Advances in Steel Structures (ICASS 2020)

Edited by

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Published by
Hong Kong Institute of Steel Construction Limited
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ON FIELD-MEASURED VERTICAL TEMPERATURE GRADIENT OF BOX GIRDER IN STEEL BRIDGES

Z.W. Zhu*, T. Qin, X.W. Chen
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 ON TRUSS TYPE STEEL REINFORCED CONCRETE JOINTS

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Abstract: This paper presents an experimental study on the structural behavior of two truss type steel reinforced concrete (SRC) joints. The objective is to characterize the mechanical behavior of SRC joints subjected to static loading. The specimens were scaled from a concrete core tube connected to a mega steel truss. Mechanical behavior of the joint zone is extremely complicated due to the complex geometry and interactive forces among the connected members. Monotonic loading tests were carried out through a self-balanced loading system. Sparse cracks were observed under design loads. Spalling concrete cover was observed for joint B1. Whereas, only a few cracks were observed in the joint D1 after testing. Based on the measured equivalent strains, the interaction zone of steel sections works elastically under 1.5 times of the design loads. This indicates that the joints have sufficient strength to meet the design requirements. The experimental results presented in this paper provides a better understanding of current truss type composite joints and offers ideas for further research based on the authors’ findings.

Keywords: Experimental test; Truss type; Steel reinforced concrete joint; Design load; Equivalent strain

DOI: 10.18057/ICASS2020.P.165

1 INTRODUCTION

Steel reinforced concrete (SRC) structures, which consist of steel, reinforced concrete and composite members, have been extensively used in recent years for their ease of handling and reliable performance when subjected to seismic behavior [1]–[3]. The steel member can provide formwork during concrete construction while concrete can prevent local buckling of the steel member and act as an insulation in case of fire. The SRC joint can lessen problems relating to field welding, reinforcing bar congestion and concrete placement over traditional steel or reinforced concrete joints. These joints are crucial to the safety and integrity of structures [4], [5]. The SRC joints are usually subjected to complex loading cases. The effects of interaction between reinforced concrete components and steel members on the mechanical behavior of a joint are not yet fully understood. For the desired integrity of a structural system, the joints must be sufficiently strong to meet the design requirements.

Complex configurations of SRC joints hinder clarification of their force transfer mechanism. Researchers have conducted laboratory tests and numerical analysis to examine the behavior of SRC joints based on beam-column connections [6]–[11]. The methodology has been verified as reliable to predict the mechanical behavior of SRC joints. Previous research on SRC joints
have mainly focused on seismic behavior of beam-to-column joints where members are perpendicular to each other[12]–[14]. The specimens are mostly subjected to moment and shear actions. With the development of construction technology and rapid economic growth, many high rise, large-scale, and multifunctional buildings have been designed and constructed. Traditional cruciform joints are not applicable for large span structures since they can introduce significant moment and shear force near a joint. Newly developed composite structural systems have been proposed to avoid predominant moment and shear force. These systems offer structural configurations similar to the truss type structures that are commonly used for bridges to resist axial loading[14]. Therefore, diagonal members are introduced and this makes the analysis and testing of joints hard to conduct [15]. The designs of these complex joints are not included in the existing specifications [16], [17].

Taking a commercial building as an example, its design features two reinforced concrete core tube structures connected by a mega steel truss. Height of the connection bridge ranges from 28.5 m to 79.5 m, as shown in Figure 1a. To avoid significant shear forces and moments that may occur under earthquake excitation, a mega steel truss replaces the conventional beam-column joint. Thus, the axial loads become the predominant issue for these steel reinforced concrete joints. Truss members were connected to a column made of cross-H shaped steel encased in a reinforced concrete core tube. This can lessen the local stress concentration and transfer its load relatively evenly to the whole tube. Based on the time history analysis of the structure, the connection truss would be subjected to huge axial loads during earthquakes. This is different from previous research that focused on joints subjected to moments or shear forces. Axial loading causes complex stress distribution in the SRC joints. Moreover, tensile loading may introduce surface cracks that are crucial to serviceability performance. It is necessary to investigate stress distributions and crack patterns by experimental tests, which have been verified as an efficient way to determine whether the joints have adequate load bearing capacity and evaluate the performance of the joints under various loading levels[18]–[20].

![Figure 1. A commercial building in Shanghai with concrete core tubes and steel truss](image)

This study was conducted to clarify the effect of several axial loads on the members of truss type SRC joints. The investigated specimens were manufactured based on a high rise building with steel members encased in concrete core tubes connected to steel truss structures. Two steel reinforced concrete composite joints were designed, constructed and tested, in which axial loads were applied to the specimens. The apparatus were carefully designed to fulfill the complex loading. Thereafter, strain histories were investigated in the interaction zone. Tests verified that steel parts of joints can work elastically under design loads introduced by seismic action. Crack
patterns were also recorded under various loading levels. This data is a valuable resource for those wanting to evaluate the serviceability of SRC joints.

2 EXPERIMENTAL PROGRAM

2.1 Specimen configuration

Two truss type SRC joints in the concrete core tube were taken as prototypes. They are joint B and joint D as denoted by circles in Figure 1a. The SRC joints consist of one H shaped steel column which is encased in the concrete tube, and two rectangular steel tube members as shown in Figures 1b and 1c. Specimens were designed based on design prototypes. They were rotated certain degrees to make the steel member NX-1 vertical since it would be subject to predominant axial loading. Based on the design loads and capabilities of the loading system, a 1/3 scale test model was designed for joint D and a 1/4.5 scale test model was designed for joint B. They are denoted as D1 and B1 for specimens, respectively. Steel members of the models were made of Q345 steel plate according to Chinese design code (Code for design of steel structures GB50017-2003). A steel column SZ-1, which was embedded in the concrete core tube, was connected by two diagonal steel tube members as shown in Figure 2. Table 1 summarizes the dimensions details of branch members. Steel parts of the joint are connected to steel base plates, as shown in Figure 2a. Two oblique reaction columns, denoted as supports in Figure 2b, were designed to support hydraulic jacks for joint B1. The joint zone details of two specimens are shown in Figure 3. Steel stiffeners, diaphragm and cover plates were welded to the steel members. Four holes were cut in the cover plates to be filled with concrete in the fabrication stage.

![Figure 2. Dimensions of two specimens (steel parts) (unit: mm)](image1)

![Figure 3. Joint zone details for two specimens (steel parts) (unit: mm)](image2)
Table 1 Dimensions of cross sections

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<td>(SKL, NX)</td>
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<td>NX-1 250×250×20×20</td>
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<td>SZ-1 533×235×26×28</td>
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<td>B1</td>
<td>SKL-2 (9F) 230×165×9×9</td>
<td>(SZ)</td>
</tr>
<tr>
<td></td>
<td>NX-1 165×165×12×12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SZ-1 355×155×18×18</td>
<td></td>
</tr>
</tbody>
</table>

I-shaped steel members SZ-1 of both joints were cast in reinforced concrete. Location of the concrete part is denoted by a dotted line in Figure 2. The cross section consists of two rectangular parts. The narrow part was designed to simulate the concrete core tube wall. The overall dimensions were 400 mm × 1335 mm and 267 mm × 890 mm for joint D1 and B1 respectively. Longitudinal bars with stirrups were arranged with the same reinforcement ratio as designed prototypes. Details can be referred in Figure 4. Covers to the transverse reinforcement were 17 mm and 15 mm for joint D1 and B1, respectively. Commercial concrete with a specified compressive strength of 50 MPa was used and cured in the room temperature for more than one month. The concrete mix proportion of cement: sand: gravel: water was 1: 1.27: 1.99: 0.35(by weight).

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Table 2 Mechanical properties of steel bars and steel plates

<table>
<thead>
<tr>
<th>Materials</th>
<th>Diameter (mm)</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Elongation (%)</th>
<th>Elastic modulus ((\times10^5)\text{MPa})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing bar</td>
<td>8</td>
<td>331.7</td>
<td>469.8</td>
<td>20.8</td>
<td>1.94</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>385.47</td>
<td>557.57</td>
<td>17.9</td>
<td>1.83</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>353.8</td>
<td>532.7</td>
<td>18.7</td>
<td>1.81</td>
</tr>
<tr>
<td>Steel plate</td>
<td></td>
<td>360.0</td>
<td>527.0</td>
<td>22.1</td>
<td>2.27</td>
</tr>
</tbody>
</table>

Longitudinal steel reinforcing bars were tested to obtain their properties, as shown in Table 2. Mechanical properties of the mild steel Q345 plates were also obtained by standard material tests according to Chinese standard GB/T 228-2002. The compressive strength and the elastic modulus of concrete were determined by the standard compression test as 49.0 MPa and
2.66×10^4 MPa, respectively for joint D1. Likewise, the concrete compressive strength and the elastic modulus were 41.8 MPa and 2.56×10^4 MPa for joint B1.

2.2 Test setup and instrumentation

Test setup was developed for two steel reinforced concrete composite joints, which can be seen in Figure 5. Monotonic loading test were applied to investigate the mechanical behavior of the joints. The design loads for joint D1 are -1,733 kN, -2,266 kN and -1,147 kN at ends A, B and C, respectively. The minus symbol indicates that all members are under axial compression, as shown in Figure 5a. The oblique compression load was fulfilled with a self-balanced loading system. It consists of a jack, upper steel plate and several steel bars. For example, a hydraulic jack with a capacity of 5,000 kN was used at end A to apply the axial compression load that was realized by 16 steel bars that connected base plates and an upper steel plate. A vertical compression load at end B was applied by an actuator with a 10,000 kN load capacity. A hydraulic jack with a capacity of 2,000 kN was used at end C to apply the axial compression load that was realized by 12 steel bars. The loading proportion was kept constant as design loads.

The design loads for joint B1 are -341.5 kN, +761.2 kN and +416.3 kN at ends A, B and C, respectively. The plus symbol indicates that the members are under axial tension, as shown in Figure 5b. With a self-balanced loading system, a hydraulic jack with a capacity of 2,000 kN was used at end A to apply the axial compression load realized by 12 steel bars. Vertical tension load at end B was applied by an actuator with a 10,000 kN load capacity. Two hydraulic jacks with a capacity of 1,000 kN were mounted on the supports and used at end C to apply the axial tension load. The loading proportion was also kept constant with each designed load.

![Test setup](image)

Figure 5. Test setup
Joint D1 had a total of 25 uniaxial strain gauges installed on longitudinal reinforcing bars in concrete and 20 strain rosettes attached on the surfaces of the steel members. Their details were illustrated in Figure 6 with cross-sections from 2-2 to 6-6. Similar arrangements of gauges were also applied to joint B1, which had a total of 24 uniaxial strain gauges installed on longitudinal reinforcing bars in concrete and 19 strain rosettes were attached on the surfaces of the steel members. During the test, loads and strains were acquired by a data acquisition system at every second.

**Figure 6. Arrangements of strain gauges**

### 2.3 Test procedure

Monotonic loads were applied to the joint based on the proportion of design loads presented in the previous section. The specimen was preliminary loaded to check the conditions of the loading and acquisition system. Thereafter, load control incremental loads were applied to three ends with each step load of -43.33 kN at end A, -57.0 kN at end B and -29.0 kN at end C for joint D1. Concrete crack initiation and propagation were observed and recorded after each loading step. In case loading, the level attained 1.5 times the value of their design loads, load at end C was kept constant while loads at end A and B were increased to 1.7 times of design loads proportionally step by step. Then all loads were unloaded smoothly to zero.

Load control incremental loads were applied to three ends with each steps load of -17.1 kN at end A, +38.1 kN at end B and +20.8 kN at end C for joint B1. In case loading level attained
2.6 times of design loads, loads at end A and C were kept constant while load at end B was increased until failure of concrete was observed near the base.

3 TEST RESULTS

3.1 General description

As the first step, general observations were presented. No obvious deformation or any sound was indicated until the end of loading for joint D1. Initial oblique crack was observed near left member A when vertical loading force was 680 kN at end B (0.3 times of design loads). Thereafter, new cracks initiated and propagated on the concrete surface. Final cracking patterns were illustrated in Figure 7a. Numbers in the figure indicates the loading levels and the crack width. It can be noted that cracks are relatively sparse in compression loads. Local buckling was not observed for steel members.

With regard to the joint B1. Initial oblique crack was observed on the interaction region near vertical member B when loading force is 456.7 kN at the end B (0.6 times of design loads). Cracks appeared and propagated randomly with increased loading. Under design loads, cracks parallel to the surface were found at the interaction region. When loading value increased to 2,283.7 kN at the end B, the test was terminated with a loud vocie. Oblique cracks were found from the right section, from base to the interaction region. Concrete spalling was also observed near the base plate. Crack patterns on one side are recorded in Figure 7b. No cracks or residual deformations were found in the encased steel members at the joint zone, which was verified by removing concrete from the interaction region for both joints.

![Figure 7. Cracking patterns of the joints](image)

3.2 Analysis of measured strains

Applied forces and strains were measured during the tests. Strain of reinforcing bars and embedded steel members were graphed separately. Strains were directly graphed with absolute applied load values at end B for uniaxial strain gauges mounted on reinforcing bars. With regard to embedded steel members, equivalent (von Mises) strains were calculated from the strains of three limbs of the rosettes based on Eq. (1). The equivalent strains were also graphed with absolute forces applied at end B.

$$\varepsilon_e = \frac{1}{1+\nu'} \left( \frac{1}{2} \left[ (\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2 \right] \right)^{\frac{1}{2}}$$  (1)
Figure 8 shows typical measured load strain curves of joint D1, illustrating the strain history of cross section 2 to cross section 6. The abscissa is the strain determined during test while the ordinate is the compression force at end B. Strain increased linearly with the increasing loads in the initial stage of the test. As loading continued, some nonlinearity was observed for some measured strains. Strains of cross section 6 were generally larger than the others. Most strains were less than the yield strain (1586 με) when loads attained 1.5 times their design loads. This indicates that the joint interaction zone was under an elastic mechanical state under design loads. All strain values of reinforcing bars were negative, indicating reinforcing bars were under compression. The rebar, concrete and steel member worked together to bear the external actions.

(a) Cross section 2-2 and 3-3

(b) Cross section 4-4 and 5-5

(c) Cross section 6-6

Figure 8. Load- strain curves of joint D1

Strains and von Mises (equivalent) strains were also plotted with loads at end B for joint B1. The trend is similar to the previous joint, linear relationships were observed within design loads for all strain gauges. However, load strain curves developed in a reverse direction for some strains of section 5 and 6, indicating extensive cracking of concrete in the interaction region.
This is consistent to the phenomena during the test. Strains of cross section 6 exceeded the yield strain with the increment of the loading.

3.3 Strain distribution

Special attention was paid to the equivalent strain values of the interaction region under various design loads. Figures 9 illustrates the relationship between equivalent strain variations and loading levels. With regard to joint D1, equivalent strains are generally proportional to the loading levels that are 0.5, 1.0 and 1.5 times the design loads. The maximum equivalent strain is on the cross section 6 with a value of 459 με under design load. The equivalent strain increased to 938 με when the design increased to 1.5 times the designed loads. The value is less than the yield strain of 1586 με. It can be concluded that the joint can work elastically and have enough margin of safety with respect to design loads.

With regard to joint B1, equivalent strains on the steel surface were compared at four load levels, 0.5, 1.0, 1.5 and 2.0 times the design loads. Most equivalent strains are less than the yield strain except one strain on cross section 6 with a value of 1750 με under design load. Measured strains increased with increased loading. Several strains on cross section 6 exceeded the yield strain when the loading level was 2 times that of the design loads. The equivalent strain distributions showed that the joint can be analyzed with elastic theory under design loads. And cross section 6 is the critical cross section.

4 CONCLUDING REMARKS

Experimental investigation was performed to clarify the mechanical behaviors of two truss type SRC joints. Their engineering background and design incentives were described. Scaled experimental tests were carried out to obtain crack patterns and strain distributions under design loads. Carefully designed apparatus were used to achieve complex loading cases. Thereafter,
mechanical behavior of the interaction region were quantified through the analysis of measured strains. The following conclusions can be drawn from this study.

- With regard to joint D1, small cracks were observed during the loading. Whereas, significant concrete spalling was observed for joint B1 since it was under tension loadings. Steel members were intact in the joint zone after removal of the concrete components for both joints. The results indicate the strength of two joints is adequate under design loads.

- The interaction zone of the steel members work elastically under design loads as indicated by test results. The maximum equivalent strains appeared on cross section 6 of both joints with values of 459 με and 1750 με for joint D1 and B1, respectively. When the loading level increased to 1.5 times of design loads for joint D1, most of the joint zone was still in the elastic range. The strains on cross section 6 gradually exceeded yield strain for joint B1 with the incremental loading until 2 times that of the design loads. The results indicate that both joints have enough of a margin of safety with respect to the design loads.

- Based on the observed strains of reinforcing bars, reinforced concrete bore part of the loads. The reinforced concrete may have prevented the local buckling of steel plates under compression. The cracking of concrete can be considered as an alert sign for steel reinforced concrete joints during service.

Current experimental research is essential to further understanding of similar steel reinforced concrete joints and to the development of design engineering methodologies.

ACKNOWLEDGEMENT

This research project was financially supported by the National Key R&D Program of China (Grant No. 2017YFB1201204)

REFERENCES


These proceedings contain the papers 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.

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