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Static and Fatigue Test on Lightweight UHPC-OSD Composite Bridge Deck System Subjected to Hogging Moment

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Abstract

A cost-effective Lightweight Composite Bridge Deck (LCBD) system, including Orthotropic Steel Deck (OSD) and lightweight Ultra-High Performance Concrete (UHPC) layer is proposed to increase the stiffness and fatigue performance of conventional OSD. Static and fatigue tests on two full-scale strip models subjected to four-point bending were carried out. The static nominal cracking stress of the UHPC layer with reinforcement spacing of 80 mm is 24.59 MPa, while it increases to 35.68 MPa when the reinforcement spacing is reduced to half (40 mm); both values are far greater than the nominal stress of 12.7 MPa obtained in the prototype bridge. Increasing the reinforcement ratio can increase the bending stiffness of LCBD and decrease the tensile strain of the UHPC layer, while the change in range is relative slight. Furthermore, the flexural strength of UHPC and the reinforcement ratio are important factors affecting the fatigue life of the UHPC layer. When the reinforcement spacing increases from 40 mm to 80 mm, the fatigue life of the UHPC layer still satisfies related code requirements. Thus, for reduction in the engineering cost and construction complexity, the reinforcement spacing can be set as 80 mm. However, the application of the UHPC as the steel deck pavement, the rib-to-diaphragm welded joint is still prone to fatigue cracks. In addition, the existing S-N curves are hard to directly use for fatigue life prediction of the UHPC layer because of the great differences in the definition of stress level and evaluation index of failure in the fatigue test, which need to be modified in further studies.
Key words: Bridge engineering; lightweight composite bridge deck; UHPC; reinforcement ratio; static and fatigue test.

1. Introduction

Orthotropic steel deck (OSD) is a typical bridge deck system composed of stiffeners (longitudinal and transverse rib) which are perpendicular to each other in longitudinal and transverse directions and together with deck plate (see Fig. 1) \(^1\)\(^{1-8}\). OSD has the advantages of high strength-to-weight ratio, superior integrity, large loading capacity and short construction period. These merits have resulted in a widespread application of conventional OSDs in bridges \(^1\)\(^{1,2}\). A typical OSD as part of a bridge is generally covered by 35 - 80 mm asphalt overlays or by 20 mm polymer-surfacing layers \(^3\). Due to insufficient stiffness of the steel deck plate, excessively heavy transportation loads, and other reasons, conventional OSD pavement might be easily damaged, leading to potential fatigue cracks occurred at the steel bridge deck \(^4\)\(^{1-8}\). To solve these problems, enhancement of the stiffness of the bridge deck has been widely recognized \(^9\)\(^{1-20}\) as one of the effective means to reduce the fatigue stress amplitude in an OSD. In 2002, Buitelaar \(^9\) proposed to strengthen OSD with 50 mm Reinforced High Performance Concrete (RHPC) connected via an epoxy resin bonding layer. In addition, it was successfully applied to Caland Bridge in the Netherlands. Different from ordinary concrete, Ultra-high Performance Concrete (UHPC) \(^21\)\(^{21-22}\) is a type of advanced cement-based composite with superior compressive (>120 MPa) and tensile strength (>8 MPa), high elastic modulus (40 - 50 GPa) and excellent durability. In 2011, Shao et al. \(^10\) have proposed a new type of composite bridge deck system composed of OSD and lightweight UHPC layer, in which the reinforced UHPC and OSD were connected via stud shear connectors. In such new type of Lightweight Composite Bridge Deck (LCBD), the thickness of UHPC layer is generally no more than 50 mm, and the thickness of the steel plate is from 12 to 16 mm.
Although LCBD has been developed for nearly 10 years in China, there is limited research existing on the basic mechanical properties and fatigue performance of such system. Previous push-out test results show that the LCBD connected by shear studs has good mechanical performance. Finite Element Analysis (FEA) and full-scale model tests have shown that the reduction of the peak stress in the steel deck of LCBD is about 50 to 80% compared with the peak stress in OSD without UHPC pavement, resulting in a reduction of the risk of fatigue cracking. In addition, increasing the reinforcement ratio of the UHPC layer can significantly reduce the potential cracking of the UHPC layer. Moreover, previous research has shown that the spacing of the reinforcement should not exceed 50 mm. The LCBD has been applied in many bridges in China, such as the Foshan-Fuchen Bridge, the Zhuzhou-Fengxi Bridge, and the Second-Dongting-Lake Bridge, etc., where the diameter of the reinforcing mesh in the UHPC layers is 10 mm, and the reinforcing spacing is 35 to 50 mm. In addition, the technical standard for lightweighted composite deck system stipulates that the spacing of UHPC reinforcement in the LCBD shall be less than 67 mm.

However, due to the small reinforcement spacing in the UHPC layers, the difficulties associated with UHPC pouring and fibre dispersion are increasing, and the quality control becomes more challenging. Furthermore, a dense reinforcement spacing will also increase the construction cost. To the best of the authors’ knowledge, the major reason for the above requirements or regulations on the reinforcement spacing may be that the flexural and compressive strength of UHPC materials adopted in previous research (and applications) are no more than 30 MPa and 140 MPa, respectively; therefore, the potential maximum structural performance of the LCBD was not reached. However, at present, UHPC with compressive strength of 160 MPa and flexural strength of 40 MPa can be easily produced. Using this kind...
of UHPC with high tensile capacity, it is possible to increase the reinforcement spacing of UHPC layer, improve the construction conditions and reduce the fabrication cost.

So far, previous research mainly focused on the static and fatigue behaviour of LCBDs with the UHPC pavement under sagging bending moment \cite{10,13,15}. There is still lack of knowledge on the static and fatigue behaviours of the LCBD under hogging bending moment \cite{16}. Since both the UHPC layer and the reinforcements resist tensile stresses under hogging moment, it is necessary to investigate the static and fatigue performance of the UHPC layer on the LCBD. Furthermore, although some scholars have carried out a small number of fatigue tests on composite beams \cite{26,27} and the LCBD \cite{11,14,16,23,24} subjected to hogging moment, there were no fatigue cracks observed in the UHPC bridge deck until the end of most fatigue tests \cite{11,16,23,24}. Consequently, the crack propagation process, failure mode and fatigue life under cyclic loading were not identified in their tests. In particular, for the UHPC layer with different reinforcement ratios, the failure mode of LCBD under fatigue load is not well understood. Therefore, it is necessary to carry out more studies to understand the fatigue crack characteristics of the UHPC layer and the relevant fatigue response of the LCBD under hogging moment.

The aims of this study are to propose an economical and practical lightweight composite bridge deck system and to evaluate the static and fatigue performance of the LCBD. Two full-scale strip models of the LCBD were designed and manufactured. Combined with nonlinear finite element analysis, the influence of the reinforcement ratio on the crack propagation process, failure mode, load-deflection relationship, and load-strain relationship under static and fatigue load is discussed. Finally, based on the fatigue test results, the fatigue life of the LCBD with different reinforcement ratios is evaluated.

2. Experimental specimens

2.1 Prototype bridge

A steel-UHPC continuous box-girder highway bridge with three spans (56 m + 103 m + 56 m), located in Foshan, China, served as the prototype bridge structure. In this bridge, the LCBD was adopted as the bridge deck system, in which the UHPC layer is connected to OSD using 13 mm-diameter 40 mm-high welded shear studs at spacing of 150 mm × 150 mm. For the OSD, the thickness of the deck plate is 16 mm. The upper opening width, the height and the thickness of the U-rib are 300, 305, and 8 mm, respectively. The spacing between the diaphragms is 4 m, and the thickness of the diaphragm’s plate is 12 mm. The OSDs are made
of Q345qc steel, which is a typical structural steel used for bridges in China with a nominal yield strength of 345 MPa\textsuperscript{[28]}. The thickness of the UHPC layer is 50 mm. In the UHPC layer, the reinforcing bars (diameter of 10 mm) are arranged at spacing of 40 mm × 40 mm. A typical cross-section of the LCBD is shown in Fig. 2.

![Cross section of the composite bridge deck system](Unit: mm).

2.2 Design and fabrication of a strip model

Based on the prototype bridge, two full-scale strip models with a total length of 3,000 mm, width of 600 mm and height of 742 mm were designed, as shown in Figs. 3 and 4. The spacing between the diaphragms is 1 m. The OSDs were made of Q345qc steel, which is the same as the steel in the prototype bridge. In order to study the static and fatigue performance of the composite deck with different reinforcement ratios, considering the symmetric geometry of the specimens, boundary conditions, and loading mechanisms, a reinforcement mesh at spacing of 40 × 40 mm was used in one half span of the UHPC layer of the strip model (this part of the UHPC layer is named as Spacing-40MM), and a reinforcing mesh at 80 × 80 mm spacing was used in the other half span (this part is denoted as Spacing-80MM). The diameter of the reinforcements in all models is 10 mm, and the yield strength of the reinforcement is 484 MPa. The arrangement of the reinforcement mesh is illustrated in Fig. 4.
2.3 Material properties

The dry mixture used in this study is composed of Portland cement, silica fume, mineral powder, and quartz sand \(^{29}\), and the mix proportion of UHPC is shown in Table 1. Hooked steel fibre with a length \((l_f)\) of 16 mm and diameter \((d_f)\) of 0.22 mm (aspect ratio \(l_f / d_f = 73\)) was adopted. The volume fraction of steel fibers \((\rho_f)\) is 3.5%. The water-to-cementitious ratio of the UHPC mixture is 0.185.

| Cementitious material | Quartz sand | Superplasticizer | Steel fiber | Water |
|-----------------------|-------------|------------------|-------------|-------|
| Cement                | Silica fume | Mineral powder   |             |       |
| 1.0                   | 0.37        | 0.25             | 1.1         | 0.04  |
|                       |             |                  | 0.36        | 0.30  |

After mixing, three cubic compression specimens (100 mm × 100 mm × 100 mm), three flexural specimens (100 mm × 100 mm × 400 mm), six prism compression specimens (100 mm × 50 mm × 50 mm), and three three-point bending specimens (100 mm × 100 mm × 400 mm) were prepared. The thickness of the UHPC layer was 100 mm. A steel plate was bonded to the bottom of the steel plate. The steel plate was 25 mm thick and 1000 mm × 1000 mm in size. The UHPC layer was placed on the steel plate. The layers were then cured for seven days before the test.
mm × 100 mm × 300 mm), and six dog-bone-shaped specimens for tensile tests were prepared to obtain the UHPC material properties. All material test specimens were casted using the same mixture as in the strip test models. After casting the UHPC mixture in the LCBD, the test models and specimens were cured for 2 days covered with plastic membranes. Then, when reaching the curing time at normal temperature, plastic membranes and formwork were removed, and the steam curing process was last for 48 hours at the temperature above 90 °C, which was the same as the prototype bridge. After curing, test specimens and strip models were placed for 60 days in the laboratory, and then the relevant tests were carried out. The results of the material tests are shown in Table 2. It can be seen that the cubic compressive strength, flexural strength and direct tensile strength of UHPC were 182 MPa, 44 MPa and 10 MPa, respectively.

| Specimen | Cubic compressive strength ($f_{cu}$) (MPa) | Flexural strength ($f_{cf}$) (MPa) | Young's modulus ($E_c$) (GPa) | Tensile strength ($f_{ct}$) (MPa) | Elastic tensile strain ($ε_{ct}$) (με) | Cracking tensile strain ($ε_{cc}$) (με) | Limited tensile strain ($ε_{pc}$) (με) |
|----------|-----------------------------------|----------------------------------|-----------------------------|-------------------------------|----------------------------------|---------------------------------|---------------------------------|
| Sp-1     | 183.20                            | 42.82                            | 45.0                        | 10.18                         | 210                              | 1,959                           | 3,234                           |
| Sp-2     | 179.64                            | 45.65                            | 44.5                        | 9.60                          | 222                              | 1,861                           | 3,366                           |
| Sp-3     | 184.53                            | 44.96                            | 46.1                        | 11.28                         | 215                              | 2,216                           | 3,618                           |
| Sp-4     | \                                | \                                | 42.0                        | 10.65                         | 208                              | 2,188                           | 3,421                           |
| Mean     | 182.45                            | 44.48                            | 44.4                        | 10.43                         | 214                              | 2,056                           | 3,410                           |
| COV      | 0.011                             | 0.027                            | 0.034                       | 0.059                         | 0.025                            | 0.073                           | 0.041                           |

Note: $ε_{cc}$ is the cracking tensile strain when the width of the crack is up to 0.02 mm.

3. Test setup and loading protocol

3.1 Static loading scheme

In order to simulate the most unfavourable situation of the UHPC layer under hogging moment, a four-point bending test was performed, where the pure bending length of the strip model is 1,000 mm, as depicted in Fig. 5. The load (P) was transferred to the diaphragms through a distributive girder, and then it was applied to the bridge deck by the diaphragms. To obtain the deformation of the strip model, five linear variable displacement transducers (LVDTs) were arranged at the supporting points (L1 and L5), loading points (L2 and L4 under the diaphragms), and mid-span section (L3). In addition, two dial indicators (Z1 and Z2) were arranged at both ends of the composite deck to measure the relative slip between the
deck plate and the UHPC layer. Also, strain gauges were installed on the UHPC surface in the pure bending region to measure the average strains on the UHPC layer, as shown in Fig. 6.

**Fig. 5** Test setup: (a) schematic diagram of static and fatigue loading test (Unit: mm), and (b) photo of static loading test.

**Fig. 6** Strain gauges arrangement at the UHPC layer (Unit: mm).
During loading process, firstly, a preloaded test under the load range of 0 - 40 kN was conducted to check test instruments. Then, the load was increased with increment of 40 kN up to 500 kN. When the load reached 500 kN, the load increment was changed to 20 kN until the failure of test specimen. During the loading process, the crack pattern, crack width, deflections and strains were recorded. To monitor the variation of the crack width, a crack observation instrument with a measurement accuracy of 0.01 mm was used.

3.2 Fatigue loading scheme

3.2.1 Fatigue stress of UHPC layer

To obtain the most unfavourable stress due to wheel loading in the prototype bridge under standard fatigue load, a linear elastic Finite Element Model (FEM) was established using the software ABAQUS [30]. In order to improve the calculation efficiency, only a half of the box girder was built due to symmetry (see Fig. 7a). In the FEM, C3D8R solid element were used to model the UHPC layer, S4R shell element were used to model the OSD, and T3D2 truss elements were used for the steel reinforcements. The global mesh size of OSD, UHPC layer and reinforcement was 50 mm, 25 mm, and 50 mm, respectively. The mesh size of the concerned area of OSD was refined to 1.0 mm (see Fig. 7b). In this way, the FEM is consisted of 429,912 shell elements, 229,200 solid elements and 85,890 truss elements in total. The UHPC material parameters were taken according to the test results in Table 2. The material constitutive model of steel rebars and OSD was based on a linear elastic model with an elastic modulus of 206 GPa and Poisson's ratio of 0.3 [28]. The local wheel load action on the UHPC layer was analysed according to the standard Fatigue Load Model 3 defined in the code of BS EN1991-2 [31], in which the single axle load is 120 kN. According to the observations during the static test, the measured relative slip between the UHPC plate and deck plate under the load of 240 kN is only 0.018 mm (Fig. 8), which is far less than 0.05 mm. In addition, the shear studs are in linear elastic stage at this moment [19]. Therefore, for reducing the computing cost and improving computing efficiency, the bond slip between steel and UHPC was not considered in this FEM, and a tie constraint was adopted between UHPC layer and deck plate [10,15]. The boundary conditions at the beam ends are that the three degrees of freedom of nodes were restricted at one end, while only the vertical freedom of nodes was restricted at the other end. In addition, symmetric boundary conditions were applied to the symmetry plane.
Fig. 7 Finite element model and loading diagram of the LCBD: (a) finite element model, (b) critical details and monitoring points, and (c) loading diagram.

Fig. 8 Relative slip curve between UHPC layer and deck plate in the static test.

Seven transverse loading conditions were obtained by moving the position of the wheel
load (L1 to L7), as shown in Fig. 7c. Along the longitudinal direction, there were 10 longitudinal loading conditions (T1 to T10) by moving the wheel load position by 100 mm. The right rear wheel was taken as the reference wheel when wheel loads were arranged according to Fatigue Load Model 3. The schematic diagram of loading mode is shown in Fig. 7. The FEA results show that the positions of T3 and L2 are the most unfavourable loading positions for the UHPC layer, in which the tensile stress is 2.17 MPa.

3.2.2 Fatigue loading protocol

The fatigue test set-up was basically consistent with the static test one (see Fig. 5a and Fig. 9). The fatigue test was employed using an electro-hydraulic servo fatigue testing machine with loading capacity of 1,000 kN.

![Photo of fatigue loading test.](image)

During the cyclic fatigue loading, the stress ratio $R$ (the ratio of minimum fatigue stress to maximum fatigue stress) was selected as 0.25 and the frequency was 2.5 to 3 Hz. The load level is determined on the basis of the FEA results under the standard Fatigue Load Model 3, and the corresponding stress level is determined according to the static test results. The fatigue loading sequence was divided into four phases with different load ranges, as described in Table 3 and shown in Fig. 10. It is worth to mention that fatigue cracks started to appear in the Spacing-40MM when the test beam was loaded to 6.25 million cycles. To assess the mechanical properties of the strip model, when the number of cycles reached 0, 10 thousand, 50 thousand, 100 thousand, 200 thousand, 500 thousand, 1.0 million, 1.5 million, 2 million, 2.7 million, 3.7 million, 4.7 million and 6.25 million cycles, the fatigue test was stopped, and static tests were conducted on the specimen. In each static test, the maximum load applied on the strip model was equal to the corresponding maximum fatigue load. In addition,
deflections and strains were recorded for each static test using the same arrangements of LVDT and strain gauges as that in Fig. 5a and 6.

![Fatigue loading protocol](image)

**Fig. 10** Fatigue loading protocol.

**Table 3** Fatigue load sequence.

| Loading Phase | Load range $P_{\text{min}}$ - $P_{\text{max}}$ (kN) | Fatigue load amplitude $\Delta P$ (kN) | Nominal tensile stress amplitude $\Delta \sigma_{\text{p,i}}$ (MPa) | Stress ratio $R$ | Number of cycles $n_i$ ($\times 10^6$) | Frequency $f$ (Hz) |
|----------------|---------------------------------------------|-----------------------------------|-----------------------------------------------|----------------|-------------------------------------|------------------|
| First phase    | 50 - 200                                    | 150                               | 6.61                                          | 6.72           | 0.25                                | 2.7              | 3.0 |
| Second phase   | 75 - 300                                    | 225                               | 9.91                                          | 10.08          | 0.25                                | 1.0              | 3.0 |
| Third phase    | 100 - 400                                   | 300                               | 13.21                                         | 13.44          | 0.25                                | 1.0              | 2.5 |
| Last phase     | 120 - 480                                   | 360                               | 15.86                                         | 16.13          | 0.25                                | 2.5              | End until cracks appeared in the Spacing-40MM |

The nominal stress of the UHPC layer under various loads was obtained using transformed section of steel-UHPC composite deck (Fig. 11). It is assumed that the cross section of the LCBD always remains plane during the bending test and the properties of all of materials are in linear elastic state. From Fig. 11, the nominal stress $\sigma$ on the bottom surface of the UHPC layer under hogging moment can be expressed as Eq. 1 to 5.
Fig. 11 Calculation diagram of the nominal tensile stress of the UHPC layer.

\[
\sigma = \frac{M \cdot (y_2 + h)}{I \cdot n_e} = \frac{P l \cdot (y_2 + h)}{2I \cdot n_e}
\]

(1)

\[
n_e = \frac{E_s}{E_c}
\]

(2)

\[
b_{eq} = \frac{b}{n_e}
\]

(3)

\[
y_2 = \frac{S_1 y_1 + S_3 y_3}{S_1 + S_2 + S_3}
\]

(4)

\[
I = I_1 + S_1 (y_1 - y_2)^2 + I_2 + S_2 (y_2 - y_z)^2 + I_3 + S_3 (y_3 - y_z)^2
\]

(5)

where $\sigma$ is the nominal tensile stress of the UHPC layer under hogging moment; $M$ and $I$ is the sectional moment and the inertia moment of the composite bridge deck, respectively; $P$ is the applied load in the test; $l$ is the span of simply supported girder ($l=900$ mm); $n_e$ is the Young's modulus ratio of steel to concrete ($E_s=206$ GPa, $E_c=44.4$ GPa); $h$ and $b$ is the height and width of the UHPC layer; $b_{eq}$ is the equivalent width of the UHPC layer; $y_i$ is the distance from the cross-sectional centroid of each part to the x-axis ($i=1, 2, 3, z$); and $I_1$, $I_2$, and $I_3$ is the inertia moment of the OSD, steel reinforcement, and UHPC layer, respectively.

Table 4 lists the basic sectional parameters of the test model. Consequently, in each loading phase, the nominal tensile stress amplitude of the UHPC layer under hogging moment can be calculated by Eq. 1 to 5, and the calculation results are illustrated in Table 3.
Table 4 Cross section parameters of the test model.

| Spacing          | b  | h  | ne  | I₁  | S₁  | y₁  | I₂  | S₂  | y₂  | I₃  | S₃  | y₃  |
|------------------|----|----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Spacing-40mm     | 600| 50 | 4.64| 1.48E8 | 15,192 | -71 | 6,869 | 1,099 | 25 | 1.35E6 | 6,466 | 25 |
| Spacing-80mm     | 600| 50 | 4.64| 1.48E8 | 15,192 | -71 | 3,434 | 549.5 | 25 | 1.35E6 | 6,466 | 25 |

4. Static test results and discussions

4.1 Crack development and nominal cracking stress

Table 5 summarizes the test results on the bottom surface of UHPC layer. When the test load reached 561 kN, the first visible crack appeared on the bottom surface of the Spacing-80MM part of the specimen, with a crack width of about 0.06 mm, as illustrated in Fig. 12a and Fig. 13. The visible crack is regarded as the crack width up to 0.03 mm. Thus, conservatively, the previous load level of 542 kN was assumed as the cracking load of the Spacing-80MM, at which the corresponding tensile strain of UHPC layer was 885 με. When the load was 696 kN, the maximum crack width of the Spacing-80MM went to 0.11 mm. When the load reached 830 kN, the first visible crack appeared on the Spacing-40MM part, with a crack width of 0.05 mm. Similarly, the previous load level of 810 kN was assumed as the cracking load of the Spacing-40MM, at which the average tensile strain was 1,215 με. When the load reached 980 kN, the load resistant of the composite deck begun to decline. When it reached the ultimate load, the failure of the composite bridge deck occurred, and then, the U-rib at the Spacing-40MM part yielded, as shown in Fig. 12b. However, the deflection continued to increase, and the UHPC layer at the loading points begun to separate from the deck plate. Fig. 13 shows the crack distribution on UHPC layer at the end of the test.
Fig. 12 Photos of the test composite deck under static load: (a) first crack on UHPC layer, and (b) steel U-rib yield.

Fig. 13 Crack distribution on the UHPC layer at failure

According to the calculation results using Eq. 1, the nominal cracking stress of the Spacing-80MM and Spacing-40MM is 24.59 MPa and 35.68 MPa, respectively, while the design nominal stress in the prototype bridge under heavy traffic is only 12.7 MPa. Moreover, the maximum nominal stress of the UHPC in prototype bridge is generally no more than 18 MPa \[^{12,32}\]. Therefore, the cracking strength of the Spacing-80MM can meet the design requirements for the prototype bridge, which has a safety margin of at least 36.6%. In addition, increasing the reinforcement ratio can greatly increase the nominal cracking stress of UHPC layer, in which the nominal cracking stress of the Spacing-40MM is 1.451 times to that of the Spacing-80MM.

Table 5 Static test results on the bottom surface of UHPC layer.

| Loading process and test results | Loading process and test results |
|----------------------------------|----------------------------------|
| Load (kN)                        | 500    542    561    696    810    830    980 |
| Moment (kN.m)                    | 225    244    253    313    365    374    441 |
| Tensile strain (\(\mu\varepsilon\)) | Spacing-40MM 683    724    780    984    1215   1306   2238 |
|                                  | Spacing-80MM 800    885    921    1198   1766   1865   /  |
| Maximum crack width (mm)         | Spacing-40MM 0      0      0      0     <0.05  0.05   0.07 |
|                                  | Spacing-80MM 0      <0.06  0.06   0.11   0.13   0.14   0.16 |

4.2 Load-deflection relationship
The load-deflection curves of the tested composite deck at the loading points and mid-span are shown in Fig. 14. Initially, the curves are approximately linear. As the load increases, these curves gradually tend to be nonlinear. Take L3 as an example, the load-deflection curves of the LCBD can be divided into four stages (Fig. 14b): (1) a linear elastic stage, in which the deflection increases linearly with the applied load, and the overall stiffness of the composite bridge deck remains constant; (2) the crack development stage, in which the cracks of the UHPC layer start to become visible and gradually develop, leading to the stiffness of the bridge deck gradually decreased; (3) the ultimate stage approaches when the applied load is close to the peak load, the load remains almost unchanged but the deflection increases rapidly. In addition, along with the increase of the crack width at the UHPC layer, the stiffness of the composite deck decreases significantly; and (4) the failure stage, in which the loading capacity drops sharply.

4.3 Reinforcement spacing in the UHPC layer

4.3.1 Nonlinear finite element model

In order to determine reinforcement spacing in the UHPC layer, a nonlinear FEM of the strip model was built in ABAQUS [30]. The UHPC layer, OSD, shear studs and distributive girder were all modelled using C3D8R solid elements, while the steel rebars were modelled using T3D2 truss elements embedded in the concrete solid elements. The concrete damaged plasticity (CDP) model [29,33] was used to define the material behaviour of the UHPC layer, which considering inelastic behaviour of UHPC under different reinforcement spacing (40, 80, 120, and 160 mm). The material parameters of UHPC are taken according to the experimental results in Table 2, and the constitutive model of UHPC in tension and
compression \cite{29} is shown in Fig. 15. The material model for steel rebars and OSD was defined using an elastic-plastic constitutive law with elastic modulus of 206 GPa and Poisson's ratio of 0.3 \cite{28}. Assuming a complete bond between the UHPC layer and shear studs, a tie constraint was used between the UHPC layer and shear studs. The contact between the deck plate and UHPC layer was defined as surface-to-surface hard contact, in which the coulomb friction coefficient was 0.5 \cite{33}. The global mesh size of OSD, UHPC layer and reinforcement was 15 mm, 10 mm, and 10 mm, respectively. The mesh size of the concerned area of OSD was refined to 1.0 mm. The details of the FEM are shown in Fig. 16.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig15.png}
\caption{Constitutive model of UHPC in tension and compression: (a) tensile strain-stress relationship, and (b) compressive strain-stress relationship ($\varepsilon_{ct}$=elastic tensile strain, $\varepsilon_{pc}$=limited tensile strain, $f_c$=tensile strength, $\varepsilon_{pc}$=strain at peak compressive stress, $f_u$=ultimate prism compressive strength (=0.85 $f_{cu}$)).}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig16.png}
\caption{Finite element model of composite deck strip.}
\end{figure}
4.3.2 Influence of the reinforcement spacing

Figs. 17a and b show the load-deflection curves and load-strain curves of the UHPC layer with different reinforcement spacing, respectively. For the load-deflection relationships, the FEA results are in good agreement with the test ones during the whole loading process, where the variations are less than 10% in the nonlinear domain. For the load-strain curves of the UHPC layer with the reinforcement spacing of 40 mm, the FEA results are in good agreement with the test ones, in which the deviation is less than 7%. For the load-strain curves of the UHPC layer with the reinforcement spacing of 80 mm, the difference between the FEA and test results is no more than 15% before the crack width of the UHPC layer less than 0.1 mm. When the crack width is larger than 0.1 mm, although the difference between the FEA and test results is up to 25.9%, the change trend of strain is still consistent. The reason for this discrepancy is that the UHPC material is assumed to be homogeneous in the FEM. However, in the actual test model, the existence of cracks will release part of the stress and redistribute the stress on the UHPC layer. Therefore, the FEM can be recognized as validated one.

![Fig. 17 The FEA and test results of UHPC layer: (a) load-deflection curves, and (b) load-strain curves.](image)

In addition, as shown in Fig. 17a, the bending stiffness of the LCBD increases with the reinforcement ratio, but the increment is rather small. Fig. 17b shows that the effect of the reinforcement on the strain of UHPC layer is insignificant in the elastic stage. In the nonlinear stage, when the reinforcement spacing increased from 40 mm to 80 mm, the maximum difference of the applied load is no more than 6% under the same strain, and when the reinforcement spacing increased from 80 to 120 mm, the maximum load difference is no more than 3%. In Fig. 17b, the cracking strains in the test are much larger than the elastic...
strain $\varepsilon_{ct}$ but are smaller than the cracking strain $\varepsilon_{cc}$ in the direct tension test. It seems that the higher the reinforcement ratio is, the larger the actual cracking strain is. Since the UHPC material is inhomogeneous in the actual test model, for the UHPC layer with less reinforcement, the effect of this inhomogeneity of the UHPC layer is more obvious. Under the visible cracking load, the nominal cracking stress of the UHPC layer can be calculated by Eq. 1. Since UHPC is a strain-hardening material, the strain rather than the stress should be used to evaluate the cracking state of UHPC in the FEM. Fig. 18 illustrates the nominal cracking stress of UHPC layer in the FEA and the test.

![Fig.18](image)

**Fig.18** Comparison on nominal cracking stress of the UHPC layer.

As shown in Fig. 18, the measured nominal cracking stress of the UHPC layer is less than 6% of the nominal cracking stress calculated by the FEM when using their actual cracking strain for evaluation. Besides, when the reinforcement spacing increases from 40 mm to 80 mm, the actual nominal cracking stress is reduced by 31.9%. Considering that the actual cracking strain may be further reduced when the reinforcement spacing is enlarged, under existing test results, it is conservatively suggested that the spacing of reinforcement should not exceed 80 mm.

5. Fatigue test results and discussions

5.1 Fatigue cracking development of the LCBD

5.1.1 Fatigue cracking in steel members

During the fatigue test, when the fatigue load reached 1.142 million cycles in the first phase, a fatigue crack was observed at the rib-to-diaphragm welded joint (Fig. 19a), where the reinforcement spacing in the UHPC layer is 80 mm. To prevent the OSD from further damage, the fatigue crack at the rib-to-diaphragm welded joint was repaired by penetration.
welding as shown in Fig. 19b. After the repair, in the second phase of the fatigue loading, the
cracked again (Fig. 19c), and the base metal of the diaphragm also
cracked. At the end of the test, the maximum crack length at the rib-to-diaphragm welded
joint was 47.6 cm (Fig. 19d).

Fig. 19 Photos of the fatigue cracking at the rib-to-diaphragm welded joint, and cut-out of the
diaphragm in the OSD

5.1.2 Fatigue cracking on UHPC layer

During the fatigue test, the fatigue crack development process on the UHPC layer was as
follows: in the first and second phase, no crack was found on the UHPC layer, showing that
the fatigue life of UHPC layer is much higher than that of the diaphragm cut-outs where the
through ribs cross the diaphragms. At the end of the third phase, a micro crack (called CR1)
with a width of 0.03 mm and length of 26 mm appeared in the Spacing-80MM near the steel
diaphragm. Through on-site evaluation, it should be noted that the CR1 crack may have been
caused by casting defects because of the crack shape is consistent with the shape of the
hooked fiber. In the last phase, when the number of loading cycles reached 4.488 million, the
crack length of CR1 expanded to 50 mm while the crack width was unchanged (Fig. 20a). At
5.297 million loading cycles, there were 7 microcracks on the side of the UHPC layer, but the

crack depth was very small. CR1 crack did not expand further, and a new tiny crack appeared
at the section of the shear span of the Spacing-80MM (the width of crack was 0.05 mm). It
can be considered that fatigue damage started in the Spacing-80MM at this time. At 5.777
million loading cycles, in the last phase, CR1 crack remained unchanged, and two new cracks
appeared on the Spacing-80MM. However, at this time, no visible cracks were found on the
Spacing-40MM. As the cyclic loading continued to 6.25 million cycles, the cracks in the
Spacing-80MM further expanded, and fatigue cracks also appeared on the Spacing-40MM.
At the end of the fatigue test (6.25 million cycles), for the Spacing-80MM, a transverse through crack with a width of 0.08 mm and depth of 20.33 mm appeared, as depicted in Fig. 20c. Thus, the UHPC layer of the Spacing-80MM was completely damaged. For the Spacing-40MM, the maximum crack width of UHPC layer was only 0.03 mm distributed in the stress concentration area near the diaphragm, and no cracks were found in any other region. The crack distribution of UHPC layer is shown in Fig. 21.

![Fig. 20 Photos of fatigue crack at the UHPC layer.](image)

![Fig. 21 Fatigue crack distribution on the UHPC layer.](image)

Based on the above analysis and the crack distribution in Fig. 21, it is concluded that the fatigue crack is more likely to appear in the stress concentration region. In the last phase, the number of fatigue cracks remained unchanged, but the length and depth of the main cracks increased sharply, resulting in the same failure mode as that in static test. The fatigue resistance of the UHPC layer is much higher than that of the OSD. The number of fatigue cracks on the Spacing-80MM is more than that on the Spacing-40MM.
5.2 Static load test results after different loading cycles

5.2.1 Moment–strain curve of UHPC layer

In order to reduce or even eliminate the adverse effects of pouring defects and test error, the average strain of two pure bending sections was taken for comparative analysis. The bending moment versus strain curves for the UHPC layer in static load tests after different loading cycles are shown in Fig. 22. Up to 90 kN·m, the average strain curve of the UHPC in the monitoring regions of Spacing-80MM and Spacing-40MM increases linearly, even if some cracks occurred. With the continuous accumulation of loading cycles, the variation of the bending stiffness is not significant before 4.7 million loading cycles, and the degree of plastic damage in the LCBD is not obvious. However, when the load reaches 6.25 million cycles, the stress is released from the main cracks in the Spacing-80MM, which leads to the measured strain decreases.

![Moment-strain curves of UHPC layer after different loading cycles](image)

**Fig. 22** Moment-strain curves of UHPC layer after different loading cycles: (a) Spacing-80MM, and (b) Spacing-40MM.
5.2.2 Moment-deflection curves

Fig. 23 shows the relationship between the deflection and moment at two loading points located at the Spacing-80MM and Spacing-40MM, respectively, in the static load tests after different loading cycles. With the accumulation of loading cycles, the relationship of the moment-deflection curves increases linearly. Before 3.7 million loading cycles, the slope of the curves almost remains unchanged, and the stiffness degradation of the LCBD is insignificant. However, after 3.7 million loading cycles, the slope of the curve increases slightly, and the stiffness of the LCBD gradually degrades.

Fig. 23: Moment-deflection curves of UHPC layer after different loading cycles.

6 Fatigue life evaluation

6.1 Stress comparison with conventional OSD

To reflect the efficiency of the LCBD, the most unfavorable stress value at the monitoring points were calculated by three linear elastic FEMs, which were the model of conventional OSD system, the model of LCBD with the reinforcement spacing of 80 mm (denoted as FEM-RS80), and the model of LCBD with the reinforcement spacing of 40 mm (denoted as FEM-RS40), respectively. These FEMs and corresponding loading conditions are based on Fig. 7. According to existing research [1-7], five typical fatigue-prone details were selected as the monitoring points (point-A to point-E) (see Fig. 7b and c). In comparison with the nominal stress method, the hot spot stress method is regarded as a more appropriate method to predict the fatigue life of complex geometries such as welded connections in OSDs [34,35]. Based on the IIW [34], the hot spot stresses at the monitoring points can be obtained by applying extrapolation approach, as shown in Eq. (6) and Fig. 24. Therefore, the maximum hot spot stress of the monitoring points can be obtained, as illustrated in Table 6.
\[ \sigma_{hs} = 1.67\sigma_{0.4t} - 0.67\sigma_{1.0t} \]  

where \( \sigma_{hs} \) is the hot spot stress, \( \sigma_{0.4t} \) is the surface stress at 0.4\( t \) away from the weld toe, \( \sigma_{1.0t} \) is the surface stress at 1.0\( t \) away from the weld toe, and \( t \) represents the thickness of steel plate.

### Table 6 The most unfavorable stress value of the monitoring points.

| Structure type     | Reinforcement spacing | Point-A (MPa) | Point-B (MPa) | Point-C (MPa) | Point-D (MPa) | Point-E (MPa) |
|--------------------|-----------------------|---------------|---------------|---------------|---------------|---------------|
| Conventional OSD   | /                     | 77.45         | 76.64         | 35.45         | 39.89         | 47.38         |
|                    | 80 mm                 | 38.77 (49.9%) | 37.88 (50.6%) | 13.21 (62.7%) | 15.19 (61.9%) | 27.36 (42.3%) |
|                    | 40 mm                 | 37.95 (51.0%) | 37.10 (51.6%) | 12.77 (64.0%) | 14.46 (63.8%) | 26.31 (44.5%) |
| LCBD               |                       |               |               |               |               |               |

Note: The values in brackets indicated the stress reduction amplitude of LCBD compared with the conventional OSD system.

**Fig. 24** Schematic drawing of hot spot stress.

Compared with conventional OSD system, the stress at monitoring points of the LCBD with spacing of 80 mm decreased of 42.3 - 62.7%, while 44.5 - 64.0% for LCBD with spacing of 40 mm. Thus, application of the UHPC as the steel deck pavement can significantly reduce the stress amplitude of the typical fatigue-prone details in OSD. Thus, enhance the fatigue life of the conventional OSD. However, when the reinforcement spacing decreases from 80 mm to 40 mm, there is slight effect on reducing the stress amplitude of the OSD, where the differences of stress amplitude are less than 5%, therefore, the mesh of rebars (40 mm or 80mm spacing) slightly influence the fatigue life of the conventional OSD. In addition, for the LCBD, the position of the maximum tensile stress is located at the rib-to-diaphragm welded joint (point-A and point-B).
6.2 Fatigue life evaluation of the OSD

For identifying the fatigue performance of the unfavorable fatigue-prone details, the calculations were conducted according to related Eurocodes \cite{31,36,37} and Siwowski et al. \cite{7}.

For the fatigue limit state, the safety level can be given by Eq. 7 to 10.

\[
\mu_{\text{fat}} = \frac{\Delta \sigma_C}{\gamma_{Mf} \gamma_{Ff} \Delta \sigma_{E,2}} \geq 1
\]  

\[
\gamma_{Ff} \Delta \sigma_{E,2} = \lambda \Delta \sigma(\gamma_{Ff}, Q_k)
\]  

\[
\Delta \sigma(\gamma_{Ff}, Q_k) = (\sum_{i=1}^{5} p_i \Delta \sigma_i)^{1/3}
\]  

\[
\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \leq \lambda_{\text{max}}
\]

where \( \mu_{\text{fat}} \) is the fatigue safety level; \( \Delta \sigma_C \) is the fatigue strength at \( N_C = 2 \times 10^6 \) cycles; \( \Delta \sigma_{E,2} \) is the equivalent constant amplitude stress range related to \( 2 \times 10^6 \) cycles \cite{36}; \( \gamma_{Mf} \) and \( \gamma_{Ff} \) is the partial safety factor for fatigue strength \( \Delta \sigma_C \) and for equivalent constant amplitude stress range \( \Delta \sigma_{E,2} \), respectively (\( \gamma_{Mf} = 1.15 \) \cite{36}, and \( \gamma_{Ff} = 1.0 \) \cite{37}); \( \Delta \sigma(\gamma_{Ff}, Q_k) \) is the equivalent stress amplitude under the Fatigue Load Model 3 considering the frequency distribution of transverse location of center line of vehicle \cite{31}, where \( p_i \) is the vehicle frequency and \( \Delta \sigma_i \) is the corresponding stress amplitude; \( \lambda_i \) is the damage equivalence factors \cite{37} (according to the general traffic record of the prototype bridge, \( \lambda_1 = 1.85 \) (minimum value), \( \lambda_2 = 1.20 \) (annual traffic volume), \( \lambda_3 = 1.0 \) (design life of 100 years), and \( \lambda_4 = 1.0 \) (effects of other lines); and \( \lambda_{\text{max}} \) is the maximum \( \lambda \)-value taking account of the fatigue limit \cite{37} (\( \lambda_{\text{max}} = 2 \)).

Suggested by IIW \cite{34}, the fatigue strength \( \Delta \sigma_C \) at \( N_C = 2 \times 10^6 \) cycles of weld toe is 90 MPa (FAT90). On the basis of the Eq. 7 to 10, the fatigue safety level \( \mu_{\text{fat}} \) of the monitoring points on the FEM-RS80 can be obtained, as listed in Table 7. In this way, all of the details of interest meet the requirements of BS EN1993-1-9 \cite{36}. However, because the fatigue safety level \( \mu_{\text{fat}} \) of point-A and point-B is up to 1, the fatigue cracking risk still exists in the rib-to-diaphragm welded joint, which was confirmed in the fatigue test. When the fatigue load reached 1.142 million cycles, a fatigue crack was firstly observed at the rib-to-diaphragm welded joint (point-B), as discussed in section 5.1.1.
From the finite element analysis of the test model (see Fig. 16), the stress distributions of the rib-to-diaphragm welded joint is shown in Fig. 25. Thus, under the load range of 50 - 200 kN in the first phase, the hot spot stress amplitude $\Delta\sigma_{p,B}$ of the point-B is 64.59 MPa (see Fig. 25b), while 51.39 MPa for the point-A (see Fig. 25c).

**Table 7** Fatigue safety calculations for the selected monitoring points.

| Monitoring points | $\Delta\sigma_{ffe, Q_k}$ | $\lambda$ | $\Delta\sigma_{E,2}$ | $\Delta\sigma_{C}$ | $\mu_{fat}$ | Fatigue safety |
|-------------------|---------------------------|----------|---------------------|------------------|------------|----------------|
| Point-A           | 38.11                     | 2.0      | 76.22               | 90               | 1.03       | Yes            |
| Point-B           | 37.24                     | 2.0      | 74.48               | 90               | 1.05       | Yes            |
| Point-C           | 13.01                     | 2.0      | 26.02               | 90               | 3.01       | Yes            |
| Point-D           | 14.68                     | 2.0      | 29.16               | 90               | 2.68       | Yes            |
| Point-E           | 26.73                     | 2.0      | 53.46               | 90               | 1.46       | Yes            |

**Fig. 25** Stress distributions and hot spot stresses of the rib-to-diaphragm welded joint: (a) stress distribution in the OSD under the load of 200 kN, (b) stress distributions in Path 1, and (c) stress distributions in Path 2.
As recommended by IIW [34] and AASHTO LRFD [38], for the fatigue life prediction of a steel structure, the fatigue resistance above the constant amplitude fatigue threshold (in terms of cycles) is inversely proportional to the cube of the stress range. Thus, the equivalent cumulative fatigue life can be calculated by Eq. 12.

\[
\Delta \sigma_E = \left( \frac{\sum n_i (\Delta \sigma_{p,i})^3}{\sum n_i} \right)^{1/3}
\]

\[
N = \sum n_i \times \left( \frac{\Delta \sigma_E}{\Delta \sigma_{E,2}} \right)^3
\]

where \( \Delta \sigma_E \) is the equivalent constant amplitude stress range; \( N \) is the equivalent cumulative fatigue life; \( \Delta \sigma_{p,i} \) is the stress amplitude in each loading phase; and \( n_i \) is the number of cycles associated with the stress amplitude \( \Delta \sigma_{p,i} \).

Based on Eq. 11 and 12, the equivalent fatigue life of point-B is 0.74 million cycles (=1.142x10^6x(64.59/74.48)^3), which is less than 2 million cycles. Therefore, application of the UHPC as the steel deck pavement, the rib-to-diaphragm welded joint is still prone to fatigue cracks.

### 6.3 Fatigue life evaluation of the UHPC layer

Table 8 lists the loading cycles and corresponding stress state of the UHPC layer in each loading phase. Similarly, the fatigue life prediction method prescribed in AASHTO LRFD [38] was also used to calculate the cumulative fatigue life of the UHPC layer by Eq. 8, 9, 11 and 12. The nominal stress amplitude \( \Delta \sigma_{p,i} \) of the test model in each loading phase was calculated by Eq. 1 to 5, and the results are shown in Table 8. Based on the FEA results of the FEM-RS40, the maximum stress amplitude of the UHPC layer (point-F) is 2.17 MPa. Through the Eq. 8 and 9, the equivalent constant amplitude stress range \( \Delta \sigma_{E,2} \) of the UHPC layer in FEM-RS40 is 4.31 MPa. In the same way, when the reinforcement spacing of the UHPC layer is 80 mm, \( \Delta \sigma_{E,2} \) is 4.52 MPa. The results of the equivalent fatigue cycles are presented in Table 8.

From Table 8, for the fatigue life of the Spacing-40MM, the equivalent fatigue cycles calculated by Eq. 12 are at least 1.743 times of that for Spacing-80MM. However, the equivalent fatigue life of Spacing-80MM reached 73.38 million cycles, which greatly exceeds the requirement in the specification [36,38]. Moreover, the fatigue life of the UHPC layer is 99

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times that of the rib-to-diaphragm welded joint. Therefore, the reinforcement spacing of the
UHPC layer can be expanded to 80 mm, so as to reduce the engineering cost and construction
complexity.

Table 8 The stress amplitude and equivalent fatigue cycles of the UHPC layer.

| Type   | Loading phase | Fatigue load amplitude $\Delta P$ (kN) | Stress amplitude $\Delta \sigma_{p,i}$ (MPa) | Number of cycles $n_i$ (10^6) | $\sum_{i=1}^{4} n_i$ (10^6) | $\Delta \sigma_{E}$ (MPa) | $\Delta \sigma_{E,2}$ (MPa) | Equivalent fatigue cycles $N$ (10^6) |
|--------|--------------|---------------------------------------|------------------------------------------|-------------------------------|---------------------------|----------------------|----------------------|-------------------------------|
| Spacing-40MM | $i=1$ | 150 | 6.61 | 2.70 |
|            | $i=2$ | 225 | 9.91 | 1.00 |
|            | $i=3$ | 300 | 13.21 | 1.00 |
|            | $i=4$ | 360 | 15.86 | 1.55 |
|           | $i=1$ | 150 | 6.72 | 2.70 |
| Spacing-80MM | $i=1$ | 360 | 16.13 | 0.597 |
|          | $i=2$ | 225 | 10.08 | 1.00 |
|           | $i=3$ | 300 | 13.44 | 1.00 |
|            | $i=4$ | 360 | 16.13 | 0.597 |

Note: The number of cycles $n_i$ after fatigue failure are not counted. Thus, in the last phase, the number of cycles for the Spacing-80MM is 0.597 million, and the number of cycles for the Spacing-40MM is 1.55 million.

7. Discussion of existing $S$-$N$ curve for UHPC plate fatigue life evaluation

7.1 Existing $S$-$N$ curve for UHPC

The nominal stress method is one of the most important methods to predict the fatigue
life of concrete structures [39-45], and the corresponding $S$-$N$ curve is indispensable. At present,
scholars have carried out a large number of flexural and tension fatigue tests on (ultra-high
performance) Fiber Reinforced Concrete (FRC) [14,39-45], and obtained corresponding $S$-$N$
curves, respectively, under their own experimental conditions, as depicted in Table 9. However, based on the summary of existing data, it is found that there are few research on the flexural fatigue properties of reinforced high performance FRC structures [14,40,41,43,45], especially for the research on the $S$-$N$ curves of LCBD. Besides, because of the great
discrepancies of definition of stress level and evaluation index of failure in fatigue test, it is
hard to directly apply existing $S$-$N$ curves for fatigue life evaluation.
Table 9 Flexural and tension fatigue test data of the UHPC or FRC.

| Type of concrete material | Stress level | Stress ratio | Fatigue cycles (10^6) | Fatigue type / Evaluation index |
|---------------------------|--------------|--------------|-----------------------|-------------------------------|
| Stress | | | | |
| $S_a$=0.49 | 0.31 | 3.1% | 2.3 | 2.734 |
| Li [14] | UHPC | $S_a$=0.49 | 0.31 | 6.2% | 2.3 | 3.74 |
| $S_a$=0.49 | 0.31 | 6.2% | 2.3 | 3.74 |
| Makita [39] | UHPC | $S_a$=0.46 | 0.1 | 3.4% | 2.44 | 10.00 |
| $S_a$=0.53 | 0.1 | 3.4% | 2.44 | 13.09 |
| LAPP [40] | UHPC | $S_a$=0.55 | 0.2 | 3.4% | 2.44 | 13.09 |
| $S_a$=0.65 | 0.2 | 3.4% | 2.44 | 0.26 |
| Sui [41] | UHPC | $S_a$=0.65 | 0.2 | 3.4% | 2.44 | 13.09 |
| $S_a$=0.65 | 0.2 | 3.4% | 2.44 | 0.26 |
| Makita [42] | UHPC | $S_a$=0.68 | 0.1 | 3.4% | 2.44 | 13.09 |
| $S_a$=0.68 | 0.1 | 3.4% | 2.44 | 0.26 |
| Parant [43] | UHPC | $S_a$=0.92 | \ | \ | \ | \ |
| $S_a$=0.82 | \ | \ | \ | \ |
| Wu [44] | FRC | $S_a$=0.4 | 0.375 | 1.62% | 0.64 | 2.0 |
| $S_a$=0.5 | 0.30 | 1.62% | 0.64 | 1.26 |

Note: $S_a$ is the ratio of maximum nominal stress to nominal cracking stress of concrete or the ratio of fatigue upper limit load to cracking load; $S_b$ is the ratio of maximum fatigue stress to ultimate static strength of concrete; $S_c$ is the ratio of maximum fatigue stress to elastic tensile strength of concrete; $S_d$ is the ratio of maximum fatigue stress to characteristic static stress of concrete.

7.2 Stress level of fatigue loading

Based on the calculation Eq. 1, the nominal stress under various loads can be obtained. Table 10 lists the nominal stress and the corresponding stress level $S$ of the UHPC layer at each cyclic loading phase.

Considering the definition of stress level and the evaluation index of fatigue life in existing fatigue tests are quite different, it is suggested that the evaluation index in fatigue test should be consistent to establish more practical $S$-$N$ curves for the fatigue life prediction of UHPC structures. For the lightweight UHPC-OSD composite bridge deck, durability of the deck structures is addressed more attention. Thus, the cracking situation in a bridge deck is

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one of the main areas of interest. The existing research results show that cracks less than 0.05 mm in a UHPC structure have no effect on the durability of the structure \([42]\). Therefore, it is appropriate to take the initial crack of 0.05 mm in the UHPC layer as the fatigue life evaluation index. Owing to the ultimate bending strength of the LCBD is mainly controlled by the yielded of steel component. For the UHPC materials, after cracking, the strain increases with the increase of load, but the stress remains basically unchanged over a period of time. Besides, the ultimate strain and visual cracking strain of UHPC are far more than the elastic strain (see Fig. 15). For these reasons, the ultimate strength and elastic strength of UHPC are not suitable for defining the stress level \(S\) of the LCBD. As shown in Fig. 14b, there is basically no degradation of bending stiffness of the composite bridge deck before UHPC layer cracks. In addition, the nominal stress can well reflect the influence of composite section parameter on the mechanical performance of UHPC layer. Thus, it may be a good way to define the stress level \(S\) as the ratio of the maximum nominal stress \((\sigma_{\text{max}})\) to the nominal cracking stress of the UHPC layer. This way was also adopted by Li \([14]\).

### Table 10 Nominal stress and stress level at each loading phase.

|                          | The first phase | The second phase | The third phase | The last phase |
|--------------------------|----------------|-----------------|----------------|---------------|
|                          | \(\sigma_{\text{min}}\) | \(\sigma_{\text{max}}\) | \(\sigma_{\text{min}}\) | \(\sigma_{\text{max}}\) | \(\sigma_{\text{min}}\) | \(\sigma_{\text{max}}\) |
| Spacing-80MM Stress level (S) | 2.24 | 8.96 | 3.36 | 13.44 | 4.48 | 17.92 | 5.37 | 21.50 |
| Spacing-80MM Nominal stress (MPa) | 8.96/24.59=0.364 | 13.44/24.59=0.547 | 17.92/24.59=0.729 | 21.50/24.59=0.874 |
| Spacing-40MM Stress level (S) | 2.20 | 8.81 | 3.30 | 13.21 | 4.40 | 17.62 | 5.29 | 21.14 |
| Spacing-40MM Nominal stress (MPa) | 8.81/35.69=0.247 | 13.21/35.69=0.370 | 17.62/35.69=0.494 | 21.14/35.69=0.592 |

Note: \(S\) is the ratio of maximum nominal stress to nominal cracking stress of UHPC layer. Under the load range of \(P_{\text{min}}/P_{\text{max}}\), \(\sigma_{\text{min}}\) is the minimum nominal stress of the UHPC layer, and \(\sigma_{\text{max}}\) is the maximum nominal stress of the UHPC layer.

### 7.3 Influencing factors of UHPC fatigue performance

According to the fatigue test of a reinforced UHPC-steel composite member and considering the influence of reinforcement ratio \(\rho_s\) and fibre characteristic parameter \(\lambda_f\), Eq. 13 for flexural fatigue \(S-N\) curve of reinforced UHPC was proposed by Li \([14]\) based on the \(S-\)
Due to limited fatigue data on the fatigue behaviour of UHPC layers in LCBDs and taking into account that the fatigue test results in Li [14] are in good agreement with the calculated results by Eq. 13, a qualitative analysis of the fatigue performance of the UHPC layer can be carried out. According to Eq. 13, the fatigue life of the UHPC layer under different stress levels in each fatigue loading phase can be calculated, and then the calculated value of fatigue life can be substituted into a linear cumulative damage method based on the Palmgren-Miner rule for fatigue life analysis, as shown in Eq. 15 [36,38]:

\[ D = \sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \cdots + \frac{n_i}{N_i} \]  

where \( D \) is the damage index, \( n_i \) is the number of cycles associated to the stress range of \( \Delta \sigma_{p,i} \), and \( N_i \) is the number of cycles calculated by \( S-N \) curve.

According to the Palmgren-Miner rule, when \( D \geq 1 \), the fatigue failure on the monitoring point occurs [36,38]. Based on the calculation results, \( D \) of the Spacing-80MM is 12.79, and \( D \) of the Spacing-40MM is 1.241. As a result, the difference between the test results of the Spacing-80MM and the theoretical value is significant. In addition, for the Spacing-40MM, the test result is also larger than the theoretical value of 1. It was found that the fatigue life of the UHPC layer in our test is larger than that of in the reference [14]. In order to understand the reason for this difference, the main differences of fatigue test parameters between this paper and Li [14] are listed in Table 11. The compressive strength and flexural strength of UHPC material used in this test are larger than those in Li [14]. In particular, the ratio of the flexural strength in present test is 1.846 times of that in reference [14]. The flexural strength is an important parameter of UHPC material, since increasing the flexural strength of the UHPC can also improve the fatigue life of the LCBD.

Table 11 Comparative results on the fatigue test parameters.

|                | \( f_{cu} \)/MPa | \( f_{cf} \)/MPa | \( E_c \)/GPa | UHPC plate thickness /mm | \( l_f \)/mm | \( \rho_f \) | \( \lambda_f \) | \( \rho_s \) |
|----------------|------------------|------------------|--------------|-------------------------|-------------|-----------|-------------|-----------|
| This study     | 182.45           | 44.48            | 44.4         | 50                      | 16          | 3.5%      | 2.55        | 1.8% and 3.7% |
| Li. [14]       | 132.9            | 24.1             | 43.3         | 50                      | 13 and 8   | 3.5%      | 2.3         | 3.1% and 6.2% |
| Ratio          | 1.373            | 1.846            | 1.025        | 1                       | \( \_ \)    | 1         | 1.11        | \( \_ \)    |
8. Conclusions

The purpose of this paper is to explore the influence of different reinforcement ratios on
the flexural static and fatigue performance of lightweight composite bridge decks. Based on
the four-point bending tests of two full-scale strip models under the static and fatigue loads
and nonlinear FE analysis, the following conclusions were obtained:

• The nominal cracking stress of the Spacing-80MM and the Spacing-40MM is 24.59
  MPa and 35.68 MPa, respectively, both of which are larger than the design stress (12.7 MPa)
in the prototype bridge. When the reinforcement spacing of UHPC decreases from 80 mm to
40 mm, the nominal cracking stress of UHPC under static load is increased by 45.1%.

• Increasing the reinforcement ratio of the UHPC layer can increase the bending stiffness
and cracking strain of the LCBD, and reduce the tensile strain of the UHPC layer, but these
variations are small. The actual cracking strain in test is larger than the elastic strain but is
smaller than the cracking strain in the tension constitutive model. It seems that the higher the
reinforcement ratio is, the larger the actual cracking strain is.

• Application of UHPC as a steel deck pavement, the stress amplitude of the typical
fatigue-prone details in OSD is significantly reduced, while the rib-to-diaphragm welded joint
is still prone to fatigue cracks. Besides, when the reinforcement spacing increases from 40
mm to 80 mm, the influence on stress amplitude of OSD can be ignored (on more than 5%).

• Increasing the reinforcement ratio of the UHPC layer can improve its fatigue resistance.
However, when the reinforcement spacing increases from 40 mm to 80 mm, the fatigue life
of the UHPC layer still satisfies durability requirements of related specification. Thus,
considering the mechanical performances and construction cost of the LCBD, the
reinforcement spacing of UHPC layer can be set as 80 mm.

• Compared with existing fatigue test data of LCBDs, increasing the flexural strength of
UHPC can improve the fatigue life of the UHPC layer. At present, research on the fatigue
cracking mechanism and fatigue S-N curve of the LCBDs is inadequate, and existing fatigue
test data is limited. Furthermore, the existing S-N curves cannot be directly used for fatigue
life prediction of the UHPC layer because of the great differences in the definition of stress
level and evaluation index of failure in fatigue test, which need to be investigated in further
studies.

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References

[1] A.H. Alavi, H. Hasni, P. Jiao, et al., Fatigue cracking detection in steel bridge girders through a self-powered sensing concept, Journal of Constructional Steel Research, 128 (2017) 19-38.

[2] I.M.E. Aghoury, K. Galal, Corrosion-fatigue strain-life model for steel bridge girders under various weathering conditions, Journal of Structural Engineering, 140 (6) (2014) 04014026.

[3] Federal Highway Administration (FHWA), Manual for design, construction, and maintenance of orthotropic steel deck bridges (No. FHWA-IF-12-027), Federal Highway Administration, Washington DC, USA, 2012.

[4] Z. Fu, Y. Wang, B. Ji, et al., Effects of multiaxial fatigue on typical details of orthotropic steel bridge deck, Thin-Walled Structures, 135 (2019) 137-146.

[5] S.T.D. Freitas, H. Kolstein, F. Bijlaard, Fatigue assessment of full-scale retrofitted orthotropic bridge decks, Journal of Bridge Engineering, 22 (11) (2017) 04017092.

[6] S. Ya, K. Yamada, T. Ishikawa, Fatigue evaluation of rib-to-deck welded joints of orthotropic steel bridge deck, Journal of Bridge Engineering, 16 (2011) 492-499.

[7] T. Siwowski, M. Kulpa, L. Janas, Remaining fatigue life prediction of welded details in an orthotropic steel bridge deck, Journal of Bridge Engineering, 24(12) (2019) 05019013.

[8] J. Maljaars, F.V. Dooren, H. Kolstein, Fatigue assessment for deck plates in orthotropic bridge decks, Steel Construction, 05(02) (2012) 93-100.

[9] P. Buitelaar, R. Braam, N. Kaptijn, Reinforced high performance concrete overlay system for rehabilitation and strengthening of orthotropic steel bridge decks. Proceedings of 1st Orthotropic Bridge Conference, California, USA-August (2004) 25-27.

[10] X. Shao, D. Yi, Z. Huang, et al., Basic performance of the composite deck system composed of orthotropic steel deck and ultrathin RPC layer, Journal of Bridge Engineering, 18 (5) (2013) 417-428.

[11] J. Cao, X. Shao, Z. Zhang, et al., Retrofit of an orthotropic steel deck with compact reinforced reactive powder concrete, Structure and Infrastructure Engineering, 12 (03) (2015) 411-429.

[12] X. Shao, J. Hu, The steel-UHPC lightweight composite bridge structures. Beijing: China Communications Press, 2015. (in Chinese)

[13] S. Zhang, X. Shao, J. Cao, et al., Fatigue performance of a lightweight composite bridge deck with open ribs, Journal of Bridge Engineering, 21 (7) (2016) 04016039.

[14] W. Li, Experimental research on static and fatigue flexural performance of UHPC layer in light-weighted composite bridge deck, (Ph.D. Dissertation, Hunan University, Changsha), 2015. (in Chinese)

[15] Y. Yuan, C. Wu, X. Jiang, Experimental study on the fatigue behavior of the orthotropic steel deck...
rehabilitated by UHPC overlay, Journal of Constructional Steel Research, 157 (2019) 1-9.

[16] S. Chen, Y. Huang, P. Gu, et al., Experimental study on fatigue performance of UHPC-orthotropic steel composite deck, Thin-Walled Structures, 142 (2019) 1-18.

[17] Z. Zhu, T. Yuan, Z. Xiang, et al., Behavior and fatigue performance of details in an orthotropic steel bridge with UHPC-deck plate composite system under in-service traffic flows, Journal of Bridge Engineering, 23 (3) (2018) 04017142.

[18] L. Dieng, P. Marchand, F. Gomes, et al., Use of UHPFRC overlay to reduce stresses in orthotropic steel decks, Journal of Constructional Steel Research, 89 (2013) 30–41.

[19] J.S. Kim, J. Kwark, C. Joh, et al., Headed stud shear connector for thin ultrahigh-performance concrete bridge deck, Journal of Constructional Steel Research, 108 (2015) 23-30.

[20] Q. Zhang, Y. Liu, Y. Bao, et al., Fatigue performance of orthotropic steel-concrete composite deck with large-size longitudinal U-shaped ribs, Engineering Structures, 150 (2017) 864-874.

[21] AFGC-Setra (French Association of Civil Engineering-French Authorities of Civil Engineering Structure Design, and Control), Ultra high performance fibre reinforced concretes, Paris: AFGC and SETRA Working Group, 2013.

[22] D.Y. Yoo, S. Kim, J.J. Kim, et al., An experimental study on pullout and tensile behavior of ultrahigh-performance concrete reinforced with various steel fibers, Construction and Building Materials, 206 (2019) 46-61.

[23] M. Liu, X. Shao, Z. Zhang, et al., Experiment on flexural fatigue performance of composite deck system composed of orthotropic steel deck and ultra-thin RPC layer, Journal of Highway Transportation Research and Development, 29 (10) (2012) 46–53. (in Chinese).

[24] N. Ding, X. Shao, Study on fatigue performance of light-weighted composite bridge deck, Journal of Civil Engineering, 48 (01) (2015) 74-81. (in Chinese)

[25] GDJTG/T A01-2015. Technical Specification for ultra-high performance light-weighted composite deck structure. Guangdong, local standards for transportation industry of Guangdong Province, 2015. (in Chinese)

[26] W. Lin, T. Yoda, N. Taniguchi. Fatigue tests on straight steel-concrete composite beams subjected to hogging moment, Journal of Constructional Steel Research, 2013, 80: 42-56.

[27] W. Lin, T. Yoda, N. Taniguchi, et al., Mechanical performance of steel-concrete composite beams subjected to a hogging moment, Journal of Structural Engineering, 2014, 140 (1): 04013031.

[28] GB/T 714-2015. Structural steel for bridge. Beijing, General Administration of Quality Supervision, Inspection and Quarantine of the People’s Republic of China and Standardization Administration of the People’s Republic of China (AQSIQ-SAC); 2015. (in Chinese).

[29] C. Li, Z. Feng, R. Pan, et al., Experimental and numerical investigation on the anchorage zone of prestressed UHPC box-girder bridge, Journal of Bridge Engineering, 25 (06) (2020) 04020028.

[30] ABAQUUS version 6.12: ABAQUUS user’s manual, SIMULIA World Headquarters, Providence, RI 02909-2499, USA; 2012.

[31] European Committee for Standardization. BS EN1991-2: 2003 Eurocode 1: Actions on Structures - Part 2: Traffic Loads on Bridges. Brussels (Belgium); 2003.

[32] Q. Tian, L. Gao, S. Zhou, Study of mechanical behaviour of composite bridge deck with ultra high
performance concrete and orthotropic steel plate, Bridge Construction, 47 (03) (2017) 13-18. (in Chinese)

[33] Y. Liu, Q. Zhang, W. Meng. Transverse fatigue behaviour of steel-UHPC composite deck with large-size U-ribs, Engineering Structures, 180 (2019) 388-399.

[34] A. Hobbacher, Recommendations for fatigue design of welded joints and components, International Institute of Welding, IIW-2259-15/ex XIII-2460-13/XV-1440-13, Paris, France; 2016.

[35] H. Abdelbaseta, B. Chenga, L. Tian, Reduce hot spot stresses in welded connections of orthotropic steel bridge decks by using UHPC layer: Experimental and numerical investigation, Engineering Structures, 220 (2020) 110988.

[36] European Committee for Standardization. BS EN1993-1-9: 2005, Eurocode 3: Design of Steel Structures-Parts 1-9: Fatigue. Brussels (Belgium); 2005.

[37] European Committee for Standardization. BS EN1993-2: 2006, Eurocode 3: Design of Steel Structures-Parts 2: Steel Bridges. Brussels (Belgium); 2006.

[38] AASHTO, AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials (AASHTO), Washington DC, 2017.

[39] T. Makita, E. Brühwiler, Tensile fatigue behaviour of ultra-high performance fibre reinforced concrete combined with steel rebars (R-UHPFRC), International Journal of Fatigue, (59) (2014) 145-152.

[40] E.S. Lappa, High strength fibre reinforced concrete: Static and fatigue behaviour in bending (Ph.D. Dissertation, Delft University of Technology, The Netherlands), 2007.

[41] L. Sui, Q. Zhong, K. Yu, et al., Flexural fatigue properties of ultra-high performance engineered cementitious composites (UHP-ECC) reinforced by polymer fibers, Polymers, (10) (2018) 892.

[42] T. Makita, E. Brühwiler, Tensile fatigue behaviour of ultra-high performance fibre reinforced concrete (UHPFRC), Materials and Structures, (47) (2014) 475-491.

[43] E. Parant, P Rossi, C. Boulay, Fatigue behavior of a multi-scale cement composite, Cement and Concrete Research, (37) (2007) 264-269.

[44] J. Wu, Experimental studies and numerical analysis of fatigue properties of steel fiber reinforced concrete beam, (Master's Dissertation, Zhengzhou University, Zhengzhou), 2013. (in Chinese)

[45] D.M. Carlesso, A.D.L Fuente, S.H.P. Cavalaro, Fatigue of cracked high performance fiber reinforced concrete subjected to bending, Construction and Building Materials, (220) (2019) 444-455.