Cyclic Reversed Loading Test of Different Structural Beam-To-Column Connections in Assembled Steel Structure

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Abstract: Three new assembled connections between an I-section beam and I-section column were proposed in this paper. To investigate the aseismic behaviour of these assembled connections, four specimens under cyclic loading were tested. The test process of the specimens was recorded and analyzed. The hysteretic properties, failure mode, ultimate load, and ductility were studied. The test results show that the specimens with assembled connections have better ductility and bearing capacity than the specimen with a welding type connection. The specimen with an “L” shaped steel plate has a strong energy dissipation capacity but low flexural rigidity. If triangular stiffeners were added to the “L” shaped steel plate, the mechanical properties of the assembled connection would be substantially improved.

1. Introduction
In the past decade, engineers in China were learning about the experience of assembled buildings from foreign countries and actively promoting the application of assembled buildings in China. Compared with traditional buildings, assembled buildings, especially assembled buildings with a steel structure, take less time to build, and produce less construction waste and noise. The Chinese government has also promulgated policies to promote the development of assembled buildings with a steel structure. In big cities, many high-rise residential and office buildings are constructed with assembled steel structures. Beam-to-column connections, as key parts of the assembled steel structure, have attracted much attention from researchers. Administrative departments have also promulgated some local standards for beam-to-column connections[1]. In these local standards, JIDC[2] (joints with interior diaphragm connections) and JESR[3] (joints with exterior stiffened ring) are recommended. But both JIDC and JESR, with their complex structure and high bearing capacity, are mainly suitable for buildings with 8 to 30 floors. Rural areas, as a major part of China, need assembled buildings of less than four stories, so JIDC and JESR are neither economical nor appropriate in rural construction. It is important to develop suitable beam-to-column connections with low cost, simple construction and sufficient bearing capacity. Three beam-to-column connection modes are proposed in this article and four specimens under quasi-static load were tested to investigate their hysteretic behaviour. The hysteretic curve, ductility, degradation of strength and stiffness, failure mechanism and characteristics were compared and analyzed.
2. Test design

2.1. Specimen design

Four specimens of beam-to-column connections were made, in which the I-section beam and I-section column were connected in a strong axis. The details of each specimen are shown in Fig. 1. Specimen J1 was a welded connection, while the other three specimens were assembled with bolts. In specimen J2, a steel beam and steel end-plate were welded in metalwork factories, and the steel end-plate was bolted to the steel column at the building sites. In specimen J3, an “L” shaped steel plate and steel column were welded in metalwork factories, and the steel beam was bolted to the “L” shaped steel plate at the building sites. Specimen J4 and specimen J3 worked in the same way, but a triangular stiffener was welded with the “L” shaped steel plate of specimen J4.

All the specimens were made of Q235B steel. According to the Metallic materials-Tensile testing-Part 1: Method of test at room temperature (GB/T 228.1-2010)[4], the yield stress and tensile stress and elongation of the steel were gauged and are shown in Table 1.

Table 1. Measured material properties

| Steel                        | Nominal t (mm) | Measured t (mm) | Yield stress $f_y$ (MPa) | Tensile stress $f_u$ (MPa) | Elongation (%) |
|------------------------------|----------------|-----------------|--------------------------|---------------------------|----------------|
| Beam webs and flanges, column webs | 4              | 3.9             | 373                      | 444.3                     | 21.5           |
| Column flanges               | 6              | 5.9             | 346                      | 429.1                     | 19.2           |
| End-plate, “L” shaped plate, triangular stiffeners | 6              | 5.7             | 355                      | 423.8                     | 19.7           |

2.2. Loading scheme

All specimens were tested under low cyclic reversed loading. The bottom of the column was connected with the base by a spherical joint. In the first stage of the experiment, a vertical steady load was exerted on the top of the column, and the axial compression ratio was 0.3. In the second stage of the experiment, vertical reciprocate loads were exerted on the beam end. The vertical load at the beam end was increased by increments of 8 kN until the specimens yielded. After that, the vertical load at
the beam end was controlled by the drift of the beam end. To study the relations between loads and drift precisely, the drift of every stage was increased by 50% of the yield drift compared with that of the previous stage. The peak of vertical load at the beam end during the entire process of testing was recorded and called the ultimate load. According to Specification for seismic test of buildings (JGJ/T 101-2015)[5], if failure of the specimen was obvious, or the vertical load in a certain load cycle could not reach 85% of the ultimate load in the stage of specimen failure, the loading would cease, and testing of this specimen would be finished.

2.3. Measurement plan
The vertical load at the beam end was measured and recorded by a force sensor (FS) during the course of the test. The drift of the beam end and other positions was measured and recorded by dial indicators (DI) and guyed displacement meters (GDM), as shown in Fig. 2. FS1 was installed between the top of the column and the hydraulic jack, and the readout of FS1 was displayed in the software system synchronously. In the process of testing, the hydraulic jack was fine-tuned to keep the readout of the force sensor invariable, so that the vertical load at the top of the column could also be invariable. GDM-1 was installed at the beam end to measure the vertical drift of the loading point. Beyond that, FS2 was installed at the beam end to measure the vertical load. The hysteretic curve was plotted from the data of FS2 and GDM-1.

3. Test process analysis

3.1. Specimen J1
Specimen J1 was designed to be the reference for other specimens. At the beginning of loading, there was no obvious deformation in the specimen given its high stiffness. When the vertical load (FS2) increased to 29 kN, small cracks were found in the weld between the column and the lower web of the beam, the width of which was about 0.5 mm. After that, the stiffness of specimen J1 slightly reduced, and the loading was controlled by the drift of the beam end. With increase of the displacement of the beam end, the load increased correspondingly. When the vertical load increased to 47 kN, reaching its peak, it then decreased rapidly at the next two loading loops. Finally, the steel plate near the weld between the column and the lower web of the beam was pulled apart, and specimen J1 failed, as shown in Fig. 3(a).
3.2. Specimen J2
The stiffness of specimen J2 was found to decrease slightly when the vertical load at the beam end increased to 24 kN. After that, with increase of the vertical load, the end-plate was found to be bent out of shape with the beam. The vertical load increased to 42 kN, reaching its peak, and then began to decline slowly. Finally, a low-cycle fatigue fracture was found at the bending position of the end-plate, as shown in Fig. 3(b).

3.3. Specimen J3
The stiffness of specimen J3 was found to decrease slightly when the vertical load at the beam end increased to 18 kN, but no obvious deformation or crack was found. When the vertical load increased to 32 kN, the first crack was found at the bolt hole of the “L” shaped steel plate near the column. Then the stiffness of specimen J3 decreased faster than before, while the bearing capacity of J3 increased slowly. When the vertical load increased to 55 kN, reaching its peak, it then began to decline. In the next two loading loops, when the bearing capacity decreased to 85% of its peak, the loading ceased, and at the same time the “L” shaped steel plate near the column was pulled off at the position of the bolt hole, as shown in Fig. 3(c).

3.4. Specimen J4
The initial stiffness of specimen J4 was clearly higher than the others, and decreased slightly when the vertical load at the beam end increased to 24 kN. In the next 10 load loops, the stiffness of specimen J4 remained constant. There was no obvious deformation or crack on the surface of the specimen. When the vertical load at the beam end increased to 56 kN, the first crack was found at the weld between the “L” shaped steel plate and triangular stiffeners, but the bearing capacity of specimen J4 continued to rise until the vertical load at the beam end increased to 70 kN. After that, the crack alternately opened and closed with the beam’s up and down movement, as shown in Fig. 3(d). The bearing capacity of specimen J4 decreased to 85% of its peak in the next loading loop and the test ceased.

4. Result analysis

4.1. Hysteretic properties

The hysteretic curve can reflect the relation between structural force and displacement at a certain point under cyclic reversed loading. The hysteretic curve is a comprehensive reflection of the seismic performance of structures [6]. The hysteretic curves of the four specimens are shown in Fig. 4. The following information can be derived from analysis of Fig. 4.

1) The hysteresis curve of J1 looked fusiform, and was plump. This shows that the beam-to-column welding type connection had a favorable energy dissipation capacity. After the vertical load reached its peak, specimen J1 failed quickly, without the yield platform stage. So the failure mode of J1 was unsatisfactory. The bearing capacity of J1 was only 47 kN, and neither was it an assembled connection.
(2) The pinching effect could be observed in the hysteresis curve of J2. This shows that the energy dissipation capacity of J2 was not as good as J1. The bearing capacity of J2 was only 42 kN. Judging from the failure mode of J2, it is clear that the bearing capacity and flexural rigidity of J2 depend on the thickness of the steel end-plate. So, it is necessary to strengthen the steel end-plate for specimen J2 to come into use.

(3) The bearing capacity of J3 was 55 kN. But the hysteresis curve of J3 looks very close to that of J2. As shown in Fig. 4(c), the load at the beam end remained unchanged when the displacement of the loading point changed from -10 kN to 10 kN. This shows that slight sliding between the beam and column occurred in the connection of specimen J3. In assembled steel structures, slight sliding is unavoidable, but can be abated by improving the machining accuracy of the component. Generally speaking, specimen J3 possessed a favorable energy dissipation capacity and higher bearing capacity, but low flexural rigidity.

(4) The hysteresis curve of J4 looks very close to that of J3, while the bearing capacity of J4 was 70 kN. Because of the similar structures, slight sliding between the beam and column occurred in the connection of specimen J4 too. Due to the triangular stiffeners in the “L” shaped steel plate, the bearing capacity of J4 increased by 30% compared with J3, and the flexural rigidity of J4 was 3% greater than J3 before yielding, and 23% greater than J3 after yielding. So, installing triangular stiffeners in the “L” shaped steel plate was an effective method to improve its mechanical performance.

4.2. Analysis of ductility
The ductility of the structure under cyclic reversed loading reflects the plastic deformation ability of the structure on the premise of maintaining its bearing capacity in its yielding stage. Ductility and strength are equally important parameters in the seismic design of structures. Generally speaking, the ductility of structures is usually measured by the ductility coefficient. The ductility coefficient refers to the ratio of ultimate displacement $\Delta u$ to yield displacement $\Delta y$ of the structure or member.

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

The ductility coefficients of the specimens are shown in Table 2.

| $\Delta y$/mm | J1 | J2 | J3 | J4 |
|---------------|----|----|----|----|
| $\Delta u$/mm | 22 | 26 | 31 | 34 |
| $\mu$         | 3.93 | 4.13 | 4.43 | 4.25 |

As shown in Table 2, the ductility coefficients of the four specimens in this paper were between 3.93 and 4.43, and therefore fairly close. The ductility coefficients of J3 and J4 were slightly higher than that of J1. This confirmed the effectiveness of the “L” shaped steel plate. Although the ductility coefficient of J3 was slightly higher than that of J4, the mechanical performance of specimen J4 is generally the best for its bearing capacity and flexural rigidity.

5. Conclusions
(1) The beam-to-column welding type connection showed a favorable energy dissipation capacity and medium bearing capacity. The disadvantage was that it is not an assembled connection, and its failure mode is unsatisfactory.

(2) The beam-to-column connection with the steel end-plate had a lower bearing capacity and flexural rigidity than that of the welded type. It is necessary to strengthen the steel end-plate in practical engineering.

(3) The beam-to-column connection with the “L” shaped steel plate had a stronger bearing capacity and is more convenient for use. If triangular stiffeners were added to this connection structure, not only the bearing capacity but also the flexural rigidity would be substantially improved.
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