better seismic performance. In addition, when the angle displacement was 0.015rad, the energy dissipation curves of the rest two specimens presented a decline segment except for specimen EBJD-1, which further showed that increasing the length of the cantilever section can improve the damage resistance of the joint.

Figure 13: Energy dissipation curves.
4.6 Rigidity analysis

According to the EC3 code [24], the initial stiffness of the joint in the elastic stage was calculated by bending moment(M)-rotation angle(θ) curves, which was used to evaluate the connection rigidity of the new type of fabricated beam-column joint, as shown in Figure 14. In Figure 14, the ordinate was $M / M_p$, the abscissa was $\theta / \theta_p$, and $\theta_p$ was calculated according to Equation 3.

$$\theta_p = \frac{M_p L_p}{EI_b}$$

(3)

Where $M$ is the bending moment, $M_p$ is the plastic bending capacity of the whole beam section, $\theta$ is the relative angle of the joint, $\theta_p$ is the theoretical value of the maximum plastic angle, $E$ is the elastic modulus of the steel, $I_b$ is the inertia moment of the beam section, and $L_b$ is the beam span.

From Figure 14, it can be known that the assembled beam-column joint with a C-shaped cantilever section was a semi-rigid joint with great initial stiffness.

5 FINITE ELEMENT ANALYSIS

5.1 Establishment of finite element models

Finite element software ABAQUS was used to simulate the reciprocating loading test of three specimens. C3D8R solid element was selected for steel beams, square steel columns, bolts and other components, as shown in Figure 15, which can better simulate the nonlinear characteristics such as slippage, extrusion deformation between plates and pretension force. Seeds were arranged densely around bolt holes for refining the meshes locally to improve the accuracy of the model. Interaction of "face-to-face contact" was defined between various components. The contact included two parts: contact between bolt rod and hole wall as well as contact between plates. The tangential contact was defined as Coulomb friction, the friction coefficient was taken as 0.4 according to the Literature [25], the normal contact was defined as hard contact, which allowed contact separation. In order to simplify the calculation model, the friction contact between nut and plate was not considered, and the degrees of freedom on the contact surface were constrained. Moreover, the connection between the cantilever section and the square steel column were directly replaced by Tie binding constraint, the end plate and the beam were combined into a whole by the merge command in ABAQUS to further simplify the calculation model.
Bilinear kinematic constitutive model was adopted for the stress-strain curve of the steel plate, considering the stress strengthening of 0.01E [25], the material properties were determined according to the material property tests. Von Mises yield criterion was adopted for calculation of the model, and the Poisson's ratio was 0.3. Moreover, the bolt pretension forces taken as 155kN were applied in three steps. The constitutive model was determined through Literature [25]. The boundary conditions and loading systems of the models were consistent with the tests except for the lateral restraint of the steel beam, which was set at 400mm away from the beam end to prevent the instability outside the beam surface.

![Finite element model](image)

**Figure 15: Finite element model.**

### 5.2 Stress analysis

The stress distribution of specimen EBJD-1 under different angular displacements were shown in Figure 16. When the angle was 1 / 250, the stress of the specimen was small, which was in the elastic stage. When the rotation displacement reached 1 / 100, the maximum stress close to the outermost row bolt holes on the beam flange plates were 234 MPa approximately, and there was stress concentration at the connection between the cantilever section and the steel column, which was consistent with the test phenomenon about the relative weld cracks. When the angle reached 1 / 50, the whole cross section of the beam on right side of the end plate almost yielded, while the steel plates of the joint domain were still in the elastic stage except for the root section of the C-shaped cantilever section, indicating that the plastic hinge was moved out. With comparison of Figure 16 and Figure 17, it can be concluded that the stress development law of specimen EBJD-2 was basically the same as that of specimen EBJD-1, which showed that increasing the length of C-shaped cantilever section had little effect on the bearing capacity of the joint.

The stress development process of specimen EBJD-3 was shown in Figure 18. Compared with the specimen EBJD-2, excessively increasing the stiffness of the lower flange cover plate caused the greater stress of lower column segment. When the angle was 1 / 100, the stress near the bolt holes of the outermost row on the top flange of the beam as well as was at the root cross section of cantilever section relatively great, and the maximum stress was about 235MPa. When the rotation angle is 1 / 50, the upper and lower flanges of the beam as well as some region of the beam web yielded, while the joint domain was still in the elastic stage. It was suggested to appropriately increase the length of cover plate on the basis of specimen EBJD-2, so as to reduce the overall rigidity of one side of the lower flange of the specimen and ensure the plastic deformation ability of the specimen according to the stress distribution of specimen EBJD-3.
Figure 16: Stress distribution of specimen EBJD-1.

Figure 17: Stress distribution of specimen EBJD-2.

Figure 18: Stress distribution of specimen EBJD-3.

5.3 Comparison of test and finite element results

The bending moment-rotation displacement curves obtained through the finite element analysis of three specimens were shown in Figures 9-10. With the comparison between test results and numerical simulation results, it can be found that curves of the two groups had the same change rule, the peak loads of the three specimens were close to the corresponding test values with little difference less than 15%, which showed that finite element models had a high calculation accuracy.

Besides, curves obtained through the finite element simulation were plumper than test results. The reason was that the slippage between components was affected by interactions between contact surfaces, resulting in no obvious pinch phenomenon in hysteresis curves. The main reason why the values obtained through the finite element analysis were slightly higher than
the test values due to without consideration of some unfavorable factors, such as the initial defects of materials, and the residual strain of welding.

6 PARAMETRIC ANALYSIS

The finite element parametric analysis of the new fabricated beam-column joint was conducted to reveal the influence of each parameter on the mechanical properties of the joint. The control parameters were mainly the thickness of flange plate $d_c$, width of the flange plate $l_c$, end plate’s thickness $t_e$ and length of lower flange cover plate $l_p$. The schematic diagram of each parameter was shown in Figure 19.

![Schematic diagram of each parameter](image)

Figure 19: Schematic diagram of each parameter.

6.1 Thickness of flange plate

Considering the importance of flange plate’s stiffness to the bearing capacity of the joint, TCB series specimens were designed to study the influence of flange plate’s thickness on the static performance of the joint, and the specific parameters were shown in Table 5.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$t_e$ / mm</th>
<th>$l_c$ / mm</th>
<th>$d_c$ / mm</th>
<th>$l_p$ / mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>TCB-1</td>
<td>16</td>
<td>350</td>
<td>13</td>
<td>400</td>
</tr>
<tr>
<td>BASE</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>400</td>
</tr>
<tr>
<td>TCB-3</td>
<td>16</td>
<td>350</td>
<td>18</td>
<td>400</td>
</tr>
<tr>
<td>TCB-4</td>
<td>16</td>
<td>350</td>
<td>20</td>
<td>400</td>
</tr>
</tbody>
</table>

Figure 20 shows the load-displacement curves of the joint in different thickness of the flange plate, from which it can be seen that when the thickness of the flange plate reduced from 16mm to 13mm, the curves presented obvious separation, and the reduction amplitude of the ultimate bearing capacity reached 3.5%, while the change amplitude of bearing capacity was less than 1% when flange plate’s thickness increased from 16mm to 20mm, indicating that thickness of flange plate should not be too small, but excessive increase of that had no significance to the bearing capacity and stiffness of the joint.
Table 6: Characteristic values of bearing capacity about TCB series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield stage</th>
<th>Δ</th>
<th>Ultimate moment</th>
<th>Δ</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_y$ / kN·m</td>
<td>$\theta_y$ / $10^{-2}$ rad</td>
<td>$M_u$ / %</td>
<td>$\theta_u$ / %</td>
</tr>
<tr>
<td>TCB-1</td>
<td>425.83</td>
<td>0.90</td>
<td>-1.55</td>
<td>-3.46</td>
</tr>
<tr>
<td>BASE</td>
<td>432.54</td>
<td>0.93</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>TCB-3</td>
<td>435.68</td>
<td>0.92</td>
<td>0.73</td>
<td>-0.4</td>
</tr>
<tr>
<td>TCB-4</td>
<td>433.28</td>
<td>0.91</td>
<td>0.17</td>
<td>-2.82</td>
</tr>
</tbody>
</table>

Figure 21 shows the stress distribution of TCB series specimens when the rotation displacement of each specimen reached 0.094 rad. The maximum stress at the connection between the C-shaped cantilever section and the column of four specimens was 307 MPa, 254 MPa, 231 MPa and 223 MPa respectively, which was 17.3%, 24.8% and 27.4% lower than that of specimen TCB-1 respectively. It shows that increasing the thickness of the flange plate can effectively improve the stress concentration at the root of the cantilever section to ensure the joint domain was in elastic stage.
6.2 Width of flange plate

The width of flange plate has great influence on the bending rigidity of C-shaped cantilever section. WCS series test pieces were designed, and the specific parameters are shown in Table 7.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$t_c$/mm</th>
<th>$l_c$/mm</th>
<th>$d_c$/mm</th>
<th>$l_p$/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>WCS-1</td>
<td>16</td>
<td>200</td>
<td>16</td>
<td>400</td>
</tr>
<tr>
<td>WCS-2</td>
<td>16</td>
<td>250</td>
<td>16</td>
<td>400</td>
</tr>
<tr>
<td>WCS-3</td>
<td>16</td>
<td>300</td>
<td>16</td>
<td>400</td>
</tr>
<tr>
<td>BASE</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>400</td>
</tr>
</tbody>
</table>

The load-displacement curves of joints in different flange plate widths were shown in Figure 22. The specific change values were selected in Table 8. When width of the flange plate reduced from 350mm to 200mm, the yield bearing capacity of each specimen reduced by 20.73%, 7.56% and 0.9% respectively, and the bending stiffness of the joint reduced by 18.1%, 9.8% and 5.8% respectively. The rotational displacement of rigid body in different degrees occurred in four specimens. The ultimate bearing capacities of the specimens reduced by 13.1%, 6.26% and 2.44% respectively with the decrease of the width of cantilever section. Therefore, width of the cantilever flange plate was of great significance to the bearing capacity of the joint, which is suggested to be same with column width.
Table 8: Characteristic values of bearing capacity about WCS series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield stage</th>
<th>$\Delta M_y / kN\cdot m$</th>
<th>$\Delta \theta_y / 10^{-2}$ rad</th>
<th>Ultimate moment $M_u / kN\cdot m$</th>
<th>$\Delta M_u / %$</th>
</tr>
</thead>
<tbody>
<tr>
<td>WCS-1</td>
<td>342.86</td>
<td>0.99</td>
<td>-20.73</td>
<td>483.90</td>
<td>-13.10</td>
</tr>
<tr>
<td>WCS-2</td>
<td>399.86</td>
<td>0.97</td>
<td>-7.56</td>
<td>521.97</td>
<td>-6.26</td>
</tr>
<tr>
<td>WCS-3</td>
<td>428.65</td>
<td>0.97</td>
<td>-0.90</td>
<td>543.25</td>
<td>-2.44</td>
</tr>
<tr>
<td>BASE</td>
<td>432.54</td>
<td>0.93</td>
<td>—</td>
<td>556.82</td>
<td>—</td>
</tr>
</tbody>
</table>

Figure 23 shows the stress distribution of WCS series specimens when the angle displacement reached 0.094 rad. All the other specimens had different degrees of rigid body angular displacement except for BASE specimen, and stresses at the root of the C-shaped cantilever section in four specimens were relatively large about 385 MPa, 340 MPa, 292 MPa and 253 MPa respectively. In addition, the plastic area gradually developed along the beam length direction with the increase of flange plate width, realizing the purpose of plastic hinge moving out.

6.3 Thickness of end plate

The TEP series specimens were designed to analyze the influence of end plate’s thickness on seismic performance of joints. Specific parameters were summarized in Table 9.

Table 9: Parameters of TEP series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$t_e$ / mm</th>
<th>$l_e$ / mm</th>
<th>$d_c$ / mm</th>
<th>$l_p$ / mm</th>
</tr>
</thead>
</table>

...
Figure 24 shows the load-displacement curve of variation of specimen bearing capacity with end plate thickness. The characteristic values of bearing capacity about TEP series specimens were shown in Table 10. It can be found that the bearing capacities of joints changed a little in different end plate thickness with the change amplitude of less than 2%. Therefore, the results showed that effect of end plate thickness on the bearing capacity of the joint was little.

![Figure 24: The effect of end plate thickness.](image)

Table 10: Characteristic values of bearing capacity about TEP series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield stage</th>
<th>$\Delta$</th>
<th>Ultimate moment</th>
<th>$\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_y$ / kN·m</td>
<td>$\theta_y / 10^{-2}$ rad</td>
<td>$M_u / M_y$</td>
<td>$\theta_u / \theta_y$</td>
</tr>
<tr>
<td>TEP-1</td>
<td>417.82</td>
<td>0.95</td>
<td>-3.40</td>
<td>2.15</td>
</tr>
<tr>
<td>BASE</td>
<td>432.54</td>
<td>0.93</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>TEP-3</td>
<td>436.53</td>
<td>0.93</td>
<td>0.92</td>
<td>0.00</td>
</tr>
<tr>
<td>TEP-4</td>
<td>438.87</td>
<td>0.94</td>
<td>1.46</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Figure 25 shows the stress distribution of TEP series specimens when beam end angles of four specimens reached 0.094rad. With comparison of each specimen stress development, it can be seen that the stress of upper flange of the beam in specimen TEP-1 was the largest, about 329.1MPa. In addition, reducing the thickness of end plate led to the rise of the stress value at the root of the cantilever section, and there was no significant effect on stress development of the specimens by increasing thickness of end plate from 16mm to 20mm. It was suggested that thickness of end plate should not be less than 16mm.

![Figure 25: Stress distribution of TEP series specimens.](image)
6.4 Length of cover plate

LCP series specimens were designed to analyze influence of cover plate length on bearing capacity of the joint, and the specific parameters were shown in Table 11.

Table 11: Parameters of LCP series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( t_c ) / mm</th>
<th>( l_c ) / mm</th>
<th>( d_c ) / mm</th>
<th>( l_p ) / mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCP-1</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>360</td>
</tr>
<tr>
<td>LCP-2</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>400</td>
</tr>
<tr>
<td>BASE</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>440</td>
</tr>
<tr>
<td>LCP-4</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>480</td>
</tr>
<tr>
<td>LCP-5</td>
<td>16</td>
<td>350</td>
<td>16</td>
<td>520</td>
</tr>
</tbody>
</table>

Figure 29 shows the load-displacement relation curves of joint bearing capacities with change of cover plate length. Characteristic values of bearing capacity about LCP series specimens were given in Table 12. It can be known that the straight line segments of the curves overlapped each other, indicating that increasing length of cover plate had little effect on the initial stiffness of the joint without variation of other parameters, and the bearing capacities of the joints had an upward trend on the whole with the increase of the length of the cover plate with an increase amplitude less than 3%, which meant that increasing the length of the cover plate had little effect on the bearing capacity of the joint.
Table 12: Characteristic values of bearing capacity about LCP series specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield stage</th>
<th></th>
<th>Ultimate moment</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_y / kN\cdot m$</td>
<td>$\theta_y / 10^2 \text{rad}$</td>
<td>$M_u / kN\cdot m$</td>
<td>$\theta_u / %$</td>
</tr>
<tr>
<td>LCP-1</td>
<td>431.02</td>
<td>0.94</td>
<td>-0.35</td>
<td>1.07</td>
</tr>
<tr>
<td>BASE</td>
<td>432.54</td>
<td>0.93</td>
<td>0.92</td>
<td>0.00</td>
</tr>
<tr>
<td>LCP-3</td>
<td>436.52</td>
<td>0.93</td>
<td>1.85</td>
<td>0.00</td>
</tr>
<tr>
<td>LCP-4</td>
<td>440.56</td>
<td>0.93</td>
<td>2.61</td>
<td>1.07</td>
</tr>
<tr>
<td>LCP-5</td>
<td>443.84</td>
<td>0.94</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 29: The effect of cover plate length.

Figure 30 gives stress distribution of LCP series specimens when the rotation angle of the beam end of each specimen reached 0.094 rad. As can be seen from Fig.28, the plastic regions of lower flange of the new fabricated joints mainly generated at one end of cover plates, which were gradually moved outward along the direction of beam length with increase of cover plate length. In addition, when the length of the cover plate was 360mm and 400mm respectively, stress distribution of relative specimens were similar. The column webs were in the elastic stage, and the maximum stress of the joint domains were all at range of 220MPa-250MPa. When the length of the cover plate increased from 440mm to 520mm, the maximum stresses in the node domains were at range of 220MPa-250MPa. Besides, the stress values at the root of the cantilever section gradually increased and spread outward as the length of cover plate increased. Therefore, it is suggested that the lower flange cover plate should not be too long.
6 CONCLUSIONS

Low cycle repeated loading tests and finite element simulation analysis of the fabricated beam-column joints with a C-shaped cantilever section were carried out in this paper to investigate the mechanical properties of the novel joints, and the following conclusions were obtained:

(1) The slippage of the joint mainly occurred between flange plates of the steel beam, the C-shaped cantilever section and the cover plate. The energy-dissipating capacity of the joint was realized through the plastic deformation of the end plate and the steel beam.

(2) The bearing capacity and stiffness of the joint was not affected with increase of the lengths of the C-shaped cantilever section the lower flange cover plate, while energy-dissipating capacity of the joint can be effectively improved, and the plastic hinge on the steel beam was guaranteed to move out, showing good seismic performance.

(3) The fabricated beam-column joint belonged to semi-rigid joint. The ductility coefficient of three test specimens were all more than 3.0, and the equivalent viscous damping coefficient were at range of 0.3 ~ 0.36, which indicating that the joint had good ductility and energy dissipation capacity.

(4) Increasing thickness of flange plates of the cantilever section can effectively improve the stress concentration at the root of cantilever section, reduce the actual stress in the node domain, achieving the seismic design requirement of "strong joint, weak component".
(5) The flange plate width had a significant impact on the initial stiffness and ultimate bearing capacity of the joint, the maximum variation amplitude was 20% approximately, while the joints appeared obvious rigid body rotation, resulting in reducing the ductility of the joints. Therefore, it was suggested that the width of flange plate should be same with column width.

(6) Thickness of end plate and length of lower flange cover plate had little influence on the bearing capacity and stiffness of the joint with the change amplitude of not exceeding 3%.

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REFERENCES


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