USING UHPFRC AS A TOPPING LAYER FOR ORTHOTROPIC BRIDGE DECKS: PROTOTYPE VALIDATION

François Toutlemonde (1), Pierre Marchand (1), Fernanda Gomes (1, 2) and Lamine Dieng (3)

(1) UPE-Ifsttar, Materials and Structures Dept, Cité Descartes, France

(2) Solétanche Freyssinet, Vinci-Construction, Rueil-Malmaison, France

(3) UNAM-Ifsttar, Materials and Structures Dept, Bouguenais, France

Abstract

Within the framework of the French national R&D project Orthoplus, an experimental and numerical study has been carried out on UHPFRC used as a topping layer for orthotropic bridge decks. Two large-scale specimens have been tested to assess the role of a thin UHPFRC layer to decrease the fatigue stress at the welded junction between troughs and deck plate under the effect of a local loading by a truck wheel. This solution also allows a dead weight reduction thanks to the very small thickness of UHPFRC compared to the traditional solution with bituminous concrete. Quasi-static loadings under different configurations, as well as fatigue tests under a central loading (up to 2 million cycles aiming to represent 100-years traffic loading) were carried out. Even though the mechanical connection between the steel plate and the UHPFRC layer turned out only partial, the efficiency of UHPFRC in reducing stress concentrations and increasing the structural stiffness of the deck over the lifespan was confirmed.

Résumé

Dans le cadre du projet Orthoplus soutenu par l'Agence Nationale de la Recherche, le BFUP a été étudié expérimentalement et numériquement pour constituer le revêtement de tabliers de ponts à dalle orthotrope. Deux modèles de grande taille ont été testés pour évaluer la contribution d'un revêtement mince de BFUP à la diminution des contraintes de fatigue à la jonction soudée entre augets et platelage, sous l'effet de la charge locale due à une roue de poids lourd. Cette configuration réduit également le poids propre étant donné la faible épaisseur du BFUP par rapport à la solution traditionnelle de revêtement en béton bitumineux. Des essais quasi-statiques selon différentes configurations ont été réalisés, ainsi que des chargements de fatigue sous effort localisé au centre (jusqu'à 2 millions de cycles destinés à représenter l'effet de 100 ans de trafic). Même si la connexion entre le platelage d'acier et le BFUP s'est avérée seulement partielle, l'efficacité du BFUP pour réduire durablement les concentrations de contrainte et augmenter la rigidité structurale a été confirmée.

1. INTRODUCTION

Orthotropic steel bridge decks represent a critical issue for highway operators. Due to their lightness they have frequently been chosen, in France like in several Northern European countries, for constituting the deck of large bridge crossings over river mouths (Normandy bridge over the Seine river, Cheviré bridge at Nantes over the Loire river, Bénodet bridge in Brittany…) and also for movable bridges (as a recent example, Gustave Flaubert Bridge in Rouen). In the past, inappropriate assembly details have resulted in important fatigue damage for such bridge decks. While recent research efforts in the Netherlands concentrated on the trough to deck welded joint over crossbeams [1], the feedback of French experience identified the trough to deck welded joint at midspan between crossbeams as most critical also, which motivated a joint R&D initiative called Orthoplus [2].

Orthotropic steel bridge decks' retrofitting has been achieved by additional plates fixing [3], or by combining a concrete overlay into a composite system [4]. Quantitative evaluation of the gain in stiffness obtained in the retrofitting operation however remains hardly available, so that it is difficult to dispense with the safe Eurocode 3 provisions in terms of minimum steel deck and bituminous concrete overly thicknesses [5]. Moreover, ultra-high performance fibre-reinforced-concrete (UHPFRC) should represent a possibility to optimize the combined advantages of reinforced ultra-high strength concrete, as employed in the Netherlands [6], and fibre-reinforced concrete, used in a prototype project in Japan, provided the durable stiffness contribution of this innovative topping layer can be quantitatively ensured. Namely, this solution also allows a dead weight reduction thanks to the very small thickness of UHPFRC compared to the traditional solution with bituminous concrete, since only a thin epoxy-gravel overlay is then required for pavement characteristics.

Within the framework of the Orthoplus project, the experimental analysis was carried out using BSI®, the UHPFRC developed by Eiffage, as a topping layer for orthotropic bridge decks [7]. After a thorough characterization of the material used in this configuration, and definition of the connection processes, two large-scale specimens were tested to assess the role of a thin UHPFRC layer to decrease the fatigue stress at the welded junction between troughs and deck plate under the effect of a local loading by a truck wheel. Quasi-static loadings under different configurations, as well as fatigue tests under a central loading (up to 2 million cycles aiming to represent 100-years traffic loading) were carried out. Demonstration of effectiveness of this UHPFRC application is detailed in this paper.

2. UHPFRC OVERLAY REALIZATION

2.1 Connecting provisions

Success of the UHPFRC deck stiffening was assumed to require mechanical connection as efficient as possible. Fresh epoxy was considered as difficult to apply on site given the concomitant possible early setting and variations in UHPFRC rheology due to thermal sensitivity. A thin epoxy-gravel coating over which UHPFRC would be cast was feared as lacking mechanical end fixations. Two different systems were then studied. The first one consisted in a welded wire mesh, welded along the longitudinal direction to the top of the steel deck. The mesh size chosen was 100 x 100 mm with ribbed bars 7 mm in nominal diameter. Alternatively, small studs welded every 0.40 m along the longitudinal axes of the

troughs were used. Deriving of classical provisions for composite bridge decks, they were supposed to introduce fewer disturbances in the fibres repartition during casting. Conversely, the first system seems advantageous to deal with cold joints at UHPFRC concreting stops.

Both systems have been characterized in push-out tests on symmetric samples with about 0.4 m² shear surfaces. Studs were found to fail by yielding at the toe, which resulted in a ductile failure mode, while the welded wire mesh exhibited a higher capacity, yet with a more brittle failure mode probably due to shear failure at the welds. In both cases the demand in shear capacity was largely reached. Thus both systems were used for the large scale samples to be tested in the project. These 2.40 x 4.00 m mock-ups, having a 10 mm or 12 mm thickness and including trapezoidal-shaped longitudinal troughs every 0.60 m, were coated by BSI® in industrial representative conditions, either in a workshop or on site (Fig. 1). The 35 mm overlay thickness was determined by technological considerations to ensure proper cover of bars and stud heads, and should result in a similar overall rigidity, with a reduced 10 or 12 mm-thick steel plate, as the conventional 70 mm bituminous concrete overlay combined with the minimum 14 to 16 mm-thick steel plate, assuming perfect bond between both layers. However, with the UHPFRC solution the pavement dead weight is divided by about two, which keeps a decisive advantage in a retrofitting operation.

Figure 1: large scale implementation of the deck strengthening using connected UHPFRC. Welded wire mesh vs. studs used for connection in 2.40 x 4.00 m mock-ups

2.2 Effective overlay characterization

Rectangular plates of real 35 mm-thickness, 200 mm-wide and 700 mm-long were tested in 4 point-bending following AFGC recommendations for thin plates' characterization [7]. Half of these plates comprised the welded wire mesh in the bottom; they had been cast in square moulds with the mesh lying on the bottom of the mould. Half of them did not comprise any bar. Results are displayed on Fig. 2. Excluding ST2-2 sample due to inappropriate casting, the plain UHPFRC exhibits an average (resp. characteristic) maximum equivalent bending strength equal to 23.3 MPa (resp. 20.0 MPa). The welded wire mesh leads to earlier cracking initiation, but bonded bars contribute to a significant increase in capacity and ductility.
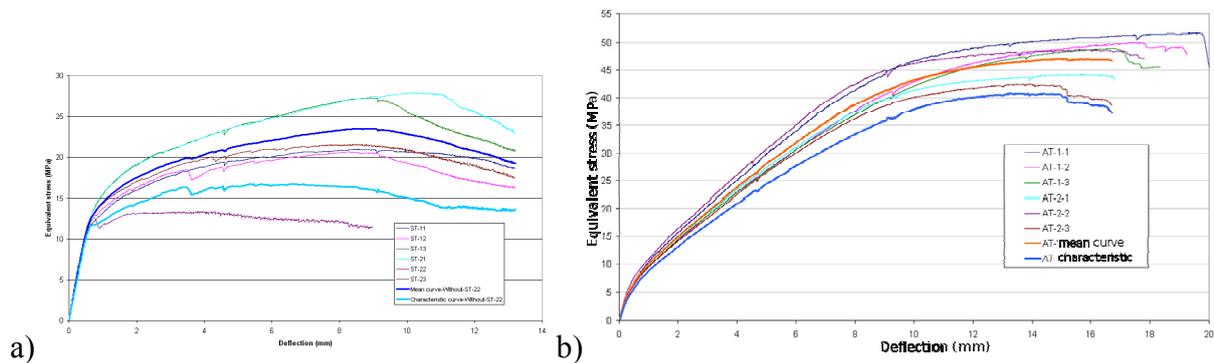
Figure 2: Equiv. bending stress (MPa) vs. deflection (mm) of plates tested in 4-point bending. In blue, characteristic design curve. a) Plain UHPFRC b) Plates with welded wire mesh

Moreover, the large mock-ups were sawn after the testing program at large scale (Fig. 3), and also tested in 4 point-bending, in order to confirm the residual UHPFRC capacity. This was made possible by the fact that in-between studs or welded bars, the UHPFRC layer proved disconnected from the steel plate, and was pulled off manually. Except for plates just in the centre of the slab directly under the load application point (curves #3 and 4 in Fig. 3-right), the UHPFRC capacity has remained stable. In such an application, the BSI® parameters [8] can thus be confirmed as Young's modulus of 57 GPa, design tensile limit of linearity of 9.6 MPa, tensile ultimate strength f_{ctf} = 8.6 MPa and ultimate strain = 7.6 mm/m.
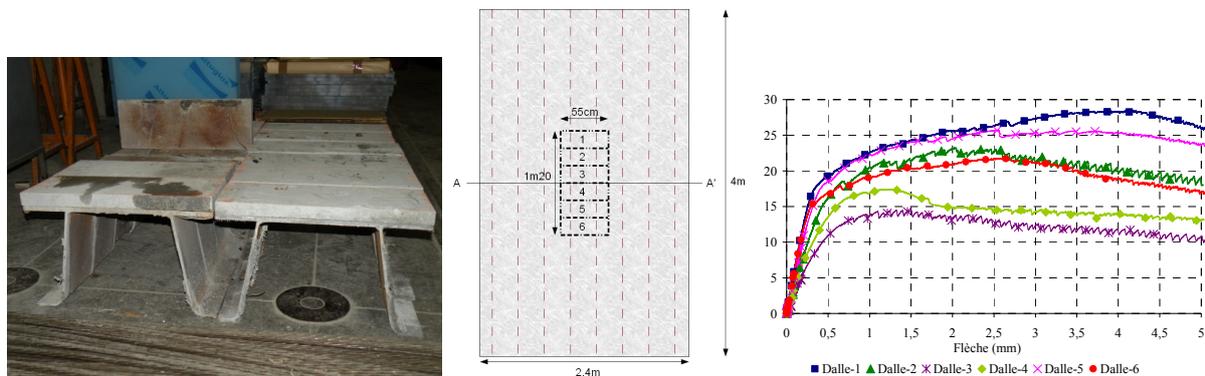
Figure 3: UHPFRC plate specimens sawn from the mock-ups for characterization of the overlay capacity after static and fatigue tests
(equivalent bending stress (MPa) vs. deflection (mm) of plates tested in 4-point bending)

2.3   Lessons drawn

Feasibility of realizing a well-controlled UHPFRC overlay exhibiting satisfactory properties (comparable to those measured in other bridge deck applications [9]) was confirmed in this project. The sensitivity of the process to possible early age plastic shrinkage in case of intense surface desiccation was also confirmed, which required immediate and thorough curing. Moreover, it was confirmed that connecting devices either provided by the studs or the welded wire mesh, although maintaining a global compatibility of the steel deck with the UHPFRC overlay, did not ensure a fully distributed connection. Quantitative analysis of the response of this partially-composite structure deserved combined experimental and modeling efforts as detailed in the following.

3.   QUASI-STATIC RESPONSE

The large-scale mock-ups were supported on both sides below the lower flange of cross-beams 3.5 m apart, and loaded in the central part (Fig. 4). Displacement sensors and strain gauges were mainly concentrated along the transverse mid-span axis to capture critical strains and evaluate the structure deformability. Although the mock-up is short and width is reduced, the span between cross-beams is representative of real situations, as well as the deck thickness. Moreover, the influence lines of an equivalent wheel load are very short (less than 1.0 m long, namely not more than one trough distance from the load in transverse direction) which makes the collected data fully usable for validating design and modeling procedures.
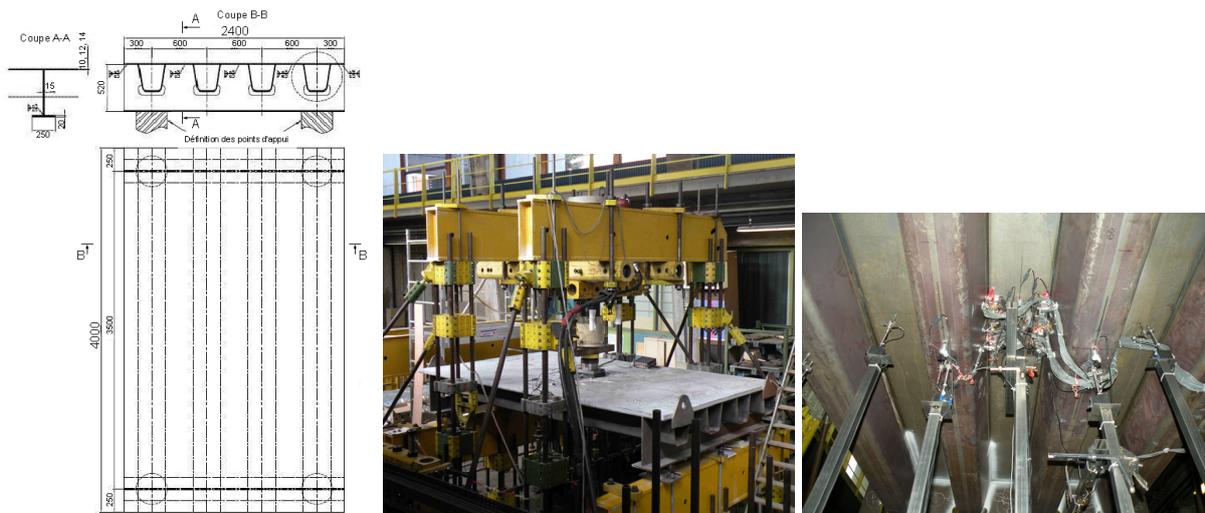

Figure 4: Central loading of the 2.40 x 4.0 m mock-ups to activate the local response and sensors to quantify the strains at welded joints at mid-span between cross-beams

3.1   Local response (plate A)

Exhaustive description of the loading program and measures collected on the mock-ups, as well as on reference samples, can be found in [10]. Evidence of the favourable stiffening provided by the UHPFRC overlay is given when analysing the mock-ups response under the "plate A" loading at mid-span in-between central troughs. This configuration derives from Eurocode idealization of traffic loads [11]. The 45 kN - load is applied over a 320 mm long by 220 mm wide surface; a neoprene layer ensures a distributed contact under the thick steel plate under the actuator, so that equally distributed pressure can be assumed (Fig. 5).

Characteristic figures of the average mock-up response are given in Table 1, after [10]. Stiffening provided by the UHPFRC overlay is clear in terms of central deflection, which possibly compensates the reduced steel deck thickness. Local plate stiffening is also clear when considering the transverse strains just below the loaded zone, with possible participation of the welded wire mesh when this connection system is used (thus the strain of 10 mm-steel mock-up is even lower). This result is all the more noticeable, that compressive strains measured on the upper side of the steel deck below the UHPFRC overlay confirm partial only connection. Average trough transverse and longitudinal deformations, just apart the loaded zone, remain however hardly affected. Finally, most interesting is the reduction of extrapolated stresses at the welded trough to deck junction both in the deck and on the trough webs, which are critical in fatigue verification.

Table 1: Deformation response of the mock-ups under plate A loading (45 kN)

mock-up	10 mm steel deck 35 mm BSI® overlay welded wire mesh	12 mm steel deck 35 mm BSI® overlay - studs	14 mm steel deck 70 mm bituminous concrete (reference)
Central deflection (mm)	1.14	0.98	1.38
Trans. strain at centre below steel deck (μm/m)	169	221	391
Trans. strain at mid-span trough lower flange (μm/m)	28	27	24
Long. strain at midspan trough lower flange (μm/m)	115	105	119
Extrapolated trans. stress [12] in the deck at welded joint (MPa)	43	22	37
Extrapolated trans. stress [13] in the trough web at joint (MPa)	41	33	54

Figure 5: Plate A loading at the centre of the mock-up

3.2   Deformability under different axle load configurations

Numerous loading configurations have been reproduced [10] in order to confirm the stiffened deck response due to UHPFRC overlay under possible traffic load situations considered in Eurocode or similar design guidelines [11]. Namely, real wheels have been used (Fig. 6) as well as twin plates representing loaded areas with variable inter-distance (from very close "wheels" to load axes above successive trough axes, which may amplify the deck counter-flexure), up to characteristic 150 kN-"wheel" load value. Major advantages of the UHPFRC overlay are illustrated on Fig. 7 for two typical situations. For loads applied straight above the trough axes (Fig. 7, left), the reference 14 mm-steel deck, either covered by 70 mm bituminous concrete or not, exhibits two maximal deflection zones (red, light blue and dark green curves) with a maximal deflection close to 1.33 mm. Conversely the specimens stiffened by the UHPFRC overlay (all other curves) exhibit a single-curvature deformed shape with a maximal central deflection close to 1.10 mm. For plates 100 mm-apart corresponding to the "wheel-B" type [11], deflection (Fig. 7, right) corresponds to a single curve in any case but the UHPFRC overlay helps reducing the deflection from 1.75 mm (reference specimen 14mm-steel deck covered by 70 mm bituminous concrete) to less than 1.50 mm even with only 10 to 12 mm-thick steel plate.
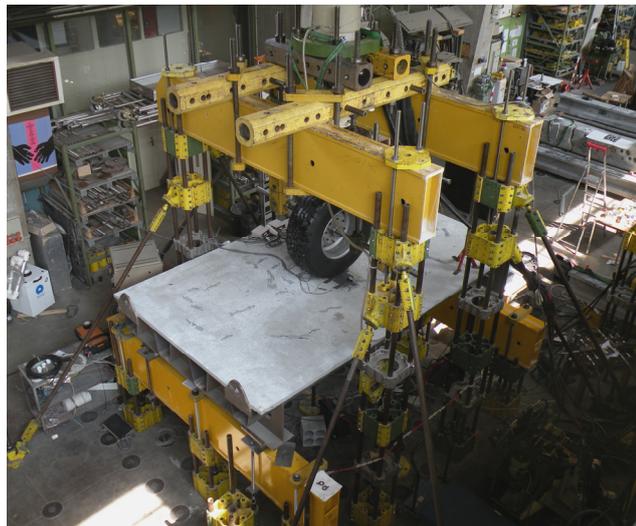

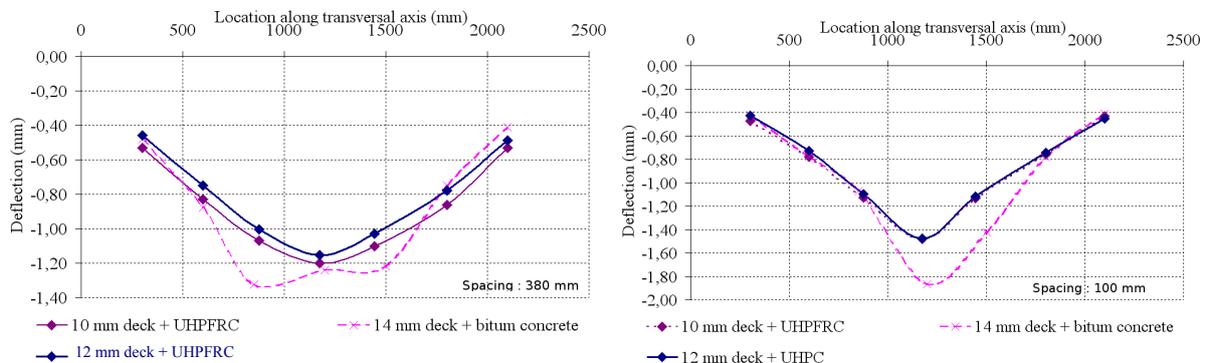Figure 6: Varied loading configurations of the stiffened orthotropic steel deck mock-ups

Figure 7: Deflections (mm) along the transverse axis of the mock-up at midspan, under twin 45 kN-loads with axes 600 mm (above the trough axes) and 320 mm-apart (type B [11])

3.3 Numerical analysis

Numerical modelling of the large-scale mock-up under centred type A loading was developed using multiple shell elements [14]. The global deformed shape and strains are reasonably well reproduced and confirm the stiffening role of the UHPFRC overlay (Fig. 8), even though a refined description of the partial connection between steel deck and UHPFRC remains to be calibrated. The model predicts significant tensile stresses in the bottom layer of the UHPFRC overlay, especially under the 150 kN-characteristic wheel loads that have been reproduced. This result seems consistent with thin cracking that has been observed *post-mortem* on the UHPFRC specimens sawn from the centre of the mock-ups [10]. However, precise quantitative calibration of the partial slip ability in the pseudo-composite element is still not gained, which prevents from a precise estimation of fatigue stresses extrapolated at the welded junction between steel plate and trough. In fact, geometrical modelling of this junction using plate elements still remains an issue for precise local stress estimation.
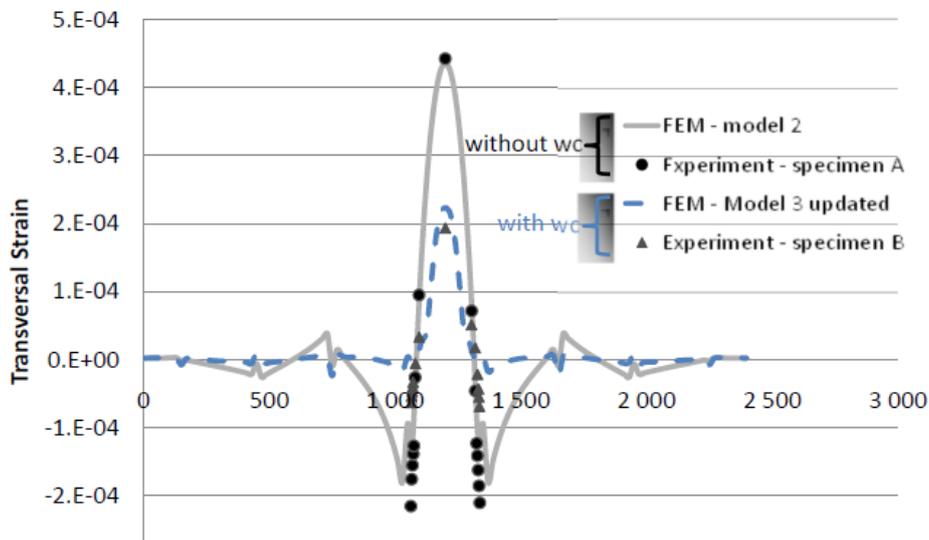

Figure 8: Transversal strain of deck bottom layer along the transverse axis at midspan. model 2 / specimen A corresponds to the 14 mm-steel deck without wearing course (wc), model 3 / specimen B to the 12 mm-steel plate plus UHPFRC overlay. See details in [14]

4.   FATIGUE RESISTANCE

4.1   Experimental validation

After the various quasi-static loading configurations, verification of fatigue resistance of the mock-ups stiffened with UHPFRC overlay was carried out in a centered type A load configuration. The load variation amplitude was derived from the typical data of heavy traffic (1 million 45 kN-wheel loads per year, over 100 years). The slopes of steel S-N diagram were used in order to find the equivalent load amplitude for 2 million cycles (Fig. 9), which is consistent with fatigue limiting mechanisms due to steel welds. This assumption contains a significant safety margin with respect to UHPFRC, since the fatigue mechanisms of such cementitious materials correspond to a much more flat S-N curve [15-17]. Under the effectively applied 110 kN-load amplitude for 2 million cycles, the critical stress variations extrapolated from strain measurements [10] result in values displayed in Table 2 (when available, two sets of values are given corresponding to symmetric trough junctions).
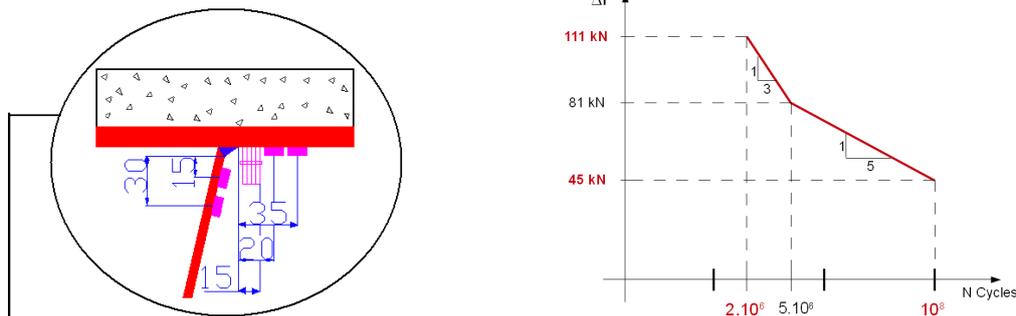
Figure 9: Determination of the load variation amplitude for fatigue testing

Table 2: Stress variations for fatigue verifications under 110 kN load variations (type A)

mock-up	10 mm steel deck 35 mm BSI® overlay welded wire mesh	12 mm steel deck 35 mm BSI® overlay - studs	14 mm steel deck 70 mm bituminous concrete (reference)
Extrapolated trans. stress [12] in the deck at welded joint (MPa)	50 to 57 / 61 to 68	12 to 24 / 15 to 28	71 to 78
Extrapolated trans. stress [13] in the trough web at joint (MPa)	71 to 80 / 80 to 88	62 to 72	115 to 121

4.2 Discussion

The mock-ups stiffened with the UHPFRC overlay hardly exhibit any significant stiffness evolution during the 2 Million fatigue cycles. Even though connection of the UHPFRC overlay with the steel deck was only partial in both cases, and even with previously damaged UHPFRC topping due to the effect of unrealistic overloads (twin characteristic 150 kN-loads closer to each other than in reality), the mock-ups were found fatigue-resistant and the critical stresses extrapolated for fatigue verifications have kept relatively stable (local variation from one side to the other turns out higher than the stress evolution during cycles). These stress values can be compared to class details according to Eurocode 3 [5], namely 100 MPa for the deck stress and 71 MPa for the stress in the trough web. Even if these values can be relaxed to 125 and 100 MPa respectively, following Kolstein's expertise [18], the UHPFRC overlay clearly provides a useful safety margin which is lacking in the reference situation.

5. CONCLUSIONS

Feasibility and efficiency of stiffening an orthotropic steel deck using a thin UHPFRC overlay has been demonstrated. Fine cracks and local nonlinearities (although non-evolving in the fatigue qualification program) were observed due to imperfect local connection between the steel deck and UHPFRC overlay, during the application of characteristic loads closer than in reality. Optimization of the connecting provisions is thus desirable for durability guarantee of the strengthening provided by the UHPFRC layer. Moreover, calibration of the simulation of the pseudo-composite system enabling precise prediction of fatigue critical stresses is still a matter of research, which would significantly help taking benefit of UHPFRC possibilities for the management and life extension of bridges with orthotropic steel deck.

ACKNOWLEDGEMENTS

REFERENCES

[1] De Jong, F., 'Renovation techniques for fatigue cracked orthotropic steel bridge decks', Ph.D. Thesis, Delft University of Technology, 2007.

[2] Le Quéré, C., 'Projet Orthoplus : ingénierie avancée des dalles orthotropes et de leur revêtement', *Travaux* n°**843**, 2007

[3] Teixeira de Freitas, S., Kolstein, H. Bijlaard, F., 'Composite bonded systems for renovations of orthotropic steel bridge decks', *Composite Structures* 92 (2010) 853-862.

[4] Battista, R., Pfeil, M., Carvalho, E., 'Fatigue life estimates for a slender orthotropic steel deck', *Journal of Constructional Steel Research* 64 (2008) 134-143.

[5] NF EN 1993-1-9, Eurocode 3, Calcul des structures en acier, Partie 1-9 : Fatigue, 2005

[6] Braam, R., Kolstein, H., Romeijn, A., Buitelaar, P., 'Concrete overlays to improve the fatigue life of movable orthotropic steel bridge', Int. Orthotropic Bridge Conf., Sacramento, 2008

[7] AFGC, 'Ultra-high performance fibre-reinforced concretes. Interim recommendations', 2002.

[8] AFGC, 'Ultra-high performance fibre-reinforced concretes. Recommendations', 2013.

[9] Toutlemonde, F., 'Les dalles en BFUP', *in* "Ponts mixtes acier-béton. Un guide pour des ouvrages innovants", Chabrolin et al. eds, Presses des Ponts, 2010, II-8, pp. 257-332.

[10] Gomes, F., 'Influence du revêtement sur le comportement en fatigue des dalles orthotropes : étude d'une solution en BFUP', Ph.D. Thesis, Université Paris-Est, 2012.

[11] NF EN 1991-2, Eurocode 1, Actions sur les structures, Partie 2 : Actions sur les ponts, dues au trafic, 2004.

[12] Hobbacher, A., 'Recommendations for fatigue design of welded joints and components', document XIII-1965-03/XV-1127-03, 2006, International Institute of Welding (IIS)

[13] Bignonnet, A., Carracilli, J., Jacob, B., 'Comportement en fatigue des ponts métalliques : application aux dalles orthotropes en acier', research report CECA n°7210 KD/317, LCPC, 1990.

[14] Dieng, L., Marchand, P., Gomes, F., Tessier, C., Toutlemonde, F., 'Use of UHPFRC overlay to reduce stresses in orthotropic steel decks', *J. of Constructional Steel Research* 89 (2013) 30-41.

[15] Parant, E., 'Mécanismes d'endommagement et comportements mécaniques d'un composite cimentaire fibré multi-échelles sous sollicitations sévères : fatigue, choc, corrosion', Ph.D. Thesis, Ecole Nationale des Ponts et Chaussées, 2003.

[16] Mouquet, D., Bernier, G., Behloul, M., 'Etude du comportement à la fatigue du BPR fibré', *Fibre Reinforced Concretes (FRC)*, BEFIB 2000, RILEM Proceedings, PR015, pp 769-779.

[17] Toutlemonde, F., Lauvin, L., Renaud, J.-C., Kretz, T., Brisard, S., 'Fatigue performance of an UHPFRC ribbed slab applied as a road bridge deck verified according to the Eurocodes', Fifth Int. Conf. on Concrete under Severe Conditions, pp 1191-1200, Tours, 2007.

[18] Kolstein M.H., 'Fatigue Classification of Welded Joints in Orthotropic Steel Bridge Decks', PhD Thesis, TU Delft, 2007