EXPERIMENTAL AND THEOREtical STUDY ON THE
BEHAVIOR OF THE LAMINATED ACTION OF STEEL-CONCRETE COMPOSITE 
BEAM IN NEGATIVE BENDING MOMENT REGION

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ABSTRACT

A new type of steel-concrete composite-laminated action beams (CLB) is developed to improve the crack resistance of top concrete slabs. It is an improved version of double steel-concrete composite beams (DCB), in which the shear stress between top concrete slab and upper steel flange is released by the uplift-restricted and slip-permitted connectors (URSP). To investigate the static mechanical behavior of CLBs in the negative moment region, an experimental test on two CLB specimens and one DCB specimen was carried out, and the theoretical study on the distribution equation of interface slip, the calculation formulas of sectional bending stiffness and ultimate bending moments was conducted. The results show that although the flexural bearing capacity of CLB is slightly lower than that of DCB, the crack resistance was markedly better than that of DCB. The slips and ultimate bending moments predicted with the simplified formulas are good in agreement with tests results.

1. Introduction

Traditional steel and concrete composite beams have been widely used in civil engineering in the past several decades because of their low weight, low cost, and the performances of the full use of the two materials. However, in the negative bending moment region of continuous beams, especially the long-span continuous beams, concrete cracking and steel flange buckling always troubled engineers [1-3]. In order to alleviate the local yield of the pressed web and the cracking bending moment of the top concrete slab can be improved, and the maximum width of cracks also can be reduced [23]. Based on the viewpoint of reducing the transmission of the shear stress while maintaining the consistent of the vertical displacement of the concrete slab and the steel beam, the uplift-restricted and slip-permitted (URSP) connector was proposed [24]. And the mechanical properties and slip performance of URSP connectors have been investigated by conducting experiment and theoretical analysis [25-26].

With the DCB and URSP connectors as the background, a composite-laminated action beam (CLB) applied in the negative bending moment region of continuous composite beams was proposed [27]. The construction of CLB is shown in Fig. 1(b), in which the connectors connecting the top concrete slab and the steel beam is URSP connectors. According to the research about CLBs applied in the practical engineering, the composite-laminated action can effectively improve the stiffness of the cross-section in the negative bending moment region of continuous composite beams, and adjust the internal force of structures, thereby improving the mechanical performance of structures [28-29].

In order to study the mechanical properties of the new structure—CLB, an experimental study of negative moment region was conducted. Two CLB specimens and one DCB specimen with the same parameters were designed and tested for comparison. The theoretically study of CLB is also conducted. The formulas for calculating the ultimate flexural capacity, bending stiffness, and slip distribution of CLB were proposed.

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2. Experimental studies

2.1. Specimens

As stated above, two CLBs (CLB1 and CLB2) and one DCB specimens were manufactured and tested. All these specimens had the same dimensions. Each specimen was 3.2 m long, and the net-span was 3.0 m with a 100-mm extent portion at each edge support. The main characteristics and nominal dimensions of test specimens are shown in Fig. 2(a), (b) and (c).

![Fig. 2 Parameters of test beams (mm)](image)

In the CLB, a laminated interface was formed between the top concrete slab and the steel beam using the URSP connectors, so the cross section was divided into top and bottom beams along the laminated interface. The top concrete slab was 80 mm thick and 700 mm wide, and the bottom concrete slab was 50 mm thick and 362 mm wide. In the top concrete slab, 8 longitudinal rebars with 12 mm diameter in two layers, and transverse bars with 6 mm diameter and 100 mm spacing were reinforced.

The steel beam was welded by 8-mm-thick SS400 steel plates. It was 180 mm high and 430 mm wide. Stiffening ribs were established at the support and loading sections, as shown in Fig. 2. The diameter and height of the connectors were 13 mm and 50 mm respectively. Fig. 3 shows the construction of the URSP and shear connectors.

![Fig. 3 The Construction of connectors](image)

2.2. Material Properties

The concrete material properties of specimens were tested on test day. The uniaxial compressive and tensile strengths were obtained based on 150-mm cubic specimens. The mean yield strength, ultimate strength, and Young’s modulus of welded steel plates and reinforcing steel bars were tested. The results are given in Table 1.

2.3. Test Setup and Loading Instrumentation

To simulate the stress state in the negative moment region, two ends of each specimen were connected to the ground though military piers. A reverse pressure transducer was applied by the hydraulic jack. The loading was controlled by force. The load increment imposed was 10 kN for each stage at the initiation of the test; when the cracking load was approached, the load increment was reduced to 5 kN to observe the development of cracks. After concrete cracking, the load increment was improved to 20 kN, and when the load reached approximately 80% of the expected ultimate bearing capacity, the load increment for each stage was reduced to 10 kN until the specimens failed. The loading was held constant for 2 min when reaching the default during each stage.

3. Test results analysis

There were three distinct stages in a typical composite beam failure process. Stage I was the stage before concrete cracking, and the materials behaved mainly elastically in this stage. Stage II corresponded to the stage in which concrete had been cracked but the steel profile had not yet yielded. In this stage, the bending stiffness of beams was weakened clearly, and non-linearity started to show in the load-deformation relationship curves.

Generally, the ending of this stage is considered as the ultimate strength of the expected ultimate bearing capacity, the load increment for each stage was reduced to 5 kN until the specimens failed. The loading was held constant for 2 min when reaching the default during each stage.
approximately 0.8 $P_u$ (where $P_u$ was the ultimate bearing capacity). In stage III, the deformation increased rapidly, and the bearing capacity began to decline at the end of this stage. Fig. 6 shows the failure states of CLB1, which exhibited "flexural failure".

3.1. Load-deflection relation

The load-deflection curves of three specimens are shown in Fig. 7. It reflects the relationship between load and deflection at the position of 200 mm distance from the mid-span. The duration time of stage I was very short in the action of negative bending moment for both CLB and DCB. When the load reached 35.7 kN, the initial crack appeared in DCB. The corresponding cracking loads of CLB1 and CLB2 were 70.71 kN and 69.43 kN respectively. The cracking loads of the two CLBs were approximately two times that of DCB.

![Fig. 6 Failure states of the CLB1](image)

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![Fig. 7 Load-deflection relationship curves](image)

Fig. 7 Load-deflection relationship curves

From load-deflection curves, the non-linearity is obvious of both CLBs and DCB. The ultimate bearing capacity of DCB was greater than those of CLBs, but the elastic ultimate bearing capacity of CLBs were basically equivalent to DCB.

3.2. Cracking

In this and the following sections, take CLB1 and the DCB as examples for discussion because the distribution and development of cracks, strain, and stress of the CLB2 were similar to those of CLB1. Fig. 8 shows the crack development at critical loads in stages I and II. The development curves of the maximum crack width as the load increased were shown in Fig. 9. The shapes of the cracks reflect that the cracks were induced by a negative bending moment. Moreover, the symmetric distribution of crack around mid-span indicates that the loads were exerted symmetrically and equivalently around the mid-spans of the specimens.

![Fig. 8 Crack formation and distribution](image)

Fig. 8 Crack formation and distribution

Due to different connector configurations, CLB1 exhibited flexural cracks, while DCB exhibited tensile cracks. More cracks were detected in CLB1 but the maximum crack width in CLB1 was always smaller than that of DCB.

It can be concluded that the crack resistance of the concrete slab could be greatly improved with the laminated action compared to the composite action. The spacing between the main transverse cracks was almost 100 mm in CLB1 and 150 mm in DCB, which was close to the spacing of stirrups in corresponding specimens.

![Fig. 9 Curves of load-maximum crack width](image)

3.3. Distribution of strain and plastic neutral axis

The mechanical properties of composite beams were depend largely on the position of neutral axis in cross sections. The strain distribution along the cross-sectional height at the position of 200 mm away from the mid-span at different loading stages were showed in Fig. 10.

![Fig. 10 Sectional strain distributions](image)

In Fig. 10(a), the slip occurred under a low load level in CLB1, and before that, the strain distribution along the section was linear and satisfied the assumption of a plane section. The top and bottom beams still worked together as a whole after slip occurred excepting for the change of neutral axis. In the middle stage of loading, the bottom of the top beam began to be compressed, and at the same time, two neutral axes appeared in the cross section. It is because the initial bond between top and bottom beams was damaged as load increasing. After that, the top and bottom beams in CLB1 were bent and deformed respectively around their own neutral axes under the premise of keeping the same vertical deformation. The strain distribution along the cross-sectional height of DCB is shown in Fig. 10(b). The difference from the strain distribution of CLB1 was that there was only one neutral axis during the entire loading process, indicating that the top and bottom beams were always a whole structure in the process of stress.

3.4. Relative slip

Measured the maximum slip—load curves were presented in Fig. 11. As shown in Fig. 11, the maximum slippages of CLB1 and CLB2 were approximately two times greater than that of DCB. The slippage distribution along the length of specimens CLB1 and DCB is presented in Fig. 12(a) and (b) respectively, where the horizontal axis is the distance from the mid-span to the measured section. It indicates clearly from Fig. 12(a) and (b) that the slippage was distributed non-linearly along the longitudinal of specimens. This is because the slippage was extremely minor and affected by many factors such as stud weld quality, compaction rate of concrete, and eccentric loading, etc. The greatest slippages of CLB1 and DCB appeared at different positions. For CLB1, the greatest slippage was at the ends of specimen, but for DCB, it was at the position of 1/4 or 3/4 of specimen’s span.

4. Analytical studies of the CLB
4.1. Sectional bending stiffness

Bending stiffness of a cross-section refers to the ability of a member to resist changes in its curved shape. It is the basis for analyzing the deflection and deformation of a structure under load and the ductility of the structure, and is also used in calculating the crack width of a concrete structural member. Therefore, the bending stiffness is very important for the structural mechanical performance analysis.

\[ \phi_B = \frac{M_B}{E_l} \]

(2)

In CLB, the bending stiffness of bottom beam is greater than that of top beam, which results in \( \phi_B \geq \phi_T \). As shown in Fig. 14, the top and bottom beams will separate under the resistance bending moment action. Under the action of URSP connectors, CLB can prevent the separation of top and bottom beams effectively, and keep the curvature of top and bottom beams consistent. Therefore, the smaller one of \( \phi_B \) and \( \phi_T \) is taken as the influence of the resistance bending moment on the bending curvature of cross-section.

![Fig. 14 Bending shape of CLB](image)

The top and bottom beams’ resistance strains \( \varepsilon_1 \) and \( \varepsilon_2 \) caused by the interfacial friction force \( f \) are:

\[ \varepsilon_i = \frac{f}{E_A} \]

(3)

The bending curvature caused by the resistance strain on the cross-section is \( \phi_i \), expressed as:

\[ \phi_i = \frac{\varepsilon_1 + \varepsilon_2}{y_1 + y_2} \]

(4)

Combining equations (3) and (4), the resistance curvature of cross section generated by friction resistance is \( \phi \):

\[ \phi = \frac{f}{y_c} \left( \frac{1}{E_A A_1} + \frac{1}{E_A A_2} \right) + \frac{f y_c}{E_l I_z} \]

(5)

where the \( y_c = y_1 + y_2 \).

The additional bending stiffness \( K_f \) caused by the friction can be expressed as:

\[ K_f = \frac{f}{y_c} \left( \frac{1}{E_A A_1} + \frac{1}{E_A A_2} \right) + \frac{f y_c}{E_l I_z} \]

(6)

The sectional bending stiffness in the negative bending moment region, \( B \) can be expressed as:

\[ B = B_1 + B_2 + B_f \]

(7)

4.2. Calculation of ultimate flexural capacity

As shown in Fig. 15, there are two plastic neutral axes in CLB, the bending moment of CLB is the bending moment sum of top and bottom beam.

![Fig. 15 Calculation model of ultimate bending moment](image)
cross section as a compact one, meaning that the steel profile would not lose stability under pressure before reaching the cross-sectional carrying capacity; and (3) without considering the natural adhesion force of laminated interface.

The whole process of stress can be divided into three distinct stages [31], the stress and strain distribution along cross-section height at each stage are shown in Fig. 15.

(1) The stage before concrete cracking ($M_0$): The top reinforced concrete slab of CLB will crack under a small negative bending moment, and the cracking moment can be expressed as [32]:

$$ M_0 = \frac{y_0 W_0 f_y}{n} $$

(8)

where $y_0$ is the resistance coefficient of section, it is 5.5 for a rectangular concrete slab; $W_0$ is the cross-section resistance, $W_0=2f_yh_yh_c$; $h_c$ is the height of top beam; $f_y$ is the tensile strength of concrete; $n$ is the coefficient of stiffness distribution, $n=B_1/(B_1+B_2)$.

(2) The stage of elasticity working ($M_0 \leq M_0 \leq M_u$): After concrete cracking, the steel beam is still in the elastic working stage. In this stage, CLB is approximately in an elastic working stage. The neutral axis moves down with the increase of moment, the deflection and rotation increase fast.

$$ M_y = M_{y1} + M_{y2} $$

(9)

where $M_{y1}$ and $M_{y2}$ are the elasticity ultimate bending moments of top and bottom beams, respectively.

$$ M_{y1} = \int_2^1 A_1 f_y \left( \frac{h_y - a}{2} - d \right) + f_y \frac{h_y^2}{8} $$

(10)

$$ M_{y2} = \int_2^1 f_y \frac{h_y - a}{2} $$

(11)

where $A_1$ is the area of reinforcing steel bar, $h_y$, $h_c$ is the width and height of top beam respectively, $h_y$ is the height of steel beam, $d$ is the diameter of reinforcing steel bar, and $a$ is the thickness of protective layer, $f_y$ is the tensile strength of reinforcing steel bar.

(3) The stage of plasticity working ($M_0 \leq M_u$): When the tensile steel profile reaches yield strength $f_y$, the bending moment $M_u$ of steel beam expands further with the increase of moment. Based on the theory of total plasticity, the ultimate moment $M_{u1}$ of top beam and $M_{u2}$ of bottom beam can be expressed as:

$$ M_{u1} = a_1 f_y h_y \frac{x^2}{2} + f_y \frac{h_y}{2} \left( h_y - x-a \right) $$

(12)

$$ M_{u2} = W_y f_y + a_1 f_y h_y \frac{x^2}{2} + \frac{2a}{2} \left( h_y - x-a \right) $$

(13)

where $x_1$ is the thickness of the compressed concrete in the top beam, $W_y$ is the cross-sectional bending coefficient of steel profile, $b_{12}$ and $b_{22}$ are the width and height of bottom concrete slab respectively, $x_1$ and $y_1$ are shown in Fig. 15(b).

4.3. Slippage distribution analysis

The bending stiffness is affected by the friction force, and the friction action will also affect the structural slippage. In this section, the theoretical analysis on the interface slip of simply supported CLB under the uniform and the reverse concentrated load is carried out to obtain the slip distribution equations along the beam’s longitudinal.

Considering the symmetry of structure and the slippage distribution, analysis is conducted by taking the mid-span as the coordinate origin, and the range of $x>0$ as the analysis object (Fig. 16).

![Fig. 16 Schematic diagram of structural analysis](image)

4.3.1. Slip under uniform load

The micro-segment deformation model for negative bending moment of CLB under uniform load is shown in Fig. 17. In Fig. 17, $T_1$ and $T_2$ are the tensile forces of top and bottom beams, respectively; $V_1$ and $V_2$ are the shear forces of top and bottom beams, respectively; and $r$ is the pressing force on laminated interface.

![Fig. 17 Micro-segment deformation model under uniform load](image)

The friction $f$ existing on the laminated interface can be expressed as:

$$ f = \mu r $$

(14)

where $\mu$ is the coefficient of friction.

The distribution of load is determined according to the bending stiffness of top and bottom beams [33-34], and the pressing force between the laminated interfaces is:

$$ r = \frac{qE_1I_2}{E_1I_1 + E_2I_2} $$

(15)

The bending moments on the left side of top and bottom beam units are taken and sorted out respectively:

$$ -\frac{dM_1}{dx} + (V_1 + dV_1) + (q - r) \frac{dx}{2} + \mu \nu_1 = 0 $$

(16-1)

$$ -\frac{dM_2}{dx} + (V_2 + dV_2) + r \frac{dx}{2} + \mu \nu_2 = 0 $$

(16-2)

Assuming that the distance of the micro-segment to the right support to be $a(x=0)$:

$$ V_1 + dV_1 + V_2 + dV_2 = -qx $$

(17)

According to the assumption (1):

$$ \phi = \frac{M_2}{E_1I_1} = \frac{M_1}{E_1I_1} $$

(18)

where $\phi$ is the curvature of cross-section, as well as the top and bottom beams.

The compressive strain at the bottom of the top beam and the tensile strain at the top of the bottom beam are:

$$ e_x = \phi \frac{T_1}{E_1A_1} $$

(19-1)

$$ e_y = \phi \frac{T_2}{E_2A_2} $$

(19-2)

Combining the above formulas, there is:

$$ \frac{d\phi}{dx} = \frac{\mu \nu_1 - qx}{EI} $$

(20)
where $E_b E_i A_s + E_f A_f$.

The expression of the first derivative of simply supported CLB slip strain is:

$$
\varepsilon_x = \frac{\mu y_x - q x}{E I} y_x + \mu r(x - 2l) [\frac{1}{2 E_i A_s} + \frac{1}{2 E_f A_f}]
$$

(21)

The boundary conditions of slip and slip strain of simply supported CLB are:

$$
\begin{align*}
\varepsilon_x |_{x=0} &= 0 \\
\varepsilon_y |_{y=0} &= 0
\end{align*}
$$

(22)

The slip distribution equation of laminated interface under uniform load can be expressed as:

$$
\begin{align*}
\frac{\mu y_x (x - 2l)}{2 E I} q x (x^2 - 3l^2) + \mu r (x - 2l) \left[ \frac{1}{2 E_i A_s} + \frac{1}{2 E_f A_f} \right]
\end{align*}
$$

(23)

4.3.2. Slip under concentrated load

Similar to the derivation under the uniform load, the slip distribution equation of laminated interface under concentrated load can be obtained:

$$
\begin{align*}
\frac{P}{2 E I} y_x (x^2 - x I)
\end{align*}
$$

(24)

where $P$ is the concentrated load.

5. Comparisons

5.1. Slip comparison

In the experiment, the friction coefficient $\mu$ between the laminated interface was not measured. According to the relevant research [35], $\mu = 0.7$ was adopted to calculate the slip distribution. Theoretical predictions and experimental responses of CLB1 were shown in Fig. 18, the “T” represents theoretical predictions and the “M” represents measured experimental responses.

From Fig. 18, it can be seen that when the load is 60kN, the measured results are only about half of the analysis results, mainly due to the initial concrete cracking, the ultimate bending capacity of CLB is almost equivalent to that of DCB.

- Concerning the cracks development of top concrete slab, it can be deduced that CLB exhibit flexural cracks, while DCB exhibit tensile cracks. Comparing with DCB, the cracking moment of CLB is nearly doubled, and the maximum crack width is reduced by about 50%.
- The slip distribution formulas which are developed by considering the interfacial friction force are effective for analyzing the slip distribution of CLB. And the formulas obtained by theoretical analysis can predict the flexural bearing capacity of CLB well.

5.2. Ultimate flexural capacity comparison

The critical bending moments obtained from tests and predicted by theoretical analysis are tabulated in Table 2. It can be found that the Eqs. (6)-(11) and the calculation formulas in reference [7] can predict the critical bending moments of CLB and DCB well.

6. Conclusions

Experimental and analytical investigation on the mechanical behaviors of the laminated action steel-concrete composite beams has been conducted in this paper. The mechanical properties including development of cracks, slippage, ultimate bearing capacity of CLB and DCB were analyzed and compared. On the basis of theoretical analysis, the calculate formulas for bending stiffness of cross-section were given. Under negative bending moment, the slip distribution equations considering frictional force of simply supported CLB were conducted. The calculate formulas for ultimate bending carrying capacity were obtained. And the theoretical predictions and experimental responses were good agreement. The following conclusions may be drawn from the present study:

- The static failure mode of the CLB in the negative bending moment region is flexural failure, similar to that of the DCB. In the inelastic stage after

**Table 1**

Experimental and Analytical Results of Critical Bending Moments of Specimens

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Eqs. (kN • m)</th>
<th>CLB Measured (kN • m)</th>
<th>Eqs. to measured</th>
<th>Eqs. (kN • m)</th>
<th>DCB Measured (kN • m)</th>
<th>Eqs. to measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking bending moment</td>
<td>54.7</td>
<td>53.0</td>
<td>1.03</td>
<td>30.2</td>
<td>26.8</td>
<td>1.13</td>
</tr>
<tr>
<td>Elastic limit bending moment</td>
<td>239.5</td>
<td>246.5</td>
<td>0.97</td>
<td>269.7</td>
<td>260.4</td>
<td>1.04</td>
</tr>
<tr>
<td>Plastic limit bending moment</td>
<td>291.9</td>
<td>297.6</td>
<td>0.98</td>
<td>318.6</td>
<td>332.0</td>
<td>0.96</td>
</tr>
</tbody>
</table>

**References**


Nie, J.G., Ma, Y., "Experimental study on the resistance performance of uplift-restricted and slip-restricted stud connectors", Special Structural Periodicals, 32(03), 6-12, 2015.


