Flexural Performance of Steel-GFRP strips-UHPC Composite Beam in Hogging Moment Region

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Research Article

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Flexural Performance of Steel-GFRP strips-UHPC Composite Beam in Hogging Moment Region

Dan Zeng ¹, Lei Cao¹, Xiaochen Luo ¹, Yang Liu², Zhaochao Li¹

Abstract:

This study aims to clarify the longitudinal flexural cracking characteristics in hogging moment regions, and propose the practical calculation method of cracking load and ultimate bearing capacity for steel-GFRP strips-UHPC composite deck structure. The longitudinal flexural behavior of two steel-GFRP strips-UHPC composite beams in the hogging moment regions are conducted through a three-point loading test method. Their failure modes and mechanism, cracks propagation and distribution characteristics are analyzed considering the influence of reinforcement ratio. Variation law of mid-span displacement, maximum crack width, strains and interface slip with load are discussed. Calculation method of cracking load and ultimate bending capacity of steel-GFRP strips-UHPC composite beams are proposed. The results show that with the increase of reinforcement ratio, the cracking load, ultimate bending capacity and interface slip value corresponding to the ultimate load are improved effectively. However, the development of cracks is inhibited, the crack width, average crack spacing and strain of reinforcement bars are reduced as the reinforcement ratio increased. The strain distribution along the height of mid-span section satisfies the plane cross-section assumption. Theoretical cracking load and ultimate bearing

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capacity of composite beams considering the tensile contribution of UHPC achieve
good agreement with the experimental values.

**Key words:** Ultra-high performance concrete; Glass fiber-reinforced polymer;
Composite beam; Flexural performance; Cracking load; Ultimate bearing capacity

1. Introduction

Glass fiber-reinforced polymer (GFRP) is a high performance material with high
specific strength, corrosion resistance, fatigue resistance. Therefore, it is a good
option for replacing steel and has been applied widely in bridges, ocean engineering,
disaster prevention and mitigation engineering [1,2]. However, due to the low elastic
modulus and high project cost for FRP, it is mainly introduced to reinforce old bridges
and replace tensile members partly. Recently, lots of research has been conducted on
GFRP-concrete composite structure, including the effect of interface connection
methods on mechanical properties of composite structures [3-6], structural
combination forms and its static and fatigue performance. Research results show that
the shear strength of the interface can be improved effectively by the treatment of
applying epoxy resin adhesive and laying crushed stone, and GFRP-concrete
composite structure can effectively reduce self-weight and welding workload, and has
good durability and fatigue performance [7,8].

Ultra-high performance concrete (UHPC) is the most innovation cement-based
material with ultra-high compressive strength, ultra-long durability, ultra-high
toughness and good crack resistance [9-11]. Currently, UHPC has been widely used in
the deck system of long-span bridge structures [12,13]. However, due to the
differences in mechanical properties and composition between UHPC and ordinary
concrete, the existing research on GFRP-UHPC composite structure is limited, and
the formula for predicting the crack width of UHPC can not accurately reflect the
Presently, researchers have carried out related research on cracking characteristics and crack width calculation methods of UHPC composite structures, such as, Kwahk et al. [14] and Deng et al. [15] conducted research on the crack resistance of UHPC beams, and the results show that the existing codes were too conservative in the design of the crack load. Luo et al. [16] investigated the crack resistance of steel-UHPC composite beams in negative moment regions, the results show that increasing the reinforcement ratio and the thickness of the UHPC layer, and reducing the spacing of shear connectors could inhibit and delay the development of cracks in the UHPC layer. Subsequently, Rahdar et al. [17], Xu et al. [18] and Jin et al. [19] analyzed the effects of reinforcement types and reinforcement ratios on the cracking strength and crack width of the UHPC beams. Choi et al. [20] conducted research on flexural behavior of inverted T-shaped steel-UHPC composite beams. The results show that the cracking strength of composite beam was improved with the reducing space of shear members and the increasing of the thickness of the UHPC layer. Wang et al. [21] designed a fully prefabricated UHPC composite beam, research results show that full dry connection joints could effectively improve the crack resistance and stiffness of the composite beams. Wang et al. [22] put forward an steel strips-UHPC composite bridge deck structure, and the fatigue life assessment method of this composite structure was proposed. Qiu et al. [23] conducted a simplified formula for calculating reinforcement stress considering the tensile contribution of UHPC. Therefore, the cracking issue of composite beams under hogging moment has been one of the main concerns for the researchers, especially the factors affecting the cracking properties of UHPC composite beams. However, the theoretical calculation methods, considering the tension contribution of UHPC, for predicting the crack load.
of UHPC composite beams required further research.

In order to realize the efficient utilization and combination of materials, solve the welding fatigue cracking of existing composite bridges, and effectively improve the crack resistance of composite bridge deck in hogging moment regions, two new materials, GFRP and UHPC, were used in the designing of composite bridges, and a steel-GFRP strips-UHPC lightweight composite beam bridge was proposed, as shown in Fig. 1. GFRP strips used in this structure were placed on the tension side to enhance the cracking strength in the positive moment regions, and the UHPC was applied to the compression side. It was an innovative bridge deck structure connected by high-strength arc-shaped shear connectors and special interface reinforcement treatment. Compared to traditional steel-UHPC and GFRP-concrete structures, this bridge deck structure has the characteristics of lightweight, good durability and crack resistance. Meanwhile, it can effectively reduce the welding workload and lower the influence of welding fatigue on structural safety [24].

![Fig.1. Steel-GFRP strips-UHPC composite bridge.](image_url)

This study investigate the longitudinal flexural cracking characteristics of the steel-GFRP strips-UHPC lightweight composite beam, and the effects of reinforcement ratio on the failure mode, mid-span displacement, interface slip, strain, maximum crack width and crack distribution of the composite beam are analyzed.
The characteristics and distribution of crack propagation are thoroughly investigated, Calculation methods are proposed for predicting the cracking load of the UHPC composite beams at the Serviceability Limit State [25], and their ultimate bearing capacity.

2. Experimental program

2.1. Specimens design

Two composite beam specimens, with the reinforcement ratios of 3.91% and 5.03% for L-1 and L-2, respectively, were designed based on Eurocode 4 [26]. The total length of composite beam was 2100 mm. The diameter of longitudinal reinforcement bar was 14 mm, and the spacing of longitudinal arrangement bars are shown in Fig. 2. The thickness of longitudinal reinforcement protective layer was 22 mm. The diameter of transverse reinforcement bar was 10 mm, and the spacing of the transverse reinforcement bar was 150 mm. The shear connectors were made of HRB400 reinforcement bars with a diameter of 14 mm, the inner diameter of arc-shape shear connector was 31 mm, and the spacing along the longitudinal arrangement of composite beams was 290 mm, the placing detail of the shear connectors are shown in Fig. 2. The type of steel fiber was straight steel fiber with a length of 13 mm and a diameter of 0.2 mm, the steel fiber content of UHPC was 1.5%.
Fig. 2. Design of composite beam specimens (unit: mm).

2.2. Materials

The UHPC layer designed for the purpose of this study consisted of portland cement, silica fume, fly ash, quartz sands, quartz powder, mineral powder, super plasticizer, steel fibers and water. The water-paste material ratio of UHPC was fixed at 0.18. Table 1 shows the mix proportions of UHPC. When the UHPC layer was cast,
three UHPC cubes (100mm × 100mm × 100mm), three prisms (100mm × 100mm × 400mm) and six prisms (100mm × 100mm × 300mm) were made and cured under the same conditions as the composite beam specimens to test the compressive strength, flexural strength and elastic modulus of the UHPC material, respectively. The weight of the GFRP strips was 19 kN/m³. The tensile strength of the steel fiber was 2850 MPa, and the elasticity modulus was 200 GPa. The mechanical parameters of the materials obtained experimentally are shown in Table 2.

Table 1 Mix proportions of UHPC

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Amount (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement (52.5R)</td>
<td>773.2</td>
</tr>
<tr>
<td>Silica fume (2-280 mm)</td>
<td>215.3</td>
</tr>
<tr>
<td>Quartz sands (450-900 μm)</td>
<td>848.4</td>
</tr>
<tr>
<td>Fly ash</td>
<td>78.2</td>
</tr>
<tr>
<td>Steel fibers (1.5% Vol.)</td>
<td>118.5</td>
</tr>
<tr>
<td>Mineral powder (S95)</td>
<td>78.2</td>
</tr>
<tr>
<td>Quartz powder (50.2 μm)</td>
<td>77.3</td>
</tr>
<tr>
<td>Super plasticize (1.5% Vol)</td>
<td>20.1</td>
</tr>
<tr>
<td>Water</td>
<td>192</td>
</tr>
</tbody>
</table>

Table 2 Main material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Elasticity modulus (GPa)</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Yield strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UHPC (steel fiber content 1.5%)</td>
<td>44.1</td>
<td>3.4</td>
<td>123.4</td>
<td>12.3</td>
<td>4.8</td>
</tr>
<tr>
<td>Q345C steel plate</td>
<td>206.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>HRB400 ribbed bar</td>
<td>206.0</td>
<td>540.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>HRB400 plain bar</td>
<td>206.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>540.0</td>
</tr>
<tr>
<td>GFRF direction</td>
<td>32.6</td>
<td>270.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GFR perpendicular fiber direction</td>
<td>11.2</td>
<td>80.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: S is the standard deviation.
2.3. Specimens fabrication

Fig. 3 shows the main construction process of steel-GFRP strips-UHPC composite beam. Initially, shear connectors arranged longitudinally was welded on the surface of the I-section steel beam with double-sided welding, the arc-shaped reinforcement shear connector was connected I-section steel beam and the GFRP strips with bolt, GFRP strips were placed on the upper flange plate of the I-section steel beam with equally spacing, and the upper flange plate of the I-section steel beam and GFRP strips were connected by the arc-shaped reinforcement shear connector, as shown in Fig. 3. Then, high-temperature resistant epoxy resin glue was smeared on the GFRP layer surface, which could work well at 150-200℃, and a layer of gravel with a particle size of 4.75-9.5 mm was spread, and the coverage rate was controlled at 50% by an equal ratio of mass to the area. After that, the steel meshes were arranged and the templates were made. Finally, the UHPC was cast and cured at room temperature for 24 h, and then cured by steam curing at a temperature of 90-100℃ and relative humidity of 95% for the next 48 h.

Fig. 3. Construction process of specimens.
2.4. Loading scheme

A three-point bending loading method was adopted for the composite beams, and a 100 T jack was utilized for loading equipment [26], as indicated in Fig.4. The test loading process was divided into the pre-loading stage and the formal stage. At the pre-loading stage, 5% of the estimated ultimate load was pre-loaded five times to eliminate the gap between components and ensure the proper function of the jack, dial gauges, and strain gauges. During formal loading, the force-based control was used to apply the static load with a 20 kN load increment. When the load reached about 70% of the ultimate load, it was changed to displacement control with 1 mm per step. Each step lasted for 5-10 min. When each step of the load was stable, the deflection at the mid-span and the end of the beam, the interface slip value of the beam end, the strain value, the crack width and length, and the crack distribution were recorded.

![Strain gauges](image1)

![Hydraulic jack](image2)

Fig. 4. Loading schematic and measuring points of composite beam (unit: mm).

3. Results and discussion

Test results are presented in Figs. 5-12 and Table 3. The failure mode and mechanism, load-deflection response, load-interface slip response, the strain distribution characteristic and the crack propagation characteristics are addressed.

3.1. Failure mode and mechanism

The failure mode of the specimens are shown in Fig. 5. During the initial loading
process, there were no cracks on the surface of the UHPC layer. With the increase in
the load, due to the stress of the UHPC layer reached its tensile strength value,
epidermal cracks appeared on the upper surface of the UHPC layer, and the cracks
mainly distributed near the mid-span position of the composite beam, during this
stage, the cracks were thin but numerous. As the load increased to 0.85% of the
ultimate value, the I-section steel beam yielded, and the number and width of cracks
on the upper surface of the UHPC layer increased and extended to the depth direction
of the UHPC layer. When the load achieved the ultimate value, the maximum crack
width for L-1 and L-2 were 2.3 mm and 1.24 mm, respectively. the cracks were
mainly distributed in the range of 300mm on both sides of the middle span of the
composite beam, and presented two main cracks. Obvious yielding deformation at the
lower flange of the I-section steel beam occurred. Table 3 summarizes the main test
results of composite beams, it can be seen from Table 3 that the ultimate bearing
capacity of specimens increased by 5.95% when the reinforcement ratio was increased
from 3.91% to 5.03%, therefore, increasing the longitudinal reinforcement ratio in
UHPC layer could improve the ultimate bending capacity of composite beams.

**Table 3** The main test results of the composite beams.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{ut}$ (kN)</th>
<th>$\omega_{ut}$ (mm)</th>
<th>$\mu$ (mm)</th>
<th>Failure characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-1</td>
<td>487</td>
<td>36.84</td>
<td>2.30</td>
<td>Two main cracks on the UHPC layer, and yielding deformation at the lower flange of the I-section steel beam.</td>
</tr>
<tr>
<td>L-2</td>
<td>516</td>
<td>35.54</td>
<td>1.24</td>
<td></td>
</tr>
</tbody>
</table>

*Note:* $P_{ut}$ is the maximum measured load, $\omega_{ut}$ is the mid-span deflection corresponding to the ultimate load, and $\mu$ is the maximum crack width corresponding to the maximum load.
3.2. Load-deflection response

Fig. 6 indicates the mid-span load-deflection curve of the composite beams in hogging moment regions. They are divided into three stages: elastic stage, crack development stage and yield stage. In the elastic stage, the load-deflection curves showed a linear trend, and no visible cracks were detected on the surface of specimens. With the increase in the load, visible cracks (crack width about 0.05mm) initiated and developed on the upper surface of the UHPC layer, and the composite beam entered the crack development stage, in this stage, fine cracks initiated and developed on the upper surface of the UHPC, the stiffness of the composite beam gradually decreased, indicated by the change slop of the load-deflection curve, at the end of this stage, the I-section steel beam yielded. Once the load reached to about 85% of its ultimate value, the composite beam entered the yield stage. During this stage, due to the steel beam has yielded, the crack width on the upper surface of UHPC layer increased rapidly. When the specimens were destroyed, there were obvious buckling deformation at the lower flange of the middle span of the I-section steel beam, and the stress of the longitudinal reinforcement bar has reached its yield strength, the mid-span deflection of L-1 and L-2 specimens were 36.84 mm and 35.54 mm, respectively. as shown in Fig. 6. Therefore, the composite beam lost its bearing...
capacity due to the successive yielding of steel beams and longitudinal reinforcement bars, meanwhile, the UHPC layer reached its ultimate tensile stress value.

**Fig. 6.** Load-deflection curve of composite beam.

### 3.3. Strain characteristic analysis

#### 3.3.1. Load-strain response

Fig. 7 (a) depicts the load-strain curves at the lower flange of I-section steel beam. It can be seen from Fig. 7 (a) that the load-strain curve of the lower flange of I-section steel beam presents two obvious stages: elastic stage and yield stage. When the load was small, the strain changed linearly with the increase of load. When the load achieved to 80%-85% of the maximum load, the strain at the lower flange of the I-section steel beam increased sharply, and the I-section steel beam yielded, meanwhile, the slope of the load-strain curves decreased.

The average value of the measured strain of each longitudinal reinforcement bar was obtained, The load-strain curve of the reinforcement bar is shown in Fig. 7 (b). It can be seen from Fig. 7 (b) that the curve of load-reinforcement bar strain presents three stages: elastic stage, crack development stage and yield stage. When the load was relatively small, the curve changed linearly, there was no cracks on the surface of UHPC layer, and the reinforcement bars and UHPC layer worked together. As the load reached to about 20% of the ultimate load, due to the UHPC layer reached its the
tensile strength and cracked, the load borne by the reinforcement bar improved, and the strain of the reinforcement bar augment quickly, while the slope of the load-strain curve decreased. When the load reached to about 85% of the ultimate load, the I-section steel beam yielded, the strain of reinforcement bars increased sharply, the load-strain curve of reinforcement bars changed nonlinearly, and the slope decreased. Comparing the load-strain curves of the two specimens, it was found that the strain of reinforcement bar for L-2, with high reinforcement ratio, was slightly less than that of L-1, therefore, increasing reinforcement ratio could reduce the strain of reinforcement bar.

![Load-strain curve](image)

(a) strain at lower flange of I-section steel beam  (b) Load-reinforcement strain

Fig.7. Load-strain curve.

### 3.3.2. Strain distribution along the height of the mid-span section

Fig. 8 depicts the strain distribution along the height of the mid-span section under different loads. It can be seen from Fig. 8 that the strain distribution along the height of the mid-span section changed linearly during the initial loading process. As the load continued to increase, the upper surface stress of UHPC layer reached its cracking strength value firstly and entered in strain hardening state. The strain distribution along the height of the mid-span section changed nonlinearly gradually, and the neutral axis moved down along the height of the composite beam. Based on
the experimental analysis results in Fig. 8, when the load was less than 80% of the ultimate load, the strain distribution along the height of the mid-span section basically conformed to the plane-section assumption. With the continues increase in the load, due to the upper surface crack width exceed the maximum value of the strain gauge, it was inability to measure the strain value corresponding to ultimate load. When the I-section steel beam was yielded, there were obvious interface slip between the I-section beam and the UHPC layer, and due to the redistribution of cross-section stress, the UHPC layer and the I-section steel beam presented obvious bending deformation around their own neutral axes respectively, and the distribution of the cross-section strain was no longer continuous. However, the cross-section strain along the height of the composite beam still approximately satisfied the plane cross-section assumption.

3.4. Load-interface slip response

The interface slip between the I-section steel beam and the UHPC layer was measured, Fig. 9 demonstrates the load-interface slip curves of the specimens. It can be seen from Fig. 9 that the change trend of load-interface slip curves were basically similar. When the load was less than 100 kN, the interface slip value was very small,
and the horizontal shear load at the interface was mainly borne by the friction force and mechanical bite force between the UHPC layer and I-section steel beam. As the load continued to increase, the UHPC layer of the composite beam cracked, the curves increased almost linearly, while the slope of the curve decreased. During this stage, the interface friction and mechanical bite force gradually quit the working state, and the horizontal shear load was mainly borne by the shear members. When the load approached the ultimate value, due to the interfacial arc-shaped shear connector yielded, the interface slip value increased rapidly. Meanwhile, from the load-interface slip curve of the two specimens in Fig. 9, it can be seen that under the same load, the interface slip value of L-1 was less than L-2. Thus, the longitudinal tensile reinforcement bar could effectively restrain the development of cracks. Under the same load, due to the specimen with high reinforcement ratio had better deformation resistance, the interface relative slip value was large.

![Load-interface slip curve](image)

**Fig. 9.** Load-interface slip curve.

### 3.5. Crack propagation characteristics

The maximum crack width on the upper surface of UHPC layer for composite beams were measured, and the load-maximum crack width curves of specimens L-1 and L-2 were obtained, as shown in Fig. 10. It can be seen from Fig. 10 that the load-maximum crack width curve is distributed in two stages: crack propagation stage
and yield stage. When the maximum crack width was less than 0.2 mm, the maximum crack width changed linearly with the increase in load. With the increase in load, the steel tended to be yielded and the load increasing rate was slow down, while the crack width increased rapidly. Comparing the crack propagation trend of specimens L-1 and L-2 in Fig. 11, it can be see that the curve slope of L-2 was greater than that of L-1, meanwhile, the maximum crack width of specimen L-1 was greater than that of specimen L-2 with the same load. When the reinforcement ratio of composite beams increased from 3.91% to 5.03%, the load values corresponding to the characteristic crack width were improved by 16.7%, 11.1%, 15.4%, 13.6% and 6.0% respectively. Therefore, increasing the reinforcement ratio could effectively inhibit the propagation of cracks and reduce the maximum crack width.

Fig. 10. Load-maximum crack width curve.

Fig. 11. Loads at characteristic crack widths.
Based on the regulation of crack width design in the ultra-high performance fibre-reinforced concretes of French [25,27], the crack distribution on the surface of the UHPC layer with main crack widths of 0.02 mm, 0.05 mm, 0.1 mm, 0.2 mm and failure were respectively measured, as shown in Fig. 12. It can be seen from the Fig. 12 that with the increase in load, the number of cracks gradually rose, the crack propagated from the mid-span to both ends of the specimen. When the maximum crack width increased to 0.2 mm, the number of cracks gradually tended to be stable and the specimen lost its bearing capacity subsequently, the cracks were mainly distributed in the range of 300mm at the mid-span, and there were two main cracks. This phenomenon was mainly due to the uneven distribution of internal force and reinforcement stress for composite beams and the layout position of shear connectors. With the action of concentrated load, the bending moment decreased gradually from mid-span to both ends of the specimen, the stress distribution of reinforcement bars were uneven, and the stress concentration occurred at the layout position of shear connectors.
4. Flexural calculation of steel-GFRP strips-UHPC composite beam

Based on the experimental results, the UHPC layer exhibited post-cracking strength, the steel fiber took the bridge role after cracking, thus, the stress of reinforcement bar was reduced to a certain extent, the development of cracks was inhibited and the cracking load value was increased. Therefore, the tensile
contribution of UHPC and the bridging role of steel fiber could not be ignored for the
calculation of cracking load and ultimate bearing capacity of steel-GFRP strips-UHPC
composite beams. Based on the existing experimental results, the tensile constitutive
model of UHPC materials could assume to be a bilinear elastic-plastic constitutive
model.

4.1. Load corresponding to 0.05 mm crack width

Since the UHPC has high elastic modulus and tensile strength, and steel fiber can
restrain the propagation of cracks, when the crack width on the surface of UHPC layer
was smaller than 0.05 mm, According to Rafiee et al. [28] and Yoo et al. [29], the
effect of crack width on tensile behavior and durability of UHPC could be ignored.
Therefore, the visible crack width of 0.05 mm was often regarded as a crack control
index affecting the long-term services of the UHPC layer. Based on the test results,
when the crack width of UHPC layer improved to 0.05 mm, deformation of composite
beam still varied linearly with the load due to strain hardening characteristics of
UHPC, and the cross-section strain distribution along the height of composite beams
changed linearly on the whole. Therefore, it can be assumed that the cross-section
strain distribution of composite beams satisfied the plane cross-section assumption.

Fig. 13. Crack load calculation model.

Fig. 13 demonstrates a calculation model of cracking load $F_{cr}$. Based on the
calculation formula of cracking strain in NFP18-710 [27] and the measured strain data
on the surface of the UHPC layer in this study, when the crack width improved to 0.05
mm, the tensile strain on the top surface of the UHPC layer could be calculated as shown in formula (1):

$$\varepsilon_{0.05} = \frac{f_{ut}}{E_t} + \frac{A_{w1}}{l_0}$$

(1)

Where, $f_{ut}$ is the tensile strength of UHPC; $A_{w1}$ is the increment of the crack width from first visible crack to 0.05 mm crack width; $l_0$ is the characteristic length related to the size of the specimen, which is considered as $\frac{2}{3}$ of the beam height.

Although the UHPC layer and the I-section steel beam has small interface slip with the increase in the load, the curvature of UHPC layer and I-section steel beam remained equal. Based on the plane section assumption and geometric deformation coordination relationship, it can be obtained that:

$$\varepsilon_{u} = \varepsilon_{c} = \frac{y_c - h_c - h_s}{y_c} \varepsilon_{0.05}$$

(2)

$$\varepsilon_{sb} = \frac{y_s}{y_c} \varepsilon_{0.05}$$

(3)

$$\varepsilon_{s} = \frac{y_c - a_s}{y_c} \varepsilon_{0.05}$$

(4)

Where, $\varepsilon_{c}$ is the strain of the lower surface of GFRP; $\varepsilon_{u}$, $\varepsilon_{sb}$ are the strains of the upper flange and the under flange of the I-section steel beam, respectively; $\varepsilon_{s}$ is the strain of reinforcement bar; $y_c$ is the distance from the neutral axis to the lower flange of the I-section steel beam; $y_c$ is the distance from the neutral axis to the upper surface of the UHPC layer.

According to the calculation model in Fig. 13, the bending moment value corresponding to the crack width of 0.05 mm can be obtained, as shown in Formula (5):
Based on the formula of structural mechanics, the cracking load of composite beams can be obtained:

\[ F_{cr} = \frac{4M_{cr}}{l} \]  

(6)

Where, \( A_{s1} \) is the area of the web above the neutral axis of the I-section steel beam; \( A_{s2} \) is the area of the web below the neutral axis of I-section steel beam; \( y_i - y_f \) are the distances from the force centroid of each part to the neutral axis; \( E_s \) is the elastic modulus of longitudinal reinforcement bar; \( E_u = E_g \) is the elastic modulus of I-section steel beam; \( E_g \) is the elastic modulus of GFRP strips; \( l \) is the calculate the span for composite beams.

4.2. Calculation of ultimate bearing capacity

Based on the above analysis of test results, when the composite beam reached the ultimate bearing capacity, both the compression side and the tension side of the I-section steel beam have yielded, the reinforcement bar reached its tensile yielding value, the GFRP strips has achieved their design tensile strength, and stress of the UHPC layer was in the plastic hardening state. Based on the plastic theory of composite structure, the calculation model of ultimate bearing capacity for composite beam under hogging moment is shown in Fig. 14.

![Fig. 14. Ultimate bearing capacity calculation model.](image-url)
On the basis of the internal force balance condition, the neutral axis of composite beam under the ultimate state can be obtained:

\[ N_c + N_g + N_s + N_{st1} = N_{st2} + N_{sw} + N_{sb} \]  

(7)

\[ f_{us} h_c h_c^2 + f_s A_s + f_{us} h_g h_g^2 + f_{st} b_0 (y_0 - h_c - h_g) = f_{st} b_0 (h_c + h_g + t_1 - y_0) + f_{sw} t_w + f_{sb} t_2 \]  

(8)

\[ y_u = \frac{f_{us} b_0 (h_c + h_g + t_1) + f_{sw} (h_c + h_g + t_1 - y_0) + f_{st} (h_c + h_g) + f_{sb} (h_c + h_g + t_1 - y_0) + f_{sw} t_w + f_{sb} t_2}{2f_{st} b_0} \]  

(9)

The moment of forces about the neutral axis location for composite beam under the ultimate state can be obtained:

\[ M_u = f_{us} b_c y_1 + f_s A_s y_2 + f_{us} b_g y_3 + f_{sw} (h_c + h_g + t_1 - y_0) y_4 + f_{st} (h_c + h_g + t_1 - y_0) y_5 + f_{sw} t_w y_6 + f_{sb} t_2 y_7 \]  

(10)

\[ F_u = \frac{4M_u}{l} \]  

(11)

Where, \( N \) is the axial force, where subscript c, s, g, st1, st2, sw and sb are the axial force components of UHPC, reinforcement bar, GFRP strips, the upper flange tension region and the upper flange compression region of the I-section steel beam, the web and the lower flange of the I-section steel beam, respectively; \( y_0 \) is the distance from on the upper surface of UHPC layer to the neutral axis of the composite beam; \( y_1 - y_7 \) are the distances from the force centroid of each part to the neutral axis; The meanings of other parameters are shown in Fig. 14.

4.3. Validation

The theoretical calculated results of the ultimate bearing capacity were obtained. In order to verified the reliability of the theoretical calculation method, the experimental data in this study and Zhang et al [30] were comparative analyzed. theoretical values without the tensile contribution of UHPC was presented. The results are listed in Table 4. It can be seen from Table 4 that the calculated values of the
method proposed in this study agree with the tested values. When the contribution of tensile strength for the UHPC was considered, the ratios of the calculated values for cracking load and ultimate load to the tested values were 0.860-1.107 and 0.863-0.973, respectively, the average values were 0.956 and 0.914, respectively, the coefficient of variation was 0.093 and 0.043, respectively. However, when the contribution of tensile strength for the UHPC was ignored, the ratio of the calculated ultimate load to the tested value was 0.806-0.893, the average value was 0.847, and the coefficient of variation was 0.040. Therefore, considering the tensile contribution of UHPC, the theoretical calculation results were in better agreement with the tested ones. Therefore, the calculation method and assumptions proposed in this study were better conformed to the engineering practice, and the theoretical formulas proposed in this study could be potentially used to predict the flexural capacity of steel-GFRP strips-UHPC composite beams. However, the authors suggest that more flexural test data for steel-GFRP strips-UHPC composite beams with different design parameters were needed to further verify the prevalent applicability of the proposed theoretical equations.

Table 4 Comparison between theoretical calculated and tested results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Calculated results ①/kN</th>
<th>Calculated results ②/kN</th>
<th>Tested results ③/kN</th>
<th>①/③</th>
<th>②/③</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-1</td>
<td>$E_c$ 163.82</td>
<td>-</td>
<td>180</td>
<td>0.910</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>$F_c$ 473.82</td>
<td>434.70</td>
<td>487</td>
<td>0.973</td>
<td>0.893</td>
</tr>
<tr>
<td></td>
<td>$E_u$ 172.07</td>
<td>-</td>
<td>200.00</td>
<td>0.860</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>$F_u$ 481.98</td>
<td>454.43</td>
<td>515.9</td>
<td>0.934</td>
<td>0.881</td>
</tr>
<tr>
<td>SU-S</td>
<td>$E_c$ 177.66</td>
<td>-</td>
<td>160.43</td>
<td>1.107</td>
<td>-</td>
</tr>
<tr>
<td>Zhang et al. 2020</td>
<td>$F_c$ 499.39</td>
<td>466.72</td>
<td>578.89</td>
<td>0.863</td>
<td>0.806</td>
</tr>
<tr>
<td></td>
<td>$E_u$ 192.78</td>
<td>-</td>
<td>204</td>
<td>0.945</td>
<td>-</td>
</tr>
<tr>
<td>SU-B</td>
<td>$F_u$ 475.30</td>
<td>434.48</td>
<td>536.89</td>
<td>0.885</td>
<td>0.809</td>
</tr>
</tbody>
</table>
Note: Calculated results ① represents the calculated ultimate flexural capacity considering the tensile contribution of UHPC; Calculated results ② represents the calculated ultimate flexural capacity without the tensile contribution of UHPC; Tested results ③ represents the experimental results in this paper.

5. Conclusion

This study conducted a experimental study to investigate the flexural behavior of the steel-GFRP strips-UHPC composite beam under hogging moment regions. The influence of reinforcement ratio on the failure mode, mid-span deflection, interface slip, strain and maximum crack width of this composite beam were investigated. The crack propagation characteristics and distribution law were clarified, and the calculation methods for the cracking load and ultimate bearing capacity of composite beam were presented, the following conclusions can be drawn:

(1) The composite beams presented obvious bending deformation and interface slip at the end of the specimens, and there were obvious buckling deformation at lower flange of the I-steel beam, and there were two obvious main cracks.

(2) Increasing reinforcement ratio appropriately could improve the cracking load, ultimate bearing capacity and interface slip value corresponding to ultimate load. Moreover, it could effectively inhibit the development of cracks, and reduce the strain of reinforcement bars, crack width and average crack spacing.

(3) When the maximum crack width was less than 0.2 mm, the maximum crack width of the specimens changed linearly with the increase in load, however, it increased rapidly once the reinforcement bar was yielded.

(4) Considering UHPC tensile contribution, the theoretical calculation method of the cracking load and flexural ultimate bearing capacity for the steel-GFRP strips-UHPC composite beam were established, and the theoretical calculation
results show a correlation with the relevant test results, indicating that they can predict flexural capacity of the composite beams.

(5) Due to the limited number of test specimens, more flexural test datas were needed to verify the applicability of the proposed calculation method of the cracking load and flexural ultimate bearing capacity for the steel-GFRP strips-UHPC composite beam. Therefore, further experimental studies, as the depth-thickness ratio of the composite beam and the UHPC layer, and finite element analysis on the flexural performance of this composite beam should be carried out in the future.

**Declaration of Competing Interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

**Data availability**

The data are presented in the paper.

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Chinese)


