PRECAST POST-TENSIONED CONCRETE TOWERS FOR HIGH POWER WIND TURBINES

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Abstract. *Recently developed high-power turbines for wind energy generators require long blades and tall towers with a large base diameter exceeding the width allowed for highway transportation. Therefore the mast must be transported in pieces, obtained by dividing it through longitudinal and transverse cuts, which must be assembled and connected on site. The important thickness, welding difficulties and other technological aspects, considerably increase the cost of a steel tower. Thus, structural concrete, either precast or cast on site on site, becomes a competitive alternative to current steel solutions for the mast structure, due to its lower cost and higher durability, especially in marine environments.*

In this paper, the concept of a 80m tall modular prefabricated post-tensioned high-performance concrete tower of trunk-conical form, designed to be connected to a 40 m tall steel tower on its upper end, for a 4 – 5 MW wind generator is presented. Te tower is composed by reinforced concrete shell elements that are monolithically connected by means of post-tensioning tendons and high strength steel bars

The most relevant topics related to the design, erection and monitoring a prototype currently under construction in Spain are highlighted, as well as the laboratory tests performed to assess the strength of the joints.

1. INTRODUCTION

The power output from a wind turbine depends linearly of the density of air, of a power coefficient, of the rotor swept area, and of the cubic power of the wind speed. The density of air is rather low, (1.225 kg/m3) and this leads directly to the large size of a wind turbine. The power coefficient describes that fraction of the power in the wind that may be converted by the turbine into mechanical work, and it has a theoretical maximum value of 0.593. Incremental improvements in the power coefficient are continually being sought, although these measures will give only a modest increase in the power output [1]. Major increases in the output power can only be achieved by increasing the swept area of the rotor or by locating the wind turbines on sites with higher wind speeds. Hence over the last years there has been a continuous increase in the rotor diameter of commercially available wind turbines. In addition, to take advantage of the increase of wind speed with height, very high towers are being used and new designs are in development.

Wind generators up to 1,5 MW power and up to 60 m height are currently supported on structural steel towers of a thin walled circular cross section, which can be transported to the wind farm in one or several trunk-conical tubular segments. Recently developed high-power wind generators, of more than 3 MW, require long blades and tall towers with a large base diameter exceeding the width allowed for highway transportation. Therefore the mast must be transported in pieces, obtained by dividing it through longitudinal and transverse cuts, which must be assembled and connected on site. Welding difficulties on site, handling, transport and erection time considerably increase the cost of a steel tower. Thus, structural concrete, either prefabricated or cast-in-place, becomes a competitive alternative for the mast structure, due to its lower cost and higher durability, especially in marine environments.

2. STRUCTURAL CONCRETE TOWERS: DESIGN OPTIONS

Concrete technology offers a variety of options to fulfil the structural requirements for wind towers, such as reinforced and prestressed concrete, either precast or cast-in-place. For a tower with given geometrical parameters and design loads, the optimal solution depends mainly on economical, technical and environmental aspects related to design, execution and service life.

Axial compressive loads are very small compared to bending moments in wind towers and, therefore, cracking is unavoidable under service loads when using reinforced concrete. Cracking facilitate the entrance of aggressive agents inside the concrete, so they may produce corrosion of the reinforcement. Cracking also reduces the structure stiffness and, therefore, modifies the dynamic response of the tower. In addition, steel and concrete variations of stresses under reversal loads are much higher in cracked than in un-cracked sections, thus reducing the fatigue strength under repeated loads. Thus, a strict crack control is needed not only for durability reasons, but also to limit deflections, to keep the tower natural frequencies within a range of values far from those that would produce resonance problems and to avoid fatigue problems under repeated loads. When a reinforced concrete solution is designed, the above limitations condition the thickness of the shell and use to require a considerable amount of mild steel.

Compressive stresses introduced by prestressing at horizontal sections contribute to avoid concrete cracking produced by to the loads and by the shrinkage and temperature stresses during concrete hardening. In aggressive environments, no tension is allowed under the frequent load combination, so the structure behaves elastically and, therefore, no changes in the natural frequencies of the tower due to cracking must be considered. Tendons are kept always in the compressed zone, so the variations of stresses in the prestressing steel under load reversals are small and, if a sufficient concrete cover is provided, the risk of steel corrosion is very small. In non-aggressive environments, a characteristic crack width of 0,2 mm is allowed under the frequent loads, although still no decompression is allowed under the quasi-permanent load combination. Crack opening reduces the concrete stiffness with respect to an un-cracked section, but much less than in reinforced concrete members, due to the axial load provided by prestressing. The variations of the steel stresses under repeated loads are usually acceptable, since the steel necessary to satisfy the stress and crack width limits is sufficient to satisfy the Ultimate Limit States (ULS).

Grouting the sheaths with cement grout provides an adequate protection to the steel and bond to the concrete. After grouting, the prestressing strands help to control the crack width and provide additional flexural capacity under increasing bending moments. When using unbonded tendons, the pre-stressing contribution is limited to that of an external compressive force only, and null or limited additional contribution in the ULS is considered. In this case, the amount of mild steel might be controlled by fatigue considerations.

Traditionally, monolithic concrete structures are associated to cast-in-place type of construction. However, the use of prestressing to connect different elements successfully allows obtaining monolithic structures made from pre-cast members. In addition, prefabrication may speed up the construction time with respect to one built on site and allows easier dismounting after the service life of the tower. Prefabrication has also the advantages of having a strict quality control of the geometry, of the materials properties and of the manufacturing process. This allows obtaining high performance concrete more easily than on site construction. I better finished elements, a more uniform quality of the pieces, and a reduction in the effective shrinkage. An important aspect to consider is the reduction of the risk of human accidents during construction.

However, handling, storage, transport and assembly of precast elements are aspects that highly affect the cost and may condition the adequacy of a pre-cast solution. Hence, the optimal solution in a precast concrete tower is the one which minimizes the cost of the whole process. This includes the amortization of the production facilities and moulds, labour, materials, handling, transport, assembly, maintenance, demolition and recycling costs. Prestressed concrete solutions result in lighter and less reinforced structures, what may be important if a precast solution is being used.

3. PRECAST-PRESTRESSED CONCRETE TOWER PROTOTYPE

3.1. Tower description

The concept of a pre-cast pre-stressed concrete tower prototype for up to 5 MW wind generators designed, constructed and instrumented in a wind farm in Spain is briefly described herein. The total height of the tower is 120m, being the lower 80 m made of a precast post-tensioned concrete trunk-conical ribbed shell (Figure 1) connected on its top to a steel tubular tower.

Figure 1. Mixed 80 m concrete and 40 m steel tower

For turbines of 4 to 5 MW, the external diameter of the tower at the base may range from 7,50 to 8,50m. The shell may be of constant thickness or may be ribbed, thus being thinner. In case they exist, vertical ribs may incorporate the vertical prestressing sheaths, provide global strength and stiffness to the structure and help to avoid the local buckling of the shell. Horizontal ribs also stiffen the shell and may lodge horizontal prestressing cables. The edge rings of each element incorporate the connection bolts between pieces of different storeys and between the tower and the foundation.
The tower is divided by horizontal cuts into segments, each segment being divided by vertical cuts into a number of identical elements or pieces with the shape of a roofing tile. The number of elements composing each segment and their length depend on economical, geometrical and constructional aspects. The width of a piece is limited by the dimension allowed for road transportation (4,3m in Spain). Access conditions to the site, and assembly difficulties under strong winds may limit the elements length. The weight of a piece is also limited by transport regulations and specially by the cost of the crane for assembly on site.
A typical tower may be composed by segments of length varying from 12 to 24 m long. The designed prototype is composed by 4 segments and 20 pieces, and may be mounted in around one week. The pieces are transported to the site and those corresponding to the lower segment are connected by means of prestressing bars to the foundation. The rest of segments are assembled at the footing level, around the first segment (figure 2).

Figure 2. Assembling of tower segments obtained from precast elements

Stressing of the horizontal tendons provides the necessary compression to the vertical joint to monolithically connect all pieces. Once assembled, the segment is elevated and put on top of the lower one, as shown in figure 2, to which it is connected by means of prestressing bars. Each segment is rotated with respect to the previous one, avoiding continuity of the vertical joints.

Long vertical prestressing tendons provide the vertical compression stresses in all the tower horizontal sections, needed to satisfy the serviceability and the safety of the tower. The sheaths are injected with cement grout, so they protect the tendons which become bonded to the concrete. High strength prestressing bars are placed crossing these joints and anchored at the two adjacent segments, to ensure stability during construction and to avoid decompression under service loads.

Horizontal prestressing consists on unbonded tendons which go through the vertical joints and span more than 180º. In order to provide additional compression in the vertical joint, high strength prestressing bars crossing the joint are placed between horizontal tendons. The compression introduced into the joint allows resisting the shear stresses in the joint at service, while the steel bars provide additional strength under increasing loads in order to achieve the safety level required by the current codes applicable to concrete wind towers [2,3,4,5,6]

3.2. Design aspects

The tower is considered as a cantilever, fixed at the bottom and free on top. The connection between the tower and the foundation is rigid, being the only rotations considered those due to the foundation movements as a consequence of the soil deformations.

The tower service life duration is 50 years. In a tower considered exposed to airborne salt, the following concrete characteristics are required: f_c ≥30 MPa (C30/37), water/cement ratio ≤ 0,50, minimum cover of reinforcement, c_{min} = 30mm, minimum cover of prestressing sheaths c_{min} = 40 mm, Typical values of the materials properties adopted for design are fy=500 MPa for mild steel, fps=1860 MPa for prestressing strands, and fps=1050 MPa for high-strength prestressing bars. Vertical prestressing is designed to ensure no decompression at any section under frequent loads. Maximum allowed concrete stresses are 0,45 f_c for quasi-permanent loads and 0,60 f_c for frequent loads. Calculations of short and long-term losses are made considering the actual prestressing sequence and the assumed age of concrete at prestressing. Tendons are stressed from the top of the concrete tower in one or several phases in an adequate sequence, to avoid producing tensile stresses during stressing.

The ULS checks must include flexure of any horizontal cross section, strength and buckling of the shell, and fatigue. Two layers of an orthogonal mild reinforcement grid and additional reinforcing bars should be placed at the shell and ribs, respectively, for strength reasons and to absorb tensile stresses during manufacturing, handling and transport. Detailing reinforcement at discontinuity zones, such as connections between segments, door opening and connection to the foundation is made by strut-and-tie models complemented by finite element analyses. Other studies made are: modal sensitivity analyses to obtain the effect of the variations of the concrete modulus of elasticity on the natural frequencies; spreading of the vertical prestressing force on the shell; distribution of the compression stresses in vertical joints induced by transversal prestressing; vortex shedding analysis during construction; stresses, deflections and optimal supports position during handling, storage and transport and transverse bending in the tower due to non-symmetric environmental thermal effects.

3.3. Experimental studies

Previous tests were made to define the adequate mix proportions of concrete and mortar for the required design properties. An evaluation of the friction losses at the unbonded tendons was made with a full scale model of a short segment, by measuring the necessary force to extract the wedges. With the aim of evaluating the shear strength of the designed joint, a number of shear tests with different levels of transverse compression were carried out at the Structural Technology Laboratory (LTE) of UPC[7], in Barcelona, Spain. The joint is laterally confined and has an indented surface, satisfying the requirements of EC2. The grout used for filling the joint had an average compression strength of 55 MPa and a flexural strength of 4,5 MPa at the age of loading (7 days). Figure 3 shows the test setup. The transverse pressure was introduced by means of prestressing bars. A total of 6 tests were done, grouped in 3 series of 2 tests corresponding to nominal transverse pressures of 0,25 MPa, 0,50 MPa and 0,75 MPa. The applied load, the joint opening, the vertical joint slip and the evolution of the transverse force along the test development, were measured by electronic devices.

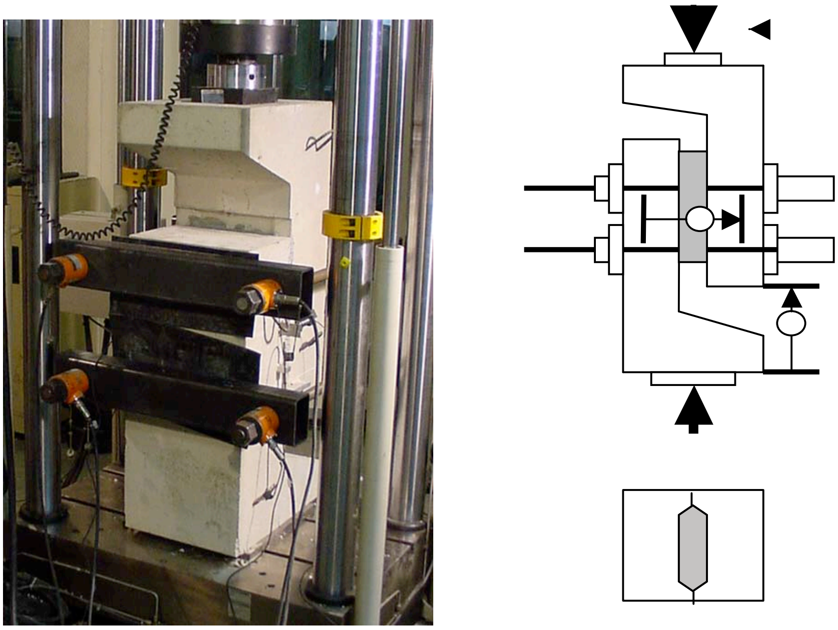

Figure 3. View and scheme of the shear test setup

The tests results showed a very stiff behaviour of the joint, with a maximum relative displacement at the peak load smaller than 0,5 mm. The failure occurred after vertical cracks tangent to the indented surface took place, showing that the strength of the shear keys governed the joint strength. The results of the joint strength experimentally obtained were in all cases higher than those provided by EC2 assuming the shear friction parameters for an indented joint, using the materials properties of the specimens tested. In addition, experimental values for the "c" and μ parameters of the shear friction model were obtained for the designed joint.
The tower and the foundation were monitored to measure accelerations, displacements, concrete and steel strains at several cross sections, footing rotations and relative movements at vertical joints, among other parameters. The results of the measures will be processed to obtain the structural response and to evaluate the actual forces introduced by the measured wind and the concomitant tip rotor speeds.

4. CONCLUSIONS

Reinforced and pretressed concrete structural elements are suitable and competitive options for towers supporting current wind turbines. The structural concept of a prototype wind tower of 120 height for wind generators up to 5 MW, already built in Spain, has been presented. The most important aspects related to its design and construction process have been underlined as well as some theoretical and experimental studies that have shown the adequacy of such type of structures. From the design and construction process experienced it can be concluded that, for high power wind energy generators, which require long blades and large base diameters, post-tensioned towers are specially suited solutions to ensure the required level of safety, durability, and functionality. Additional advantages, related to construction speed, small maintenance and easiness of disassembling, make pre-cast post-tensioned solutions competitive for economical, technical, environmental and aesthetical reasons.

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