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Design of 38m Span Post-tensioned Ultra High Performance Fiber-Reinforced Concrete (UHPFRC) Composite Bridge

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Abstract. Although ultra-high-performance fibre reinforced concrete (UHPFRC) has become commercially available in many countries, there are still limited design standards for UHPFRC structures. In 2016, the French Standard Institute has published the world first design standard for UHPFRC structures (NF P18-710) which is read conjunction with the UHPFRC material specification code (NF P18-470). This standard is the national complement to Eurocode 2 for the design of UHPFRC structures. To date, there have several UHPFRC bridges been designed by using this French Standard. The aim of this paper is to demonstrate working example on the design calculation of 38m span post-tensioned UHPFRC composite bridge using DURA® Grade140 which was recently constructed in Kuala Terengganu. The working example mainly focus on (i) the material properties input in the structural analysis software – Midas Civil; (ii) the output results on the design forces at the critical sections (mainly the design moment effect, M_{Ed}, and design shear force effect, V_{Ed}); (iii) the stresses of the post-tensioned UHPFRC U-girder at transfer, at different construction load history and at service stage (iv) the design moment resistance (M_{Rd}) and design shear resistance (V_{Rd}) of the composite section.

1. Introduction

Since the millennium, over 200 bridges or bridge components have been constructed using UHPFRC all around the world which includes Australia, Canada, China, France, Germany, Japan, Malaysia, Netherland, Spain, South Korea, United States and many others. The world first UHPFRC road bridges with the total length of 44 m and 12 m width were constructed in France in 2001 [1]. To-date, Malaysia has built the most number of UHPFRC bridge with more than 100 UHPFRC bridges have been built since year 2010. Ninety over percent of these bridges are owned by Malaysia governmental agencies (such as JKR, JPS and KKLW) and the UHPFRC girders come with different features and sizes are manufactured and supplied by DURA Technology Sdn Bhd (DTSB). The Malaysia first UHPFRC composite road bridge built with 50m long precast/ prestressed U-girder and composited with 4.5 m wide and 200mm thick RC deck was constructed in 2011 crossing Sg. Linggi [2], Negeri Sembilan. Since then, the Sungai Nerok Bridge comes with three-30 m long-spans and 15 m width using 30 numbers of UHPFRC decked bulb-tee girders was completed in 2012; and the Rantau-Siliau Bridge with a single span of 52 m long and 18.3 m wide using 5 pieces of UHPFRC U-beams was built in 2014 in Malaysia [3]. The world's longest single span (i.e. 100 m) road bridge using UHPFRC segmental box girder crossing Sungai Perak was built in 2015 [4]. To-date, the world's longest multiple span UHPFRC road bridge (i.e. 10 spans x 42m = total 420m) crossing an estuary between Kampung Baharu and Kampung Teluk at Ayer Tawar, Perak, was constructed in 2016. The traffic loading used was according to the specification of the bridge design code BS5400 [5]. Although there are several design guidelines and recommendations by different countries (e.g. Australia, Switzerland, Germany, Japan, South Korea and etc) for design of UHPFRC structures, but they are generally not acceptable by the authorities due to the documents are not a full code. However, in 2016, the French Standard Institute broke the iceberg by publishing the first design and material specification standards, NF P18-710 [6] and NF P18-470 [7], respectively. Today, the Malaysia Public Work Department is adopting these two standards to design and tender some of their bridge projects. This paper presents an example for the design of a 38-meters span U-shape post-tensioned UHPFRC road bridge according to the French standards.

2. General information of the project

The road bridge presented in this paper is known as KT-Bypass ST3 located at the urban area of Kuala Terengganu (GPS Location: 5.304631 N, 103.124018 E). This elevated bridge was recently completed and was designed as a simply supported span and linked slab from each span to the another spans. The overall length of the bridge is 191.6 m with a total width of 11.5 m x 2 sides which consists of five spans of 38.5 m (see figure 1a and 1b). The superstructure of the bridge is made of two numbers of precast UHPFRC post-tensioned U-beams for each span as shown in figure 1c. All the recast beams were seated on elastomeric bearings which placed on the in-situ piers and abutments.

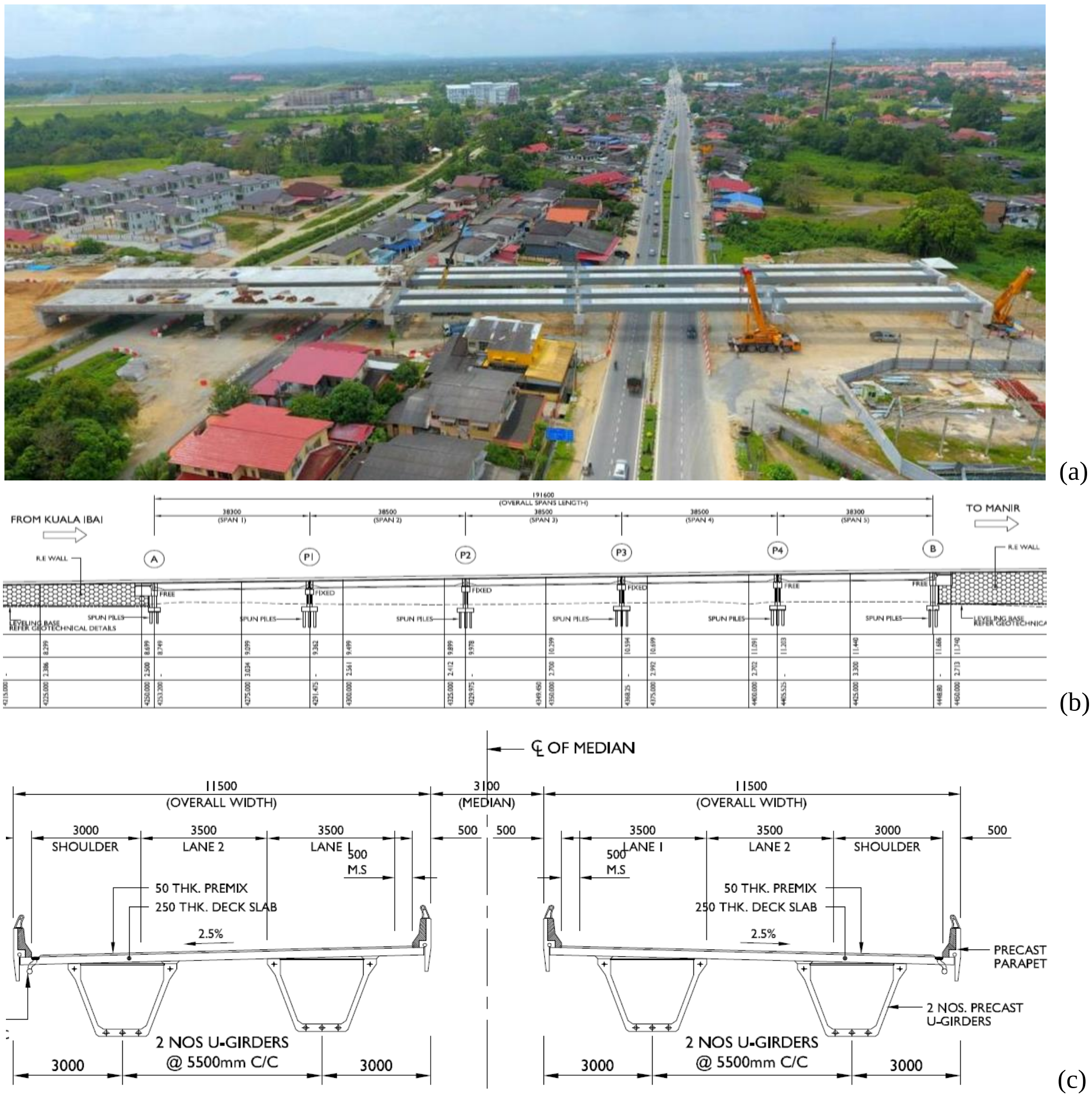

Figure 1. KT-Bypass ST3: (a) Partially constructed with (b) elevation and (c) cross section views.

2.1. Details of UHPFRC U-girder

Each of the 38m long UHPFRC U-girder consists of five U-segments, that is three numbers 8 m long internal segments (19 tonnes each), and two 7 m long anchorage end segments (16.6 tonnes each) as shown in figure 2. In the UHPFRC U- girder there is no stirrup in its thin webs unlike conventional concrete girders. The only conventional reinforcements used are the bursting reinforcement at the anchorage zone and the horizontal shear studs at the top flanges where connection is needed with the reinforced concrete deck. The segments are assembled and joined with post-tensioning forces. The top tendons consist of 2-7K15 and the bottom tendons consists of 2-22K15 and 2-27K15. A total of 112 number of strands are used in a single 38m long UHPFRC U-girder. After the tendons were stressed, the corrugated ducts were grouted with cement grout.

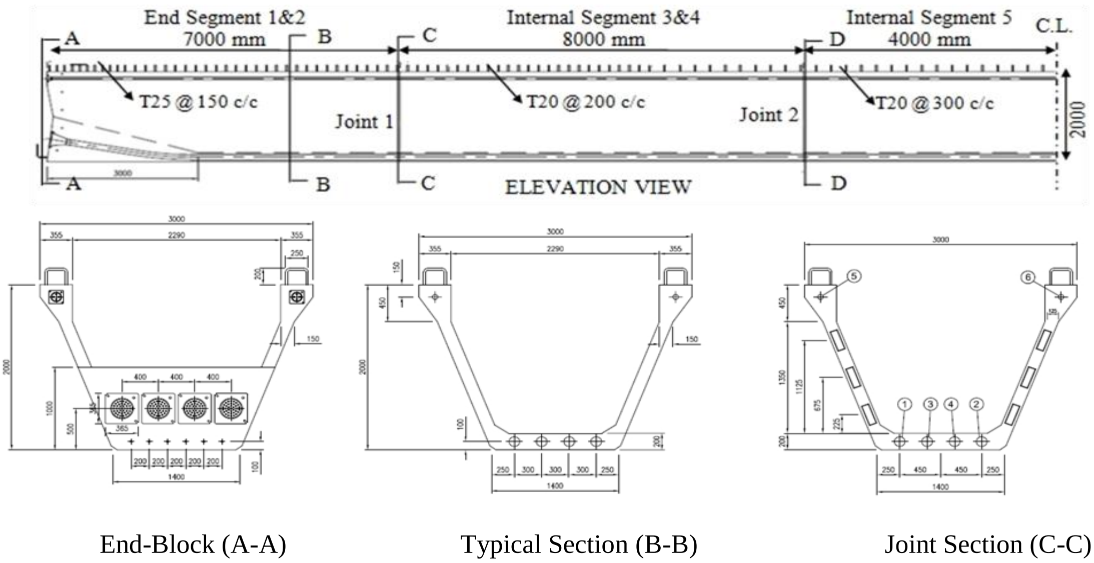

Figure 2. Details of UHPFRC DURA® UBG2000 girder.

2.2. Mechanical Properties of DURA® UHPFRC

UHPFRC is a type of advanced cementitious based composite material which has superior strength and durability. Table 1 shows the QA/QC test results on the mechanical strength of the twenty U-girders, where $f_{cm,cu}$ is the mean cube compressive strength at 1 and 28 days; $f_{ctm,el}$ and f_{ctfm} are the mean tensile limit of elasticity and mean post-cracking tensile strength, respectively [7]. The term $f_{ctm,fl}$ is the equivalent elastic flexural strength which was measured using 100 mm prism under four point test [7], where an example on the experimental curves of the flexural strength test are presented in figure 3. The type of UHPFRC used in the structural analysis/design has a design characteristic compressive strength of Grade140/155.

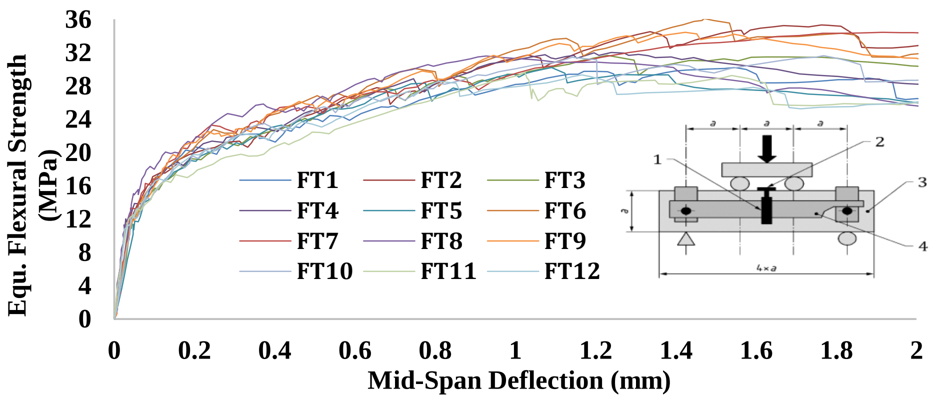

Figure 3. Experimental curves on the equivalent flexural strength of UHPFRC (Girder 3)

Table 1. Test results on material properties of UHPFRC use at U-girders (in MPa)

Girder	$f_{cm,cu,1d}$ [1]	$f_{cm,cu,28d}$ [2]	$f_{ctm,el}$ [3]	f_{ctfm} [4]	$f_{ctm,fl}$ [3]	Girder	$f_{cm,cu,1d}$ [1]	$f_{cm,cu,28d}$ [2]	$f_{ctm,el}$ [3]	f_{ctfm} [4]	$f_{ctm,fl}$ [3]
1	92	169	8.67	9.30	32.6	**11**	84	172	11.3	10.8	32.4
2	85	165	10.0	9.70	32.4	**12**	90	175	9.27	10.7	31.2
3	82	173	8.40	9.80	32.8	**13**	82	175	10.27	10.7	30.2
4	83	162	8.93	10.3	34.3	**14**	88	170	8.73	10.3	31.0
5	89	162	8.80	10.5	34.8	**15**	89	174	8.27	10.2	27.9
6	92	169	8.40	9.00	29.0	**16**	84	179	10.1	9.60	30.7
7	91	163	8.27	9.90	29.8	**17**	96	172	8.60	10.2	32.4
8	92	164	8.20	11.1	36.3	**18**	86	177	9.07	9.90	31.5
9	93	165	9.53	10.0	30.0	**19**	92	170	9.00	9.90	29.7
10	85	163	9.60	10.5	35.4	**20**	86	182	8.93	10.3	28.9
Sample Size (n)							**300**	**300**	**240**	**240**	**240**
Mean							**87**	**170**	**9.1**	**10.1**	**31.7**
Standard Deviation, S.D.							**6.5**	**7.5**	**0.8**	**0.5**	**2.3**
Experimental Characteristics Value							**76**	**157**	**7.8**	**9.3**	**27.9**

Notes:1. For 1-day mean strength, each beam tested with 15 nos. of 100mm cubes (before heat curing).
2. For 28 days mean strength, each beam tested with 15 nos. of 100mm cubes (after heat curing).
3. For 28 days mean strength, each beam tested with 12 nos. of 100mm prisms (after heat curing) under 4 points test.
4. For 28 days mean strength, each beam tested with 12 nos. of 100mm prisms (after heat curing) under 3 points test.

Table 2 gives the general material properties of UHPFRC used in the design software. Moreover, the basic creep coefficient at 28 days ($\varphi_{b,28d}$) and after cured shrinkage (ε_{sh}) are obtained as 0.2 and zero, respectively.

Table 2. Material properties of UHPFRC used in the design.

Mechanical Properties	UHPFRC140/155
Characteristic cylinder compressive strength, f_{ck} (MPa)	140
Characteristic cube compressive strength, $f_{ck,cube}$ (MPa)	155
Characteristic tensile limit of elasticity, $f_{ctk,el}$ (MPa)	7.0
Characteristic post-cracking tensile, f_{ctfk} (MPa)	8.0
Characteristic modulus of rupture, $f_{ctk,fl}$ (MPa)	20
Mean value of Modulus of elasticity, E_{cm} (GPa)	50
Poisson's ratio of UHPFRC, υ	0.2
Post-cured shrinkage, ε_{sh}	0

3. Construction stages

The following briefly gives the construction stages of the UHPFRC composite bridge.

3.1. Fabrication of UHPFRC U-girders

Manufacturing of the U-girder began in early 2017 as shown in **Figure 4a**. All the segments undergone heat treatment of 90°C and 100% humidity for a period of 48 hours as recommended by NF P18-470 [7]. The total weight of the full girder is approximately 95 ton.

3.2. Assembling of UHPFRC U-girders

Pole trailers were used to transport the unassembled segments to the site (see figure 4b). As shown in figure 4c due to the lightness of each segments, only one unit 45 tones mobile crane was used to unload and align the U-segments to form a full straight 38m long beam.

3.3. Post-tensioning of UHPFRC U-girders

The installation of strands and fitting the anchorage heads is shown in figure 4d. This picture shows the technicians are fitting the anchorage blocks of the four ducts at 27K15 tendons (bottom row) and the two ducts at 2-7K15 (top row). At total numbers of 112 strands with diameter of 15.24 mm were passed through the ducts of each 38m U-girder. Post-tensioning (PT) was carried out by Freyssinet PSC Malaysia using 70,000kN capacity hydraulic jack as illustrated in figure 4e. Total jacking prestressing forces of 19110 kN and 2730 kN were applied to the tendons at bottom and top rows, respectively. At the end of the PT work, the midspan instantaneous hog deflection was measured to be approximately 56 mm hog. At the end the grout was pumped into the ducts.

3.4. Launching of UHPFRC U-girders

Figure 4f shows the transportation of the fully assembled UHPFRC U-girder for beam launching. Two 160 tonnes mobile cranes were used to lift the UHPFRC U-girders (see Figure 4g). Figure 4h shows all the beams after positioned on the substructures.


(a) Casting of 8m long internal U-segment


(b) Transporting U-girder segments using trailer


(c) Unloading and assembly of U-girders


(d) Installation of strands and anchorage heads


(e) Post-tensioning of tendons in progress


(f) Transporting U-girder for beam launching


(g) 2 units 160 tonnes mobile cranes used


(h) All UHPFRC U-girders were launched

Figure 4. Construction stages of five spans UHPFRC bridge

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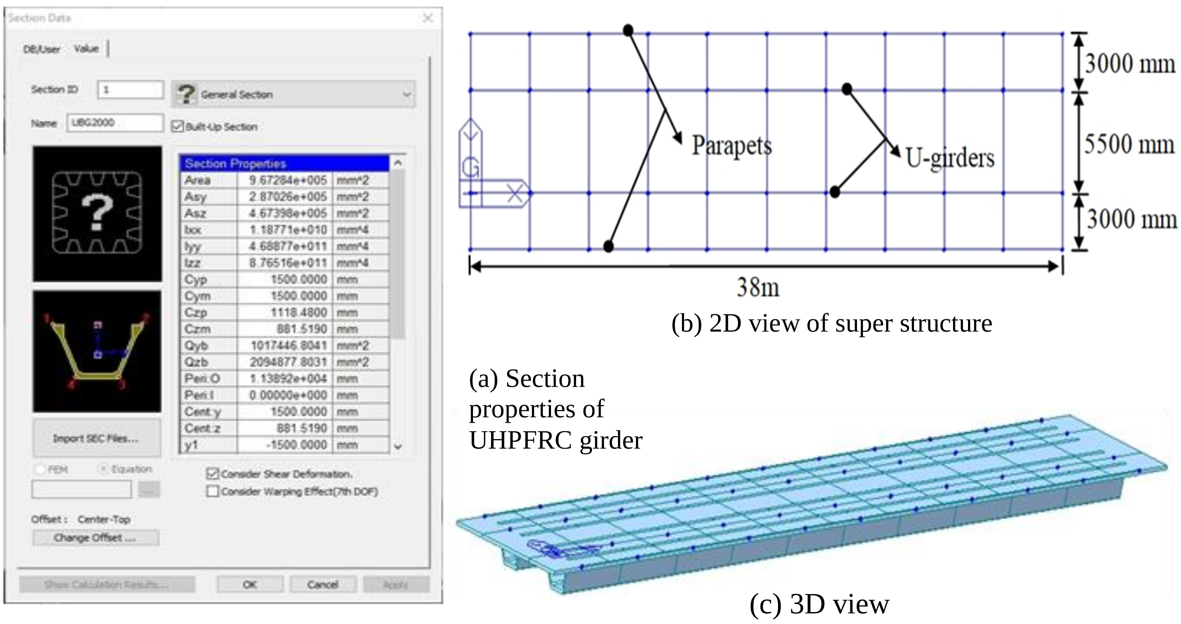
3.5. In-situ conventional concrete components
To enhance the durability of concrete structure the minimum concrete grade for all the in-situ concrete structure considered as Grade40. The cast-in-place deck was built from normal Grade 40 concrete with 250 mm thickness which has a transversal slope of 2.5%. In addition, the superstructure considers the use of cast in-situ reinforced concrete diaphragms over piers and abutments but no intermediate diaphragms at midspan. This bridge was classified under the exposure class of XC3 with moderate humidity according to the EC2 [8].

4. Design method

4.1. Design Forces under SLS and ULS
Grillage analysis was used to calculate the structural response of the simply supported UHPFRC composite bridge. For this purpose, commercial bridge design software called Midas Civil was utilized to analyze the UHPFRC girders (refer to figure 5). The 3D view of the simply supported 38 m bridge is depicted in figure 5c.

Static loads imposed on the bridge due to self-weight (SW) of the UHPFRC U-girders, 250mm thick RC deck, super-imposed dead load (i.e. due to premix) and RC parapet are shown in Figures 6a-d, respectively. The composite bridge is designed to withstand 45 units HB loading and HA + KEL loading for the ultimate limit state (ULS). However, for the service limit state (SLS), 30 units HB loading and HA + KEL loading is applied when calculating crack widths in accordance with the BS5400 [5]. Figure 6e shows the notional lanes during moving loads analysis. For this bridge, the notional lane width is 3.5 m in accordance with BS 5400 [5]. figures 7 and 8 show the unfactored bending moments and shear forces of the UHPFRC girders respectively. Table 3 presents the design forces due to static dead loads and moving loads.

From the structural analysis output, the design shear force effects at the supports under SLS and ULS loadings were calculated as $V_{SLS} = 2880$ kN and $V_{Ed} = 4463$ kN respectively. On another hand, the design moment effects at SLS and ULS are equal to $M_{SLS} = 27035$ kNm and $M_{Ed} = 41205$ kNm.



(b) 2D view of super structure

(a) Section properties of UHPFRC girder

(c) 3D view

Figure 5. Beams arrangement and sectional properties in Midas Civil software

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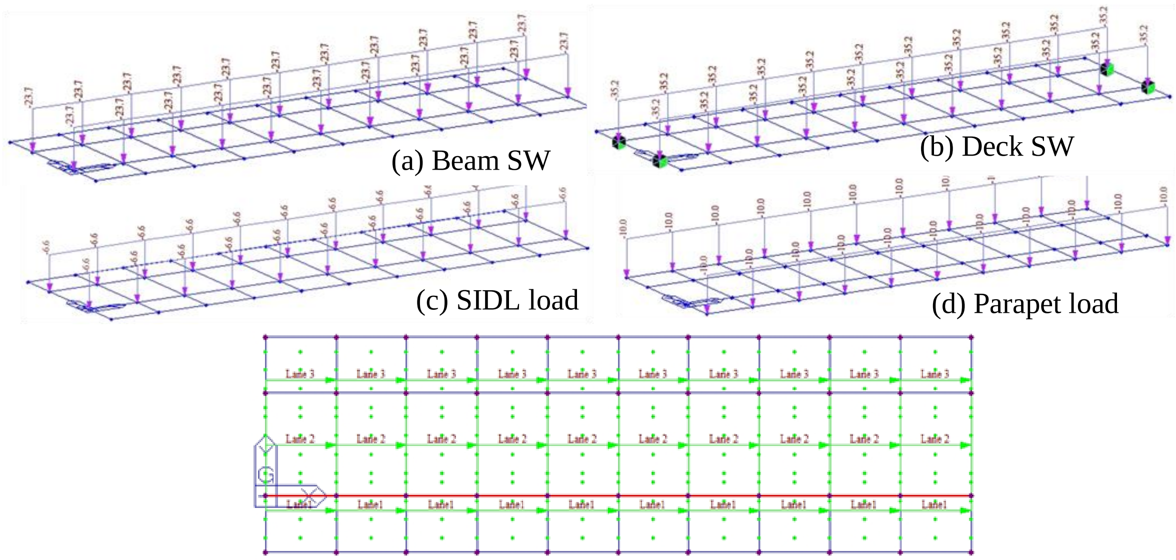
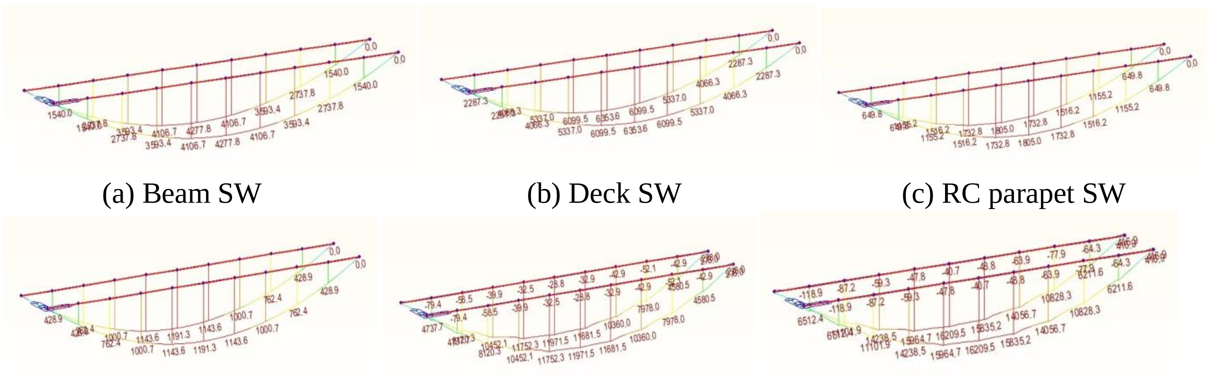


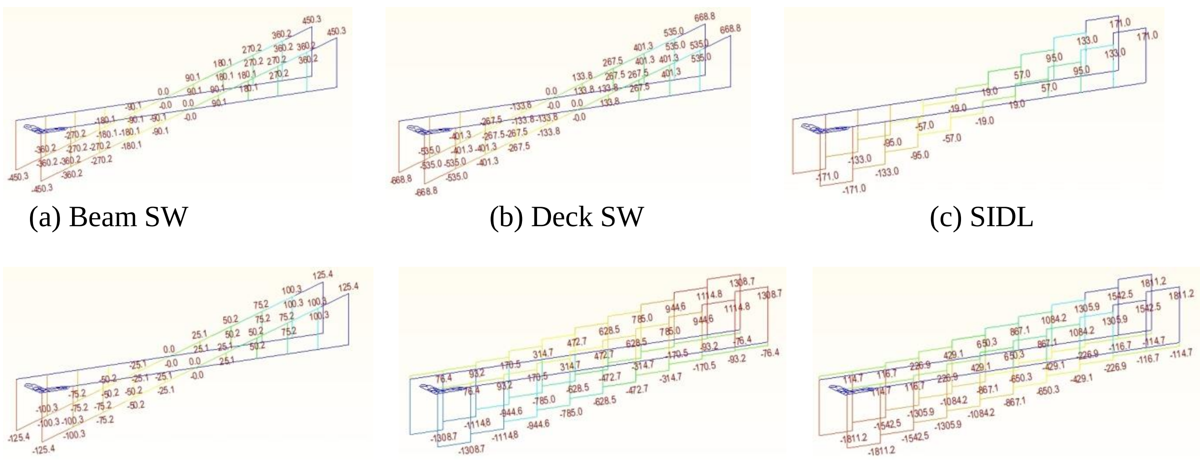
(a) Beam SW (b) Deck SW (c) SIDL load (d) Parapet load (e) Number of notional

Figure 6. Different loads applied on the bridge



(a) Beam SW (b) Deck SW (c) RC parapet SW (d) SIDL (e) HA+30HB (SLS) (f) HA+45HB (ULS)

Figure 7. Unfactored bending moments diagrams.



(a) Beam SW (b) Deck SW (c) SIDL (d) Parapet SW (e) HA+30HB (SLS) (f) HA+45HB (ULS)

Figure 8. Unfactored shear force diagrams

Table 3. Design shear forces and bending moment effects at SLS and ULS

Member ID	Unfactored Moment at mid-span (kNm)	Unfactored Shear at support (kN)	SLS	ULS		SLS		ULS	
			γ_{fL}	γ_{fL}	γ_{f3}	Factored Moment (kNm)	Factored Shear (kN)	Factored Moment (kNm)	Factored Shear (kN)
U-girder	4277.8	450.3	1	1.15	1.1	4278	450	5411	570
Deck	6353.6	668.8	1	1.15	1.1	6354	669	8037	846
Parapet	1805	171	1	1.15	1.1	1805	171	2283	216
SIDL	1191.3	125.4	1.2	1.75	1.1	1430	151	2293	241
Com.HA&30HB	11971.5	1308.7	1.1	1.3	1.1	13169	1440	-	-
Com.HA&45HB	16209.5	1811.2	1.1	1.3	1.1	-	-	23180	2590
Total	-	-	-	-	-	**27036**	**2881**	**41204**	**4463**

For SLS check, the sectional modulii used for the transformed section is taken as $Z_{top,beam}$ = 436.83 x 10^6 mm^3 and $Z_{bot,\ beam}$ = 584.99 x 10^6 mm^3; wherease for the composited section is take as $Z_{top,bridge}$ = 1,477.1 x 10^6 mm^3 and $Z_{bot,\ bridge}$ = 868.33 x 10^6 mm^3. Table 4 presents the stresses at the top and bottom extremem fibersof U-girder, stress at top slab and the midpsan deflection of the UHPFRC U-girder at transfer and different load histories during the construction. Calculation shows the precast U-beam will experience an instantaneous net hog deflection of 53.6 mm wherase the top and bottom extreme fibers will have stresses of -5.1 MPa and -31.9 MPa respectively. After the RC deck is casted, the precast U-beam will experience a net hog deflection of 15.4 mm wherase the top and bottom extreme fibers will have stresses of -19.6 MPa and -21.1 MPa, respectively. During casting of the wet topping, it is assuming only the none-composited precast U-beam is taken the full dead load of the RC deck. The next stage will be the additional load due to the parapet and the wearing course. Calculation shows at this stage the composited beam will experience a net hog deflection of -7.5 mm wherase the top and bottom extreme fibers will have stresses of -21.1 MPa and -17.3 MPa respectively. Lastly, precast U-beam will experience a net sag deflection of 24.7 mm wherase the top and bottom extreme fibers will have stresses of -27.4 MPa and -2.2 MPa respectively during full service stage.

Table 4. Stresses and deflection at midspan at SLS

Load Cases	**Load histories**	**Section Type**	**Stress at U-girder (MPa)**		**Stress at top slab (MPa)**	**Midspan deflection (mm)**
			Top	**Bottom**		
(1)	SW of girder	None-Composited	-9.8	7.3	-	26
(2)	Prestressing force	None-Composited	4.7	-39.2	-	-79.4
(3) = (1) + (2)	After transfer	None-Composited	-5.1	-31.9	-	-53.6
(4)	Incremental In-situ RC deck	None-Composited	-14.6	10.9	-	38.2
(5) = (3) + (4)	After RC deck Casted	None-Composited	-19.6	-21.1	-	-15.4
(6)	Incremental SIDL + Parapet	Composited	-1.5	3.7	-1.5	7.9
(7) = (6) + (5)	Under Total Sustained DL	Composited	-21.1	-17.3	-1.5	-7.5
(8)	At Service HA + 30 HB	Composited	-6.2	15.2	-6.2	32.2
(9) = (7) + (8)	At Full Service Stage	Composited	-27.4	-2.2	-7.8	24.7

4.2. Design moment resistance and design shear resistance

The calculation of the design moment resistance (M_{Rd}) of the UHPFRC composite bridge is similar to the conventional concrete bridges. The theory of strains compatibility and forces equilibrium acting on the cross-section of the bridge were used and the resultant strains and resisting internal forces of each components are presented in figure 9. The calculated neutral axis depth is X = 221.3 mm, which is located at the RC deck. Because the full 38m U-beam is come with five segments which were then joined by using prestressing, the weakest sections are the segmental joint sections, therefore the sections are consider do not developed any tensile stress at any level of the sections. Thus, the tensile force generally is taken purely by the tendons. The composited RC deck comes with four layers of reinforcements whereas the longitudinal direction is reinforced with T12-125mm c/c and the transverse direction is reinforced with T25-125mm. The concrete cover used is 30 mm. For linear strain compatibility, the concrete top extreme fibre strain is a taken as 0.0035 as per EC2 [8].

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The top and bottom reinforcement strains can be written as $\varepsilon_{s1} = 0.0035 \times \frac{(X-61)}{X} = 0.00254 > 0.002\ (yielded)$ and $\varepsilon_{s2} = 0.0035 \times \frac{(X-189)}{X} = 0.00051 < 0.002\ (not\ yielded)$ respectively. The top and bottom tendon strains can be expressed as $\varepsilon_{p1} + \varepsilon_{pe1} = \left(0.0035 \times \frac{(de-X)}{X}\right) + \frac{1860\times0.75\times0.95\times0.95}{195000} = 0.00848 > 0.0083\ (yielded)$ and $\varepsilon_{p2} + \varepsilon_{pe2} = \left(0.0035 \times \frac{150}{X}\right) + \frac{1860\times0.75\times0.95\times0.95}{195000} = 0.03694 > 0.0083\ (yielded)$ respectively. The terms ε_p and ε_{pe} are the flexural strain and effective pre-strain in tendons respectively. A 5% immediate and 5% long term losses were considered to calculte the design moment resistance. It is obvious that the top layer of the reinforcement has yielded and the bottom layer of the reinforcement still in the elastic range. However, both the top and bottom tendons have the reached their maximum design capacity. Therefore the internal forces can be calculated as: compressive force of RC deck, $C_C = 5750 \times 22.7 \times 0.8 \times 221.3/1000 = 23108.6kN$; compressive force of top reinforcement, $C_{S1} = 39 \times 113 \times 460/(1.15 \times 1000) = 1762.8kN$; compressive force of bottom reinforcement, $C_{S2} = 39 \times 113 \times 200{,}000 \times 0.00052/1000 = 450.3kN$. Therefore the total internal compressive force is $\mathrm{C} = 25321.7\ kN$. The tensile force of the top tendon, $T_{P1} = 14 \times 260/1.15 = 3165.2kN$; and the bottom tendon is $T_{P2} = 98 \times 260/1.15 = 22156.5kN$. The total internal tensile force is calculated as $\mathrm{T} = 25321.7\ kN$. Therefore, the sum of forces is equal to zero. Lastly the design moment resistance can be calculated by taking moment about the top extreme fiber which gives M_{Rd} = 45995 kNm > M_{Ed} = 41205 kNm, which is greater than the design moment effect. Thus the section has adequate flexural resistance.

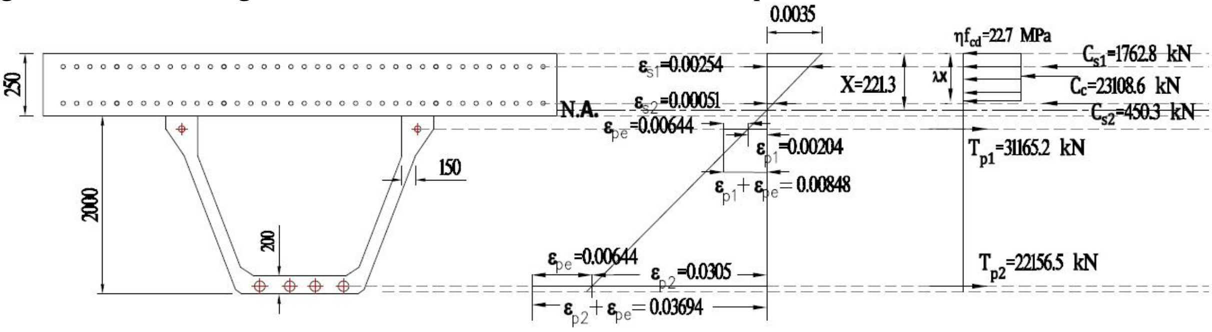

Figure 9. Strain distribution and equilibrium of the forces acting on the section

Since no shear reinforcement is used in the vertical web of the U-girders, the design shear resistance (V_{Rd}) can be calculated from the design provision as given in the French Standard [6]. As explained in clause 6.2 of French Standard for UHPFRC [6], V_{Rd} can be calculated as follows:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} + V_{Rd,f} \tag{1}$$

where $V_{Rd,c}, V_{Rd,s}$, and $V_{Rd,f}$ are design shear resistances provided by UHPFRC, steel stirrups, and steel fibers, respectively. Due to there is no stirrup used in the UHPFRC girder, thus the term $V_{Rd,s}$ is zero.

Design shear resistance provided by UHPFRC ($V_{Rd,c}$) is calculated as:

$$V_{Rd,c} = \frac{0.24}{\gamma_{cf}\gamma_E}\ k\ f_{ck,UHPFRC}^{1/2} b_w.z = \frac{0.24}{1.5} * 1.436 * 140^{1/2} * 300 * 1935 = 1578\ kN \tag{2}$$

where material safety factor, $\gamma_{cf}\gamma_E = 1.5$; $b_w = 150\ mm \times 2\ sides = 300\ mm$ is the total web thickness; lever arm of external force, $z = 0.9d = 0.9 \times 2150mm = 1935\ mm$; $d = 2000 + 250 - 100 = 2150\ mm$ is the effective depth of composite section, and k factor is determined as:

$$k = 1 + 3\,\sigma_{cp}\,/f_{ck,UHPFRC} = 1 + 3 * 20.3/140 = 1.436 \tag{3}$$

where σ_{cp} is average confining stress due to prestress and is equal to $\sigma_{cp} = N_{Ed}/A_c = 19711/0.967 = 20.3\ MPa$. Total axial force due to prestressing in the cross-section is $N_{Ed} = (2 \times 7 + 2 \times 27 + 2 \times 22) \times 260\ kN \times 0.75 \times 0.95 \times 0.95 = 19711\ kN$ and $A_c = 0.967\ m^2$ is gross cross section area of UHPFRC girder.

Design shear resistance provided by steel fibers ($V_{Rd,f}$) is determined as:

$V_{Rd,f} = A_{fv}\sigma_{Rd,f}cot\theta = 300 * 1935 * 4.923 \cot(30) = 4950\ kN$ (4)

where A_{fv} is effective vertical web area and is equal to $b_w \times z$, θ is angle of inclination of the main compression stress on the longitudinal axis (which is taken as $\theta = 30°$), $\sigma_{Rd,f}$ which is the design value of post cracking strength and calculated as below.

$\sigma_{Rd,f} = f_{ctfk}/K.\gamma_{cf} = 8.0/1.25 * 1.3 = 4.923\ MPa$ (5)

where $K = 1.25$ is global fiber orientation factor and $\gamma_{cf} = 1.25$ is the partial factor of UHPRFC under tension.

Finally, the design shear resistance is determined as:

$V_{Rd} = V_{Rd,c} + V_{Rd,s} + V_{Rd,f} = 1578 + 0 + 4950 = 6528\,\text{kN} > \text{V}_{\text{Ed}} = 4463.4\,\text{kN}$.

Therefore, the girder has adequate shear resistance (V_{Rd}) to withstand the design shear force (V_{Ed}). Furthermore, according to the French Standard [6], V_{Rd} must be smaller than the design limit force for the compressive strength of UHPFRC ($V_{Rd,max}$) which is calculated as follows:

$$V_{Rd,max} = 2.3\frac{\alpha_{cc}}{\gamma_c}b_w z f_{ck,UHPFRC}^{2/3}tan\theta = 2.3\frac{0.85}{1.5} * 300 * 1935 * 140^{2/3} * \tan(30) = 11177kN \quad (6)$$

5. Conclusion

The aim of this study was to demonstrate a working example on the design calculation of a 38m long post-tensioned UHPFRC composite deck bridge using DURA® Grade 140 in accordance with the new French Standard. The traffic loading used was according to the specification of the bridge design code BS5400 [5]. General information of the chosen road bridge project and detail of the UHPFRC U-girder were presented. Furthermore, the construction stages of the UHPFRC composite bridge were briefly explained and demonstrated in this paper. Finally, the design moment effect (M_{Ed}) and design shear force effect (V_{Ed}) were calculated and compared to the design moment resistance (M_{Rd}) and design shear resistance (V_{Rd}) to show the UHPFRC bridge has adequate resistance to withstand during full service load.

6. References

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