EXPERIENCES WITH TWO OF THE WORLD’S LARGEST WIND TURBINE TOWERS

Dr.-Ing. Marc Seidel
GE Wind Energy GmbH
Holsterfeld 16, D-48499 Salzbergen, Germany
Tel.: +49-5971-980-1130, Fax -2130
Web: http://www.gewindenergy.com
Mail: Marc.Seidel@ps.ge.com

ABSTRACT:

As the size of wind turbines and the overall structure increases, more and more attention is paid to the support structures. This paper summarises experiences with two large towers for the GEWE 3.x series. The hybrid tower for the GEWE 3.6s prototype with external prestressing in the 70m long concrete section and the steel tower for the GEWE 3.2s prototype are discussed w.r.t. design and fabrication. The main advantage of the concrete tower is ability to manufacture it at nearly every site, but with the drawback of high costs. The concrete structure has to be designed carefully for dynamic properties and considering fatigue assessment for concrete and tendons. The pure steel tower is relatively inexpensive, but requires transports with large diameter towers which imposed restrictions on sites.

Keywords: Wind Turbines (HAWT)-Towers, Concrete, Steel

1 INTRODUCTION

Towers for wind turbines are an important part of the overall structure, both with regard to structural integrity and to costs. This paper deals with two large towers for Multi-MW-turbines. Economical and technical aspects are discussed.

2 HYBRID STEEL-CONCRETE TOWER

2.1 Design concepts for concrete towers

Several design concepts exist for concrete towers. For all variants, the prestressing cables (tendons) are installed after the concrete has been casted.

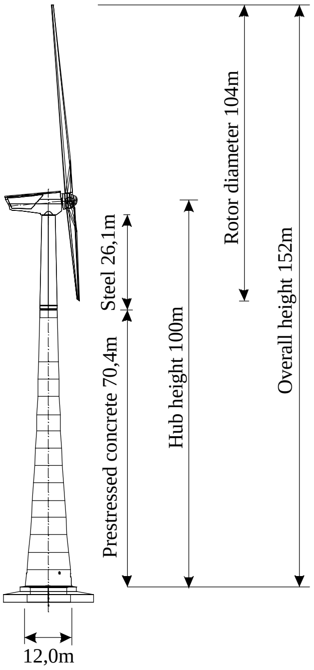

Fig. 1: Hybrid tower for GEWE 3.6 prototype

- **Post-tensioned towers:** The sheaths are filled with grout afterwards to prevent corrosion and to achieve a combined load carrying behavior of concrete and tendons. While this has some advantages with respect to static strength, there are also many disadvantages: Bond is often not perfect along the tendon, stresses are high at the anchorage of the cables and the cables may not be inspected or exchanged.
- **Post-tensioned towers without bond:** The difference to the first variant is that the sheaths are not filled. Obviously, this increases the difficulties and uncertainties for corrosion protection, but prevents highly stressed anchoring areas.
- **Externally post-tensioned towers:** This alternative uses pretensioning cables that are placed outside of the concrete wall (usually inside the tower).

External Prestressing (unbonded) has the following advantages against posttensioning (bonded prestressing):

Design:
- Higher permissible steel stress
- Lower friction losses (lower steel consumption)

Construction:
- Very few tendon supports required
- Concreting is not impaired by tendons
- No need for grouting (winter-proof construction)

Use/ Maintenance:
- Higher tolerance for dynamics loading (greater elongation)
- Corrosion protection of the tendon can be checked by visual inspection
- Prestressing force can be verified
- Prestressing force can be adjusted (re-stressing or stress relief)
- Tendons can be exchanged (individual strands or whole tendons)
- Easier demolition of the structure at the end of its service life.

2.2 Design considerations

Hybrid and concrete towers share some peculiarities that have to be considered in the design process [1]:

- **Dynamic characteristics:** Concrete towers are usually stiffer, but have the disadvantage of less well known material properties. The Young's modulus of concrete may vary about 30% around the mean value given in design codes, which results in an equal difference in natural frequency if the tower is entirely made from concrete. Thus, concrete properties have to be observed and defined more closely than in normal civil engineering if strict stiffness requirements exist.
- **Fatigue:** Fatigue for concrete structures is non linear, prohibiting fatigue check with simple load spectra. For concrete fatigue, Markov- or Rainflow matrices of the fatigue loads have to be used. Fatigue assessment is performed acc. to Model Code 90 [2].
- **Connection between concrete and steel:** This is a crucial detail in the total structure. Flanged solution, which are often used, have severe disadvantages due to the eccentric load especially for internally prestressed towers with bond.

Fig. 2 shows the connection detail as it was built for the Barrax prototype. Special features include:

- **Small eccentricity:** The lever arm between the centre line of the tower wall and the prestressing tendons was reduced with special anchors to achieve minimum additional stresses in the tendons.
- **Free movement of the tendons:** The tendons are guided through a steel pipe with large radius transitions to achieve minimum friction and lateral bearing. This does also serve to ensure small extra stresses in the tendons, leading to good fatigue performance.

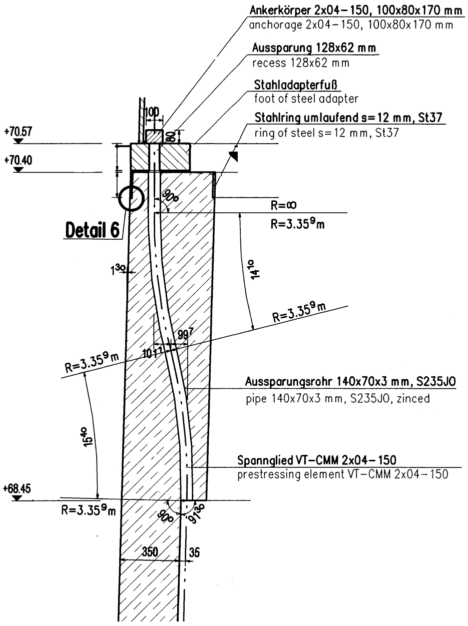

Fig. 2: Detail at tower top (hybrid connector)

3 STEEL TOWER

The tower for the GEWE 3.2s prototype with 100m HH is a conical/tubular steel tower with five sections (Fig. 3). The overall weight is app. 290t, the highest section weight is 70t.

The tower base diameter is 5700mm, the connection to the foundation is established by an anchor cage. The lower part of the tower is cylindrical with 5700mm constant diameter to meet strength and stiffness requirements.

Design driver for this tower was mainly fatigue assessment, but stiffness was also a consideration as a soft-stiff tower was aspired. A wall thickness of 40mm was required at the tower bottom due to fatigue assessment of the door section which was performed with local concepts acc. to Eurocode 3 (Fig. 4).

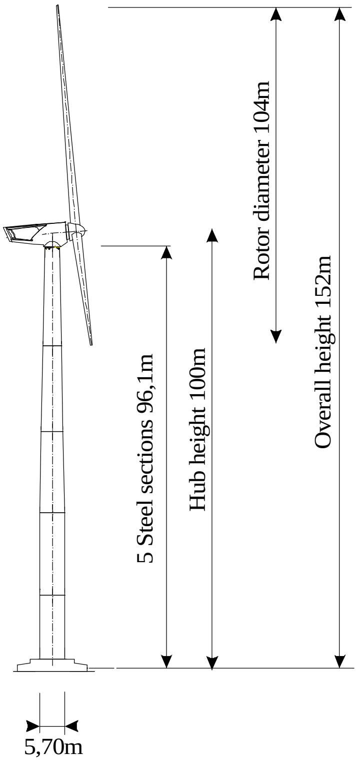

Fig. 3: Steel tower for GEWE 3.2s prototype

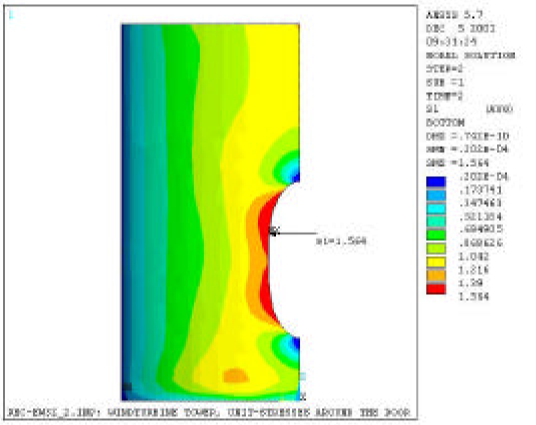

Fig. 4: Local stresses at doorframe

Using large steel towers for Multi-MW-turbines is mainly limited by transport and manufacturing capacities. Ring flanges are available up to 6500mm in diameter, but these towers can hardly be transported on streets. Nevertheless, steel towers with big diameters can be an economic solution despite high transportation costs, as concrete towers are much more expensive.

Increasing attention is paid to the flatness of the flanges as recent research has shown the negative impact of gaps between the mating surfaces of the flanges before tightening of the bolts [3]. For the 3.2 steel tower, the flanges were required to be flat within a 2mm tolerance. The manufacturer for this tower was able to achieve this tight tolerance despite the large dimension of the tower. The tolerance was measured with a laser device („Easy Laser") which is available at many manufacturers today. An example of the achieved tolerance is shown in Fig. 5.

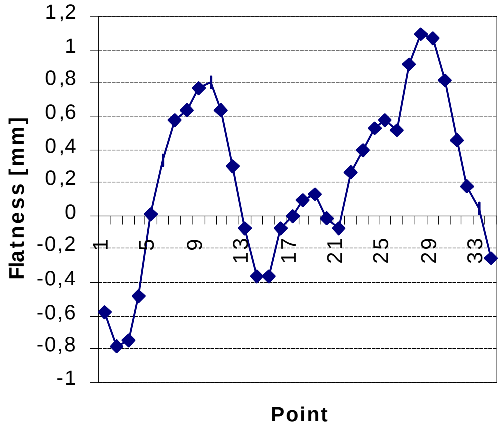

Fig. 5: Example for flange flatness reading

In Germany, high-strength friction grip bolts (HV bolts) acc. to DIN 6914 are only defined up to 36mm diameter (M36). This is not sufficient for large towers, though. The German suppliers for HV bolts (August Friedberg, Peiner, Fuchs) have extended their scope of delivery to M48 bolts, increasing the possibilities for large towers. Bolts with even larger size are already in discussion.

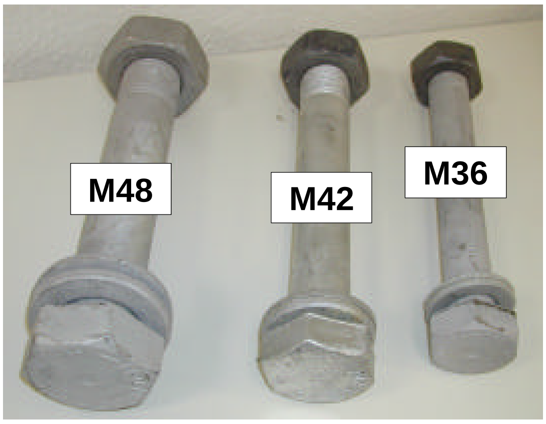

Fig. 6: Large HV bolts

Other design features include:

- **Elliptical door:** As the wall thickness at the tower bottom is driven by fatigue design, it is essential to keep stress concentrations low. This is achieved with an elliptical door shape.
- **Buckling assessment:** Buckling was assessed using latest methods from Velickov [4] (at the door opening) and Speicher/Saal (for buckling under bending loads). These assessment methods will also be included in the new DIBt-Guidelines, which will be released this year.

4 MEASUREMENT RESULTS

4.1 Natural frequency

The calculated first natural frequency of the 100m hybrid tower for the 3.6s is shown for different elastic modulus of the concrete and varying soil stiffness in Fig. 7. The measured first natural frequency of 0.57 Hz corresponds very well to the calculated value for the probable rotational stiffness of the foundation and the mean value for the elastic modulus of the concrete. As the real tower top mass is a little bit higher then the mass used for the calculation, the concrete is probably slightly stiffer than anticipated.

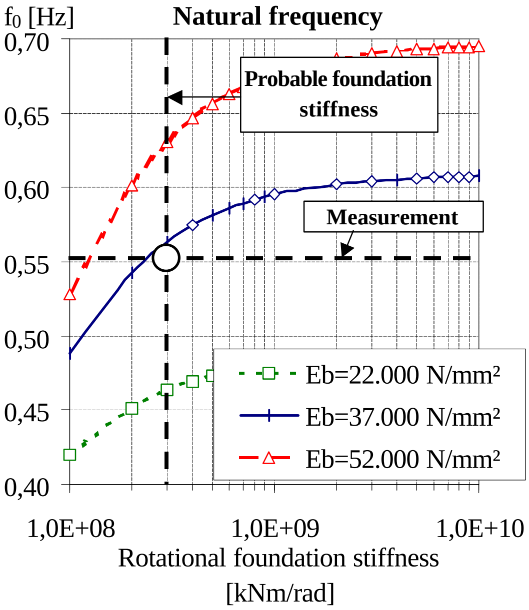

Fig. 7: First natural frequency of the hybrid tower against measurement

5 COST ASPECTS

Towers and foundations have a considerable impact on the overall material price for a large wind turbine. The overall tower/foundation cost can vary considerably, which is illustrated in Fig. 8. The total cost of the conventional steel tower is only 50% of the cost for the hybrid tower! Even though this comparison is not accurate due to the different requirements for the towers (esp. with respect to stiffness), the tendency is clear that concrete towers are much more expensive then steel towers. If transport to site is possible with large diameters above 4300mm, this will probably be the most economic choice.

Fig. 9 shows the share of tower and foundation costs for three turbines from GEWE:

- For the 1.5sl with 100m HH, tower and foundations costs are app. 31% of the total material costs (plus blades, transformer, nacelle and hub).
- For the 3.2s with 100m HH, the economic solution with a tubular steel tower leads to a decrease to 21% of the overall costs.
- For the 3.6s with 100m HH, the overall share of tower and foundation costs rises to 34% due to the rather expensive concrete part (which includes the foundation).

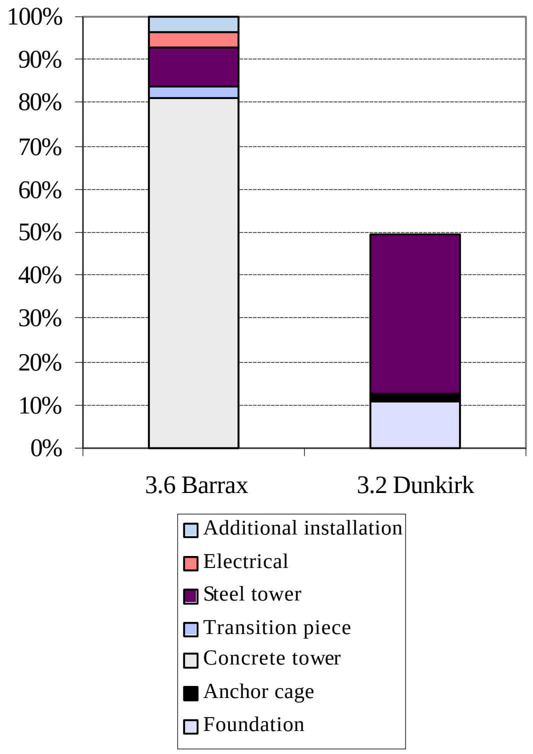

Fig. 8: Cost comparison for two 3.x towers

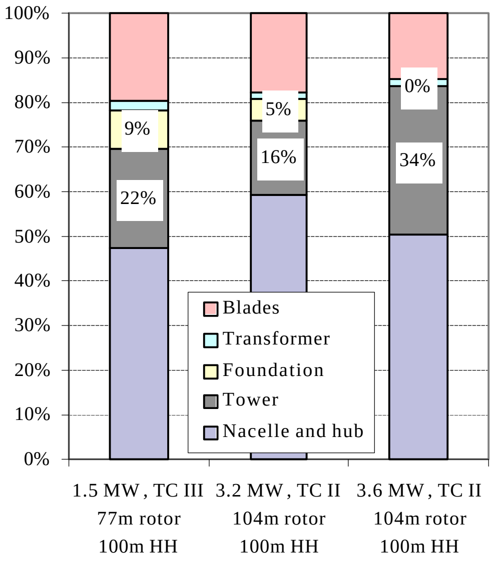

Fig. 9: Share of tower and foundation costs of total material costs

6 SUMMARY

Large towers for wind turbines include new challenges for civil engineers. Economic structures can be achieved most easily with steel towers when transport is not a problem. Hybrid towers are an alternative for other sites, but at much higher costs. Other structures are under consideration, but have not evolved to a technically feasible alternative yet.

7 REFERENCES

[1] Seidel, M.: Auslegung von Hybridtürmen für Windenergieanlagen. Beton- und Stahlbetonbau 97 (2002), Heft 11, S. 564-575.

[2] Comité Euro-International du Béton, Bulletin d’Information No. 203, CEB-FIP Model Code 1990, Final Draft, Chapters 4-10, Lausanne 1991.

[3] Jakubowski, A.: Ermüdungssichere Bemessung geschraubter Ringflanschstöße in turmartigen Stahlbauten unter besonderer Berücksichtigung von Flanschimperfektionen. Göttingen: Cuvillier Verlag 2003.

[4] Velickov, D.: Stabilität stählerner Kreiszylinderschalen mit unversteiften und umlaufend randversteiften Mantelöffnungen unter Axialdruck. Göttingen: Cuvillier Verlag 2000.