HUAXIN CEMENT / UHPC DUCTAL® FM PRE-TENSIONED BEAM

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Abstract

This paper has been drawn up in the framework of the construction studies for a pre-tensioned high performance concrete beam in UHPFRC DUCTAL® FM, located in Wuhan, China. The purpose of the paper is to present the structural design associated to this project. The particularity of this structural design is to have to adapt concrete pres stressed design regulations to UHPFRC DUCTAL® FM material characterized, principaly, with a very high compression stress capacity.Also the slenderness of the UHPC section, designed as a "I", is closer from a steel beam rathern than a concrete one. In this context all the classical steel structural design craterias such as lateral stability, local buckling or shear stress are critical.

Résumé

Cet article a été élaboré dans le cadre des études de construction d'une toiture dont l'ossature primaire est réalisée par un ensemble de poutres précontraintes réalisée en béton à ultra haute performance UHPFRC DUCTAL® FM. L'ensemble est situé à Wuhan, en Chine. Le but de l'article est de présenter la conception structurelle associée à ce projet. La particularité de cette conception est d'avoir adapté les règles du béton précontraint au béton armé à Ultra Haute Performance caractérisé, principalement, par une contrainte de compression élevée. De plus, la finesse de la nouvelle section nous ramène à des proportions plus proches de celles d'un profilé acier que d'une pièce issue de l'industrie du béton. Dans ce contexte, toutes les critères classiques de conception des structures métalliques telles que la stabilité latérale le voilement ou la résistance au cisaillement sont critiques.

1 GENERAL PRESENTATION

The pre-tensioned UHPFRC DUCTAL® FM beam (around 140 units in total) is designed for a one-story concrete frame factory located in Wuhan, Hubei, China (Fig. 1). The building's plan dimensions are 177 m by 50 m, with column spacing of 6 m along the short dimension and 25 m along the long dimension. The story height is about 22 m. The global design includes an expansion joint every 44 m of the long dimension (Fig. 2). The global bracing of the system is done by considering that the columns are fixed in both longitudinal and transversal direction with the foundations. Within this hypothesis, the UHPFRC beams are considered as isostatic. The connections between the column and the UHPFRC beam is modeled as pinned. The roof supported by the beams is done with pre-stressed concrete ribbed panels which are welded on the top of the beam.

The particularity of this structural design is to adapt concrete pre-tensioned design regulations to UHPFRC material characterized, principally, with a high compression stress capacity. The high compression resistance of the material and the advantage of a pre-tensioned system allow to optimize the cross-section and finally to obtain a design which is geometrically closed from a typical "I" steel section (Fig. 3). In this situation shear analysis and lateral stability become critical points of the analysis. Also, the behavior of the beam at early stages is part of the discussion in particular during the loosening of the pre-stressing. The substantiation of the structure is carried out using the SOFISTIK non-linear design software program.

Figure 1: View of the Mock-up beam (Source: Huaxin Cement, Wuhan)

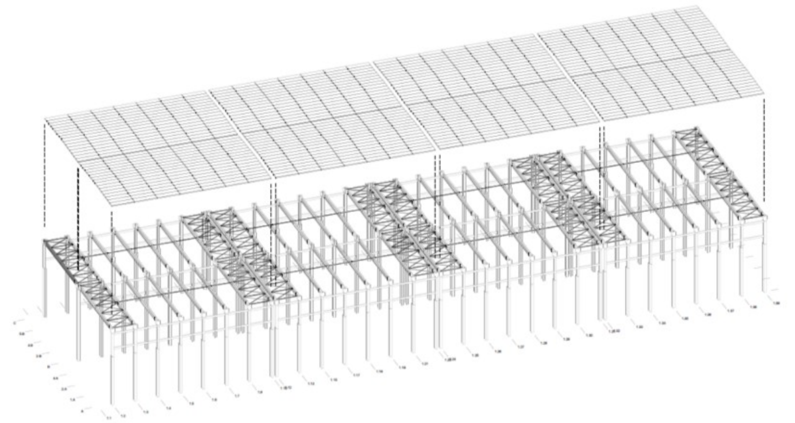


Figure 2: Building axonometric view (Source: C&E)

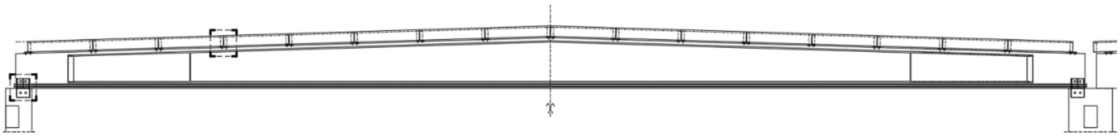


Figure 3: Side view of the beam (Source: C&E)

2 MATERIAL HYPOTHESES

The design calculation methods for the pre-stressed Ductal structural design are based on the National addition to Eurocode 2 NF P 18-710 [1]. The constitutive laws (Fig. 4) at the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) are defined as follows:

- A linear elastic stage limited by a stress value $f_{ct,el}$
- A post-cracking stage characterized by a stress-crack width law.

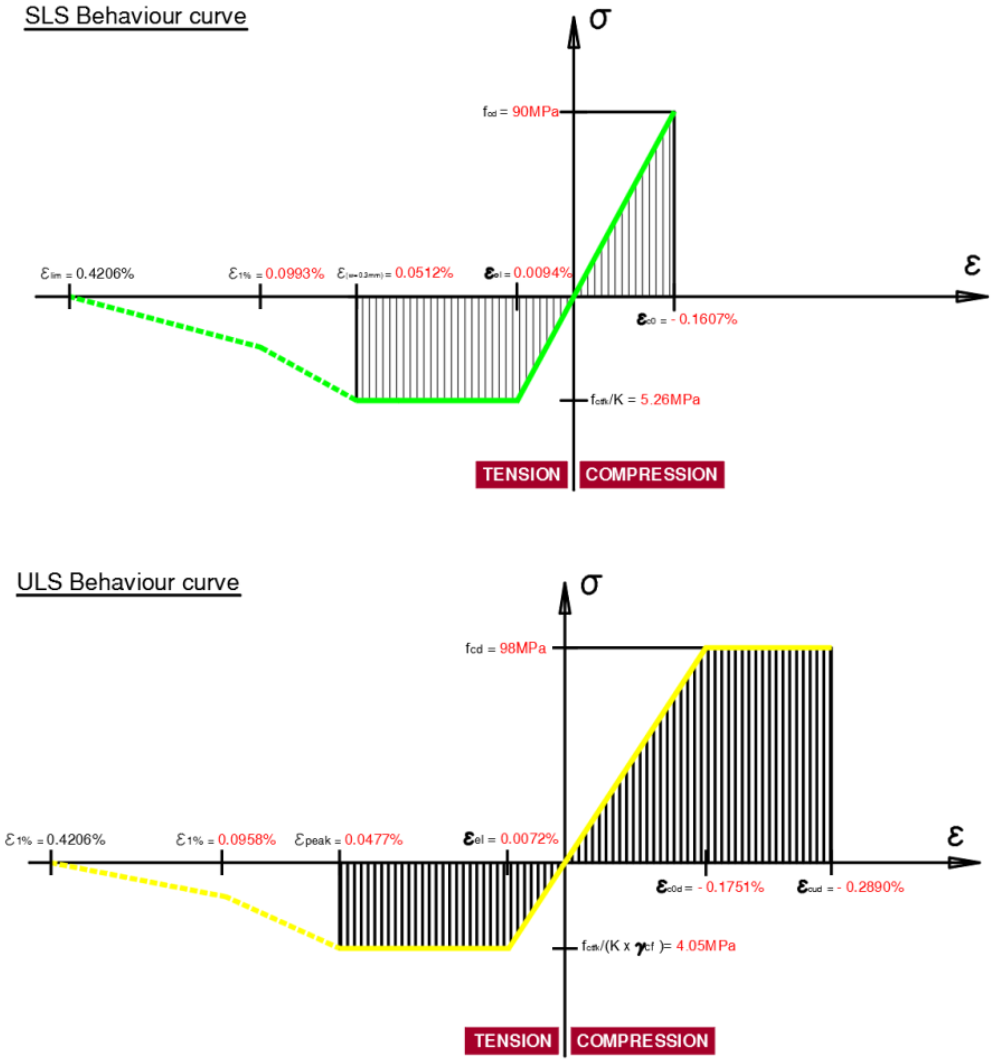


Figure 4: Design constitutive laws of Ductal FM® [2]: SLS and ULS (Source: C&E)

Fourteen units of pre-tensioned bonded tendons with diameter of 15.2 mm are used. The initial tensile stress and the initial pre-stressing force in the pre-tensioned bonded tendons are calculated as the following:

- Tension stress in the tendon after transfer of pre-stressing: 1395 MPa
- Resulting tensile force: 1395 MPa x 140 mm² = 195.3 kN

Requirements for durability: The good compactness and the quality of UHPFRC can optimize the concrete cover. In the design of this project, it is supposed that the concrete inside buildings is subjected to moderate or high air humidity and is exposed to industrial waters containing chlorides. That means the exposure classes following French regulations is XC3, XD2. The minimum concrete cover is 36.4 mm and the spacing of bars is 30.4 mm according to NF P18-710 §4.4 [1] and Eurocode 2 §8.10.1.2 [3-4].

3 GEOMETRY AND MODELLING

The height of the beam is varying from 880 mm to 1248 mm which is corresponding to a slope of 3 %. The 14 units of pre-tensioned tendon with diameter of 15.2 mm are placed straightly in the bottom flange of the beam. Two types of cross-sections are designed (Fig. 5):

- Full Rectangular cross-section (from 0 to 2 m from the support);
- I-shaped cross-section with web thickness 70 mm (from 2 m to mi-span 12.27 m).

The beam characteristics are the following:

- Span: 24.54 m
- Maximum height: 1.248 m; Minimum height: 0.88 m
- Width of the upper and lower flange : 0.34 m
- Thickness of the upper (resp. lower) flange: 0.08 m (resp. 0.14 m); Web thickness: 0.07 m
- Current section of the pre-tensioned tendons: Diameter 15.2 mm

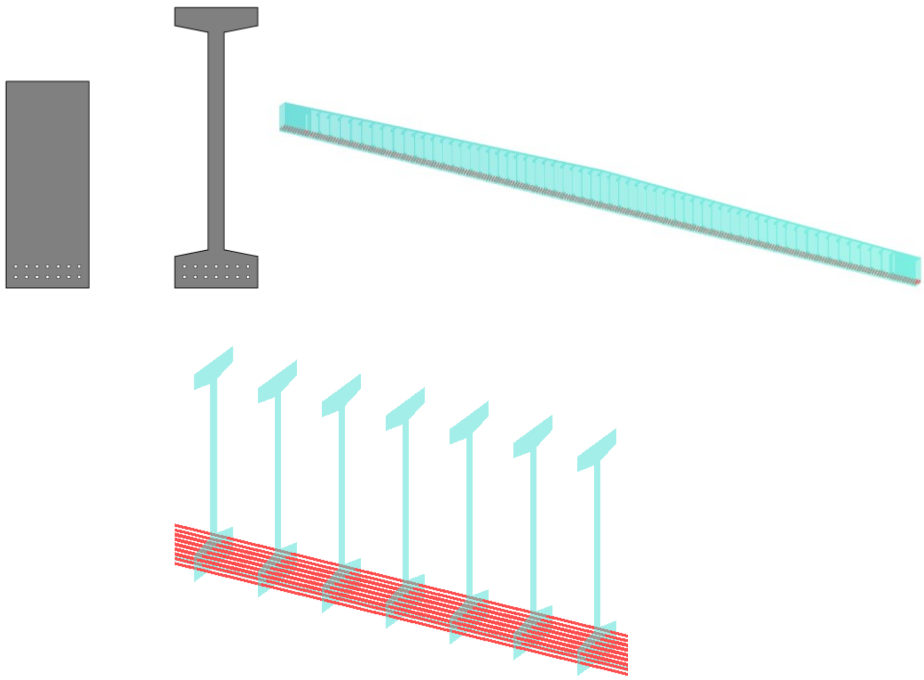


Figure 5: Two typical cross-sections and the model including the tendons

4 ACTIONS ON THE STRUCTURE

The following loads are included in the calculation:

- Dead load including roof panels (2.5 kN/m^2).
- Technical live load (0.5 kN/m^2).
- Wind load following Chinese codes (-0.27 kN/m^2 / vertical axis)
- Snow load following Chinese codes (0.7 kN/m^2 / vertical axis)

The building global is assigned to fortification intensity 6 following Chinese codes [5-6]. The seismic checking of cross section of structural members is permitted not to be carried out.

5 RESULTS

The analysis results are divided within the three following steps:

- Global analysis of the building;
- Beam analysis during loosening of the pre-stressing after 24 h at early stage;
- Beam analysis at the final stage.

6 GLOBAL ANALYSIS OF THE BUILDING

The global bracing of the building is ensured by the fixed foot of the columns. UHPFRC DUCTAL® FM beams are placed on a self-stable "Box" located at the top of the column (Fig. 6). The lateral force from the façade is transferred directly to the columns and then the foundation of the building (Fig. 7).

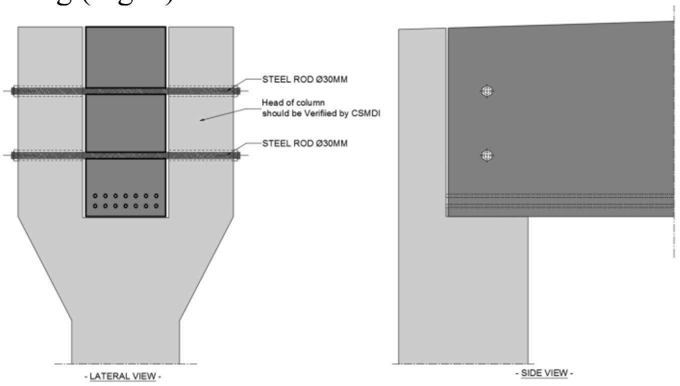


Figure 6: Connection beam/column

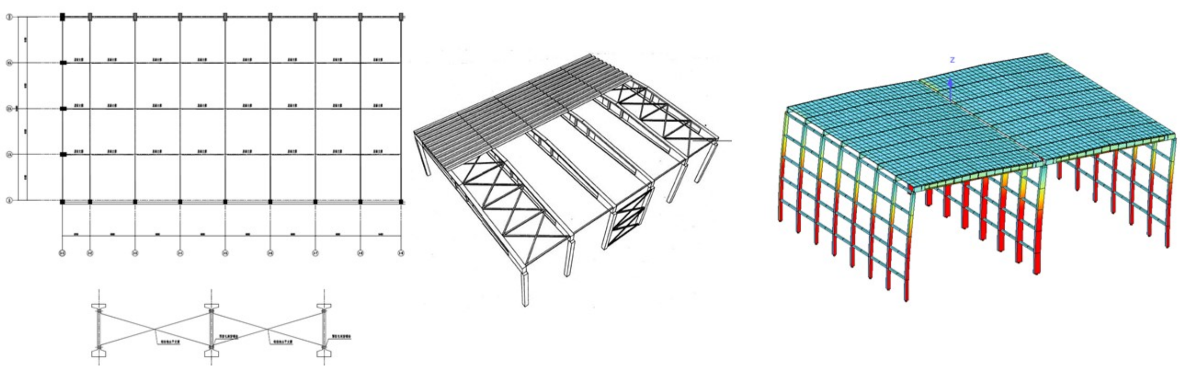


Figure 7: Global view of the building with lateral roof stability built with steel elements

The buckling risk can affect the top flange of the beam. The solution is to have a horizontal bracing system every 6 m (Scissors) connected with horizontal trusses under the concrete roof in order to maintain the beams with the fixed ended columns and finally the foundations.

7 RELEASE OF THE PRE-STRESS

A critical point in the design concerns the optimization of the time after which it is possible to release the pre-stressing tendons. This analysis makes it possible to optimize the production sequence of the beams. It requires to establish the approximate constitutive law of UHPFRC at the early age (Fig. 8) in order to verify whether the values are compatible with the stress capacity needed (Fig. 9). The assumptions are as follows:

- Tensile force at the release of the pre-stress P_2: 190 kN
- Initial tensile force P_0 = 195.3 kN
- Losses before the release 2.71 %
- Tensile force at the release of pre-stress P_2 = 190 kN
- Upper and lower characteristic factor : r_{sup} = 1.05, r_{inf} = 0.95

Crack opening is obtained by the maximum elongation of the fiber the most strained fiber (Fig. 10). The compressive strength of the material is acceptable from 24 hours. Concerning the tensile stresses, the crack openings at 24 h remain below or equal to the maximum value (0.3 mm), they are between 0.1 mm and 0.2 mm. But they remain considerable for a provisional phase.

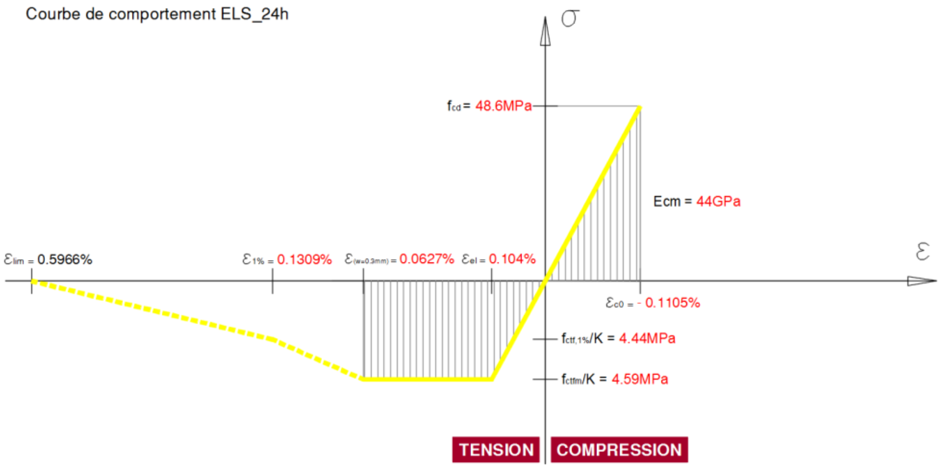


Figure 8: Average constitutive curve at 24 h for SLS (strain in %, stresses in MPa)

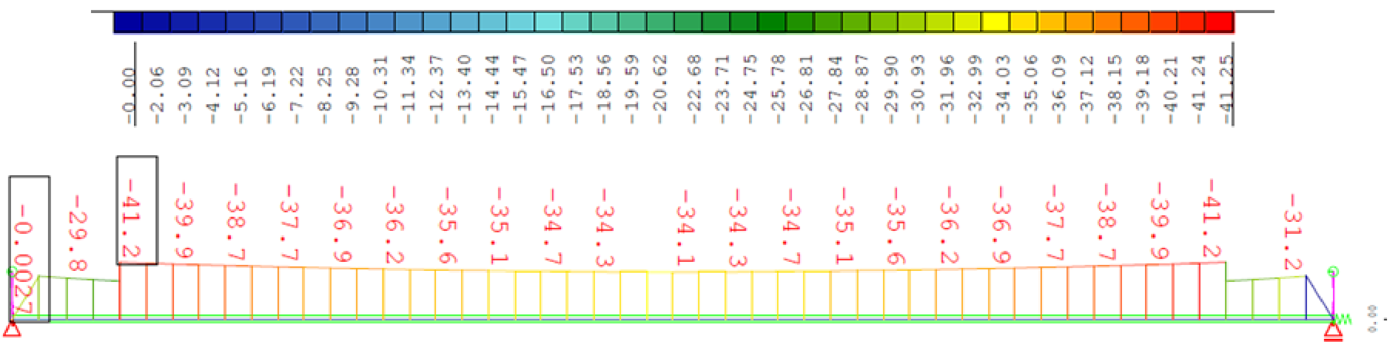


Figure 9: Maximum compressive stress under P+G _ Release at 24 h

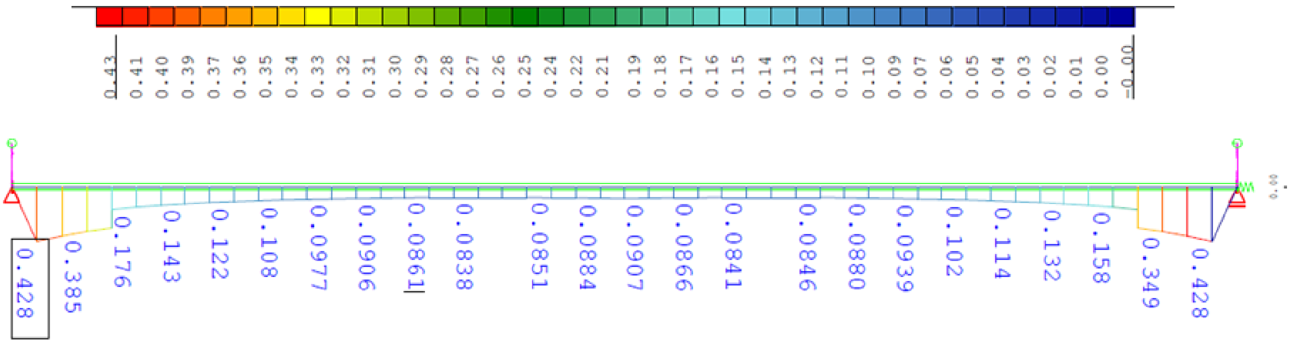


Figure 10: Maximum elongation under P+ G _ Release at 24h (en ‰)

8 FINAL STAGE ANALYSIS

Regarding the Service limit state (SLS), the stresses in the UHPFRC material are under the limits described in section §2 (Fig. 11-12).

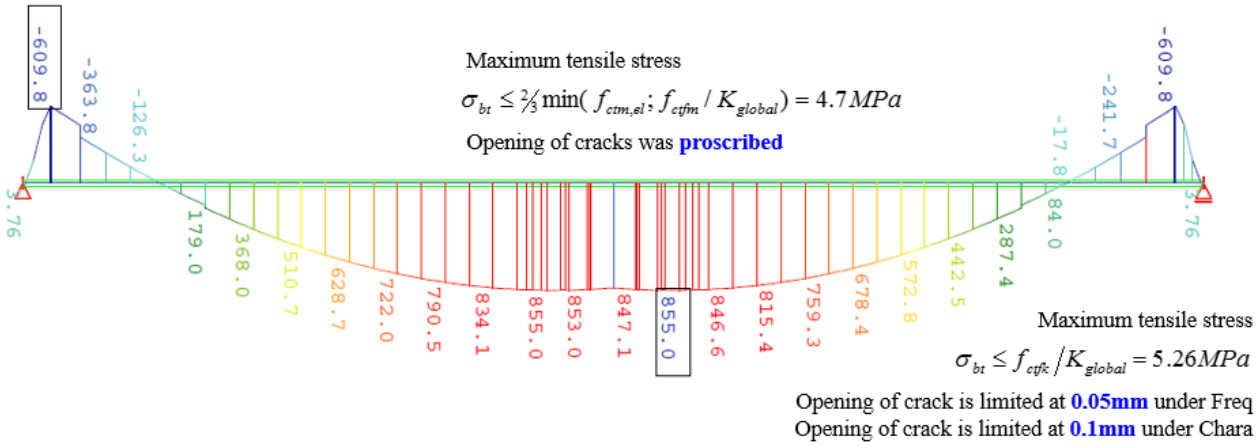


Figure 11: Bending moment under SLS combination and crack control criteria

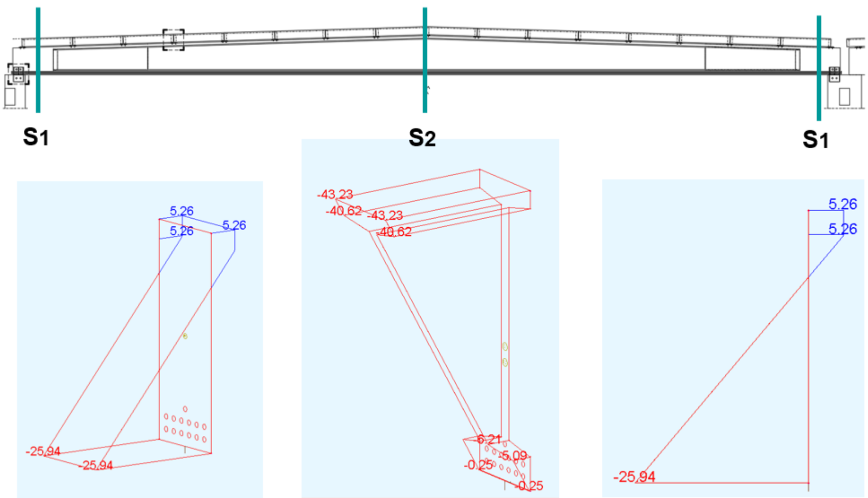


Figure 12: Maximum tensile stress under SLS combination

Regarding Ultimate Limit State (ULS), globally the level of utilization of the UHPFRC material in the element is lower than the failure limits described in chapter §2. The non-linear distribution of stress is shown in Fig. 13.

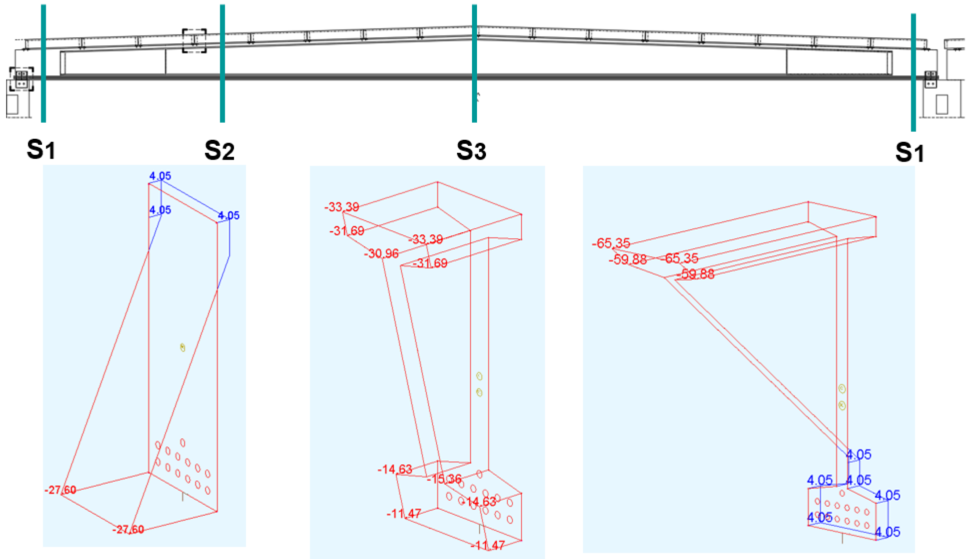


Fig. 13: Non-linear stress distribution in the 3 critical cross-sections under ULS combination

9 LATERAL STABILITY ANALYSIS

A plate model is defined to analyze the lateral buckling and plate buckling risk of the beam. The question to be solved is lateral buckling of the top flange in compression and the plate buckling of the web in shear. An additional rod bracing system is implemented to avoid the lateral stability of the beam. A constraint of lateral displacement is then modelled in the calculation every 6 m. These support conditions are illustrated Fig. 14. As a result, the lateral buckling analysis conform to the criteria given from experience: Buckling factor > 5 (Fig. 15).

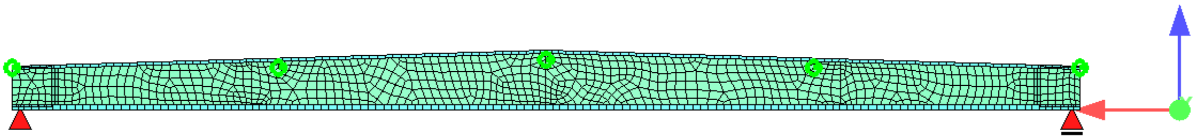


Figure 14: Plate model of beam with the visualization of mesh

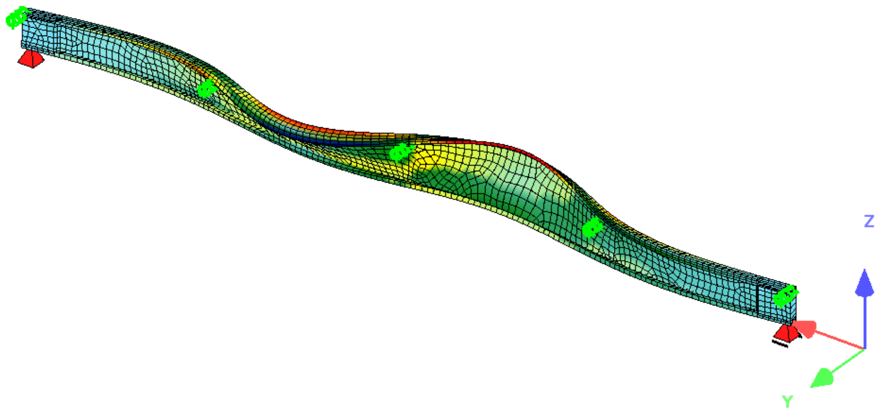
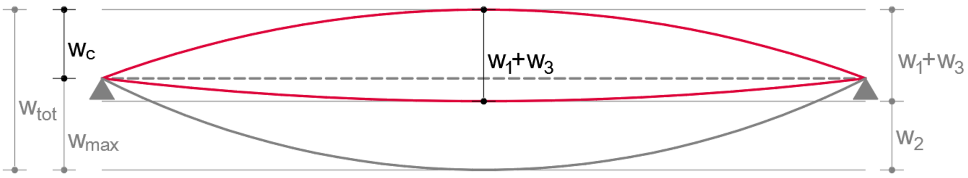


Figure 15: First mode of lateral buckling with a buckling factor = 6.95 ≥ 5 OK

10 TESTING PROCESS

Figure 16: View of final rupture due to testing process



	Theoretical analysis	Testing results
	[mm]	[mm]
Precamber under prestressing force W_c	**70.7**	**50.00**
Deflection under characteristic combination W_1+W_3	**111.6**	**67.72**
Deflection under characteristic combination taking into account the precamber W_1+W_3-W_c	**40.9**	**17.72**

Figure 17: Comparison of the deflection values

A full loading test is carried out with a mock-up scale 1 (Fig. 16). The bubbles with a depth of about 5 mm appear on the surfaces of the upper and lower flange. The bubbles of depth of about 5 mm also appear punctually inside the beam according to the observations of the photographic results of the tests. These manufacturing defects must be reduced and eliminated by the improvement of the mixing, concreting and ground, more generally, the implementation procedures. The differences of displacement were observed between the theoretical calculation and the tests (Fig. 17). The deformation values observed during the tests are less important than those found by the calculation. This discrepancy may be related to the consideration of significant safety coefficients when dimensioning the beam such as the partial safety factor applied to the Ductal FM (1.3) or the scatter in the coefficient of orientation and distribution of the fibers (1.35). During the tests, a brutal shear rupture at 2 meters from the support was observed under a load equivalent to 2 times the ULS load while the first expected rupture is a ductile rupture due to the bending moment. The rupture occurs in the area where, prior to the tests, there is cracking due to the 2 steps of concreting. This initial state results in a redistribution of shear stresses. This result confirms a fragility highlighted during the numerical analysis. At the second stage of the design, the beam web thickness was generalized to 7 cm on the entire beam.

11 CONCLUSION

This paper has been drawn up in the framework of the construction studies for a pre-tensioned high-performance concrete beam in UHPFRC DUCTAL® FM, located in Wuhan, China. The calculation process was followed by a testing process to improve the result. The following conclusions can be drawn from the comparison of both processes:

- The prestressing permits to use the full potential of the material.
- The geometrical optimization is driving the concrete design process towards typical steel design process [7] characterized by the apparition of lateral stability questions such as buckling.
- Due to the optimization of the sections the connexion design of the concrete element can become critical.
- Finally, the limitation of the optimization process is given by shear stress analysis. This parameter was understandable during the testing process.

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- Siqing Wen, **CITIC General Institute of Architectural Design and Research**
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[3] NF EN 1992-1-1 Eurocode 2 - Design of concrete structure (October 2005)

[4] National annex to NF EN 1992-1-1 - General rules and rules for buildings (Mars 2007)

[5] National Standard of PRC: Load Code for the Design of Building Structures (建筑结构荷载规范 GB 50009-2012)

[6] National Standard of the People's Republic of China: Code for Seismic Design of Building (建筑抗震设计规范 GB 50011-2001)

[7] EN 1993-1-8 Eurocode 3 – Design of steel structures – Part 1-8: Design of joints (May 2005)