Research Article
Experimental Study on Mechanical Performance of U-Shaped Steel-Encased Concrete Composite Beam-Girder Joints

Zhangqi Hu,1,2 Ran He,1 Yukui Wang,1 Weirong Lv,3 and Jingchao Li1
1College of Civil Engineering, Hunan City University, Yiyang 413000, China
2Key Laboratory of Key Technologies of Digital Urban-Rural Spatial Planning of Hunan Province Hunan City University, Yiyang 413000, China
3College of Civil Engineering, Hunan University of Science and Technology, Xiangtan 411201, China
Correspondence should be addressed to Zhangqi Hu; huzhangqi0413@163.com

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1.Introduction
Beam-girder joints are widely used in frames, and the construction types relate directly to the reliability of the structure and the construction period. Three of the commonly used beam-girder joints [1, 2] are shown in Figure 1. These traditional beam-girder joints face problems, such as complex construction, slow progress, and insufficient ductility. Therefore, researchers proposed some improved beam-girder joints, such as the reinforced concrete beam-girder composite joint [3], shown in Figure 2(a); the composite RC (reinforced concrete) beam-girder joint with rebar lapping in the pressed sleeve [4], shown in Figure 2(b); the joint of a concrete girder and an inserted steel secondary beam [5], shown in Figure 2(c). The beam-girder joints shown in Figures 2(a) and 2(b) are prefabricated concrete structures, and the indicators of ductility, stiffness, and bearing capacity reflect the unsuitability of the two structures for large-span and spatial structures, which developed rapidly in recent years.

Steel-concrete composite beams have the advantages of high bearing capacity, light weight, reduced beam depth, and good seismic performance, which can meet the demands of large-span and spatial structures on space and mechanics performance, and have thus been widely used in many buildings and bridges [6–8]. Figure 2(c) shows an improved steel beam-concrete girder joint, in which the steel beam and the concrete girder can be constructed simultaneously, shortening the construction time compared with that of traditional steel beam-concrete girder joints. However, instability may still occur if the structure is subjected to
negative bending moment [9], which will affect the mechanics performance of the overall structure.

U-shaped steel-encased concrete composite beams possess the advantages of short construction period and strong stability compared with traditional steel-concrete composite beams and thus have extensive application prospects and high use value. The application of U-shaped steel-encased concrete composite beams can reportedly shorten the construction period by 10%–20%, thereby producing good economic benefits [10]. Hence, significant research has been conducted on U-shaped steel-encased concrete composite beams through finite element analysis. Eight thin-walled U-shaped steel beams filled with demolished concrete blocks (DCBs) and fresh concrete (FC) and four thin-walled U-shaped steel beams filled with FC alone were tested by Wu and Ji [12] to investigate the influences of the replacement ratio of DCBs, the thickness of the U-shaped steel, and the longitudinal reinforcement ratio on the flexural behavior of the composite beams. The test results showed that adding of longitudinal bars was an effective way to improve the flexural capacity. Zhang et al. [13] tested three continuous composite beams and studied the internal force redistributions of the composite beams. A good rotation ability and ductility were observed with each composite beam, and the formula of the relationship between moment redistribution coefficient and force ratio was then put forward. Shear connectors are always used to prevent the shear sliding failure between concrete slabs and steel girders of composite beams. Thus, several forms of shear connectors have been reported for enhancing the flexural behavior or eliminating the constitution difficulty of composite beams [14–22]. Park et al. [14, 15] used diagonal rebars and welded rebar connections to strengthen the beam-column joints. Keo et al. [16] proposed a novel composite beam with angle connectors welded to the top flange of the U-shaped girder (shown in Figure 3(a)) and found that the angle connectors could provide ductile connection between concrete and U-shaped steel girder. Guo et al. [17] and Liu et al. [18] performed further studies to investigate the effects of angle connectors, intervals, and geometries on the flexural behavior of this kind of composite beam. Ductile failure mode and high strength were observed with all specimens, and the depth of U-shaped girder was found to be the most sensitive affecting parameter on the flexural behavior of the composite beams. Zhou et al. [20] and Liu et al. [21] discussed the flexural behavior of the rebar truss stiffened cold-formed U-shaped steel-concrete composite beams, in which the interface between the concrete slab and U-shaped girder was enhanced with inverted U-shaped rebars. Through experimental results, the mid-span deflection of the composite beams can reach up to $L_0/20$, reflecting a good deformability. This research team [22] proposed a novel U-shaped steel-encased concrete composite beam, in which the inserted bars acted as the shear connectors (shown in Figure 3(b)), and performed static loading tests on two specimens to study the mechanical performance of composite beams under positive and negative bending moments. The results showed that the composite beam exhibited an extremely strong deformation performance under a positive bending moment, and no instability was observed on the specimen under a
negative bending moment. Though researchers have performed numerous studies on composite beams to improve the flexural behavior or simplify the construction of composite beams, and some improved beam-girder joints were also proposed, the available beam-girder joints are not suitable for large-span and spatial structures, or threatened by the instability failure. Moreover, the stress states of beams are quite different from that of beam-girder joints. During structure design, girders are usually considered as the supports of beams, and the beam ends are always subjected to negative bending moments, while the girders are subjected to positive bending moments, subjecting the slabs in the joint zone to complex stress states, which affects the stress distributions and failure patterns of the girders. However, the slab effects are always ignored by researchers, and the available tests cannot reflect the actual force condition of beam-girder joints well.

On the basis of the previous study of this research team [22], this paper proposes a novel U-shaped steel-encased concrete composite beam-girder joint. Two full-scale specimens with slabs were tested to verify the reliability and investigate the failure modes, bearing capacity, deformation performance, and strain distributions of the novel composite beam-girder joints. The two specimens were only varied in beam section reinforcements.

2. Specimen Design and Material Properties

2.1. Specimen Design. Two full-scale specimens were labeled BGJ1 and BGJ2. The specimens had the same dimensions and were constructed similarly, except for the beam section reinforcements, as shown in Figure 4. The U-shaped girder (referred to herein as the girder) was 300 mm deep and 150 mm wide, and 20 mm of the girder was embedded into the 100-mm-thick slabs. Each U-shaped beam (referred to herein as the beam) was 250 mm deep and 150 mm wide but not embedded into the slabs. The openings were distributed equidistantly on the top flanges of the girders and beams, and the inserted bars were embedded in them through the slabs. A rectangular opening was arranged on the web of each girder, through which the shaped sleeve is inserted. The U-shaped beams at the two sides (L and R) were inserted into the shaped sleeve, and the U-shaped girder and two U-shaped beams were connected by the shaped sleeve through welding. The thickness of all the girders, beams, and shaped sleeves were 4 mm. The two specimens were only varied in beam section reinforcements, GBJ2 was equipped with 3Ø16 additional bars in the beams based on Specimen BGJ1, and no stirrup was installed in both the specimens. For simplicity, the slabs are divided into five zones and four interfaces, as shown in Figure 4(a), namely, the node, beam, and girder zones and the beam and girder interfaces.

2.2. Material Properties. The two specimens were constructed simultaneously and thus share the same concrete strength grade of C35, the measured cubic compressive strength of concrete \( f_{cu} \) is 38.6 MPa, and the mechanical properties of the steel plates and the reinforcements are listed in Table 1.

3. Test Setup and Measuring Scheme

3.1. Test Setup. The test setup is shown in Figure 5. Each specimen is supported on the two girder ends by piers and fixed by two jacks. Synchronous vertical loads are imposed at both cantilever beam ends of the specimen by the other two jacks.

3.2. Measuring Scheme. Force control was adopted before the peak with an increment of 10 kN each step, and displacement control was employed after the peak. The loads were sustained for 10 min to observe the crack development and record the progression of displacements and strains. Figure 6 shows the arrangements of the displacement meters and the strain gauges. Displacement meters D8 and D9 were used to record the slip deformation between slabs and beams, and the beam deflections were recorded by displacement meters D1–D7. Strain gauges C1–C21 and R1–R13 were arranged to measure the strains of the slabs and the reinforcements, respectively; the strains of beams were measured with strain gauges S1–S6 and S19–S24; strain gauges S7–S12 and S13–S18 were used to measure the strains of the shaped sleeves. For each specimen, the loading point was 1075 mm away from the beam interface and 1400 mm away from the girder edge.

4. Results and Discussion

4.1. Crack Progression and Failure Pattern. For Specimen BGJ1, both the beam interfaces cracked at the 30-kN loading...
Figure 4: Continued.
Bending cracks appeared in each beam zone, 100 mm away from the nearby beam interface, when the loads reached 60 kN. At the 70 kN loading step, the first node zone crack appeared right on the girder’s central axis, and beam interface cracks developed into an arc shape, shown in Figure 7(a). Two beam zone cracks and one node zone crack developed when the loads from the actuators reached 80 kN, each beam zone crack was 200 mm from the nearby beam interface, and the node zone crack was 100 mm away from the nearby beam interface. From 90 kN to 100 kN, more cracks developed in the beam zones, and the beam interface cracks extended to the bottom of the slabs. The existing cracks widened, and no more crack developed in the node or beam zones. When the loads reached 120 kN, the bottom flange of each beam yielded in compression. Visible diagonal cracks developed on the slab webs of the girder zones when the loads reached the peak (211 kN at the L side, and 198 kN at the R side), as shown in Figure 7(b). Thereafter, the actuator loads decreased with deflection increasing, and the diagonal cracks widened to 10 mm when the cantilever-end deflections reached 45 mm. Specimen BGJ1 failed as both the arc-shaped and diagonal cracks extended and combined (shown in Figures 7(c) and 7(d)) when the deflection at the R side reached 55.13 mm, and the deflection at the L side reached 59.30 mm, respectively.

For Specimen BGJ2, both beam interfaces cracked at the 30 kN loading step. Bending cracks were first observed in the beam zones, and each was 80 mm away from the nearby beam interface when the loads came up to 50 kN, shown in Figure 8(a). The beam interface cracks developed into an arc shape when the loads reached 80 kN, and the first node zone crack also appeared at this loading step, right on the girder’s central axis. Four bending cracks were observed at the 100 kN loading step (the measured actuator load at both sides were 97 and 100 kN), the cracks on the L side beam zone were 250 and 400 mm away from the L side interface, respectively, and the cracks on the R side beam zone were 170 and 360 mm away from the R side interface, respectively, shown in Figure 8(b). The longitudinal reinforcements started to yield at the 130 kN loading step. When the loads reached 140 kN, the shaped sleeve (bottom flange) yielded by compression, and three more bending cracks were observed. One crack was in the L side beam zone, 260 mm away from the L side beam interface, and two cracks were in the R side beam zone, 410 and 550 mm away from the R side interface, respectively. Since then to destruction, the exiting cracks widened, but no more crack appeared in node and beam zones.

**Table 1: Mechanical properties of steel and reinforcement.**

| Type of steel     | Yield strength $f_y$ (MPa) | Ultimate strength $f_u$ (MPa) | Elastic modulus $E_s$ (MPa) | Yield strain $\varepsilon_y$ (×10^-6) |
|------------------|---------------------------|-------------------------------|----------------------------|---------------------------------------|
| Reinforcement (Φ8) | 457                       | 665                           | $2.0 \times 10^5$          | 2285                                  |
| Reinforcement (Φ16) | 428                       | 577                           | $2.0 \times 10^5$          | 2140                                  |
| Steel plate (4 mm) | 382                       | 608                           | $2.06 \times 10^5$         | 1854                                  |

$\varepsilon_y = \frac{f_y}{E_s}$. 

**Figure 4:** Dimensions, reinforcements, and construction of the tested specimens. (a) Dimensions and reinforcements of slabs. (b) Section 1-1 of Specimen BGJ1. (c) Section 1-1 of Specimen BGJ2. (d) Section 2-2 (Specimen BGJ1 and BGJ2). (e) Connection pattern of girder and beams. (f) Seams of beam and shaped sleeve. (g) Seams of girder and shaped sleeve.

**Figure 5:** Test setup.
zones. The R side beam yielded by compression at the 180 kN loading step (the measured value was 181 kN), and the L side beam yielded when the load reached 189 kN (190 kN loading step). Cracks developed on the slab webs of the girder zones when the loads reached the peak (213 kN at the L side, and 219 kN at the R side). Since then, the loads decreased with the increase of deflection, and displacement control was employed. The cracks on the slab webs of the girder zones developed to an L shape when the deflections reached 40 mm, as shown in Figure 8(c). Specimen BGJ2 failed as the arc- and L-shaped cracks extended and combined, as shown in Figure 8(d).

The failure patterns of the two specimens are similar, as shown in Figures 7 and 8. The arc-shaped cracks developed at the beam interfaces, and the specimens failed as the cracks extended to the girder zones, while no weld fracture was observed.

Figures 9(a) and 9(b) show the failure patterns of beam-girder joints, which were proposed by Xu et al. [3] and Han et al. [4], respectively. Bending cracks and diagonal cracks were observed in these beam-girder joints, while the bending cracks on beams did not extend to the girders, because the slab effects were not considered, leading to the different failure patterns with the specimens in this article. When considering the slab effect, the girders may crack and yield before prospect due to the beam crack extension, and the girders should not be simply considered as supports of beams.

4.2 Load-Deflection Curves. Figure 10 shows the load-deflection curves (P–Δ curves) of the two specimens, where the load from the L side actuator corresponds to the deflection differences of D1 and D4, and the load from the R side...
actuator corresponds to the deflection differences of D7 and D4. The load, deflection, and chord angle \( \theta \) \((\theta = \Delta/1400)\) of each characteristic point are shown in Table 2, where \( \Delta_{cr} \) and \( \theta_{cr} \), respectively, represent the deflection and chord angle, at which the first visible crack (cracks) appeared; \( P_{cr} \) is the cracking load; \( \Delta_{y}, \theta_{y}, \) and \( P_{y} \) represent the yield deflection, chord angle, and load, respectively; \( \Delta_{p}, \theta_{p}, \) and \( P_{p} \) are the peak deflection, chord angle, and load, respectively; \( \Delta_{m} \) and \( \theta_{m} \) are the respective maximum deflection and chord angle, where \( \Delta_{m} \) is taken as the deflection when the load drops to 85% \( P_{p} \). \( \Delta_{y} \) and \( \Delta_{m} \) are calculated with the method shown in Figure 11.

Figure 10 and Table 2 indicate that both specimens exhibited good ductility. The load decreased slowly after the peak, proving that the novel composite beam-girder joints are reliable. The loading capacity of Specimen GBJ2 increased by 5.6% (the average value) compared with Specimen BGJ1, while the maximum deflection and ductility of Specimen GBJ2 decreased by 8.2% and 7.2%, respectively.

4.3. Deflection Distributions. The chord angles can be obtained by dividing the deflections of the measuring points by the corresponding length to investigate the nonlinear characteristics of the novel composite beam-girder joints. The chord angles at the measuring points of D1 – D3 and D5 – D7 can be calculated by \((\Delta_{1}-\Delta_{4})/1400\), \((\Delta_{2}-\Delta_{4})/400\), \((\Delta_{3}-\Delta_{4})/200\), \((\Delta_{5}-\Delta_{4})/200\), \((\Delta_{6}-\Delta_{4})/400\), and \((\Delta_{7}-\Delta_{4})/1400\), respectively. \( \Delta_{1}-\Delta_{3} \) and \( \Delta_{5}-\Delta_{7} \) denote the deflections of D1 – D3 and D5 – D7, respectively. The chord angle-location curves are shown in Figure 12, and both specimens exhibited obvious nonlinearity along the beam. The chord angles of D2 and D6 are generally smaller than that of D1, D3, D5, and D7, because the displacement meters of D2 and D6 were
near the beam interfaces where the discontinuity of stiffness occurred. The relationship between locations and deflections were always ignored by researchers [3–5], while the stiffness change is important to beam-girder joints, especially when the slab effect is considered. Through investigation in this paper, we can better understand the deflection distributions of beam-girder joints.

\[ \mu = \frac{\Delta_m}{\Delta_y}, \text{which denotes the ductility factor.} \]

4.4 Load-Strain Curves. Figure 13 compares the load-tensile strain curves of the two specimens, and Figure 14 shows the comparison of the load-compressive strain curves, where the strains of R6, S18, and S24 versus the load from the R side actuator, and the strains of R10, S6, and S12 versus the load from the L side actuator, the strain of R8 corresponds to the average load from the two side (L and R) actuators.
Figure 13 illustrates that Specimens GBJ1 and GBJ2 exhibited the similar load-tensile strain curves. The strains of the longitudinal reinforcements suddenly increased after the slab cracked, and the strain growth of R6 and R10 slowed after the specimen yielded. A decrease was observed when the actuator loads were near the peak, while the strain of R8 showed a different increment trend, which continued to grow with the load decrease.

Specimen BGJ1 showed uniform strain distributions in the shaped sleeve and both beams, as shown in Figure 14. For
Specimen BGJ2, the strains of the shaped sleeve increased rapidly with the loads, and the strains of S12 and S18 were significantly higher than those of S6 and S24 under the same loading step. As described in Section 4.1, the shaped sleeve of Specimen BGJ2 yielded in advance, while the yield of beams postponed compared with Specimen BGJ1.

The specimens have the same yielding mechanism, that is, their longitudinal reinforcements yielded under tension, their sleeves yielded under compression, and their beams yielded under compression.

4.5. Strain Distributions. Figure 15 shows the tensile strain distributions of the slabs. The strains were large near the edges and small in the middle. The phenomenon can be explained by the bidirectional force state of the node zone slabs, which were subjected to negative bending moments along the beam axis and positive bending moment along the girder axis. Tensile and compressive stresses were correspondingly generated, and the girder section is subjected to greater positive bending moments nearer the middle of the girder. Under the combined effects of the positive and negative bending moments, the strains distributed non-linearly along the section depth (measurement points S7–S11 and S12–S17).

During the same loading step, a much higher strain was observed at the bottom flange (measurement points S12 and S18) of the shaped sleeve than the lower web (S11 and S17). The previously mentioned phenomenon can be mainly attributed to the insertion of the beams into the shaped sleeves, and connected by welding, a “leverage” effect could be formed on the shaped sleeves, making the shaped sleeves subjected to the pressure $q$, while the bottom ends of the shaped sleeve subjected to the shear force $V$. Under the combine action of $q$ and $V$, the shaped sleeves were exposed to complex stress states and additional tensile stress formed on the web of shaped sleeves, while stress (compressive stress) concentration happened nearby the bottom ends, shown in Figure 18. Hence, a much higher strain was made at the bottom flange (measurement points S12 and S18) than the lower web (measurement points S11 and S17). The

Figure 14: Load-compressive strain curves. (a) BGJ1 and (b) BGJ2.
Figure 15: Strain distributions of slabs. (a) BGJ1 and (b) BGJ2.

Figure 16: Continued.
Figure 16: Strain distributions of GBJ1. (a) Beam section-L. (b) Beam section-R. (c) Shaped sleeve section-L. (d) Shaped sleeve section-R.

Figure 17: Continued.
“leverage” effect can also explain the nonlinear strain distributions along the section depth of the shaped sleeves. The additional bars (3Φ16) can increase the “leverage” effect along with the stiffness of the beam, resulting in high stress concentration on the bottom flange of the shaped sleeves, thereby decreasing the deformability and ductility of Specimen GBJ2.

5. Conclusions

Static load tests were performed on two full-scale U-shaped steel-encased concrete composite beam–girder joints in this research, and the following conclusions can be drawn.

1. The yield sequence of the two specimens is as follows: the longitudinal reinforcement yielded under tension, the shaped sleeve yielded under compression, and the beam yielded under compression. The specimens were destroyed as arc-shaped cracks developed at the beam interfaces and extended to the girder zones, but no weld fracture was observed.

2. Additional bars can slightly increase the loading capacity but will also increase the “leverage” effect on the shaped sleeves, thus increasing the stress concentration on the bottom flange of the shaped sleeves, thereby decreasing the deformability and ductility of the structure. The “leverage” effect resulted in the nonlinear stress distributions along the section depth of the shaped sleeve and the increase of the bottom flange strain. The beam strain distributed linearly with the section depth, which is consistent with the plane section assumption.

Figure 17: Strain distributions of GBJ2. (a) Beam section-L. (b) Beam section-R. (c) Shaped sleeve section-L. (d) Shaped sleeve section-R.

Figure 18: Force state of shaped sleeve. (a) Welding joints distribution and (b) stress concentration.
(3) Both specimens exhibited obvious nonlinear deflection distributions along the beams, that is, large on the edges and small in the middle.

(4) The novel composite beam-girder joint proposed in this paper has the advantages of easy construction, high bearing capacity, and good deformability and can thus be applied to large-span and spatial structures.

Data Availability

No data were used to support this study.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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