Study on seismic behavior of reinforced concrete column-steel beam side joints

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\textbf{ABSTRACT}
In this paper, a steel beam insert RCS side joint specimen is designed, and the low-cycle reciprocating loading test has been studied by designing a similar form of side joint specimen. At the same time, the hysteretic curve, skeleton curve, ductility, energy consumption, stiffness, bearing capacity degradation, and other seismic performance indexes are studied. The test results show that the failure mode of the specimen under the low-cycle reciprocating loading is the failure of the steel beam plastic hinge, which conforms to the seismic conceptual design principle of “strong column and weak beam”. The hysteretic curve of the specimen is relatively full, and the equivalent viscous damping coefficient is higher than that of ordinary reinforced concrete joints, showing better ductility and energy dissipation capacity. On the basis of experiments, Combined with ABAQUS finite element software, the variables of five parameters including insert length of steel beam, width of steel beam end plate, height of beam, axial compression ratio and embedded carrier board are analyzed, and the influence of these factors on the seismic performance of joints is also researched. By the method of variable analysis, the optimum value of each parameter is provided, which provides a reference for joint design.

\textbf{1. Introduction}
In the 1980s, the United States developed a composite frame consisting of reinforced concrete column and steel beam, in short of RCS. Reinforce concrete and steel show their advantages in RCS composite frame as two traditional building materials: steel has high strength, good plasticity and toughness, good ductility and tensile strength. At the same time, steel beam has good bending resistance, strong ductility and energy consumption, and light weight, which can effectively reduce the beam section height, is suitable for large-span structures and save the formwork, easy to construct. Reinforced concrete columns are easy to take, and have high rigidity, good fire resistance and durability. As a column member, it is also more excellent in stability.

Foreigner scholars started earlier on RCS composite joint research. Sheikh, Deierlein, and Yura et al. (1989a, 1989b) conducted static and reciprocating loading tests on 15 2/3 ratio RCS composite frames commonly used in 1989, including face-supporting plates, embedded carrier plates, overhang type surface bearing plates, built-in steel column type and built-in stud type, etc., studied the influence of node failure mode, strength, stiffness and structural measures of the core area of the node on the mechanical performance of the joint. Kanno (1993) carried out low-cycle reciprocating loading seismic tests of 11 RCS composite structural joints in 1993. The following four failure modes are obtained, namely plastic hinge failure of steel beam, concrete shear or pressure failure in joint zone, concrete shear failure in joint zone and plastic hinge failure of steel beam occur simultaneously, and plastic hinge damage at the end of the column. Bugeja, Bracci, and Moore (2000) carried out low-cycle reciprocating loading tests on six 2/3 ratio RCS space nodes with concrete slabs. The main considerations are the influence of concrete slabs, face plates, pre-buried steel columns, coamings, and cover plates on the seismic performance of the joints. At the same time, the nodes are designed according to the principle of \textit{strong columns and weak beams}. The experimental results show a good seismic performance for the RCS composite joint. And the failure mode is basically the plastic hinge failure of the steel beam. In addition, the test also shows that the effective width of the slab in the joint is basically consistent with the calculation method of AISC-LRFD(1993). Parra-Montesinos, Liang, and Wight (2003) conducted experimental research on two specimens, which showed that the effective design can control the deformation mode and failure mode of the joint, which made the plastic hinge first appeared at the beam end in RCS frame.

Domestic scholars’ research on RCS composite nodes basically began after 2000. Jianjiang and Zhijun (2001) carried out low-cycle reciprocating loading tests on four beam-through RCS nodes, studied the strength and
deformation properties of the joints. And the parametric analysis is carried out for the axial compression ratio, beam-column section size and joint construction measures. And the formula for calculating shear capacity of RCS composite nodes is proposed. Chen Lin (2003) carried out a low-cycle reciprocating loading test on four 1/3 RCS nodes in the context of actual engineering, studied the hysteretic curve, ductility performance and shear capacity of the batch of nodes. The simulation results are compared with the test results, and the mechanical model of the RCS combined node is constructed. Xiaolei, Guoliang, and Weizhong et al. (2008) used ANSYS finite element software to calculate the mechanical properties of the beam-through RCS nodes with different structural forms and axial compression ratios. Men Jinjie, Huijuan, and Xiaodan et al. (2014) used ABAQUS finite element software to calculate and analyze RCS composite nodes of six different structural forms, including face plate, cylindrical steel plate, flat steel hoop, extended surface bearing plate, X cross reinforcing steel and the orthogonal short beam. Comparing the results with several existing formulas for calculating shear capacity of RCS composite nodes, it is found that the existing shear capacity formula has greater limitations and cannot accurately reflect the design requirements of composite nodes with different structures. And more research and improvement are

Figure 1. RCS node design.
needed. Literature (Jinjie, Jun, and Qiuwei et al. 2014; Jinjie, Zhifeng, and Xuanshi et al. 2014; Jinjie, Peng, and Zhifeng et al. 2015) conducted pseudo-static tests and OpenSees finite element analysis on 6 RCS composite nodes, focusing on studying the impact of different structures on the seismic performance of RCS composite nodes and built resilience models of different structures based on the test results. Ling Yuhong, Jianjia, and Hongwei et al. (2016) put forward a kind of new steel secondary beam – concrete main beam penetration joint of which the steel secondary beam welds the anchor end plate at the end, and then full penetration into the concrete main girder. The static loading test was carried out on the joint specimen, and the test results show that the failure is manifested as the tensile yield of the upper flange of the secondary steel beam, and the flexural capacity of the secondary steel beam can be fully exerted. During the trial, the joint area is well anchored and the joint has good deformation capacity. Huang Xinhuang (2017) carried out the pseudo-static test of specimen under different axial pressure ratio for RCS space beam-column composite with common bolt connected to the end plate, acquired the influence of axial pressure ratio on the anti-seismic property of the space beam-column composite, and proposed the restoring force model of this type of RCS beam-column composite.

The above studies have confirmed that the RCS combination node has good anti-seismic performance. According to different needs to adopt different construction and construction technology, the structure can have good comprehensive performance. However, no systematic normative provisions have been formed due to the current research status, so it is necessary to further study the mechanical and seismic performance of RCS composite structures. In addition, although some node structures can improve the seismic performance of the nodes, the node structure is too complex. So it is necessary to seek a simple structure, convenient construction of the node.

In addition, some scholars have pointed out that the research methods of modern structural engineering have been extended from the previous theoretical plus test (including engineering practice) to the combination of theory plus test plus calculation. Therefore, this paper uses the research method combining experiment and finite element analysis, which is conducive to improving research efficiency.

In this paper, it proposes a new style of plug-in RCS edge node, which adopts the structural form that the steel beam is directly inserted into the concrete column, and the hoop is inserted through the hole at the web of the steel beam for the placement of the column stirrup. The advantage of this node is that the structure is simple, the construction is convenient, and it has good mechanical performance according to the existing domestic engineering projects such as Meizhou World Merchants Cultural Center. Therefore, the seismic performance of the nodes with such structures is tested and simulated in the study.

2. Experiment design

2.1. Node design

The steel beam steel is made of Q345 steel and has a length of 1.9 meters, of which 0.3 meters are inserted into the reinforced concrete column, and the end of the cantilever end is 0.1 meter. The section size of the steel beam is \( H_1 \times B_1 \times T_1 \times B_2 \times T_2 \times t_w = 400 \times 120 \times 10 \times 120 \times 10 \times 6 \) (unit: mm). Considering that the stirrup of the concrete column will be cut off by the steel beam. In the production stage of the test piece, the corresponding round holes are pre-positioned on the steel beam for the hoop to pass through. At the same time, three transverse stiffeners are added in the longitudinal direction of the steel beam to prevent the steel beam from being unstable or distortion damage during loading. The thickness of the stiffener is \( T = 6 \) mm. In addition, an end plate is added to the end web of the steel beam joint zone to enhance the joint stiffness. The end plate width is 120 mm, and the end plate thickness is 6 mm.

The concrete column section size is 400 mm × 400 mm, the column concrete strength grade is C30, the concrete protection layer thickness is 20 mm, the reinforced concrete column receives the axial force, the bending moment and the axial force transmitted from the steel beam. The column produces an axial force of 0.3 axial compression ratio \( N = 0.3fc_A = 686.4\text{kN} \). The axial force transmitted by the steel beam to the column can be calculated from the design value of the beam end moment and the length of the steel beam. After obtaining the bending moment design value and the axial force design value of the column, according to the formula of the compressive bearing capacity of the rectangular section eccentric compression member in Code for design of concrete structures (GB 50010-2010 2015), the column reinforcement can be obtained. The RCS node design is shown in Figure 1.

2.2. Test loading and measurement

The loading device of this test is the MTS loading device of the structural laboratory of South China University of Technology. The upper and lower ends of the column are hinged for the purpose of hinge. The lateral support is applied on both sides of the steel beam to prevent the steel beam from being twisted and out-of-plane. The top of the column is subjected to axial pressure by hydraulic jacks. Finally, a low-cycle
A reciprocating load is applied to the loading end of the steel beam through the MTS servo loading system. Figure 2 is the sketch map of the test device.

The test reciprocating loading system refers to the relevant provisions of the *Specification of Testing methods for Earthquake Resistant Buildings* (JGJ101-2015) and the Seismic Provisions for Structural Steel Building in the United States (AISC/AISC 341-10-2010). Considering that the calculation of the yield load may have deviated, and the loading device is not easy to control in the force control mode, the test uses displacement control to load. Combined with the trial calculation, it is determined that the reciprocating cycle is performed once the beam end displacement is 5 mm and 10 mm. After the beam end displacement reaches 15 mm, the member approaches the yielding state, the displacement increment is 15 mm. The load of each stage is repeated three times, the displacement is gradually increased until the test piece is broken. Figure 3 shows the beam end loading system.

The test measurement contents include column top load, beam end load, loading point displacement and beam-column angle. Figure 4 is a schematic diagram of the displacement meter arrangement.

### 3. Seismic performance analysis of test specimens

#### 3.1. Test phenomena and failure modes

When the loading displacement reached 10 mm, the steel beam was in the elastic stage. The first fine horizontal micro-crack appeared at the joint of the steel beam, which was caused by tension, as shown in Figure 5. When the loading displacement was between 15 mm and 30 mm, a large number of cracks
began to appear on the connection surface, and the transverse cracks extended to other surfaces, while the oblique cracks continued to develop on each surface. At the same time, some oblique cracks intersected due to the opposite loading direction, as shown in Figure 6. Due to the development of cracks, the load-displacement curve decreased, and the steel beam was about to enter the yield stage. When the loading displacement reached 26/-22 mm, the load-displacement curve showed an obvious inflection point, and the steel beam entered the yield phase. The original crack continued to develop and new cracks appeared. The maximum crack width was 0.3 mm. At the same time, the concrete surface at the joint cracked obviously, with small pieces peeling off. Cracks continued to develop as the load continued. When the loading displacement reached 80 mm, the load was lower than the maximum load of 60 mm cycle. It can be considered that after the specimen reached the peak load, the hysteresis curve entered the descending section. Then stopped loading. At this time, after buckling of the compression flange of the steel beam, there was a large deformation, the steel beam web also appeared to bulge, the concrete at the joints was spalling, and the maximum crack width was 0.54 mm. However, the steel beam did not pull out, which proved that the anchorage state was good. The ultimate failure mode was the plastic hinge failure of the steel beam, as shown in Figure 7.

Figure 4. Schematic diagram of the displacement meter arrangement.

Figure 5. 10 mm load level.

Figure 6. 15 mm load level.
3.2. Hysteretic curve and skeleton curve analysis

The hysteretic curve shows the stiffness variation and bearing capacity variation of the structure during the stress process (Parra-Montesinos, Liang, and Wight 2003). According to the Specification of Testing methods for Earthquake Resistant Buildings (JGJ101-2015 2015), the envelope curve of the structure connected to the peak of the first cycle of each loading stage is utilized to extract the skeleton curve of the structure. And the stiffness degradation, bearing capacity degradation and energy consumption change characteristics of the structure are analyzed. Figures 8 and 9 show the hysteretic curve and skeleton curve of the test piece. From the figure, we can draw the following conclusion: the hysteretic curve of the beam-column joint specimen develops in a bow shape, and the skeleton curve changes in an S-shape, which is roughly the same as the regular node, and can be divided into the following four stages: During the elastic stage, there is almost no change in the stiffness of the specimen; in the cracking stage, the concrete cracking leads to a decrease in

![Figure 7. Displacement of the steel beam.](image)

![Figure 8. Load-displacement curve of the test piece.](image)

![Figure 9. Skeleton curve of the test piece.](image)
stiffness; in the yielding stage, the end flange portion of the steel beam first yields, and the slope of the skeleton curve is significantly reduced; in the failure phase, when the 80 mm loading displacement level is reached, you can see that the hysteresis curve has fallen back. It is judged that the specimen reaches the peak load. The skeleton curve falls back, and the bearing capacity begins to decrease, at this time the specimen enters the destruction phase and finally reaches the limit state.

### 3.3. Displacement ductility analysis

In the seismic design of the structure, the structure will have great deformation ability in the later if it has good displacement ductility, and can absorb certain energy after reaching the yield or maximum bearing capacity to avoid the occurrence of brittle failure. This important index is generally expressed by the ductility coefficient. In this paper, the displacement ductility coefficient $\mu$ of the steel beam loading end is used to study the ductility characteristics of the joints. The calculation formula is shown in Equation (1).

$$\mu = \frac{\Delta u}{\Delta y}$$

In the formula, $\Delta y$ is the yield displacement of the specimen, and $\Delta u$ is the ultimate displacement of the specimen.

In this paper, the energy equivalent method (Ping, Kaicheng, and Xiongwei 2011) is adopted to confirm the yield point of the test piece, and since the falling section of the skeleton curve does not reach 85% of the peak point in this test, the final state measured by the test is selected as the reference value of the limit state. The measured values of load-displacement at each stage of the test piece are shown in Table 1.

Table 2 shows the values of calculation of the ductility coefficients for this joint.

### 3.4. Stiffness degradation analysis

The stiffness degradation reflects the increasingly poor ability of the structure to resist deformation. In this test, the ring stiffness ($K_j$) under the displacement amplitude of the same level is used to reflect the stiffness degradation law of the specimen with the change of the load displacement level ($\Delta/\Delta y$). $K_j$’s formula is given by Equation (2).

$$K_j = \frac{\sum_{i=1}^{n} P_j}{\sum_{i=1}^{n} u_i}$$

Where $P_j$ is the peak load value of the $i^{th}$ loading cycle when the loading displacement level is $j$; $u_i$ is the peak point deformation value of the $i^{th}$ loading cycle when the loading displacement level is $j$; $n$ is the number of cycles.

Figure 10 shows the stiffness degradation curve of the RCS node test piece. At the initial stage of loading, the stiffness degradation curve is steeper with the increase of loading displacement, indicating that the specimen has obvious stiffness degradation before yielding. After the steel beam begins to yield, the stiffness degradation rate of the specimen decreases. The curve tends to be gentle, and the whole exhibits the property of plastic deformation. After reaching the ultimate bearing capacity, the stiffness of the specimen degrades to approach zero.

### 3.5. Bearing capacity degradation analysis

The bearing capacity degradation curve reflects the influence of cumulative damage caused by plastic development on the degradation of bearing capacity. The same load carrying capacity degradation curve used in this test is the ratio $\lambda_j$ of the peak point load obtained from the last cycle of the same loading stage to the peak point load obtained by the first cycle of the stage under the condition that the displacement amplitude is constant. The curve is shown in Figure 11.
When the loading displacement increases, the $\lambda_j$ of the test piece has a slight decrease. The bearing capacity is basically unchanged before the steel beam didn’t yield at the initial stage of loading. When the specimen is loaded to the end of the steel beam and the flange is yielded, and the deformation is large, the bearing capacity under the same loading can be obviously reduced, but the overall change is smaller, and the $\lambda_j$ of the test piece keep above 0.9, indicating that the node has good resistance to damage.

3.6. Energy consumption analysis

The power consumption performance of the node is related to the size of the area surrounded by the hysteresis curve. The better energy consumption performance of the component with larger hysteresis loop enclosing area, the more energy dissipates of the structure, the less likely of the structure will be destroyed. In this paper, the equivalent viscous damping coefficient $h_e$ is used to evaluate the energy dissipation capacity of the node, which is defined by Equations (3) and (4) and Figure 12.

$$\hbar = \frac{E_d}{2\pi}$$  \hspace{1cm} (3)

$$E_d = \frac{S_{ABC} + S_{CDA}}{S_{OBE} + S_{ODF}}$$  \hspace{1cm} (4)

Among them, the energy dissipation coefficient $E_d$ is the ratio of the total energy of the hysteresis loop of the test piece to the elastic energy, $S_{ABC} + S_{CDA}$ is the actual area under a complete hysteresis loop of the test piece, and $S_{OBE} + S_{ODF}$ is the elastic energy.

Figure 13 shows the equivalent viscous damping coefficient $h_e$ of the RCS node specimens at different beam end displacements. The equivalent viscous damping coefficient $h_e$ of the test joint member is 0.26 when the peak load is reached. The equivalent viscous damping coefficient of the reinforced concrete joints is generally about 0.1. And the equivalent viscous damping coefficient of the section steel concrete joints is generally about

![Figure 10. Stiffness degradation curve of the test piece.](image1)

![Figure 11. Bearing capacity degradation curve of the test piece.](image2)
Figure 12. Schematic diagram of calculation method of energy dissipation coefficient $E_d$.

Figure 13. Equivalent viscous damping coefficient.

Figure 14. Concrete uniaxial tension and uniaxial compression stress-strain relationship curve.
Therefore, the equivalent viscous damping coefficient can reflect that the steel beam inserted RCS edge node used in this test has good energy dissipation capacity.

Figure 15. Steel constitutive curve.

Figure 16. Comparison of calculation results and test results.

0.3 (Qijing 1991). Therefore, the equivalent viscous damping coefficient can reflect that the steel beam inserted RCS edge node used in this test has good energy dissipation capacity.
4. Numerical simulation study

4.1. Numerical simulation reliability verification

4.1.1. Model establishment

In this paper, the concrete plastic damage model in finite element software is used for calculation and analysis. The concrete and steel beams are simulated by C3D8R unit, an eight-node hexahedral linear reduced-integral three-dimensional solid element. Under the same grid density, its calculation accuracy is more accurate than that of the complete integral unit. However, the stress characteristics of the steel bars are only axial tension and no bending moment. So two-node three-dimensional truss element T3D2 is adopted for simulation.

In terms of constitutive relations of materials, the proposed formulas in Appendix C.2.3 and C.2.4 of code for design of concrete structures (GB 50010-2010 2015) are adopted to construct the constitutive relationship of concrete. The uniaxial tension constitutive curve and uniaxial compression constitutive curve are shown in Figure 14. Concrete needs five material parameters defined by the user, which are: a) Dilation Angle adopts the value of 30°. b) the Eccentricity uses the default value of 0.1. c) the biaxial and uniaxial initial yield strength ratio $f_{0d}/f_{0c}$ adopts the default value of 1.16. d) the ratio $K$ of the second stress invariant on the tension and compression meridional plane adopts the default value of 0.6667. e) the Viscosity parameter adopts the value of 0.005. The constitutive model of steel adopts the double fold line follow-up strengthening model, and the constitutive curve is shown in Figure 15. According to the actual measurement, the yield stress of steel is 400MPa and the ultimate stress is 600MPa.

In the Boundary Condition module, hinged constraints were applied to the top and bottom of the column, only the rotation in the XZ plane direction of the component and the Z-direction displacement under the pressure of the column top axis were released. At the same time, a reference point is added above the loading end of the steel beam, which is coupled with the surface of the beam end pad (that is, the loading surface) for the subsequent application of reciprocating loads at the loading end.

In terms of meshing, this paper focuses on the stress and strain development of the core region of the node and the plastic hinge region of the steel beam. The calculation accuracy, convergence and calculation efficiency are comprehensively considered. After many trials, the mesh size of the steel beam is taken as 30 mm. The grid

![Figure 17. Simulation analysis of steel beam insertion length.](image-url)
The size of the concrete column is taken as 40 mm. The concrete area intersecting with the web and the flange of the steel beam is subdivided according to the corresponding thickness. The grid size of the steel is also 40 mm.

When the interface interaction is set, the steel frame is embedded into the concrete column using the Embedded command. The contact between the steel beam and the stiffener is tied together using the tie command without relative displacement. The contact between the steel beam and the concrete column in the joint region uses the tangential behavior and normal behavior in the Interaction module to define the contact relationship between them. The tangential behavior is defined by the penalty function, and the friction coefficient is taken as 0.2, normal behavior is defined as a hard contact.

4.1.2. Comparison of finite element results with experimental results

Figure 16 is a comparison of the hysteresis curve, the skeleton curve, the stiffness degradation curve, and the bearing capacity degradation curve obtained from Figure 18.

**Figure 18.** Simulation analysis of steel beam end plate width.
the finite element calculation results and experimental results.

It can be seen from the above figure that the hysteresis loops and the skeleton curve of the two are completely coincident in the initial elastic phase of loading, and as the loading displacement increases, the calculated value and the experimental value of each loading level are basically consistent, and the error is small. At the same time, the equivalent viscous damping coefficient of the simulated node and the experimental node is the same, which indicates that the ABAQUS finite element simulation can accurately reflect the energy dissipation capacity of the node.

The maximum error of the stiffness degradation curve of the two is 8%, and the maximum error of the bearing capacity degradation curve is 3.4%, which indicates that the finite element simulation can accurately reflect the plastic development of the joint, and the material constitutive model selection is reasonable.

The reason for the termination of the test is that the steel beam is twisted, and the finite element analysis results show that the steel beam still has not reached the limit state at the 60 mm displacement level, and more load levels should be set in the subsequent finite element analysis.

4.2. Study on the influencing factors of seismic performance of joints

4.2.1. Selection of influence factors

After verifying the reliability of the finite element model, five kinds of research parameters were selected, and their computational models were established by using the experimental nodes as prototypes. The control variable method is used to study the aseismic performance. Table 3 lists the basics of the five research parameters.

| Research Factor                      | Variable control situation                                                                 |
|--------------------------------------|-------------------------------------------------------------------------------------------|
| Steel Beam Insertion Length          | 200, 250, 300, 350 (mm)                                                                   |
| Steel Beam End Plate Width           | 0, 120, 170, 270 (mm)                                                                     |
| Steel Beam Height                    | 400, 450, 500 (mm)                                                                        |
| Axial Compression Ratio              | 0.3, 0.4, 0.5, 0.6, 0.7                                                                   |
| Embedded Carrier Board               | Increase carrier plate construction                                                       |

The concrete column section size is 400 × 400 mm, and the steel beam flange width is 120 mm.

4.2.2. Steel beam insertion length analysis

Figure 17 shows the hysteresis curve and stress cloud of the steel beam with different insertion lengths. When the steel beam insertion length is increased from 200 mm to 250 mm, the bearing

Table 3. Research factors summary.

![Figure 19. Steel beam height analysis.](image)
capacity of the joint is increased by 33%, the hysteresis curve of the joint becomes fuller, and the plastic deformation ability is enhanced. However, as the insertion length exceeds 300 mm, the increase of the node performance becomes inconspicuous. And from Figure 17(b), as it shows that when the insertion length is 200 mm, the steel beam flange first begins to yield, and the web stress is small, the material strength is not fully exerted, and the insertion length is too small, the steel beam is easily pulled out, so the recommended length of the steel beam insertion is 250 to 300 mm (i.e. 5/8 to 3/4 of the column width).

4.2.3. Steel beam end plate width analysis
Figure 18(a,b) shows the hysteresis curve and stiffness curve obtained by simulating different widths of the steel beam end plate. The increase of the width of the steel beam end plate can greatly improve the stress performance and seismic performance of the joint. However, when the end plate exceeds 120 mm (i.e., the flange width), the node performance does not change significantly with the increase of the end plate width. Figure 18(c) shows that when the width of the end plate is the same as the width of the flange, the end plate and the flange are in good joint with uniform force. When the width of the end plate is 170 mm, stress concentration tends to occur in the angular region. And when the width of the end plate is expanded to 270 mm, The effect on stress does not continue to increase. The increase in the width of the end plate has no obvious effect on reinforcement stress. The recommended value for the end plate width is the same as the steel beam flange width.

4.2.4. Steel beam height analysis
Figure 19 is the hysteresis curve and bearing capacity degradation curve that simulated in different insertion lengths. Due to the change of the section size of the steel beam, the moment of inertia of the steel beam will increase, which will inevitably lead to a significant increase in the bearing capacity and stiffness of the steel beam, an increase in the hysteresis loop, an increase in the energy consumption of the joint. Moreover, the increase in the height of the steel beam has no obvious adverse effect on the bearing capacity degradation of the component. Therefore, in the case of insufficient bearing capacity, the node design can be carried out by increasing the height of the steel beam. For the node of this paper, it can be larger than 500 mm (1.25 times of the column width), and the aspect ratio of the steel beam is greater than 25/6, which also meets the application requirements of the node in the large-span structure.

4.2.5. Axial compression ratio analysis
Figure 20 is the hysteresis curve and stiffness curve that simulated in different axial compression ratios. When the axial compression ratio is less than 0.7, an increase of the axial compression ratio has tiny influences on the hysteresis curve and stiffness degradation curve, only the increase of column longitudinal reinforcement stress can be inspected. In the following research, it is possible to increase the axial compression ratio and decrease column longitudinal reinforcement stress setting, observing whether the concrete in the joint zone is more prone to shear and pressure damage and whether the ductility of the joint is reduced.

4.2.6. Embedded carrier board analysis
Embedded carrier board is one of the common structural forms of RCS composite nodes. It adds a surface bearing plate at the side of the core area, whose height is the same as that of the steel beam, and width is the same as that of the flange of the steel beam. The embedded carrier board structure enables the joints to form concrete diagonal bars within the width of the flange. Figure 21 is the hysteresis curve and stress cloud that simulated by adding an embedded carrier board.
board. The structure of the node area embedded carrier board has less impact on the strength and stiffness of nodes. But it can effectively restrain the concrete in the joint zone, enhance the integrity of the node, reduce the slippage, and the hysteresis curve of the node is fuller. At the same time, the bearing plate, the end plate and the area enclosed by the upper and lower flanges of the steel beam restrain the concrete in the core area, improve the shear resistance of the concrete in the core area, and the ability to limit the slip energy consumption of the steel beam has been significantly improved, and the construction of the load plate is convenient. So it is recommended to be adopted.

5. Conclusions

From the above study, the following conclusions can be drawn:

(1) The plug-in RCS combined edge joint is characterized by the ductile failure mode of the steel girder flange yielding outside the core of the joint under the low-cycle reciprocating load test, and the steel beam is not pulled out, the anchoring state is good, and the hysteresis curve is full. The ductility coefficient is greater than 3.64, and the equivalent viscous damping coefficient is 0.26, which is greater than the general reinforced concrete joints. The overall performance of the reinforced concrete joints is good, including seismic performance and energy consumption. The stiffness degradation curve of the test nodes gradually becomes gentle, and the ratio is maintained above 0.92, showing good plastic deformation ability and damage resistance.

(2) The calculation results obtained by the ABAQUS finite element analysis model established in this paper are compared with the seismic performance indexes of the test results. It is found that the error is small with the increase of the loading displacement on the hysteresis curve, the skeleton curve, the stiffness degradation curve and the bearing capacity degradation

![Figure 21. Board simulation analysis of embedded carrier.](image)

- **a)** Hysteresis Curve
- **b)** Embedded Carrier Board Stress Cloud
In this paper, five kinds of research parameters are selected, and their computational models are established by using the experimental nodes as prototypes. The seismic behavior of low-cycle reciprocating loads is studied by the control variable method. The final recommended value for each parameter is: the length of the steel beam insertion length is 5/8 – 3/4 of the section width of the column; the width of the end plate of the steel beam is the same as the width of the flange; the height of the steel beam is greater than 1.25 times of the width of the column; increase the embedded carrier board construction. It provides some references for engineering design.

Disclosure statement

No potential conflict of interest was reported by the authors.

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