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DESIGN OF OFFSHORE WIND TURBINES WITH UHPC

François-Xavier Jammes (1), Xavier Cespedes (1), Jacques Resplendino (1)

(1) Setec TPI, Paris, France

Abstract

Ultra-High Performance Concrete (UHPC) has impressive characteristics in terms of fatigue and durability which seem perfectly suited for the erection of wind turbine masts. A two-stage feasibility study is carried out to come up with an efficient design for the tower. First, an approach using linear finite elements based on the beam theory enables to define the global geometry. Then the latter is checked locally through shell elements in order to perform the necessary updates. Finally, a construction sequence shows the advantage of such a material when combined with prestressing for the erection process.

Résumé

Le Béton à Ultra-Haute Performance (BFUP) a des propriétés impressionnantes en termes de fatigue et de durabilité qui semblent parfaitement adaptées à la construction des mâts d'éoliennes. Une étude de faisabilité conduite en deux étapes propose une conception rigoureuse de la tour de l'éolienne. Tout d'abord, un modèle éléments finis basé sur la théorie des poutres permet de définir la géométrie globale de la structure. Ensuite, cette dernière est vérifiée localement afin d'assurer les mises à jour nécessaires. Enfin, un phasage de construction illustre l'avantage de ce matériau au niveau du processus de construction lorsqu'il est combiné avec de la précontrainte.

1. INTRODUCTION

With a current worldwide capacity of 200 GW constantly increasing, wind energy (Fig. 1) is becoming one of tomorrow's main sources of energy [1]. To assure the competitiveness with regard to traditional energies, a race towards more productivity and efficiency has been launched. The current tendency consists of building offshore wind farms to benefit from the best wind conditions available. It requires

Figure 1: View of a typical offshore wind farm (Siemens Press Picture)

massive structures designed for heavy loads in complex environments, making the design difficult when using traditional materials such as steel or concrete. Until now, the tower of the wind turbine, which represents by far the largest part of the structure, has been almost exclusively made of steel. The use of an Ultra-High Performance Concrete for the erection of the mast, revealing promising properties for fatigue and offshore conditions, could represent a breakthrough. It could be a more adapted material than steel or concrete and could push back the current limitations.

2. UHPC CHARACTERISTICS IN TERM OF FATIGUE AND DURABILITY

2.1 Fatigue

The fatigue of UHPC has been investigated for many years, mostly on bridges and offshore structures [2], [3]. This complex phenomenon is directly provoked by the cyclical and repetitive loadings acting on the material. Even if decisive researches are always under development, some preliminary but important conclusions can be drawn. On the one hand, the fatigue behaviour of UHPC in compression is generally considered as outstanding until the design compressive stress is reached. On the other hand, the fatigue behaviour in tension becomes problematic only when the stresses exceed 0.5 to 0.6 times the design tensile stress of the material [4]. In addition to that, if the formworks are sufficiently well realized, UHPC behaves well at the joint level between two precast sections [2], [5]. With regard to the three aforementioned criteria, the reliability of a UHPC structure for fatigue is basically achieved when the material mostly behaves in compression. In other words, as for traditional concrete, UHPC has to be designed for compression and its impressive properties in tension are penalized by fatigue limitations. Besides, fatigue represents one of the major current restrictions for the erection of offshore wind turbines. Assuming that the designer provides a structure that mostly behaves in compression, the intrinsic characteristics of UHPC clearly represent an advantage over traditional materials like normal strength concrete or steel [6].

2.2 Durability

Durability primarily refers to the direct impact of the environment on the material. Experiences have demonstrated the interesting characteristics of UHPC when compared to conventional materials. The material shows interesting abilities in harsh experimental environments very similar to offshore environments. For instance, many studies were conducted with a particular focus on chloride ion penetration, abrasion, alkali-silica reaction, freeze-thaw and scaling resistances [7], [9]. All illustrate the adaptation of the UHPC material to its surroundings. Some even reveal that UHPC could increase its properties when subjected to more unfavourable conditions. Besides, UHPC is a partially hydrated material. When subjected to aquatic environments, its hydration increases, thus decreasing its already low porosity explained by the ultra-dense C-S-H packing [8]. UHPC high rate in Portlandite also accounts for a carbonation delay. These particular aspects could have significant impacts on the future design of offshore structures in general. Substituting steel with UHPC would make any corrosion problematic obsolete and the longevity of offshore structures prolonged. However, these assumptions have to be confirmed by additional and systematic investigations.

3. AN INNOVATIVE DESIGN FOR A UHPC TOWER

3.1 Basic Assumptions

UHPC Design Parameters

According to common specifications, the values considered for the UHPC design parameters are listed in Table 1 [10].

Table 1: UHPC Design Parameters

Design Parameters for UHPC	Values
Density	2500 kg/m^3
Compressive strength	150 MPa
Flexural strength	30 MPa
Direct tensile strength	8 MPa
Young's modulus	50 GPa

Design Loads

The loads acting on the mast were considered according to static but conservative values (Fig. 2). Such an approach presents the advantage to avoid any tedious aeroelastic model to account for the turbine dynamics. The values given in Table 2 are relative to onshore conditions and were validated by expert from the National Renewable Energy Laboratory (NREL) and Det Norske Veritas (DNV).

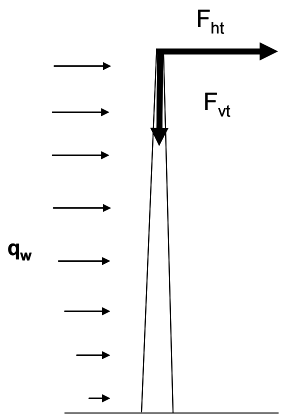


Figure 2 : Tower geometry

Table 2: Tower Design Loads

Design Load Characteristics	Load Values
Dead load	2500 kg/m^3
Lumped mass of the nacelle/rotor F_{vt}	4000 kN
Horizontal thrust at the rotor level F_{ht}	1200 kN
d Load along the tower height $q_w(h)$	Eq. (1) to (3)

The wind load was computed with equations (1) to (3) taking C=1, ρ=1.23 kg/m^3, h_0 = 0.05 m and $V_{50}(10)$ = 26 m/s.

$$q_w(h) = \frac{1}{2}\rho V(h)^2 CB \quad (1) \qquad V(h) = 1{,}5\overline{V_{50}(h)} \quad (2) \qquad \overline{V_{50}(h)} = \overline{V_{50}(10)}.\ln(\frac{h}{h_0})/\ln(\frac{10}{h_0}) \quad (3)$$

Tower Geometry

The shape is entirely defined by five parameters: the tower height h, the thickness and the outside radii at the bottom and at the top; respectively $t_{w,bottom}$, $t_{w,top}$ $r_{o,bottom}$, $r_{o,top}$. Then, the thickness and outside radii evolve linearly all along the tower height.

Based on the current dimensions for the biggest MW steel wind turbines, the following values are considered: h = 120 m, $t_{w,bottom}$ = 0,12 m, $t_{w,top}$ = 0,06 m, $r_{o,bottom}$ = 4 m, $r_{o,top}$ = 2 m.

The structure is not monolithic but composed of 40 segments of 3 m high bonded together. The link is achieved through unbonded post-tensioned cables all along the height of the tower and anchored twice every two sections. In addition, every section is locally prestressed longitudinally via bars also anchored every two sections. Finally, for construction reasons each segment is made of two-half segments linked together via an orthoradial prestressing force. (Figure 3).

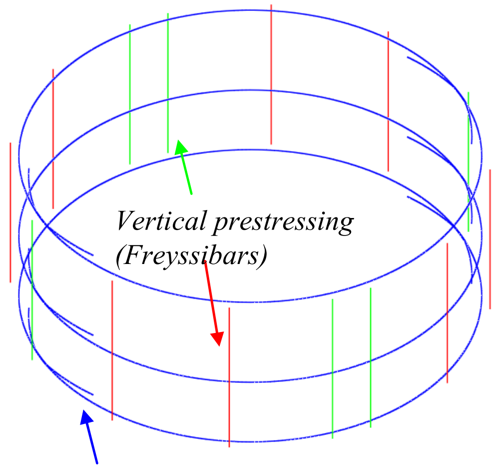


Figure 3: Layout of the local prestressing

Design Combinations

The focus being on the design of a concrete tower, a more traditional civil engineering approach, known as the Limit State Design approach, has been preferred to the International Electrical Commission recommendations based on the Design Load Cases (DLC) [11]. The design is carried out through a Serviceability Limit State (SLS) and an Ultimate Limit State (ULS). For ULS, the load coefficients are 1.35 for permanent loads and 1.5 for live loads.

3.2 Global Analysis

Model Characteristics

The tower is characterized as a cantilever beam according to the beam theory. The global prestressing force is modelled as a centred point load per segment whose value linearly increases from the top to the bottom of the mast. The maximum force at the bottom is taken so that there is no tension for a SLS scenario. The local prestressing forces are not considered in the global model.

SLS Combination

The characteristics of the main prestressing cables are directly obtained from the maximum value of the prestressing force Fp after having considered the instantaneous and time-dependent losses of pre-stress. Here Fp=155 MN requires 50 cables 13 T15S [12]. The normal stress σ obtained after prestressing is represented on Fig. 4. The design resisting stress for compression of UHPC with safety factors is given by $0.85\,fck\,/\,\theta\,\gamma b$ where fck is the compressive strength of the material. With the data from Table 1 and taking $\theta\gamma b$=1.3, it gives a design resisting stress of 98 MPa for compression [13]. According to Fig. 4, the normal stress remains between 0.3 MPa and 82 MPa. Therefore, the constraints for SLS are satisfied both in tension and in compression.

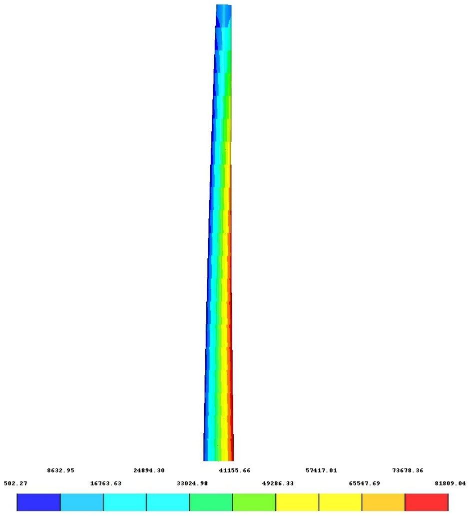


Figure 4: σ_{SLS} with prestressing

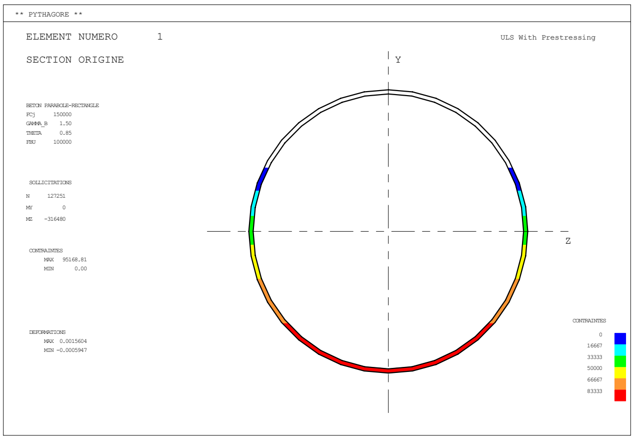


Figure 5: σ_{ULS} in a particular section obtained with a conventional linear constitutive law for UHPC

ULS Combination

For ULS, because the structure cannot resist any global tension due to the discontinuity levels at the joints, an analysis has to be carried out by assuming that the material only behaves in compression. In other words, the sections have to equilibrate the loads applied through compression exclusively without exceeding the design compressive strength of the material. The constitute law for ULS regarding the compressive behavior is taken as provided by the AFGC Interim Recommendations: a conventional linear constitute law has been considered with a yield plateau fixed at a maximum stress of $0.85\ f_{ck} / \theta\ \gamma_b = 98$ MPa (same value as the one given in the precedent subsection). For a given joint section, the compressive stresses are illustrated in Figure 5. Every section satisfies the equilibrium without exceeding the ultimate stress.

3.3 Local Analysis

Model Characteristics

Following the global analysis, the structure has to be checked in detail. The shell theory enables to describe better the effects of 3D efforts concentrated on intensely loaded thin elements. A governing section (containing the anchorage blocks of a final segment and mentioned as a level 2 section in §0) is isolated from the rest of the tower so that it can be precisely defined with the Pythagore FE software developed by Setec tpi (Figure 6). This section is considered as simply supported and contains two anchorage blocks of the longitudinal prestressing. The latter are modelled via rigid zones linked to the two corresponding points of application of the cable force. These forces are directly applied as point loads in the FE model. Concerning the local prestressing forces, the orthoradial

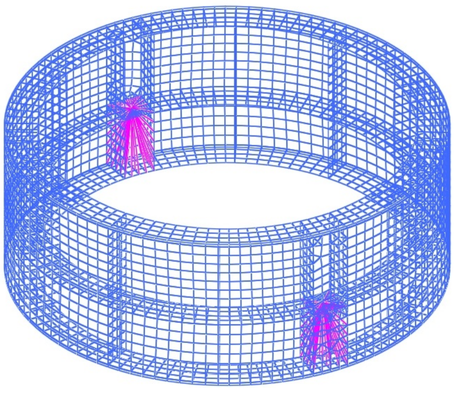


Figure 6: Geometry of the section

prestressing is defined by cable strands located into the horizontal reinforcement slabs called ribs and the longitudinal prestressing is precisely set through bars cast into the vertical reinforcement (Figure 3). The verification is carried out on two governing configurations.

Analysis of the Construction Phase

The first configuration occurs during the construction phase when the final segment is added on top of the temporary structure: the two prestressing cables are pulled from the anchorages of the segment until the basis of the tower. This phase is critical as the tensioning of the global prestressing creates tension over the anchorage and the section does not benefit yet from the global compression due to the upper segments. However, the results shown in Fig. 7 and Fig. 8 reveal that the tension never exceeds -7 MPa and remains much smaller than the current limitation on the material fixed at -30 MPa without safety factors (Table 1).

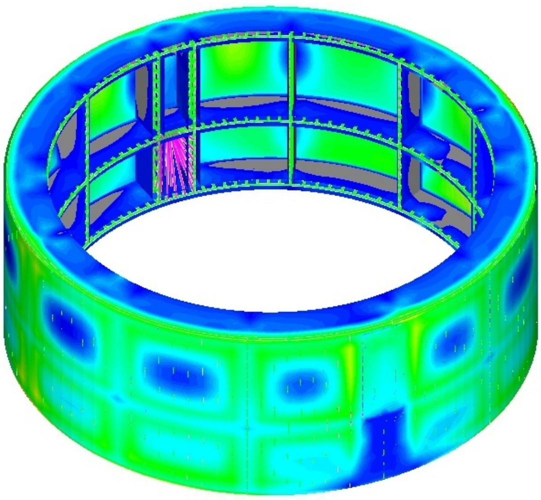


Fig. 7: σ_{xx} during the construction phase

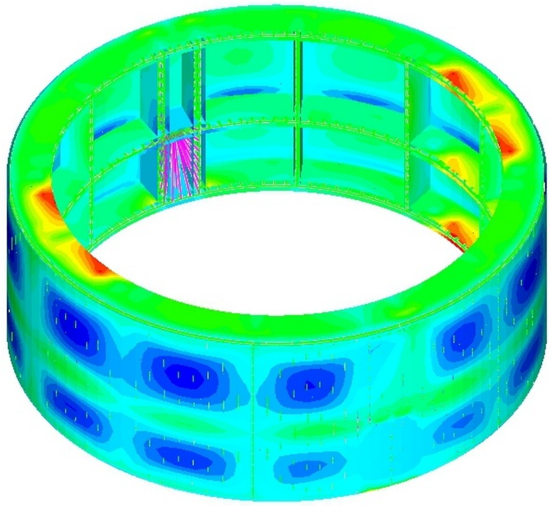


Fig. 8: σ_{yy} during the construction phase

Analysis of the Final Phase

The second phase is the final one. The loads coming from the structure above have to be modelled. This effect is computed by the application of the stress tensors –normal stress and shear- on top of the section obtained from the global model (Fig. 9). The latter is converted into forces acting on a fictive beam. These forces already contain the participation of the global prestressing not already anchored. Because of the symmetry of the structure, it is sufficient to successively consider this resulting force over two perpendicular directions to account for the random orientation of every section. These directions are the one facing any anchorage (configuration 1) and its perpendicular corresponding direction (configuration 2).

The results of the FE analysis show that the configuration 2 is the most disadvantageous (Fig. 10). It can be explained by the fact that the bending of the tower is perpendicular to the axis of the anchorages which prevents the prestressing from releasing any tension in the section. The minimum stresses provoked by a local bending are equal to - 9 MPa which roughly corresponds to 30 % of the tension design stress of the material. Most importantly, it enables the structure to behave correctly with regard to fatigue limitations (see §2.1).

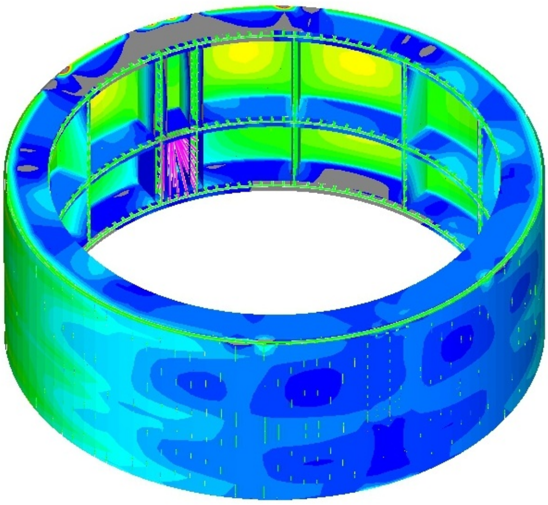


Fig. 9: σ_{xx} during the final phase
(ULS Combination – configuration 1)

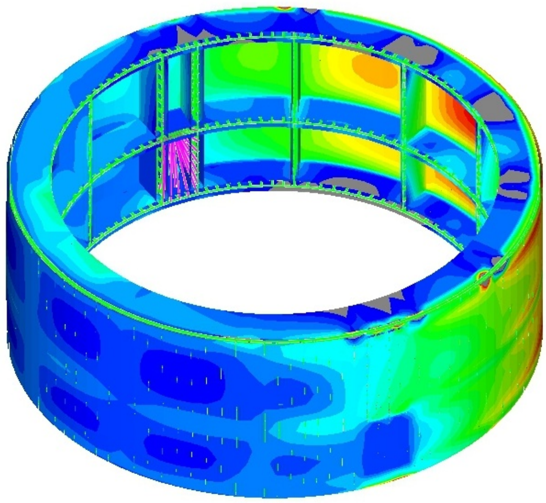


Fig. 10: σ_{xx} during the final phase
(ULS Combination – configuration 2)

3.4 Summary of the Prestressing

The characteristics of the prestressing are summarized in Table 3. The global prestressing is given by the global beam model while the local prestressing is computed from the local shell model.

Table 3: Summary of the Prestressing Characteristics

Prestressing Denomination	Prestressing Characteristics
Global longitudinal prestressing	-50 strands 13 T15S at the bottom and 10 strands 13 T15S at the top of the tower with a linear repartition -2 global anchorages per final segment (defined in the FE model)
Local longitudinal prestressing	-1 Freyssibar for every vertical reinforcement wall anchored every two temporary segment (Freyssibars Φ 40 and Φ 26.5 per reinforcement for the lower final segment) -2 local anchorages per reinforcement wall per final segment
Local orthoradial prestressing	-Monostrands for every horizontal ribs anchored at the junction section (2x10 Monostrands T15S per rib for the lower final segment i.e. 60 T15S per section) -2 local anchorages per rib

4. A REVOLUTIONARY CONSTRUCTION PROCESS

The construction sequence is of major importance in the feasibility of any project, and it is more particularly true for the erection of the highest wind turbines. Many studies have been conducted in the case of globally prestressed towers made of reinforced concrete segments [14]. The latter have been adapted to masts made of UHPC (Fig. 11). What remains essential is the easiness of the construction process with regard to transport and speed of erection.

The sequence is composed of three phases (see Figure 12). First, the half-segments cast in the shop are bonded together with the orthoradial prestressing strands in order to compose a full section or full segment. Two similar kinds of half-segments are present (level 1 and level 2 half-segments), the only difference being that the anchorage blocks are only on level 2 segments. Second, a level 1-full segment is assembled with a level 2-full segment via the local longitudinal prestressing in order to compose a two-level final segment. Finally, all these final segments are linked together through the global longitudinal prestressing anchored twice at every anchorage level (level 2 of every final segment) to compose the final structure (Fig. 11).

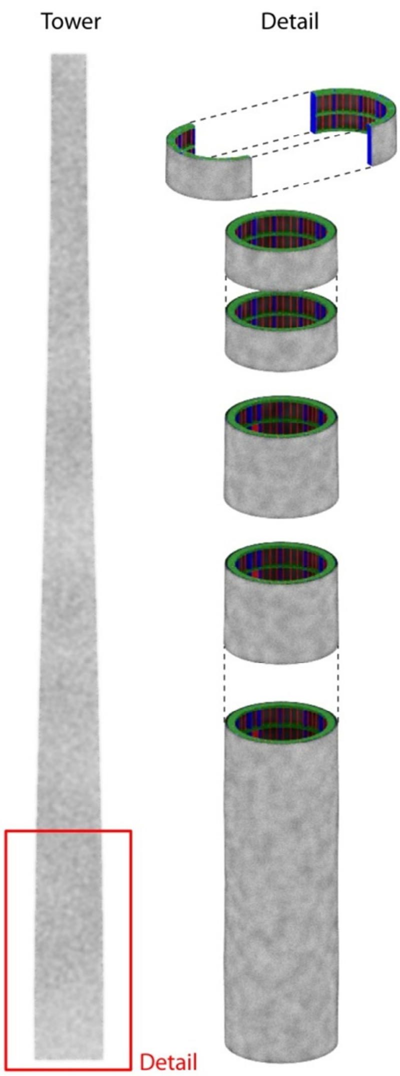
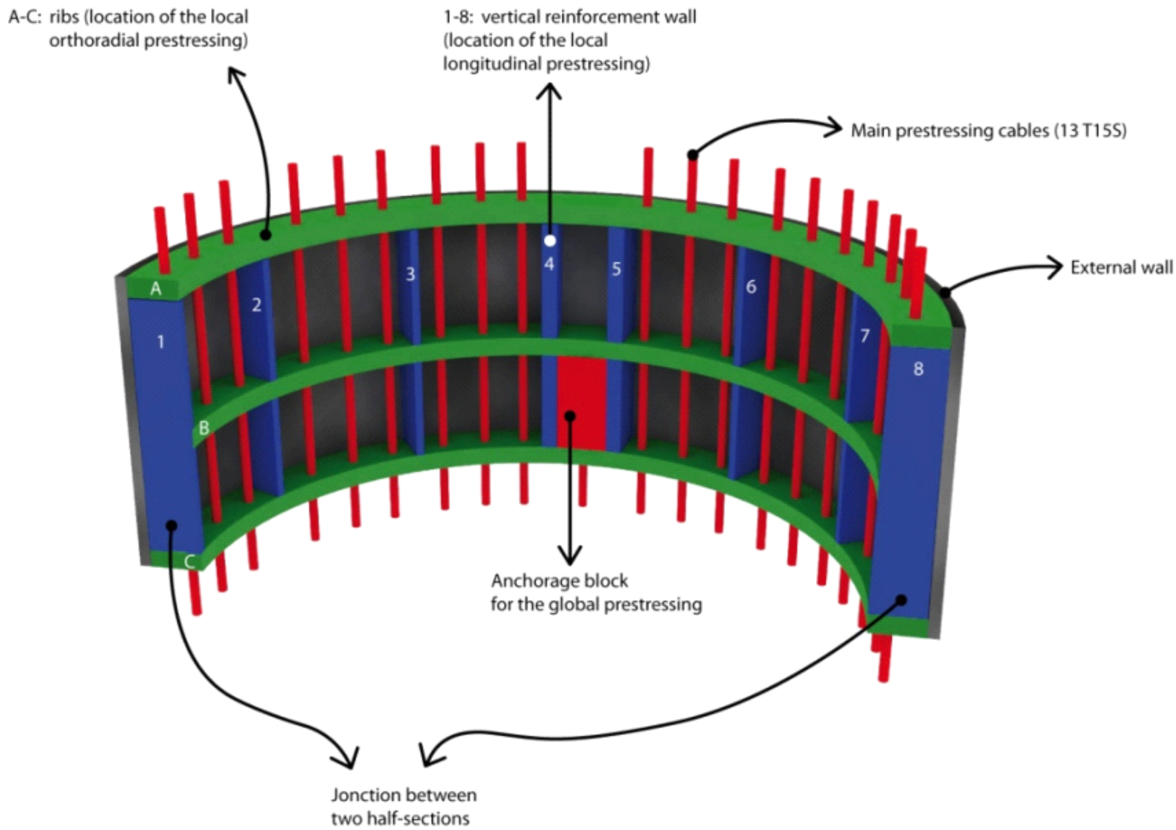


Figure 11: Construction sequence of the tower

5. CONCLUSION

Being a revolutionary material, UHPC has been used for many different civil applications in the last decade. Considering its promising properties in terms of fatigue and durability, the material seems particularly suited for the erection of the current offshore wind turbine masts. This feasibility study developed in the specific case of onshore conditions could be easily updated for later studies and more particularly for the construction of a project in an offshore environment. It would require further research to take into account more precisely both the behavior of the material (shrinkage, creep...) as well as the effects of wind gusts or wave loads.



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REFERENCES

[1] "Global Wind Report – Annual market update 2010", Global Wind Energy Council, pp.3.
[2] Toutlemonde F., *et al.*, "Experimental validation of a ribbed UHPFRC bridge deck", Second International Symposium on Ultra High Performance Concrete, 2008, pp. 771-778.
[3] Moeller A., "Use of UHPC in offshore wind turbine foundations", Second International Symposium on Ultra High Performance Concrete, 2008, pp. 863-870.
[4] Parant E., "Mécanisme d'endommagement et comportements mécaniques d'un composite cimentaire fibré multi-échelles sous sollicitations sévères : fatigue, choc, corrosion", Ph.D. Thesis, ENPC, 2003.
[5] Graybeal B., "Behavior of Ultra-High Performance Concrete connections between precast bridge deck elements", FHWA, 2010.

[6] Fitik B., Zilch K., Niedermeier R., "Fatigue Behaviour of Ultra-High Performance Concrete under Cyclic Stress Reversal Loading", Second International Symposium on Ultra High Performance Concrete, 2008, pp.529-536.
[7] Graybeal B., Strength and durability of Ultra-High Performance Concrete, PSI Inc., 2003.
[8] Ulm F.J., Acker P., "Nanoengineering UHPC Materials and Structures", Second International Symposium on Ultra High Performance Concrete, 2008, pp.3-10.
[9] Scheydt J.-C., Müller H.C., Herold G., "Long Term behaviour of Ultra High Performance Concrete under the Attack of Chlorides and Aggressive Waters", Second International Symposium on Ultra High Performance Concrete, 2008, pp.231-240.
[10] "Ductal – Technical Characteristics – Grey Ductal-FM Formulae 3GM2.0", Lafarge Ciments, 2009.
[11] "Wind turbines – Part 1 : Design requirements", IEC, 2005.
[12] "European Technical Approval No. ETA-06/0226 – Post-tensioning kit for prestressing of structures", SETRA-EOTA, 2007.
[13] "Ultra High Performance Fibre-Reinforced Concretes – Interim Recommendations", AFGC-SETRA, 2002, pp.16.
[14] "Concrete Towers for Onshore and Offshore Wind Farms – Conceptual Design Studies", Gifford/The Concrete Center, 2005.