RESEARCH AND APPLICATION OF THE COMPOSITE DECK SYSTEM COMPOSED OF ORTHOTROPIC STEEL DECK AND THIN UHPFRC LAYER

Cheng Liu (1), Wenhao Pan (1), Jiansheng Fan (1), Jianguo Nie (1), Jianhua Hu (2), and Jianfeng Cui (2)

(1) Department of Civil Engineering, Tsinghua University, Beijing, China

(2) Hunan Provincial Communications Planning, Survey & Design Institute, ChangSha, China

Abstract

The widely used orthotropic steel deck system is vulnerable to two diseases, the fatigue cracking of steelworks and the deterioration of pavement. To overcome these two diseases, a novel composite deck system composed of an orthotropic steel deck (OSD) covered by a thin ultra-high performance fibre reinforced concrete (UHPFRC) layer was recently adopted in a super long-span suspension bridge in China. Firstly, the design essentials and details of the proposed OSD-UHPFRC composite deck was illustrated. Then a comparative study by FEM was conducted to demonstrate the stresses reduction effect of the additional UHPFRC layer under vehicle load. After that, the reliability of the OSD-UHPFRC composite deck were explored experimentally: the tensile behavior of the wet joints between precast UHPFRC layer and the cast-in-situ UHPFRC layer was investigated by tensile tests; the fatigue performance of the headed stud encased in UHPFRC layer was verified by fatigue tests. Finally, the key design and research points of the OSD-UHPFRC deck system is concluded.

Résumé

Les ponts métalliques à dalle orthotrope, très répandus, sont soumis à deux pathologies, la fissuration par fatigue et la détérioration de la couche de roulement. Pour y remédier, un nouveau système mixte constitué de la dalle orthotrope associée à un revêtement mince en béton fibré à ultra-hautes performances (BFUP) a été adopté pour un pont suspendu de très grande portée en Chine. Le principe et les détails du système mixte sont présentés. Puis une étude aux éléments finis a été menée pour démontrer comparativement la réduction des contraintes sous charge de trafic permise par le revêtement BFUP. La fiabilité du tablier mixte a été testée expérimentalement : des essais de traction ont permis d'explorer le comportement des joints entre éléments BFUP préfabriqués et couches de BFUP coulées en place, la tenue en fatigue des goujons dans la couche de BFUP a été testée. On conclut sur les points clés de ce concept et des recherches associées.

1. INTRODUCTION

The orthogonal steel deck (OSD) system is widely used in modern long-span bridges, especially in cable-stayed and suspension bridges, for its light weight, high strength, good stiffness and construction convenience [1-3]. However, the conventional orthogonal steel deck is subject to two major durability problems: the fatigue cracking of steelworks [4-6] and the wearing of surfacing [6, 7]. One major reason for the durability problems of conventional OSD is the excessive large local deformations and stresses caused by the wheel load due to insufficient stiffness of the steel deck plate [6].

Ultra-high performance fibre reinforced concrete (UHPFRC) is an advanced cementitious composite with remarkable compressive and flexural strength, excellent ductility and good workability that offers new opportunities for current and future infrastructure development. By covering the orthogonal steel deck with a thin UHPFRC layer and connecting the UHPFRC layer and steel deck with shear connectors, a novel composite deck system, namely the OSD-UHPFRC deck system, is formed [8, 9]. This OSD-UHPFRC deck system is expected to solve the durability problems of the conventional OSD system for several reasons: 1) the composite action between the steelworks and UHPFRC layer greatly improves the stiffness and load-transferring ability of the deck system and thus reduces the stress amplitude under service vehicle load; 2) the bond strength between the UHPFRC layer and asphalt surfacing is strong enough to avoid delamination failure; 3) the ultra-high tensile strength of UHPFRC prevents the cracking of concrete and corresponding corrosion problem; 4) the light-weight advantage of conventional OSD is still maintained.

This paper aims to comprehensively demonstrate some design and research issues of the OSD-UHPFRC deck system adopted in a super-long span suspension bridge in China. Firstly, the design essentials and details of the OSD-UHPFRC desk system is illustrated; secondly the tensile behavior of the wet joint between precast UHPFRC layer and cast-in-situ UHPFRC layer is investigated by tensile tests; thirdly, the fatigue performance of the headed-stud encased in UHPFRC layer is verified by fatigue tests; finally, the key design and research points are concluded.

2. DESIGN OF COMPOSITE DECK SYSTEM

A super long-span suspension bridge with a main span of 1480-m named the Dongting Lake Second Bridge will be built in Hunan Province of China. A deck-truss composite stiffening girder is used for this suspension bridge (Figure 1), where the deck works together with the stiffening truss to provide sufficient vertical stiffness. To overcome the durability problems of the conventional OSD, a novel OSD-UHPFRC deck system is utilized in this bridge, as depicted in Figure 2. The steelworks in the OSD-UHPFRC deck system are similar to that in the conventional OSD system, except that open ribs are adopted for material saving and easier welding procedure. A 50-mm UHPFRC layer is laid upon the steel deck plate, where headed studs spacing 250mm×250mm are used to provide reliable bond connection. Beyond the UHPFRC layer, a 35-mm conventional asphalt pavement is utilized instead of the costly epoxy asphalt pavement.

The mix proportion of the UHPFRC material is shown in Table 1 and the material property is shown in Table 4. A mixture of two types of steel fibres is applied in the UHPFRC material. One had a diameter of 0.12 mm and a length of 8 mm, and the other had a diameter of 0.2 mm and a length of 13 mm. Their volume ratios are 1.5% and 2%, respectively. The tensile strength of both types of steel fibers is 2,700 MPa. To improve the crack resistance capacity, the UHPFRC layer is reinforced with a single-layer rebar mesh, which has a diameter of 10 mm and a spacing of 35 mm. The early-age treatment of UHPFRC layer is a two-stage process : 1) After initial setting, a humidity maintenance is adopted until the final setting of UHPFRC layer (for about 1~2 days); 2) then the UHPFRC layer is under a 90°C hot steam curing for 4 days. Previous study [8] showed that this early-age treatment can effectively decrease the autogenous tensile stress due to early-age shrinkage.

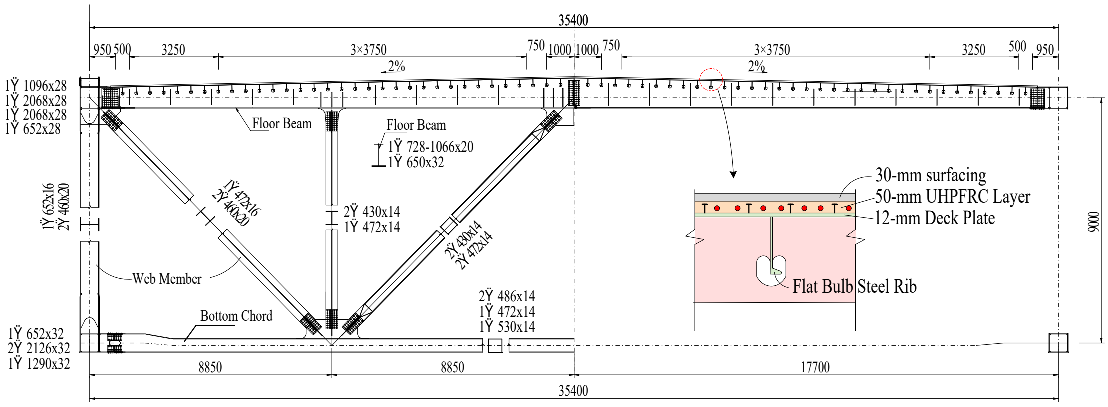


Figure 1 : Cross-section of the stiffening girder of the Dongting Lake Second Bridge

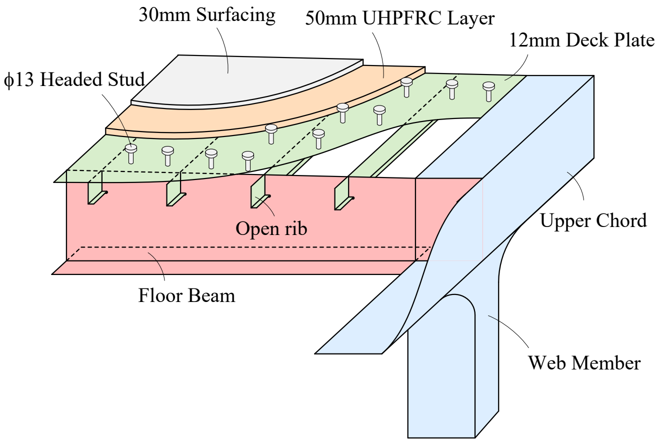


Figure 2 : Details of the OSD-UHPFRC deck system in the Dongting Lake Second Bridge

Table 1: Mix proportion of the UHPFRC material

Cement	Quartz sand	Silica fume	Fly ash	Quartz powder	Water	Water reducer	Steel fibre L=8 mm	Steel fibre L=13 mm
1	1.1	0.2	0.1	0.2	0.19	1.5%	1.5%	2%

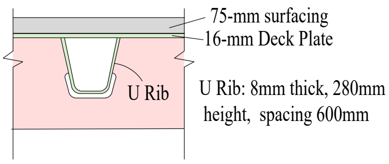
Note: Proportions of the water reducer and steel fibres are volume ratio.

3. STRESS REDUCTION EFFECT OF THE UHPFRC LAYER

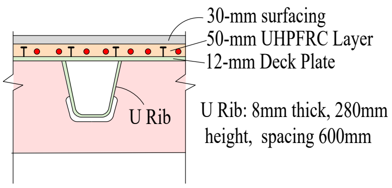
A comparative investigatiogn by finite element method (FEM) is carried out to quantify the stress reduction effect of the additional UHPFRC layer on the Dongting Lake Second Bridge. Segmental models with different deck details are established as shown in Figure 3.

Detail I represents the conventional OSD ; detail II represents the OSD-UHPFRC deck system with U-shaped rib ; detail III represents the OSD-UHPFRC deck system with flat bulb steel rib, which is the same as in Dongting Lake Second Bridge. In the FEM, the truss members are modelled with 1D truss element and the OSD are modelled with shell and solid elements (Figure 3d). The elastic modulus of steel and UHPFRC are taken as 210GPa and 42GPa, respectively. Self-weight and vehicle load defined in Chinese bridge specification JTG D60-2004[10] (Figure 3e) are considered.

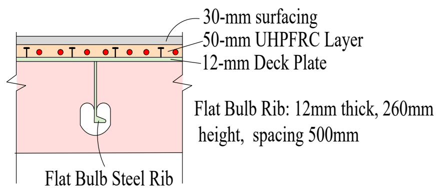
The local deflection of deck, stress of floor beam and longitudinal rib are calculated for different details as shown in Table 2. With the UHPFRC layer (detail II and III), the Von Mises stresses in the floor beam and longitudinal rib are cut down to 50%~70% of the conventional OSD (detail I), while the stress reduction effect of the closed rib (detail II) and open rib (detail III) are close. Since the open rib has simpler weld procedure and costs less material, the OSD-UHPFRC deck in the Dongting Lake Second Bridge takes the flat bulb steel rib as the longitudinal stiffener.



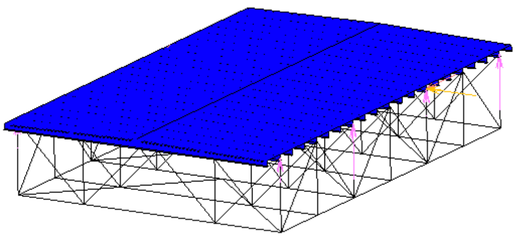
(a) Detail I



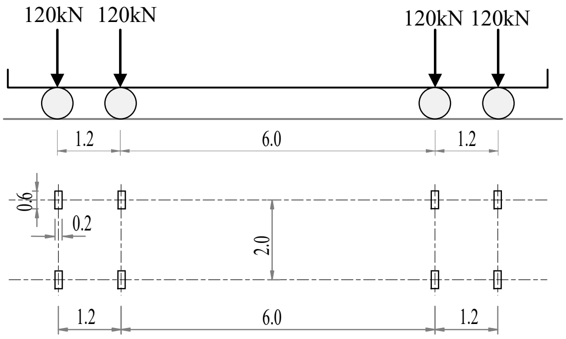
(b) Detail II



(c) Detail III



(d) Overview of the FEM model



(e) Vehicle load in JTG D60-2004[10]

Fig. 3: FEM models of the OSD-UHPFRC deck system in the Dongting Lake Second Bridge

Table 2: FEM results for different deck details

	Local deflection (mm)	Von Mises Stress (MPa)	
		Floor beam	Longitudinal rib
Detail I	23.8 (1.00)	225.1 (1.00)	263.7 (1.00)
Detail II	21.8 (0.92)	158.4 (0.70)	154.4 (0.59)
Detail III	22.6 (0.95)	115.1 (0.51)	172.4 (0.65)

Note: The number in the parenthesis represents the ratio with respect to detail I.

4. TENSILE BEHAVIOR OF THE WET JOINT

To ensure the construction quality, the stiffening girder segment including the UHPFRC layers is prefabricated for the Dongting Lake Second Bridge. During the field assembly, the steel members in two adjacent prefabricated segments are bolted together first, after which the precast UHPFRC layers are connected by a cast-in-situ UHPFRC layer. Thus, a joint is formed between the precast UHPFRC layer and the cast-in-situ UHPFRC layer, and are defined as the wet joint. Since few steel fibres lie in the interfaces between the precast and the cast-in-situ UHPFRC layer, a typical UHPFRC wet joint with a vertical, plane interface (Figure 4a) is weak in crack resistance and can have potential durability problems. Therefore, wet joint details with better mechanical properties and crack resistance performance need to be developed.

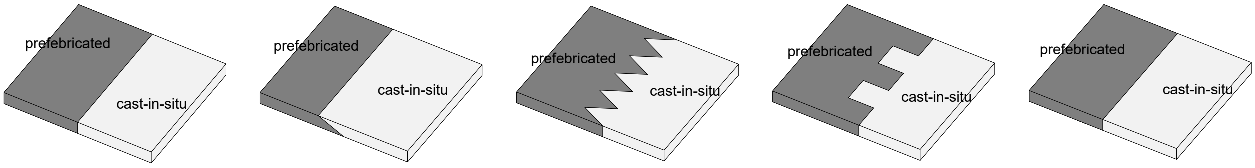
4.1 Experimental programs

Six different UHPFRC wet joint models are designed as shown in Table 3 and Figure 4. Tensile tests are carried out to compare their tensile performances.

Table 3 : Parameters of the different test models

Wet joint	Details	Reinforcing rebar ratio in wet joint (%)	Reinforcing steel plate ratio in wet joint (%)	Panel Label
integral casting detail	integral casting	3.84	0	T-1
conventional wet joint	vertical, plane interface	6.86	0	T-2
reinforcement-enhanced wet joint	conventional wet joint enhanced by larger full-length reinforcements	10.54	0	T-3
inclined wet joint	inclined interface	6.86	0	T-4
sawtooth wet joint	sawtooth interface	6.86	0	T-5
rectangular wet joint	rectangular interface	6.86	0	T-6
steel plate-enhanced wet joint	conventional wet joint enhanced by a 14-mm thick special-shaped steel plate	3.84	8.96	T-7

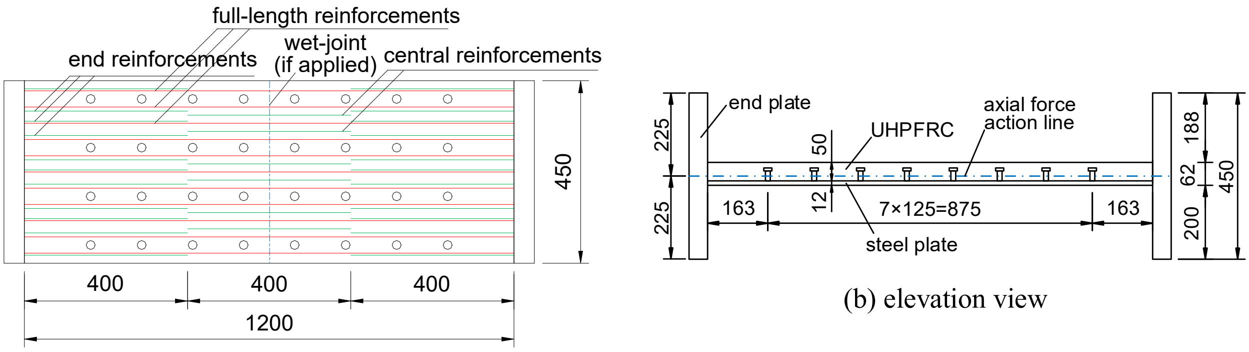
Details of the tested panels are shown in Figure 5. The tested panels are 1200×450×62 mm in size, with a 12-mm thick steel plate and a 50-mm thick UHPFRC layer. At two ends of the panels are end plates for the test setup. The steel plate and the reinforcements in the UHPFRC layer are all welded to the end plates. As shown in Figure 5a, three types of longitudinal reinforcements are placed in the UHPFRC layer, i.e. full-length reinforcements, central reinforcements and end reinforcements. The central reinforcements placed in the central wet joint region are designed to improve the tensile behaviour of wet joints, while the end reinforcements are designed to transfer loads from the end plates to the UHPFRC layer. Headed-studs spacing 125×120 mm are used to ensure the composite action between the steel plate and the UHPFRC layer.



(a) T-2/T-3 (b) T-4 (c) T-5 (d) T-6 (e) T-7

Figure 4 : Different wet joint details

The test setup is shown in Figure 6. During the tests, the maximum crack width in the UHPFRC surface is measured; the nominal UHPFRC tensile stress-maximum crack width curves is compared to study the crack resistance performance.



(a) plan view (b) elevation view

Figure 5 : Details of the tested panels (Units : mm)

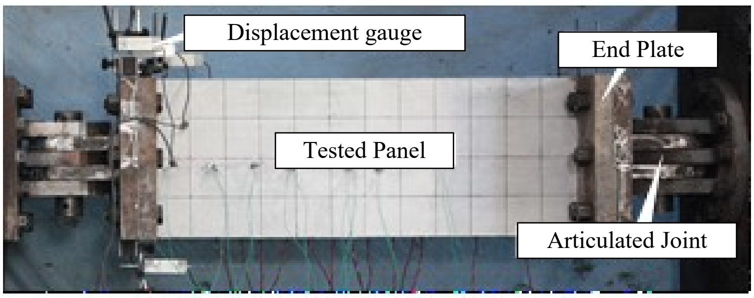


Figure 6 : Setup for the tensile test

4.2 Test results

The material test is conducted first and summarized in Table 4.

The test phenomena of the tensile tests are as follows: As the load increased gradually, initial tensile cracks formed in the transverse direction of the UHPFRC surface. The cracking loads, cracking stresses and cracking strains of these panels are shown in Table 5. After initial cracking, new cracks started to appear and develop continuously. The crack development is accompanied by a small sound, which is probably caused by the micro-cracking of the UHPFRC matrix and the debonding between steel fibres and UHPFRC at local regions. In all test panels, failure occurred shortly after the sudden rupture of the reinforcement-end plate welding. Due to the dispersion of the welding process, the failure loads ranged from 1109 to 2405 kN.

The nominal tensile stress in the UHPFRC surface is calculated under the plane section assumption. The nominal tensile stress versus maximum crack width curves for different panels, which represents the crack resistance capacity, are illustrated in Figure 7. The crack resistance capacities of wet joints with different details differ greatly. The sawtooth wet joint (T-5), rectangular wet joint (T-6) and steel plate-enhanced wet joint (T-7) have substantially better crack resistance performances than the conventional wet joint, and their crack resistance performance even match the panel casted integrally (T-1). The reason is that the sawtooth wet joint and the rectangular wet joint prevented the direct application of the tensile load on the wet joint interfaces, while the special-shaped steel plate enhances its wet joint greatly. In contrast, the conventional wet joint (T-2), inclined wet joint (T-4) and reinforcement-enhanced wet joint (T-3) have smaller cracking stresses because of their plane interfaces. Comparing the conventional wet joint (T-2) and the reinforcement-enhanced wet joint (T-3), the larger full-length reinforcements do help in limiting the crack development.

Table 4 : Material properties

	Thickness (mm)	f_y (MPa)	f_u (MPa)		Diameter (mm)	f_y (MPa)	f_u (MPa)
steel plate	12	392.2	552				
	22	346.4	551.3		Φ10	509.3	611.2
	f_{cu} (MPa)	f_r (MPa)	f_{cr} (MPa)	rebar	Φ12	530.5	654.3
Precast UHPFRC	155.2	29.5	13.6		Φ14	441.7	597.6
cast-in-situ UHPFRC	155.7	28.6	12.9		Φ16	467.5	581.9

Note: f_y = yield stress of steel and rebar; f_u = ultimate stress of steel and rebar; f_{cu} = UHPFRC cubic compressive strength determined by 100×100×100 mm specimens; f_r and f_{cr} = UHPFRC flexural strength and cracking stress determined by four-point bending test of 100×100×400 mm specimens, respectively.

Table 5 : Tensile test results

Panel	P_{cr} (kN)	σ_{cr} (MPa)	ε_{cr} (με)	$P_{failure}$ (kN)
T-1	450	11.8	370	1109
T-2	100	2.6	94	1714
T-3	300	7.9	272	1446
T-4	250	6.6	223	1804
T-5	375	9.8	306	1849
T-6	400	10.5	315	2262
T-7	400	10.5	310	2405

Note: P_{cr} = cracking load; σ_{cr} = cracking stress, i.e. the nominal tensile stress at cracking load; ε_{cr} = cracking strain measured by a strain gauge; $P_{failure}$ = failure load.

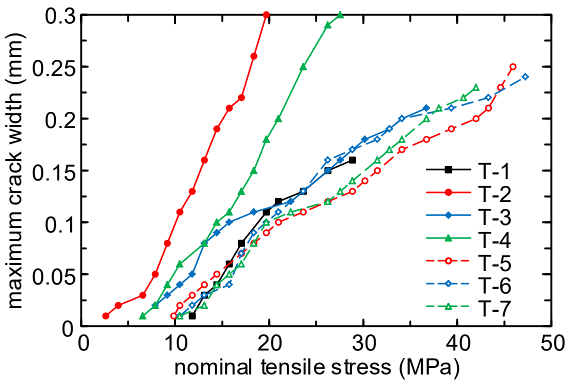


Figure 7 : Nominal tensile stress-maximum crack width curves of tested panels

5. FATIGUE PERFORMANCE OF THE HEADED STUDS

The headed studs connecting the steel deck plate and the UHPFRC layer are crucial to the fatigue lifetime of the OSD-UHPFRC deck system. Since the UHPFRC layer is usually very thin (50 mm thick for the Dongting Lake Second Bridge), the headed stud in the OSD-UHPFRC deck system has to be short (a height of only 35mm for the Dongting Lake Second Bridge as shown in Figure 8) and thus can behave differently from the normal headed stud used in the conventional composite girder. Besides, the stud encased in the UHPFRC layer subjects to sophisticated two-way shear forces as the UHPFRC deck is essentially a two-way slab under vehicle loads. Therefore, fatigue performance of the headed stud in the OSD-UHPFRC deck system needs to be investigated.

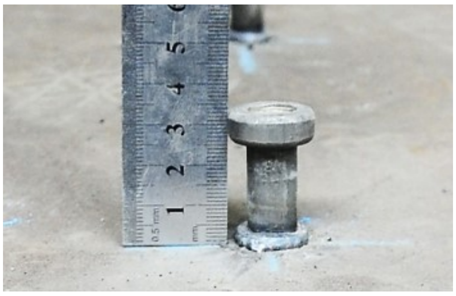


Figure 8 : Short headed stud (Units : mm)

5.1 Experimental programs

Fatigue performance of the headed stud is tested by a beam-bending test. Two specimens namely the positive-moment and negative-moment specimen are designed respectively, taken from the deck strips in the positive and negative moment region in the OSD-UHPFRC deck prototype, as shown in Figure 9.

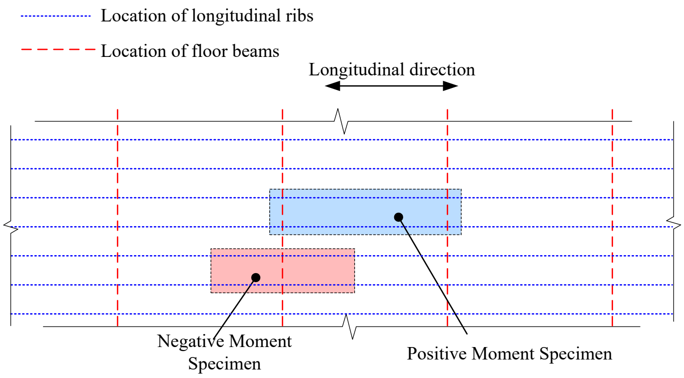
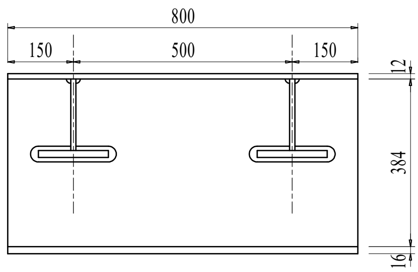
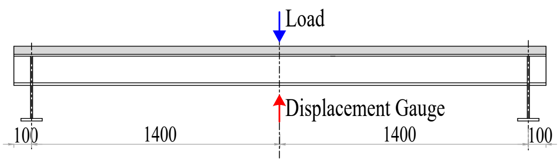


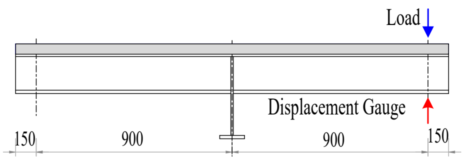
Figure 9 : Positive-moment and negative-moment region in the OSD-UHPFRC prototype



(a) Cross section of the specimens



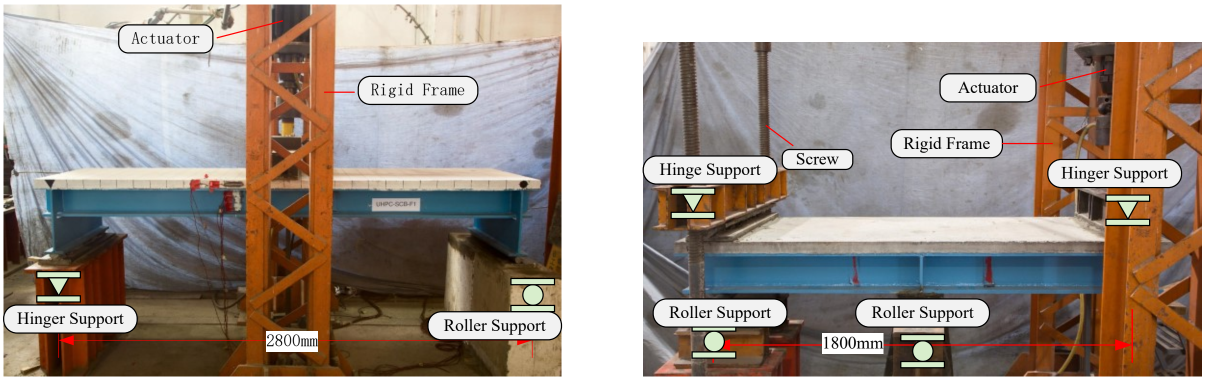
(b) Positive-moment specimen



(c) Negative-moment specimen

Figure 10 : Dimensions of the specimens (Units : mm)

The material, reinforcement, and welding procedure of the specimens keep the same as that of the prototype structure. Instead of the flat bulb rib in the prototype structure (see Figure 3c), T-shaped ribs with equivalent cross-sectional properties are used in the tested specimens. Four rows of headed studs with diameters of 12 mm and spacing 250 mm are welded to the top plate of the steel girder. The dimensions of the two specimens are illustrated in Figure 10. The material properties are the same as in the tensile test as in Table 4. The test setup is illustrated in Figure 11. Progressively-increasing cyclic load is applied on the specimen by an actuator. If the specimen doesn't failure in a given load level by 2×10^6 cycles, the load will be raised to the next level. To monitor the fatigue damage, static loading and unloading are performed by 1×10^4, 1×10^5, 5×10^5, 1×10^6, and 2×10^6 cycles, at a specific load level. During the static and fatigue tests, strains and displacement are measured by strain gauges and displacement gauges.



(a) Positive-moment specimen (b) Negative-moment specimen

Figure 11 : Dimensions of the specimens

5.2 Test results

The nominal shear stress range $\Delta\tau_c$ of the headed stud is used as the indicator of the load level. The positive-moment specimen are firstly loaded at a load range of 80 kN ($\Delta\tau_c$ =59.8 MPa) and no failure phenomenon is observed by 2×10^6 cycles. Then the load level is raised to 160 kN and again no failure phenomenon is observed by 2×10^6 cycles ($\Delta\tau_c$ =119.5 MPa). After that, the actuator applied at the mid-span is moved to the 1/4-span to further increase the fatigue shear stress applied on the headed-stud ($\Delta\tau_c$=201.8 MPa). The test halts by about 1.6×10^4 cycles because of the failure of the roller support. The negative-moment specimen are firstly loaded at a load range of 50 kN ($\Delta\tau_c$ =72.0 MPa) and no failure phenomenon is observed by 2×10^6 cycles. Then the load level is raised to 150 kN ($\Delta\tau_c$ =216.0 MPa). The test halts by about 5.0×10^5 cycles because of the failure of one skew bar.

Although the specimens don't failure in the tests, the largest load level that the specimen undergo 2×10^6 cycles still provide useful information and can be taken as the lower limit of the fatigue strength. The lower limit of the fatigue strength of the stud is shown in Table 6.

Table 6 : Lower limit of the strength of the headed stud

	Positive-moment specimen	Negative-moment specimen
$\Delta\tau_c$ (MPa)	119.5	72.0

6. CONCLUSIONS

The proposed OSD-UHPFRC deck system is a promising structure to overcome the durability problems of conventional OSD structure. Some design and research issues of the OSD-UHPFRC deck system adopted in a super-long span suspension bridge in China are explored in this paper. And the conclusions are as follows:

- The additional UHPFRC layer, even thin as 50mm, can effectively reduce the stress under vehicle load and thus improve the fatigue lifetime of the deck system;
- The sawtooth wet joint, rectangular wet joint and steel plate-enhanced wet joint have reliable crack resistance performance and are recommended in for deck design;
- The fatigue resistance of the short headed stud in the OSD-UHPFRC deck system can at least reach 119.5 MPa for positive-moment region and 72.0 MPa for negative-moment region.

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