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Chapter 20

Design of Prestressed UHPFRC Girder Bridges According to Canadian Highway Bridge Design Code

20.1. Introduction

The construction of new bridges and the renewal of aging highway bridges using ultra high performance fiber-reinforced (UHPFRC) concrete can lead to the construction of structurally-efficient long life bridges that will require minimum maintenance resulting in low lifecycle costs. UHPFRC is a newly developed concrete material that provides very high strength and very low permeability. UHPFRC could enable major improvements over ordinary concrete (OC) and high-performance concrete (HPC) bridges in terms of structural efficiency, durability and cost-effectiveness over the long term.

A slab on precast girders is one the most common forms of structural systems used for the construction of highway bridges in North America due to their good long-term performance and cost-effectiveness [LOU 93, LOU 97]. Over the past century and throughout a progressive improvement process, the precast/prestressed industry has standardized the girder sections for use with conventional concrete and HPC. The use of standard precast/prestressed girder sections is a popular and cost-effective choice for the construction and replacement of short- and medium-span bridges, as well as the construction of long-span bridges using segmental construction and splicing girders by post-tensioning [LOU 97].

Throughout the last four decades there was a considerable growth in the use of high strength concrete/HPC (HSC/HPC) in highway bridges. With a compressive strength up to 85 MPa and tensile strength up to 3 MPa, the benefits of using HPC/HSC to extend the span length capability or reduce the weight of slab-on-precast girder bridge systems reach their limit at about 50 MPa. Beyond this there are only marginal improvements, as the governing design criterion is the condition of no cracking at service [LOU 97]. The development of UHPFRC represents a major innovation in the concrete construction industry that can help overcome some of the shortcomings of OC and HPC/HSC, such as strengths that are at least twice the compressive and tensile strengths of HPC/HSC, and permeability to chlorides that is orders of magnitude lower than that of HPC/HSC. The use of UHPFRC in slab-on-girder bridges could lead to a considerable reduction in the number of girders and girder size, and could enable the construction of long-life bridges.

As of now, several bridges have been designed and built using UHPFRC in Europe:

- four highway bridges in France [HAJ 04, HAN 06, RES 06, MAT 08];
- the Kassel pedestrian bridge in Germany [MED 03];
- a pedestrian bridge in Seoul, Korea;
- several highway bridges and pedestrian bridges in Australia and New Zealand;
- a highway bridge and a pedestrian bridge in Canada;
- several highway and pedestrian bridges were opened to traffic recently in Japan.

There is a need for comprehensive structural evaluation of the behavior of such bridge systems, as well as the development of design methodologies for this type of construction. The first UHPFRC road bridge, which is a highway overpass bridge [HAJ 04], was designed and constructed in France and opened to traffic in 2001 with two simply-supported spans of 22 m. At the same time, another UHPFRC bridge was constructed in Italy, with a span of 11.8 m. More recently, a 33.8 m-span UHPFRC bridge was designed and constructed in Iowa and opened to traffic in late 2005 [MED 03], and a 47 m single-span road bridge was built in France in 2005 [RES 06].

The only available design guidelines for UHPFRC structures are the French interim recommendations [AFG 02], which provide modifications to the existing French design standards for reinforced and prestressed concrete structures. The recent draft recommendation of the Japan Society of Civil Engineers for the design and construction of UHPFRC structures [JAP 06] proposed some modifications compared to the earlier French recommendations. Neither recommendations are

developed specifically for highway bridge structures. They are not necessarily compatible with the Canadian bridge design code safety requirements and traffic load models. Hence there is an urgent need to develop a procedure for the design of UHPFRC bridges according to the Canadian Highway Bridge Design Code [CAN 06] and using the available standard Canadian Prestressed Concrete Institute [CAN 96] precast/prestressed I-girder sections. An iterative analysis and design procedure was proposed for concrete slab on precast/prestressed UHPFRC girders [ALM 07] and a preliminary evaluation of the structural performance of this type of bridge and comparison to typical OC bridges was carried out [ALM 08].

The objectives of this chapter are twofold:

- (i) propose a simplified design approach for slab on ultra high performance concrete UHPFRC girder bridges; and
- (ii) compare its structural efficiency to a conventional concrete slab on OC girder bridges.

20.2. Mechanical properties of UHPFRC

Based on advances in nanotechnology, UHPFRC was developed to achieve very high durability through ultra high dense packing by means of refined mix-design involving a minimum water/cement ratio ($w/c < 0.2$), high percentages of cement and silica-fume, fine sand and no coarse aggregates [ULM 08]. UHPFRC is reinforced at the microlevel by means of uniformly distributed short fibers with a percentage that can vary from 2–12% (by volume of concrete). The fiber length is either constant or variable, ranging from 1–20 mm. Depending on the fibers' length and volumetric proportion, there are three major types of UHPFRC:

- UHPFRC with high proportions of short fibers, introduced in Denmark in 1987 [ROS 08];
- UHPFRC with an intermediate proportion of long fibers, introduced in France 1995; and
- UHPFRC with very high proportion of fibers of various lengths, introduced in France 2000 [PAR 07, ROS 08].

The fiber-reinforcement of UHPFRC, heat treatment and the material's high homogeneity (due to the use of very fine aggregate only) contribute to eliminate the initiation of extensive early age cracks that are the major disadvantage of HSC/HPC. The superior macrolevel mechanical properties of UHPFRC, such as very high compressive and tensile strengths, high modulus of elasticity, high ductility and high fatigue strength, could enable the development of lighter bridge superstructures that would reduce the number of girders and/or support longer spans than conventional

HPC/HSC. The compressive strength of UHPFRC can vary in a very wide range from 120–400 MPa, its direct tensile strength can vary from 8–30 MPa, and its modulus of elasticity is in the range of 60–100 GPa [ACK 04, BUI 04]. Typical stress-strain relationships and tensile (bending) stress versus displacement of UHPFRC are compared to those of a typical HPC in Figures 20.1a and b, respectively. Figure 20.1a also shows the conservative elastoplastic approximation of compressive behavior of UHPFRC that is assumed in design.

a) Stress-strain relationships

b) Flexural stress-displacement

Figure 20.1. Mechanical properties of UHPFRC and OC

The uniform distribution of the fibers in the UHPFRC matrix is hard to achieve and different fiber orientations are observed in practice due to the casting process, element size, geometry, thickness and distribution of non-prestressed and

prestressed reinforcement throughout the element length [PAN 08, SCH 08, KIM 08]. Since the fiber alignment could result in local or overall anisotropy in UHPFRC, the mechanical properties are affected locally or throughout the entire structural element, depending on the affected region, location and size. The material properties could be improved in some directions and lowered in other directions, which could result in weak regions that would develop cracks under tensile stresses or fail under compressive stresses well below the anticipated levels. Furthermore, the anisotropic behavior adds more complications in the structural analysis of UHPFRC systems. Despite this, in some situations the anisotropy could improve the overall performance of the structural element if it is properly accounted for in the structural design. To simplify the analysis and design procedure and given the lack of comprehensive experimental data on UHPFRC behavior, it is generally recommended to use a reduction factor applied to the homogenized properties from the standard material tests as in Association Française de Genie Civil, (AFGC) [AFG 02] and Japan Society of Civil Engineers [JAP 06].

General design recommendations for the use of UHPFRC in reinforced and prestressed concrete structures were first developed in France by AFGC [AFG 02], followed by the Japanese recommendations [JAP 06], and a new set of design recommendations for UHPFRC structures has also been released. The *fédération internationale du béton (fib)* is also developing a first set of design rules for UHPFRC [WAL 08]. However, no recommendations have yet been developed in North America. On the other hand, no specific recommendations for the use of UHPFRC in bridge design have been developed in Europe or North America.

20.3. Design approach for prestressed UHPFRC girders

20.3.1. *General*

Most existing structural concrete design codes and standards (including the bridge design codes) limit the concrete compressive strength to a maximum of 80–85 MPa. The use of UHPFRC with its very high compressive and flexural strengths is investigated in the present study, with reference the French and Japanese UHPFRC design recommendations [AFG 02] and [JAP 06], Canadian Highway Bridge Design Code (CHBDC) [CAN 06] as much as possible and uses engineering judgment. The material reduction factor at ultimate limit state (ULS) for UHPFRC must be calibrated based on a lifetime target reliability index of CHBDC [CAN 06].

A rigorous reliability-based analysis is needed to determine the appropriate reduction factor for UHPFRC to ensure lifetime reliability indices that are consistent with the requirements of the bridge design code for flexural and shear design. However, given the lack of data on strength variability and performance to failure of

RC beams, the material reduction factors are assumed conservatively in this to be the lowest of the values given by the French recommendations [AFG 02] ie CHBDC [CAN 06]. The derivation of more appropriate factors for UHPFRC ng studied.

2. Serviceability limit states

Given the lack of data on the performance of UHPFRC girders in the field, the gn of highway bridges will be based on the criteria of no crack at transfer and at iceability limit states (SLS) by keeping the tensile stresses below the cracking t for both cases, i.e. assuming fully prestressed girders. Compressive strength of t-treated UHPFRC gains almost 95% of its full strength within the first few days , long before transfer [AFG 02, JAP 06]. A conservative limit on the compression ngth at transfer of 85% of the 28-day strength is assumed in the present chapter, $f'_{ci} = 0.85f'_c$. The allowable compressive stress at transfer is taken conservatively $f_{ci} = 0.6f'_{ci}$ [CAN 06] and (Cl 6.1.12) [AFG 02]. Using both the CHBDC [CAN] and AFGC [AFG 02] recommendations, the allowable tensile strength at nsfer, f_{cti} , is assumed equal to:

$$f_{cti} = 0.4\sqrt{f'_{ci}} \quad [20.1]$$

The allowable compressive stress f_c at SLS is equal to 60% of the UHPFRC ompressive strength (see Figure 20.2), i.e.:

$$f_{ci} = 0.6f'_c \quad [20.2]$$

The maximum tensile strength is equal to the first crack strength, f_{cr} . Linear elastic analysis is to be carried out at SLS with the assumption that plane sections remain plane and stresses are linearly proportional to strains. The static deflection of the bridge under gravity loads and the maximum deflection for superstructure vibration should satisfy the bridge design code limits.

Although the CHBDC [CAN 06] and AFGC [AFG 02] recommendations allow tensile stresses that exceed the cracking limit at SLS, the tensile stress is limited to the cracking limit in this chapter. Hence, the stiffness calculations at SLS involve the entire cross-sectional area. Using Cl 6.1.11 of the French recommendations [AFG 02], the allowable tensile strength of UHPFRC at the serviceability limit state is given by:

$$f_t = \frac{\sigma(w_{0.3})}{K} \quad [20.3a]$$

where f_t is the allowable tensile stress, $\sigma(w_{0.3})$ is the tensile stress corresponding to a crack width of 0.3 mm (see Figure 20.2) and represents the basis for fiber tensile strength, and $1/K$ is an orientation coefficient that accounts for the actual variability in fiber orientation due to placement. K is equal to 1.25 for all loading other than local effects (which is used in this chapter), and equal to 1.75 for local load effects.

Figure 20.2. Assumed tensile and compressive behavior of UHPFRC for the design

Expressing the allowable tensile strength in terms of $\sqrt{f'_c}$ (to be consistent with the CHBDC [CAN 06]) and substituting this in equation [20.3a], the allowable tensile strength at serviceability limit state is equal to:

$$f_t = 0.4 \sqrt{f'_c} \quad [20.3b]$$

This value will be the design cracking limit at SLS, which is much lower than the direct tension test result of UHPFRC (see Figure 20.2).

20.3.3. Proposed approach for flexural design of UHPFRC girders at ULS

At the ULS, a bi-linear stress-strain relationship is assumed as shown in Figure 20.1a that includes:

(i) a first line from zero stress and zero strain up to strength of f_{cu} and a strain of (f_{cu} / E_{uhpfc}) ; and

(ii) a second line that is horizontal up to the ultimate strain of $\epsilon_{cu} = 0.003$.

The ultimate strength, f_{cu} is given as [AFG 02] and adapting it to the CHBDC [CAN 06] yields:

$$f_{cu} = \frac{0.85\phi_c f_{cj}}{\theta} \quad [20.4a]$$

where:

– f_{cj} is the cylinder compressive strength at age j , which is taken in the present study equal to 28 days;

– θ is a factor related to the probability of the load application period or rate of loading $\theta = 1.0$ for loads with application period equal or exceeds 24 hours. It is $\theta = 0.9$ for loads applied over a period between one hour and 24 hours, and $\theta = 0.85$ for the loads with period of application of less than one hour;

– ϕ_c is a coefficient that takes into account the variability of UHPFRC resistance as well as localized effects.

The UHPFRC resistance factor is $\phi_c = 1/\gamma_b$, which is equal to $\phi_c = 0.75$ for load combinations 1 and 2 and $\phi_c = 0.95$ for exceptional combinations, as given in Cl 6.2.1 [AFG 02]. Substituting the factor values in equation [20.4a] conservatively yields:

$$f_{cu} = 0.65 f'_c. \quad [20.4b]$$

For a conservative estimation of the flexural strength of pre-tensioned UHPFRC elements, the effect of tension stiffening of the fiber-reinforced UHPFRC is ignored. At ULS, the factored bending moment and shear force should be less than or equal to the factored flexural strength and shear strength, respectively. A ductile failure at ULS should be ensured so that the tensile stresses in the prestressing steel are kept below the factored ultimate tensile strength (i.e. $0.95 f_{pu}$) by ensuring that the prestressing steel tensile strain at failure is beyond the yield strain. In CHBDC [CAN 06], this is ensured by checking that the relative neutral axial depth (c/d_p) is < 0.5 , where c is the distance from the extreme compression fiber to neutral axis and d_p is the distance from the extreme compression fiber to the centroid of the tendons, as shown in Figure 20.3.

Assume that the slab acts compositely with the girders through a section formed from a concrete slab of t_{slab} thickness and UHPFRC prestressed girder of H_s depth (see Figures 20.3a and b). The equivalent slab width is equal to the effective slab

width [CAN 06] divided by the modulus of elasticity ratio (E_{girder}/E_{slab}). Assuming a linear strain distribution over the composite section depth and a bilinear stress distribution for UHPFRC under compression, the tensile resistance of UHPFRC under tension is ignored. From Figure 20.3, equilibrium conditions dictate that $C_c = T_p$, where $C_c = \sum C_{ci}$ is the sum of all the internal compressive force components on the cross-section, and T_p is the tensile force in the prestressing steel at ULS.

Three major cases (A–C) can be identified as follows, including sub-cases A-1 to A-3, B-1 and B-2, and C:

Figure 20.3. Strain and stress distribution at ULS of flexural action in UHPFRC girders

20.3.3.1. *Case A: neutral axis in the girder web - $c \geq h_{ft} + t_{slab}$*

As shown in Figure 20.3a, the depth of the fully-stressed compressive zone, or the zone under constant compressive stress of (f_{cgu}), is given by:

$$r_{fu} = \left(\frac{\epsilon_{cu} - \epsilon_{bc}}{\epsilon_{cu}} \right) \times c \quad [20.5]$$

Comparing r_{fu} with t_{slab} and $t_{slab} + h_{ft}$, three secondary cases can be considered, as follows in the sections below.

20.3.3.2. *Case A-1: depth of fully stressed zone - $r_{fu} \geq t_{slab} + h_{ft}$*

The compressive force component in the slab is equal to:

$$C_{c1} = b_e f_{cgu} t_{slab} \quad [20.6a]$$

and the resulting compressive force component in the top flange is given by:

$$C_{c2} = b_{ft} f_{cgu} h_{ft} \quad [20.6b]$$

The compressive force component in the web is given by:

$$C_{c3} = b_w \left[\left\{ r_{fu} - (t_{slab} + h_{ft}) \right\} f_{cgu} + r_{fo} \left(\frac{f_{cgu}}{2} \right) \right] \quad [20.6c]$$

20.3.3.3. *Case A-2: depth of fully stressed zone - $t_{slab} \leq r_{fu} \leq t_{slab} + h_{ft}$*

In this case, the slab is under maximum compressive stress, f_{cgu} , while only a portion of the top flange with area equal to $[(r_{fu} - t_{slab}) \times b_{ft}]$ is under the maximum compressive stress, f_{cgu} . The rest of the top flange is under stress that varies linearly from f_{cgu} to $f = f_{cgu} \times (r_{fo} - h_{cw})/r_{fo}$, where $h_{cw} = t_{slab} + h_{ft} - r_{fu}$. The compressive zone of the web and the added strip of the flange are under linearly varying stress from f_{cgu} to zero at the neutral axis. The compressive force component in the slab is equal to:

$$C_{c1} = b_e f_{cgu} t_{slab} \quad [20.7a]$$

$$C_{c2} = b_{ft} f_{cgu} (r_{fu} - t_{slab}) + (b_{ft} - b_w) h_{cw} \left(\frac{f + f_{cgu}}{2} \right) \quad [20.7b]$$

and the web component (plus the added strip) is equal to:

$$C_{c3} = b_w (c - r_{fu}) \left(\frac{f_{cgu}}{2} \right) \quad [20.7c]$$

20.3.3.4. *Case A-3: depth of fully stressed zone - $r_{fu} \leq t_{slab}$*

$$f_2 = f_{cgu} \times \frac{h_{cw2}}{r_{fo}} \quad [20.8a]$$

The compressive force component in the fully and partially stressed zones of the slab is equal to:

$$C_{c1} = b_e f_{cgu} r_{fu} + b_e h_{cwl} \left(\frac{f_{cgu} + f_1}{2} \right) \quad [20.8b]$$

The compressive force component in the flange is equal to:

$$C_{c2} = b_f h_{ft} \left(\frac{f_1 + f_2}{2} \right) \quad [20.8c]$$

The compressive force component in the web is equal to:

$$C_{c3} = b_w (c - t_{slab} - h_{ft}) \left(\frac{f_2}{2} \right) \quad [20.8d]$$

20.3.3.5. *Case B: neutral axis in top flange of girder, i.e. $t_{slab} \leq c \leq t_{slab} + h_{ft}$ (as shown in Figure 20.3b)*

$$r_{fo} = \frac{\epsilon_{bc}}{\epsilon_{cu}} \times c \quad [20.9a]$$

$$\epsilon_{bu} = \frac{f_{cgu}}{E_{cg}} \quad [20.9b]$$

$$r_{fu} = \left(\frac{\epsilon_{cu} - \epsilon_{bc}}{\epsilon_{cu}} \right) \times c \quad [20.9c]$$

Two secondary cases can be derived as follows in sections 20.3.3.6 and 20.3.3.7.

20.3.3.6. *Case B-1: $t_{slab} \leq r_{fu} \leq t_{slab} + h_{ft}$*

The slab is under stress equal to f_{cgu} . The top flange is divided into two zones: a compressive zone under a stress varying from f_{cgu} to zero and a possible fully stressed portion, and a tensile stress zone over the remaining top flange depth, i.e. $(t_{slab} + h_{ft}) - c$. The compressive component from integrating the stresses in the slab is given by:

$$C_{c1} = b_e f_{cgu} t_{slab} \quad [20.10a]$$

The compressive force component in the flange is given by:

$$C_{c2} = b_{ft} (r_{fu} - t_{slab}) f_{cgu} + b_{ft} (c - r_{fu}) \left(\frac{f_{cgu}}{2} \right) \quad [20.10b]$$

20.3.3.7. *Case B-2: $r_{fu} < t_{slab}$*

A portion of the slab is under stress equal to f_{cgu} , while the other part is under a linearly varying stress from f_{cgu} to $f = f_{cgu} \times (r_{fo} - h_{cw}) / r_{fo}$ at the lower surface of the slab, where $h_{cw} = t_{slab} - r_{fu}$. The top flanges' compressive stress zone is under stress varying from f to zero. The compressive force component in the slab is given by:

$$C_{c1} = b_e f_{cgu} r_{fu} + b_e (t_{slab} - r_{fu}) \left(\frac{f_{cgu} + f}{2} \right) \quad [20.11a]$$

The compressive force component in the flange is:

$$C_{c2} = b_{ft} (c - t_{slab}) \left(\frac{f}{2} \right) \quad [20.11b]$$

20.3.3.8. *Case C: neutral axis is located in the slab, i.e. $c \leq t_{slab}$*

Then the slab is divided into two zones, a compressive stress zone under stress will vary from f_{cgu} to zero and a tensile stress zone. The entire girder will be under tensile stress. The compressive force component of the fully stressed zone is given by:

$$C_{c1} = b_e f_{cgu} r_{fu} \quad [20.12a]$$

while the compressive force component of the linearly varying stress zone is given by:

$$C_{c2} = b_e r_{fo} \left(\frac{f_{cgu}}{2} \right) \quad [20.12b]$$

For any of the cases – A, B or C shown above – the resultant compressive force is given by:

$$C_c = \sum_{i=1}^m C_{ci} \quad [20.13]$$

where m is the number of compressive force components according to the suitable case given above. The location of the resultant C_c from the top compression fiber of the composite section is given by:

$$a_c = \frac{\sum_{i=1}^m C_{ci} \times a_{ci}}{\sum_{i=1}^m C_{ci}} \quad [20.14]$$

where a_{ci} is the location of force component, C_{ci} , from the top compression fiber of the composite section.

The centroid of the prestressing steel tendons is then calculated for the critical flexural sections. Through a trial-and-error process the equilibrium of the internal forces is satisfied ($C_c = T$). Hence, the flexural resistance of UHPFRC is given by:

$$M_r = C_c (d_p - a_c) \quad [20.15]$$

where d_p is the depth of the prestressed steel from the top compression fiber.

Two limits for the moment resistance are to be checked, which are:

- $M_r \geq 1.2 M_{cr}$ for minimum reinforcement, where M_{cr} is the cracking moment; and
- $c/d_p < 0.5$ for maximum reinforcement to ensure ductile failure [CAN 06].

20.3.4. Shear design of UHPFRC girders at the ULS

It is essential to simulate the load-carrying mechanism in one expression rather than dividing it into components, which are not representing the real UHPFRC shear failure mechanism. After UHPFRC cracks, the randomly distributed fibers provide most of the shear capacity. However, there is a lack of knowledge and data at the present time on how to include the effect of the fiber-reinforcement into such an expression of shear strength.

A simplified approach is used to estimate the shear strength of UHPFRC (with at least 2% volumetric fiber content), which is based on the proposed AFGC [AFG 02] and JSCE [JAP 06] models. The shear resistance of UHPFRC or the shear load-carrying mechanism is hypothetically divided it into three components:

- (i) the first component represents the composite action of the matrix and fiber;
- (ii) the second component represents the shear capacity provided by the average fiber tensile resistance (before fiber pull-out) acting along the diagonal cracks; and
- (iii) the third component is shear capacity provided by stirrups and prestressing.

The overall shear resistance is given by:

$$V_r = (V_c + V_f) + V_p + V_s \quad [20.16]$$

The limited experimental results available [USD 06] are in agreement with AFGC [AFG 02] estimation; however, more research is needed to assess the shear resistance of UHPFRC.

$$V_c = \frac{0.24\phi_c}{\gamma_E} \sqrt{f_{cj}} b_0 z \quad [20.17]$$

The coefficient γ_E characterizes the current uncertainty regarding the possibility of extrapolating to UHPFRC the design equations established for HPC for which $f'_c \leq 85$ MPa and is taken equal to 1.15 for CHBDC ultimate limit states 1 and 2. f_{cj} is the concrete compressive strength at j days (or 28 days in the present chapter). b_0 and z are the effective web shear width and depth, respectively. The concrete contribution to shear strength is therefore $V_c = 0.16\sqrt{f'_c} \cdot b_0 z$, and the fiber contribution, V_f is given by:

$$V_f = \frac{S\sigma_p}{\gamma_{bf} \tan\beta_u} \quad [20.18]$$

where σ_p is the residual tensile strength, which is given by:

$$\sigma_p = \frac{1}{K} \frac{1}{w_{lim}} \int_0^{w_{lim}} \sigma(w) dw \quad [20.19]$$

where:

- $w_{lim} = \max(w_u, 0.3 \text{ mm})$ where $w_u = l_c \cdot \epsilon_u$, ϵ_u is the ultimate strain of 0.003;
- $l_c = \frac{2}{3}h$, where h is the total height of the section;
- $\sigma(w)$ is the experimental characteristic post-cracking stress corresponding to a crack width w ;
- w_u is the ultimate crack width;
- S is the area of the fiber effect, $S = 0.9 b_0 d_p$ or $S = 0.9 b_0 z$;
- K is the fiber orientation coefficient for general effect ($K = 1.25$);
- $\gamma_{bf} = 1.3$ and β_u are the angle of the compression struts and lower-bounded to 30° as per Cl 7.3.3 [AFG 02].

The shear reinforcement contribution, V_s , is calculated in the same way as for OC girders following CHBDC [CAN 06]. It is important to mention here that the French Recommendations [AFG 02] allow the use of shear reinforcement with UHPFRC structural elements in a similar manner to conventional concrete elements where $f'_c \leq 80$ MPa. On the other hand, the Japanese recommendations [JAP 06] provide more restrictions on the use of non-prestressed reinforcements with UHPFRC, as the use of reinforcement with UHPFRC would result in disturbance of fiber orientation that would cause cracks. Some cracks may also develop due to the internal constraints of UHPFRC due to shrinkage unless the effects of the constraint are properly evaluated and measures to prevent cracks are properly taken into account [JAP 06].

20.4. Illustrative example – slab on prestressed UHPFRC girders bridge

20.4.1. Proposed iterative design procedure

The iterative design procedure proposed for a prestressed UHPFRC bridge is illustrated in Figure 20.4. The procedure involves two steps:

- (i) a simplified semi-analytical approach to obtain the preliminary feasible superstructure design; and then
- (ii) a refined finite element-based analysis is carried out to check that the preliminary design is acceptable and if modifications are required. The refined analysis generates a detailed stress distribution in all girders that enable us to identify the zones of maximum stresses and to optimize the girder section and prestressing steel area and layout.

In order to verify the structural efficiency of UHPFRC bridge girders, a comparative study is conducted between two girder bridges. One bridge system has a conventional concrete slab compositely integrated with OC girders. A slab of the same geometrical and material properties is compositely integrated with UHPFRC girders in the second system. Both bridges are designed to have the same capacity, i.e. support the same traffic load and superimposed dead loads. The major parameters in this comparison are the number of girders (or the girder spacing); the girder size; stresses at SLS; the girder deformations under service loads; and the ultimate bridge load capacity. Other parameters – such as slab thickness, span length, number of lanes, traffic speed, prestressing system pattern and boundary conditions – are assumed to be the same for both bridges. The traffic load and bridge design complies with all serviceability limit states (SLS) and ULS requirements of the CHBDC [CAN 06], however the material reduction factors are as shown earlier. The bridge girders are designed for no crack at transfer and serviceability limit states.

Figure 20.4. Design procedure for UHPFRC and OC bridges

The bridge center-to-center span is 25 m and the total width of the deck including the barrier walls is 11.6 m. The slab thickness for both bridges is 175 mm. Two types of live loads are applied on the deck surface: (i) a lane loading; and (ii) a single moving truck. For multi-lane loading, modification factors of 0.9 and 0.8 are applied to the two lanes and three lanes, respectively.

Low-relaxation, seven-wire strands, Grade 1,860, with a nominal diameter of 12.7 mm, nominal area of 98.7 mm^2 and tensile strength (f_{pu}) of 1,860 MPa are used. CHBDC [CAN 06] limits the minimum effective stress in tendons to $0.45 f_{pu}$; the maximum stress at jacking to $0.78 f_{pu}$; the maximum tensile stress at transfer to $0.74 f_{pu}$; and the maximum stress at ULS to $0.95 f_{pu}$. The total prestress losses are estimated to be 17% of the ultimate strength. The tendons for the OC and UHPFRC girders are arranged in straight and conventional deflected strand-pattern groups. The straight tendons provide 50–60% of the total prestressing steel area, depending on the maximum stresses in the girder. There was no need to debond the strands near the supports as the tensile stresses remained below the allowable values for both UHPFRC and OC bridges.

The material properties of the OC bridge are selected to match the properties of similar existing bridges. The compressive strength of the slab concrete is taken as $f'_{cs} = 30 \text{ MPa}$. For OC girders, $f'_{cg} = 40 \text{ MPa}$ and at transfer $f'_{cgt} = 30 \text{ MPa}$. Five CPCI 1400 (see Figure 20.5) are required for the OC bridge with a girder spacing of 2.5 m. It is found that the use of 20 straight tendons at the bottom span and 10 tendons linearly deflected on one-third of the girder span are sufficient. The centroid of the straight tendons is 100 mm from the bottom fiber while the centroid of the deflected tendon in the girder ends is 850 mm and in the middle third is 110 mm.

The allowable compressive stress at transfer is $0.6 f'_{cgt}$ (or 18 MPa) and the allowable tensile stress at transfer is $0.2 \sqrt{f'_{cgt}}$ (or 1.1 MPa). At SLS, the allowable compressive stress is $0.6 f'_{cg}$ (or 24 MPa) and the first crack strength is $f_{cr} = 0.4 \sqrt{f'_{cg}}$ (or 2.53 MPa).

The OC bridge design is governed by the tensile strength or cracking limit at SLS. Short- and long-term static deflections are very low and the vibration of the bridge superstructure satisfies CHBDC [CAN 06] requirements. For the ULS, the reduced flexural capacity of the girder is 7,300 kNm, which is higher than the factored moment (5,440 kNm) and much higher than the cracking moment (3,100 kNm). The girder design is checked for ductile failure as the moment resistance is developed with $c/d_p = 0.074$, far less than 0.5. The shear reinforcement at the critical shear section is provided by 10M stirrups at 125 mm spacing.

Figure 20.5. Comparison of UHPFRC and OC precast prestressed girder bridges

For the UHPFRC bridge, the compressive strength of the deck slab is assumed to be $f'_{cs} = 30$ MPa. The compressive strength of the UHPFRC of the girders is $f'_{cg} = 175$ MPa and at transfer $f'_{cgi} = 0.85 f'_{cg}$ (or 149 MPa). Three CPCI 900 girders (see Figure 20.5) are found to be sufficient, with a spacing of 4.0 m.

In order to use a small girder size and maximize the benefits of the very high strength of UHPFRC both in compression and tension, the optimum prestressing pattern is found when the ratio of deflected tendons to straight tendons is between 0.8 and 1. In the present example, 36 straight tendons and 30 deflected tendons are sufficient. The centroid of the straight tendons is 90 mm from the bottom fiber, while the centroid of the deflected tendons at the girder ends is 600 mm and in the middle third is 220 mm. The allowable compressive stress at transfer is $0.6f'_{cgi}$ or (87 MPa) and the allowable tensile stress at transfer is 4.88 MPa. At SLS, the allowable compressive stress is 105 MPa and the cracking strength is 5.3 MPa.

Similar to the OC girder, the design of UHPFRC girders is also governed by the cracking limit at SLS. Short- and long-term static deflections are within the acceptable limits and the vibration of the bridge superstructure satisfies CHBDC [CAN 06] requirements. For the ULS, the reduced flexural capacity of the UHPFRC girder is found to be higher (10,110 kNm) than the factored moment (8,260 kNm) and is much higher than the cracking moment (3,470 kNm).

The girder design is checked for ductile failure as the moment resistance is developed with $c/d = 0.085$ being far less than 0.5. The shear reinforcement at the critical shear section is provided by 10 M stirrups at 100 mm spacing.

20.4.2. *Finite element modeling of a UHPFRC bridge*

A linear elastic 3D finite element model (FEM) is developed to determine the stress distribution in all girders that make up the two investigated bridges. This 3D FEM model enables more accurate predictions of the stresses in all girders than the simplified analysis approach of the CHBDC [CAN 06]. Both the deck slab and girders are modeled using shell elements, while the prestressing tendons are modeled using cable elements. The prestressing losses, deformations and relaxation are accounted for in the model.

The FEM enables us to predict the stresses in every girder of the bridge and then optimize the prestressing steel area and profile for better stress distributions. The FEM results indicate that the maximum stresses are found in the central girders for both OC and UHPFRC in the case of two-lane loading. On the other hand, the maximum stresses are found in the external girders in the case of three-lane loading. In general, the results show that the maximum stresses for the three-lane loading case are less critical than those of the two-lane loading case. It also shows that the most critical girders are the central girder (G3) for the OC bridge and the internal girders (G2 or G3) for the UHPFRC bridge, as shown in Figure 20.5. The required area of prestressing steel from the FEM is found to be 10–15% lower than the value obtained by simplified analysis method, and the deflected strands profile is slightly different. The results of the finite element model also show that the compressive stresses at ULS in the top fibers at midspan and bottom fiber of the support span of the critical girders identified above are well below the USLs for both OC and UHPFRC girders.

20.4.3. *Comparison of structural efficiency of UHPFRC and OC bridges*

The use of UHPFRC enables a considerable reduction in the volume of concrete by up to two-thirds when compared to OC. The number of girders is reduced from five to three and the size of the girder is also reduced from CPCI 1400 to the minimum provided size CPCI 900. The weights of the girders per unit deck area are 427 kg/m^2 for the OC bridge and 158 kg/m^2 for the UHPFRC bridge. The total weights per unit area of the superstructure, including the deck slab, are 847 kg/m^2 for the OC bridge and 578 kg/m^2 for the UHPFRC bridge. Consequently, the use of UHPFRC results in a 32% reduction in the total weight of the superstructure and 63% reduction in the girders' weight. If the cement used for the OC girders bridge is assumed equal to 380 kg/m^3 and for UHPFRC is $1,114 \text{ kg/m}^3$ [HAJ 04], then there is a 7% reduction in the total cement needed to cast five CPCI 1400 OC girders compared to the three CPCI 900 UHPFRC girders. The reduced weight of UHPFRC superstructure will lead to a reduction in the size of the substructure.

It is clear that a reduction in the weight of the superstructure will lead to a reduced size of the substructure (piers and abutments) and foundations, and reduced overall cost of the bridge. Furthermore, a reduction in concrete consumption will have considerable environmental benefits through the reduction of energy consumption and greenhouse gas emission associated with the production of cement, extraction and transportation of raw materials to the construction site [LOU 07].

20.5. Conclusions

A simplified design approach of a concrete slab on UHPFRC girders bridge was proposed. CHBDC guidelines were used, the current recommendations for designing UHPFRC elements were investigated, and a simplified flexural design approach has been developed for prestressed concrete beams. A simple and conservative sectional analysis procedure is proposed that is similar to that of the CHBDC [CAN 06], using a bilinear stress-strain model for UHPFRC under compression and ignoring the UHPFRC contribution to the tensile strength.

For the investigated cases of two- and three-lane bridge 25 m-long, it is found that UHPFRC in precast/prestressed concrete girders yields a considerable reduction in the number of girders and girder size when compared to a conventional OC girder bridge, and hence results in a significant reduction in concrete volume and a slight reduction in cement consumption. This weight reduction leads to a more efficient design of the superstructure that reduces the weight on the substructure, which is very important for the safety of aging bridge substructures.

Further experimental work is needed to validate the proposed flexural design approach as well as shear strength of UHPFRC. Furthermore, the shape of girder sections can be improved for optimum use of the UHPFRC material.

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