OPTIMAL DESIGN OF UHPC HIGHWAY BRIDGES BASED ON CRACK CRITERIA

CONCEPTION OPTIMISEE DE PONTS ROUTIERS EN BFUP SUR LA BASE D'UN CRITERE DE LIMITE DE FISSURATION

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ABSTRACT – Industrial implementations of Ultra High Performance Concrete (UHPC) require specific design guidelines to fully exploit its extraordinary properties, such as the ultra high strength and the quasi-ductile tensile behaviour. Nowadays, UHPC has been employed in several worldwide pilot projects and shows promising potential for designing more durable bridge structures.

This paper presents an optimization method to design a cross-section (i.e., box, double-tee and girder) of medium span bridges based on crack opening criteria. We propose an optimization design problem that maximizes the characteristic cross-section resistance capacities up to the design loads for some design limit states. In particular, the classical section capacity approach is adapted for both maximum crack opening criteria and the material non linearity. Finally, we account for the beneficial effect of the 3D material behaviour by a two-phase brittle fracture-plasticity model specifically developed for UHPC materials.

RÉSUMÉ **-** L'application industrielle des Bétons fibrés ultra-performants (BFUP) nécessite d'accumuler une expérience croissante dans l'utilisation de règles de conception appropriées permettant de valoriser leurs propriétés hors du commun : ultra-haute résistance et comportement quasi-ductile en traction. Dès à présent, les BUFP ont été employés de par le monde dans plusieurs projets innovants, et présentent un potentiel prometteur pour concevoir des structures de ponts plus durables.

Cet article présente une méthode d'optimisation pour concevoir la section transversale d'un pont de moyenne portée (caisson, double-T ou multi-poutres) sur la base d'un critère de limitation de l'ouverture de fissure. Un problème d'optimisation est posé pour maximiser les capacités caractéristiques de résistance de la section, jusqu'à atteindre les efforts sollicitants, sous les états limites considérés. L'approche classique de calcul des sections est ainsi adaptée en tenant compte de la fissuration maximale admise et de la non-linéarité matérielle. Enfin, il est tenu compte des effets tridimensionnels bénéfiques du comportement du matériau grâce à un modèle biphasique rupture fragile – plasticité mis au point spécifiquement pour les BFUP.

1. Introduction

The combination of steel fibres with a dense cementitious matrix endows Ultra-High-Performance Concrete (UHPC) with an extraordinary compressive strength (fc ~ 150 to 220 MPa) and the ability to delay crack localization. Moreover, the quasi-ductile tensile behaviour can be taken into account when designing UHPC structures, avoiding, to some extent, the need for passive reinforcement.

Figure 1 shows some recent applications of UHPC in bridge engineering. The first UHPC highway bridge in the United States was completed in May 2006 in Iowa's Wapello County: a single span bridge with three 33 m girders without any shear stirrups.

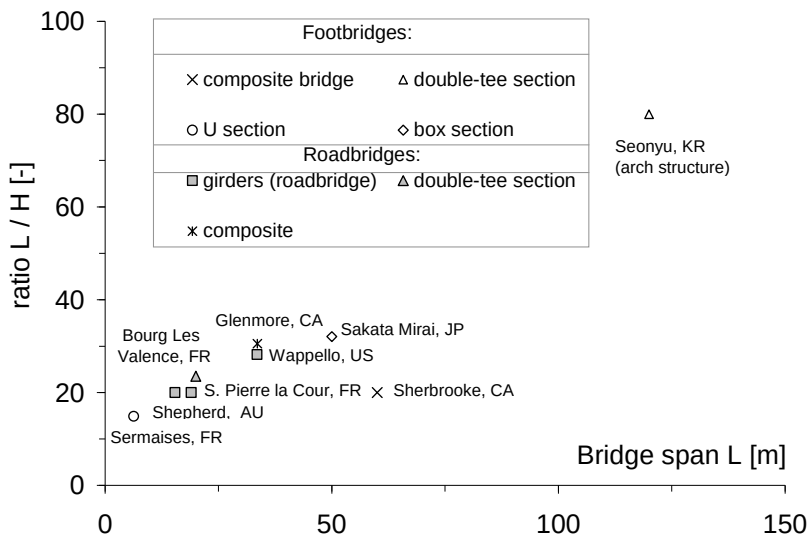


Figure 1. Some of the UHPC applications to footbridges and roadbridges.

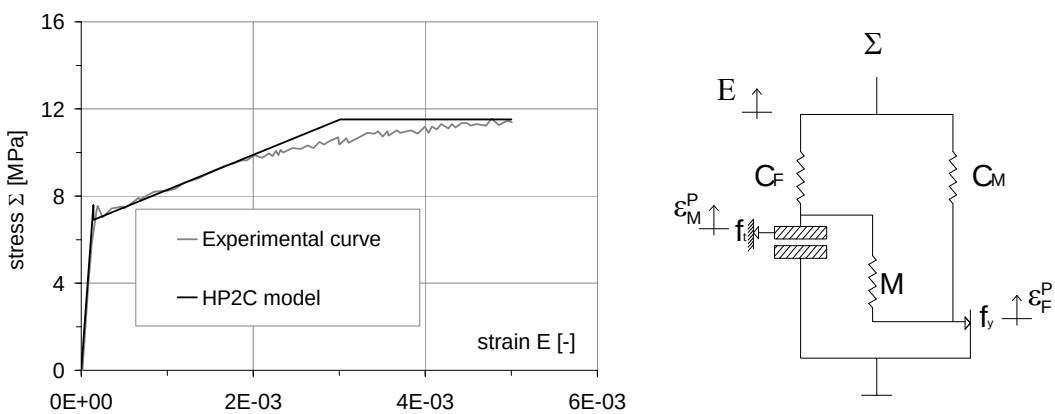


Figure 2. (left) Overall stress (Σ) and strain (E) relation in tension (Boulay et al., 1997); (right) 1D Think model for UHPC material (Chuang and Ulm, 2002).

Chuang and Ulm (2002) developed a two-phase damage-plasticity model based on the action of the composite phases that accurately predicts the non linear overall stress-strain relationship in tension (Figure 2). The rheological model consists of a brittle-plastic device (stiffness C_M and strength f_t) representing the cementitious matrix, in parallel with an elasto-

plastic device (stiffness C_F and strength f_y) representing the composite fibres. Matrix-fibre bond mechanisms (e.g., debonding and pull-out of fibres) are lumped into the coupling spring (M).

Combining UHPC with prestressing technology allows us to design more slender structures with longer service life. This is especially important for highway bridge design, given that approximately twenty-one percent of such bridges in North America are made of prestressed and precast material (Collins and Mitchell, 1991).

2. Features of appropriate UHPC design methods

Unlike normal concrete, where "no tension" criteria are often implied by codes of practice, the remarkable properties of UHPC allows one to include the strength capacity in tension, provided that the maximum cracking is controlled by crack opening criteria. This has been recognized in recent design guidelines for UHPC:

- The *Association Française de Génie Civil* (AFGC) design recommendations (2002) capitalizes on the first industrial feedback of UHPC implementation, and accounts for the UHPC tensile strength in beam structure design, while restricting the admissible crack opening for the Service State and the Ultimate Limit State; namely "zero crack" opening for the Serviceability Limit State (SLS) under frequent load combination, consistently with French code for Prestressed concrete design "class II", and the following maximum crack opening $[[w]]_{max}$ at Ultimate Limit States (ULS) for un-reinforced and reinforced or pre-stressed beam-type structures:

$$[[w]]\max = \begin{cases} \min\left(\dfrac{Lf}{4}, \dfrac{H}{100}\right) & \text{reinforced or pre-stressed structure} \\ 0.3\,\text{mm} & \text{unreinforced structure} \end{cases} \tag{1}$$

where L_f [mm] is the fibre length and H [mm] is the section depth.

- The Japanese recommendations JSCE (2005) defines a bilinear stress-crack opening relationship up to 4.3 mm crack opening based on a very ductile class of UHPC materials with polymeric fibres Uchida *et al.* (2005).

In the following, we propose a design approach for optimizing bridge cross-sections that accounts for the UHPC ultimate admissible crack opening within the context of the LRFD (2002) design method.

3. Highway UHPC bridge design optimization

3.1 Methodology

The UHPC design approach developed here adopts the LRFD method of the AASHTO standard specifications (2002) together with the maximum crack opening criteria for UHPC structures:

$$\sum_{1}^{N}\left(\alpha_i \psi_i \gamma_i Q_i\right)_j \le \left(\phi R\right)_j \qquad J = SLS, ULS \tag{2}$$

The left hand side of Eq.2 represents the factored design load, with Q_i = nominal loads (dead loads and live loads), α_i = load factors; ψ_i = load combination factors; γ_i = importance factors. The AASHTO (2002) design codes for highway bridges define the necessary design loads as the bridge's self-weight (25 kN / m^3), the future wearing surface (1.20 kPa), the traffic load of a truck (i.e., a total load of 320 kN distributed over three axles distanced by 4.27 m), and a uniform load over traffic lanes (2.60 kPa). On the right hand side of Eq.2 stands the section's characteristic resistance, where R = mean load capacity at the maximum crack opening; ϕ = design strength reduction factor. This approach is thus consistent provided the load-crack opening characteristic curve can be considered as continuously increasing up to the maximum admitted crack opening, which may be realized for UHPC under bending. The material behavior of an UHPC is described by the following non linear stress-strain uniaxial relations:

$$\begin{cases} 0 \geq \sigma = K_0 \varepsilon \geq -f_c^{'} & \text{(compression)} \\ 0 \leq \varepsilon \leq \varepsilon_{SLS}: \quad \sigma = K_0 \cdot \varepsilon & \text{(tension)} \\ \varepsilon_{SLS} \leq \varepsilon \leq \varepsilon_{ULS}: \quad \sigma = \Sigma_1^+ + K_1\left(\varepsilon - \varepsilon_{SLS}\right) \leq \Sigma_2 & \end{cases} \tag{3}$$

where $f_c^{'}$ is the UHPC compressive strength, Σ_1^+ is the strength immediately after cracking, and K_1 is the post-cracking stiffness (Figure 2b). The material parameters were calibrated from tensile tests on Ductal® notched specimen (Chuang and Ulm, 2002) as listed in Table 1.

Table 1. Material 'identity card' of Ductal®.

	Symbol	Mean Value [MPa]
Compressive strength	$f_c^{'}$	158.58
Initial stiffness	K_0	53918
Post-cracking stiffness	K_1	1606
Cracking strength	Σ_1^-	7.58
Post-cracking strength	Σ_1^+	6.90
Ductile strength	Σ_2	11.51
Fiber diameter	ϕ_f	0.2 mm
Fiber length	L_f	12.7 mm

The section design capacity, i.e. the right hand side of Eq.2, is based on the section equilibrium for a given maximum crack opening $[[w]]_{max}$ as defined by Eq.1. This method assumes the following three principles:

1 The section equilibrium in terms of force (N_R) and moment (M_R):

$$N_R = \int \sigma dA = 0; \qquad M_R = \int y \cdot \sigma dA \tag{4}$$

where σ is the longitudinal stress in the cross-section. The strand pre-stress is taken into account by considering an elasto-plastic relation:

$$\sigma_p = \gamma \cdot f_y^p + E_T \cdot \varepsilon_p \quad \leq \quad f_y^p \tag{5}$$

where γ is the pre-stress level (after pre-stress losses due to creep), f_y^p = 1,860 MPa is the yield stress, E_T = 200 GPa is the elastic stiffness, and ε_p is the tendon strain after prestressing;

2 The Navier-Bernoulli hypothesis of plane section;

3 The maximum tensile strain in the bottom flange is restricted by either the maximum tensile strength (Σ_1^-) or the maximum crack criterion of Eq.1. The section moment capacity for either SLS or ULS is evaluated by solving Eq.3 and Eq.4, and is formally written as:

$$M_R^J = F\left(\text{Materials, Section, Prestressing}\right); \quad J = SLS, ULS \tag{6}$$

The design moment capacity accounts for a factor accounting for scatter (ϕ_M) that reduces the mean moment section capacity (M_R^J) to its characteristic value:

$$\phi_M \cdot M_R^J = M_R^J - 1.75 \cdot s_M^J \tag{7}$$

where s_M^J is the standard deviation of the moment capacity. From flexural tests on Ductal® small beams, this factor was determined equal to 0.85 for both SLS and ULS (Chuang and Ulm, 2002; Chanvillard, 2003).

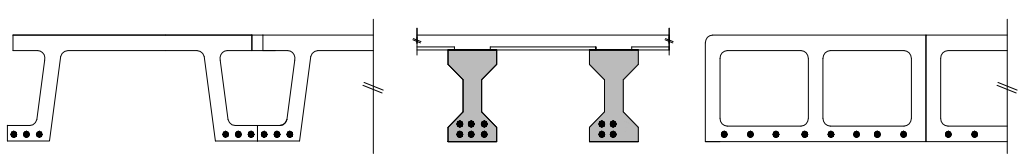
For the optimization problem, we define the following four efficiency factors for bending (M) and shear (V) at both SLS and USL states:

$$\eta_{i,j} = \frac{\sum_{k=1}^{N}\left(\alpha_k \psi_k \gamma_k Q_{ik}\right)_j}{\left(\phi R_i\right)_j} \qquad i = M, V; \quad j = SLS, ULS \tag{8}$$

where each efficiency factor η_{ij} has to be less than 100% to be accepted as a design solution. We consider the 100% efficiency factors as an optimum design solution, thereby proposing the following objective function for a minimum problem:

$$f = \sum_j \left[(\eta_{Mj} - 1)^2 + (\eta_{Vj} - 1)^2 \right] \qquad j = SLS, ULS \tag{9}$$

In the optimal problem, we keep the material properties as constant (as reported in *Table 1*), and we minimize the objective function Eq.9 by varying the section geometry parameters and the pre-stress level. In a parametric fashion, we account for five different span lengths (24.38, 27.43, 30.48, 33.52, 36.57 m) and three different cross-sections (double-tee, girder, box section, as shown in Figure 3).



(a) (b) (c)

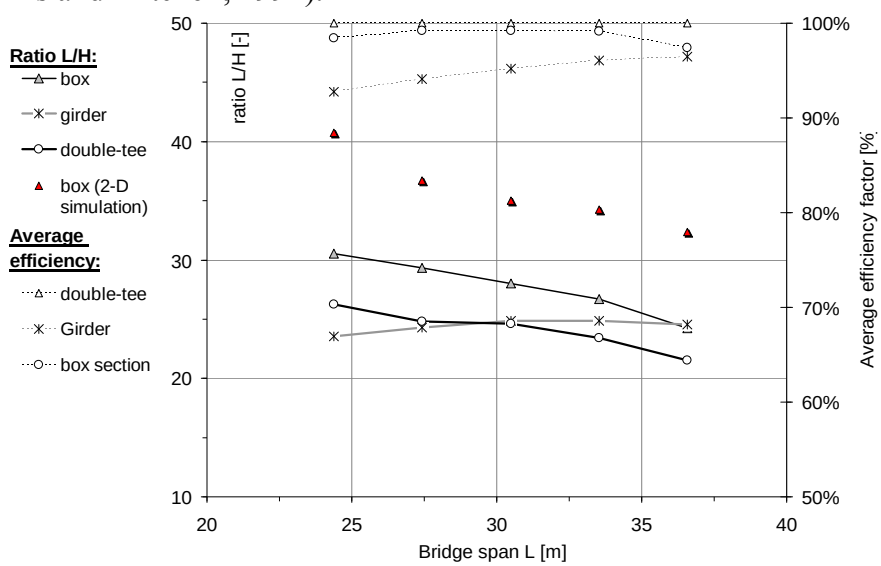
Figure 3. Bridge sections considered in the optimization problem: (a) double-tee section, (b) girders with a slab of normal concrete, and (c) box girders.

Additional design constraints are assumed in the optimum design, such as: (i) the minimum thickness for the web and slab are about 76 mm and 102 mm, respectively; (ii) at the time when the prestressing load is applied, the maximum tensile stress is no more than

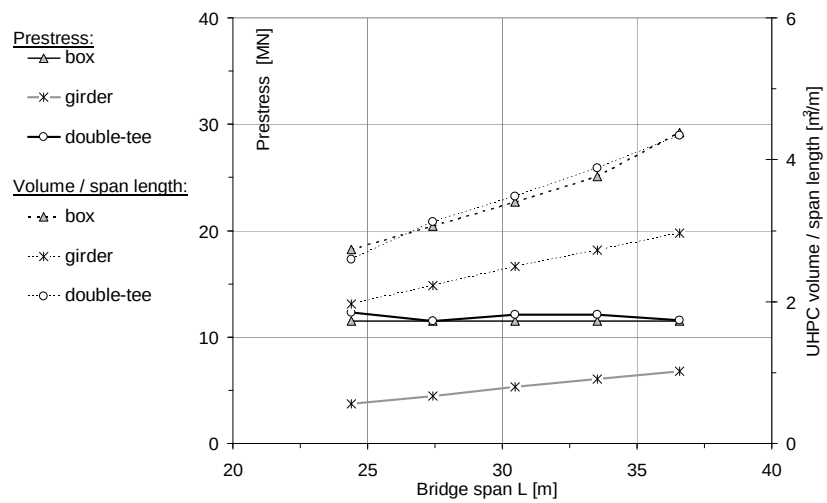
50% of the material tensile strength (Σ_1^-); (iii) the bottom flange has to be large enough to accommodate the total number of strands, in accordance with required minimum spacing and cover distances (both equal to 25 mm); (iv) room exists only for four girders with a maximum girder width of about 1.1 m, for two box sections, or for three double-tee sections; (v) for all cases, the total bridge width is about 6.88 m to accommodate two traffic lanes.

3.2 Optimized 1D design solutions based on crack criteria

The optimized solutions are plotted in Figure 4 in terms of the span-to-depth ratio (solid line) and the average efficiency (dashed line) against the bridge span length. As aimed by the optimization problem, the average efficiency factors of Eq.8 are close to 100% (dashed lines of Figure 4a). The optimum design results show that the box solution allows a significant reduction of section depth (~ 12%) for bridge span lower than 35m. On the other hand, the girder solution saves UHPC volume per span length (~ 50%) and prestressing cables (~ 40%). As compared with prestressed normal concrete, the span-to-depth ratio (~ 25) of UHPC girder is significantly higher than the typical values for highway bridge (~18; Collins and Mitchell, 1991).



(a)



(b)

Figure 4. 1D design bridge optimal solutions plotted in terms of (a) span-depth ratio vs. bridge span, and (b) prestressing force and UHPC volume vs. bridge span.

3.3 3D model-based optimization

Adopting a 3D description of the material behaviour may allow further optimization of the cross-section. We used a two-phase damage-plasticity model implemented in the finite element code CESAR-LCPC (Chuang and Ulm, 2002). The objective of this optimization is to design a structure that reaches the ULS design load and the maximum crack width at the same time, while the "no crack" condition at SLS state is still verified (Park *et al.*, 2003). The 2D finite element mesh consists of about 740 linear quadrilateral elements and 2360 nodes.

The computation results of the box section are plotted in Figure 5 in terms of the ULS safety factor (i.e., the inverse of our efficiency factor defined in Eq.8) and the normalized crack width. Optimized results according to this model are obtained with a box-section depth reduced by 20-25 %. For the sake of comparison, these results are also plotted in Figure 4.a (dashed triangles) with the 1D design solutions. The main difference of this computation corresponds to the distribution of the matrix plastic strain (right side of Figure 5), which is smeared out on a bridge length much longer than the one assumed in the 1D simulation (i.e., the AFGC characteristic length of 2/3H).

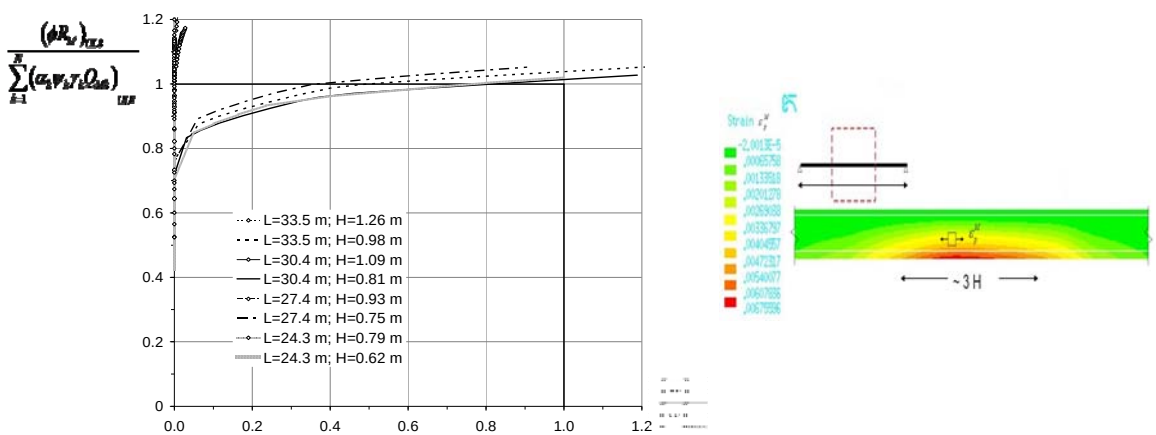


Figure 5. 2D computation results in terms of the efficiency factor (SLS and ULS) and the normalized crack opening and matrix plastic strain distribution at the peak load.

4. Concluding remarks and future research

Successful UHPC industrial implementations rely on shared experience of appropriate design methods that exploit the extraordinary compressive strength, account for the tensile pseudo-ductile behaviour, and optimize the structural durability with respect to the maximum crack width. We propose a design approach based on crack criteria able to optimize the cross-section of highway bridges of medium span between 23 and 36 m. The following conclusions can be drawn:

- The optimized box sections allow the highest span-to-depth ratios (between 25 and 30), while the girder sections save significant UHPC material volume and prestressing cables;

- Precisely accounting for the effective matrix crack distribution appears as critical for possible further optimization.

The current research is focusing on the size effect implications of the proposed design procedure and on the risk of the early age cracking due to autogeneous shrinkage and thermal effects (Park *et al.*, 2003; Shim and Ulm, 2004).

5. Acknowledgement

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