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CASE STUDY: TESTS ON TWO UHPC I-GIRDERS AND ANALYSIS OF FLEXURAL BEHAVIOUR

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Abstract

This article describes the details of a case study on the flexural behaviour of two UHPC I-girders, which have been tested in collaboration with the precast company Ergon. The *experimental phase* focuses in a first step on material characterization with small scale test specimens, and the flexural behavior of full-scale tests with UHPC prestressed I-girders. A specific UHPC-type, with an average compressive strength of about 150 MPa and 1 % fibre mix volume, was used to fabricate two 7 m girders containing 9 prestressing strands. The *analytical phase* of this research analyzes the results from the experimental phase and compares with standard EN 1992-1-1 recommandations (Eurocode 2). The full scale test results are compared with the standard Eurocode 2 predictions. In a next step, the test results on small scale specimens are used to correct and extrapolate Eurocode's material laws, in order to provide a more accurate prediction of the flexural behavior.

Résumé

Cet article décrit en détail un cas d'étude consistant à évaluer le comportement en flexion de deux poutres en I réalisées en BFUP, en collaboration avec l'usine de préfabrication Ergon. La *phase expérimentale* consiste en la caractérisation du matériau à l'échelle de laboratoire et en la réalisation d'essais à grande échelle visant à évaluer le comportement en flexion de poutres en I précontraintes. Une formulation spécifique de BFUP, de résistance en compression moyenne d'environ 150 MPa et contenant 1 % en volume de fibres d'acier, a été utilisée pour fabriquer deux poutres de 7 m de portée contenant chacune 9 torons. Dans la *phase analytique* de cette recherche, les résultats issus de la phase expérimentale sont analysés et comparés aux recommandations de la norme EN 1992-1-1 (Eurocode 2). Les résultats des essais à grande échelle sont tout d'abord comparés aux prédictions de l'Eurocode 2. Ensuite, les résultats des essais sur les échantillons de laboratoire sont utilisés pour corriger et extrapoler les lois de comportement du matériau dans le but de fournir une prédiction plus précise du comportement en flexion.

1. INTRODUCTION

Eurocode 2 is the current design standard for pretensioned prestressed normal strength concrete, this standard had provided safe and economical structures in the past. When using UHPC, these procedures should be verified for general use since the specific characteristics – as for instance the low porosity and high performance of the concrete - have an influence on all concrete properties. In a first stage, this will result in the requirement for a case-by-case specification and material check. When sufficient experience is gained, this should lead to an extension of the Eurocode 2.

This research project aimed at an optimization of UHPC for precast elements and at an evaluation of some of the material laws of Eurocode 2 (*EN 1992-1-1:2004, abbreviated "*EC2" in this article). The research has been carried out in three phases.

In the first phase, lab research on small scale elements allowed for the characterization of the mechanical and durability properties of the material, and for a comparison of these different mechanical performances with the behaviour as described in Eurocode 2.

Then, the flexural behavior of these UHPFRC girders from the initial elastic loading to failure is analyzed thought full-scale tests to determine the contribution of this concrete to the overall flexural behavior of the girder.

In the third step, EN 1992-1-1 models and material laws are used to predict the flexural behaviour of prestressed UHPC beams, even though they are not valid any more for the tested material (compressive strength about 150 N/mm²). Still in this phase, another prediction is made using the information gathered from lab tests.

Finally these predictions are compared to the results gathered in the second stage. The comparison is made in order to judge if it is feasible to extrapolate Eurocode 2 material laws and models and if the information gathered from lab tests could be used to obtain a better prediction, indicating which material laws should be changed.

2. MECHANICAL PROPERTIES

2.1 UHPFRC Mix design

Only one mix was used in this experimental investigation, indicated as M2. This UHPFRC contains quartz sand, with a grain size between 0 and 2 mm, and porphyry, with a grain size between 2 and 4 mm, as coarse aggregate. The powder fraction (≤ 0.125 mm) is composed of ordinary Portland cement CEM I 42.5 R, combined with silica fume and ultrafine quartz powder. Silica fume is used as an aqueous suspension with a solids content of 50 % by weight. A fiber cocktail was used at a dosage of 1 % in volume, composed by: 70 % short fibers (6 mm length and Ø 0.16 m) and 30 % longer fibers (30 mm length and Ø 0.38 mm).

2.2 Compressive strength

Practical use of UHPFRCs requires knowledge of the basic compressive behavior of this concrete. The results presented in this section are based on compressive test of 100 mm and 150 mm side cubes loaded in axial compression, the tests were completed according to NBN EN 12390-3. The cubes were demoulded 1 day after casting and were stored in a humid laboratory environment (T = 20°C, RH > 95 %) from demoulding until testing.

Overall 18 cubes were tested: six with 100 mm side at respectively 28 and 56 days after casting and twelve with 150 mm side at 1, 7, 28 and 56 days after casting, generally three samples for each age were tested (see Table 2). Figure 1 presents a compilation of the compressive strength data for 150 mm side cube tested between 1 and 56 days. The extrapolation of the formulation proposed by Eurocode 2 – which normally is not valid any more for UHPC – still accurately describes the strength evolution for these tests (equation 3.2). Furthermore, based on additional tests, the conversion factor between results on cubic specimens (side 150 mm) and cylindrical specimens (diameter 150 mm, height 300 mm) can be estimated at 0.95 (first results documented in [10]).

Table 1 Mix design of the UHPC used

Component - Type	**M2 [kg/m³]**
Cement - CEM I 42.5 R HSR LA	830
Quartz powder - dav = 7 µm	83
Quartz sand - 0/0.5	335
Porphyry - 1/4	776
Silica fume (SF) *(in 50% suspension)*	332
Water *(besides water in SF)*	12
Superplasticizer *(Polycarboxylate 30 %)*	24
Air	0
Fibers - 60 mm and 30 mm	78
w/c value [-]	0.23
w/b value [-]	0.20

Table 2: Compressive strength at various ages after casting

Specimens dimensions	**Age**	**f_{cm}**	**$\sigma(f_{cm})$**
[mm]	[d]	[N/mm²]	[N/mm²]
Cubes *(150 x 150 x 150)*	1	43.2	1.4
	7	110.3	4.9
	28	142.6	0.9
	56	152.0	5.4
Cubes *(100 x 100 x 100)*	28	158.4	3.3
	56	154.2	5.4

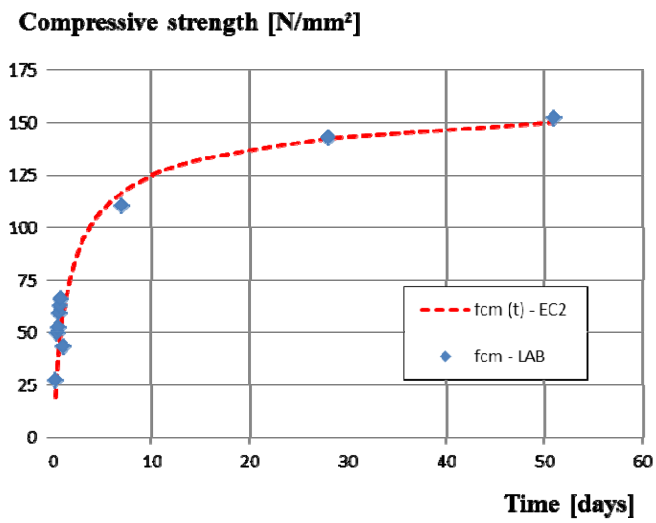

Figure 1: Comparison of compressive strength as a function of time after casting obtained from experimental results and Eurocode 2 formulation

2.3 Elastic modulus

The tests were conducted according to NBN B 15-203. Six prisms 100 mm in side, by 400 mm in height were tested, half at the age of 1 day and the others at 51 days, after the elastic modulus's measurements, the same prisms were crushed by a compressive test. The testing resulted in an UHPFRC modulus of elasticity of 43.3 GPa at 51 days.

Table 3: Modulus of elasticity at various age after casting

Specimens dimensions	Tests date	Age	E_{cm}	$\sigma(E_{cm})$	$f_{cm.prism}$	$\sigma(f_{cm})$
		[d]	[GPa]	[-]	[N/mm^2]	[-]
Prisms (100 mm x 100 mm x 400 mm)	19/4/2012	1.00	21.8	0.47	38.57	0.48
	08/06/2012	51.00	43.3	0.45	143.43	3.51

As shown in Figure 2, the variation of the secant modulus of elasticity with time proposed by Eurocode 2 doesn't seem to give satisfying results on UHPFRC (see table 3.1. of EC2). Research results on a number of additional tests – besides those of this case study - confirmed this difference (first results documented in [10]). A common relationship, which is often found in literature and standards (i.e. ACI 318), consists in relating the square root of the compressive strength to the modulus of elasticity through a scalar factor. According to the results of the concrete M2 carried out in this experimental program, this relationship could be approximated by: $E_{cm} = 3485 \cdot \sqrt{f_{cm}}$.

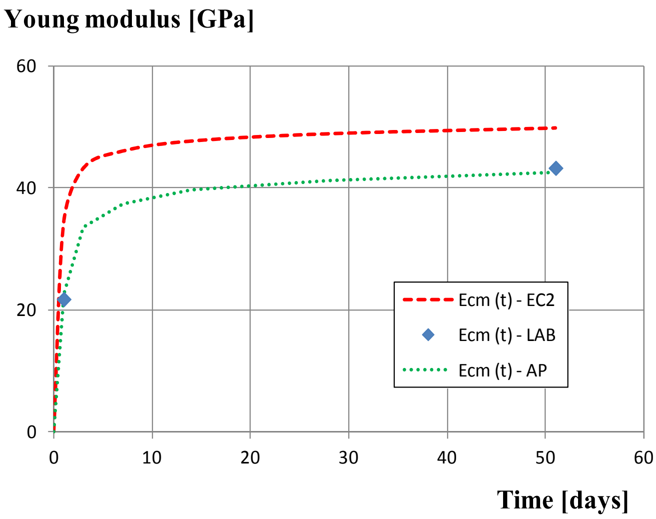

Figure 2: Comparison of modulus of elasticity as a function of time after casting obtained from: experimental results, approximation and Eurocode's formulation

2.4 Shrinkage measurements

Long-term shrinkage testing was completed according to RILEM TC 107-CSP, total shrinkage was measured on three prisms 70 mm by 70 mm by 280 mm. The specimens, after casting, were kept into laboratory environment, protected with a plastic sheet. Initial readings start after demoulding at 1 day, after which the storage conditions are 20 ± 2 °C and 60 ± 5 % relative humidity. From the experimental set-up, the development of shrinkage strain is measured as a function of time, measurements of the changes in length were recorded automatically for 25 days, then manual ones were taken.

Figure 3 gives both the results for specimens made at the precast plant and for specimens that have been made in the lab with the same composition. The specimens from the precast plant show less deformation compared to those realized in the lab, no specific explanation could be found since both mix design and storage conditions after demoulding are identical.

The comparison of the prediction according to EC 2 with the deformation of specimens tested gives first evidence that total shrinkage is underestimated (EC2, equation 3.8). Figure 3 shows that the measurements deviate from this reference, the EC 2 prediction after 220 days is around 430 µm/m while the lab measurements vary from 450 µm/m (plant specimens) to 650 µm/m (lab specimens). This deviation becomes very large when the deformations during the first 24 hours are included, showing an important autogeneous shrinkage, up to 450 µm/m (lab measurements on specimens with M2). It seems that Eurocode's formulations have not been calibrated starting from very early age for this type of concrete.

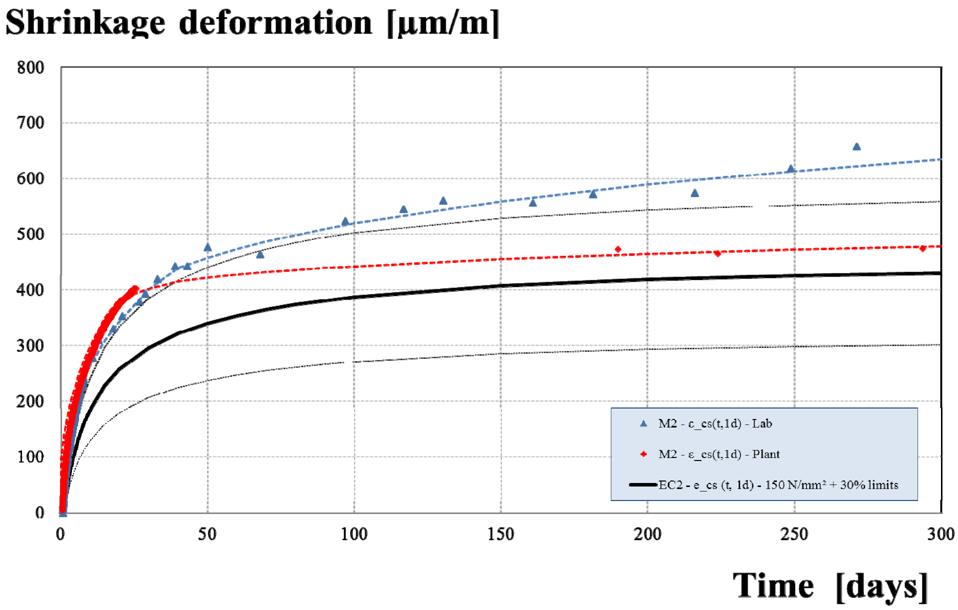

Figure 3: Total shrinkage measurements (started after demoulding at 1 day)

3. TEST ON TWO PRESTRESSED I-GIRDERS

3.1 Geometry and reinforcement of the girders

Two identical 7 m girders were fabricated at Ergon, in Lier, Be. The girders were allowed to cure in the ambient atmosphere until they had gained sufficient strength to resist the forces imparted by strand release. After the strands were released, at 24 hours, remained in the laboratory with temperature of more or less 20°C and relative humidity of approximately 60 %. The bending tests have been carried out 51 days after casting. At that moment, the compressive strength of the UHPC is 142.6 N/mm² and the Young modulus is 43300 N/mm².

The cross section of the girders is shown in Figure 4. The girder is 630 mm deep and has a 240 mm wide top and bottom flange. The web is 280 mm deep and is 70 mm thick. These beams were each prestressed with eight 12.5 mm diameter, 1860 MPa, low-relaxation prestressing strands in the bottom bulb (type 1/2" with seven wires) and one 9.3 mm diameter, 1860 MPa, low-relaxation prestressing strand in the top bulb (type 3/8" with seven wires), resulting in a total strands area of 796 mm²Before testing, calculated strand deformations vary from 6.71 ‰ (top strand) to 6.32 ‰ (bottom strand). Moreover minimum shear reinforcement was disposed. These 1860 MPa low-relaxation strands are stressed to approximately 80 % of their ultimate strength (1486 MPa). The initial prestressing force was 1183 kN (120.7 ton) reduced to 1056 kN after initial losses (16 kN relaxation losses and 111 kN elastic losses).

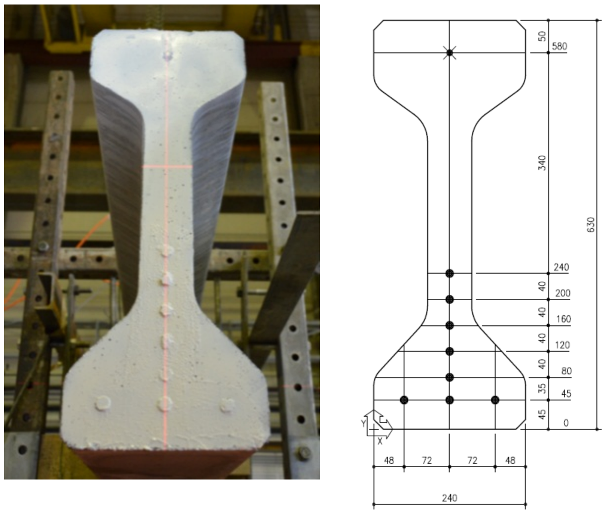

Figure 4: Girders cross section and strand pattern

3.2 Test configuration

The 7 m long prestressed girders is tested in a four-point isostatic bending configuration with a 5 m span. A schematic diagram of the test set-up can be found in Figure 5: two point loads F/2 each located 0.50 m from the midspan and two roller supports (support A and B: roller with diameter 45 cm and top plate with 80 mm width) each located 2.5 m from the midspan. The horizontal displacement of support A is blocked, to create a hinge on one side and a free horizontal displacement on the other side.

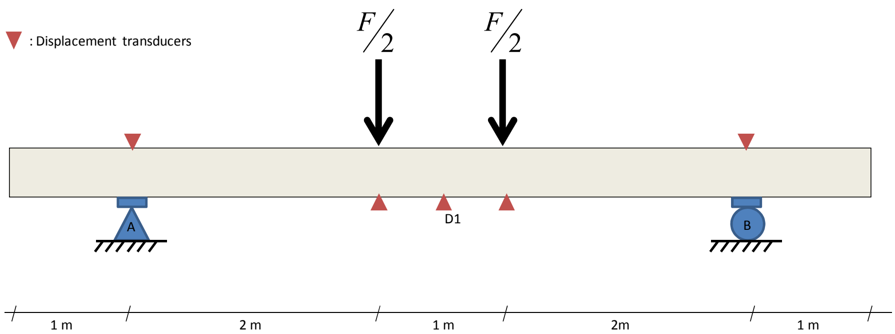

Figure 5: Schematic diagram of test configuration

Loads are applied vertically in the plane of the strong axis of each girder, with one hydraulic jack of 900 kN. A beam split the single load provided by the hydraulic jack into two loads. The deflection is measured, with five displacement transducers: at the two supports, at the two loading points, and at the midspan centerline. The two girders were loaded at a constant rate of 25 kN/minute until failure.

Figure 6: Test configuration, photo during test

3.3 Test results

Figure 7 shows the applied load versus the net midspan vertical deflection response, from initial load application to the peak load. Girders “test 1” and “test 2” began to soften between 350 and 410 kN at respectively a deflection of 5.6 and 5.7 mm. After that, they exhibited significant additional load-carrying capacity, reaching a peak load between 741 and 754 kN at a deflection of 60 and 80 mm.

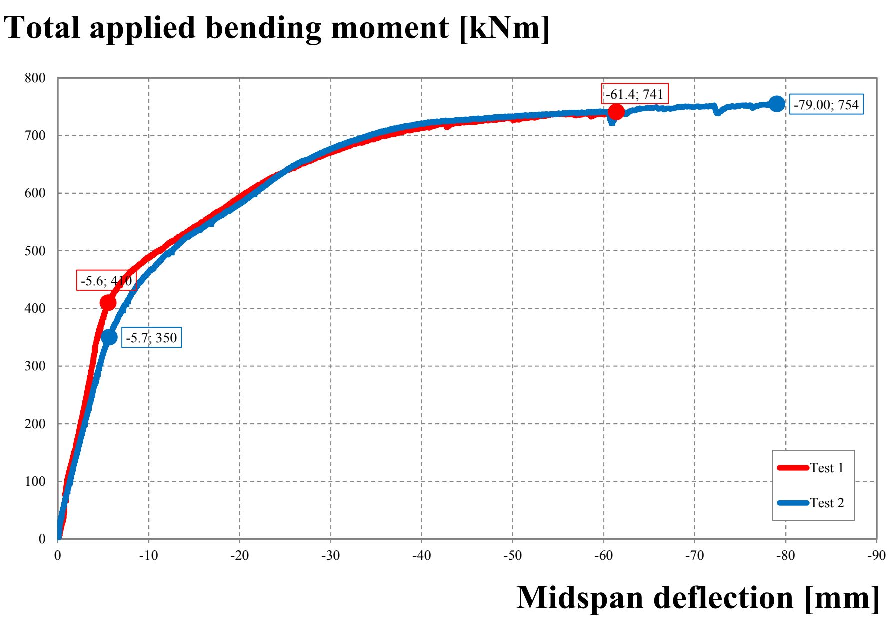

Figure 7: Load versus midspan deflection of the two girders

At the end, beams reached a distinctive plateau in the load/deflection curve, which indicates yielding of the reinforcement. Test beam 1 failed by crushing of the concrete in the top region, test beam 2 failed by rupture of the prestressing strands. In both cases distinctive bending cracking and shear cracking could be noticed (see Figure 8 and Figure 9). No strain measurements on strands or beam are performed.

Table 4: Results from bending test

	Crack flexural capacity		Ultimate flexural capacity	
	Load [kN]	Midspan deflection [mm]	Load [kN]	Midspan deflection [mm]
Test beam 1	410	5.6	741	61.4
Test beam 2	350	5.7	754	79.0

Figure 8: Crack pattern after failure of beam 1

Figure 9: Beam 2 after testing

4. PREDICTIONS OF THE GIRDER FLEXURAL CAPACITY

4.1 Crack flexural capacity

Before the application of flexural live load, the only stresses in the beam are caused by the prestressing force and the dead load. An analysis was completed to determine the stresses in the UHPC and the prestressing strands before and during the test. The basic assumptions in this analysis included: linear elastic behavior of materials, sections within the cross section remain plane and that strain-compatibility between the strands and the concrete is maintained.

The prediction of the crack flexural capacity, the moment when the first crack is expected and the beams will start to soften, is based on the tensile strength of UHPC, in the same way as Eurocode 2 proposes for normal strength concrete: stress limits are proposed for compressive stress and tensile stress. In this case, concrete stress in upper compression zone should be smaller than 0.6 x $f_{ck}(t)$, and concrete stress in lower tensile zone should be smaller than $f_{ct,eff}$, which can be interpreted as f_{ctm}.(= 5.8 N/mm² = 2.12 ln(1 + f_{cm}/10), relation confirmed for this UHPC-composition with a number of splitting tensile tests [10]). When these limits are passed, cracking and softening of the beam will start with increasing load. For this type of beam and test configuration, the tensile limit for the lower fiber is nearly always reached before that on compression of the higher zone. The prestressing force at 51 days is 873 kN (long term losses = creep + shrinkage + relaxation)]. Theoretically the total force F at which the beams should start to soften is equal to approximately 375 kN.

4.2 Ultimate flexural capacity

In order to determine the peak load, a standard pure flexural analysis of the beam was made; this calculation was conducted with the hypothesis that UHPC carries no tensile forces after cracking, and supposing a linear strain distribution over the section depth. Two different analyses are carried out: a first one based on the Eurocodes formulations for stress-strain behaviour, creep, shrinkage, etc. A second analysis is based on the test results of a large number of small scale samples (not only those made together with the beam in the precast plant, [10]). These test results allowed for a correction of most of the Eurocodes material constitutive laws.

It was supposed that the girder failed when at least one of the strain limits is reached (tension or compression): the tensile strain limit was set equal to 35 % typical strands elongation under maximal load (instead of the 2.2 % suggested by EC2), while the compression strain limit was derived from experimental stress-strain curves (peak stress-strain diagrams at 4.5 ‰).

The flexural analysis of the girder indicates that the moment capacity should be 670 kN.m and 671 kN.m, with the neutral axis located 70 mm and 67 mm down from the top of the girder at failure, respectively for behavior the real and Eurocode stress-strain respectively. As we can see in Table 5, which resumes the results of the predictions, according to this simulation, the theoretical capacity of the beams for concrete failure or strands failure are very close, this means that neither of the two types of failure is particularly privileged over the other.

Table 5: Summary of the predictions of peak load and corresponding applied moment and neutral axis based on Eurocode 2 and experimental data, and relative errors between these predictions and the test results.

	Type of failure	Total load	Moment	Neutral axis	Rel. Error
		[kN]	[kNm]	[mm]	[%]
Prediction based on Eurocode 2	Concrete failure	680	686	67	11.0
	Strands failure	666	672	67	
Prediction based on experimental UHPC-data	Concrete failure	677	683	67	11.2
	Strands failure	664	670	71	

5. COMPARISONS AND CONCLUSIONS

The estimated crack flexural capacity is 370 kN, which corresponds very well with the experimental results obtained in Test 1 and Test 2 respectively of 350 kN and 410 kN.

In regard to the ultimate flexural capacity, Eurocode's prevision deviates about 11 % from the observed one, approximately the same results is obtained using the experimental UHPC characterization of the material. This should not mislead and suggest that the Eurocode 2 provides a perfect representation of the real behavior, there are actually two phenomena that counter each other: on the one hand the prestressing losses are underestimated among others due to the underestimation of shrinkage and creep and on the other hand, the full potential of UHPC is underestimated as well by using the rather low concrete strain at peak stress. Altogether, it can be concluded that common design code-based calculation method provides a good representation of the final flexural resistance, but best representation of the overall behavior should include detailed material characterization.

Furthermore considering that the concrete has no tensile resistance, neglects the tensile properties of UHPC and the difference between the theoretical predictions and the test results may be in part attributed to this phenomenon.

To sum up the following conclusions could be drawn:

The use of prestressed UHPC for the realization of I-girders can be considered safe and reliable, this material is a viable substitute for normal concrete and HPC;

Experiments on creep and shrinkage have shown a distinctive deformation capability of UHPC, especially in the young concrete age which should not be disregarded in design codes for UHPC constructions;

Based on experimental data and extrapolation of Eurocodes material laws, the overall behavior of pretensioned girders made with UHPFRC is shown to be sufficiently well predicted (relative error of 11 %).

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