DESIGN OF LARGE SCALE WIND TURBINE TOWERS IN SEISMIC AREAS

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1. ABSTRACT

In the present paper, the design of the prototype of large tubular steel wind turbine tower in earthquake areas is presented. For the simulation of the tower's structural response, two computational models have been developed, a linear model and an overall Finite Element model in which all the details of the structure are included (flange connections, door opening, foundation, anchoring details). The tower has been designed for plastic and buckling limit states, for wind loading and for seismic loading as well, using both calculation models according to the provisions of the relevant Eurocodes. A geometric, material and boundary condition non linear analysis (including global and local shell imperfections) of the tower is performed for the wind loading. In addition, a buckling analysis to the perfect shell, and a limit load analysis to both perfect and imperfect shells has been carried out for the whole structure. Finally, the three methods proposed by the Eurocodes for the local buckling design of the shell have been compared. Concerning the design against earthquake, an eigenvalue analysis along with a response spectrum analysis has been performed according to the Eurocode 8 specifications. The behavior of the tower for earthquake loading is compared to the corresponding for wind loading one regarding both computational models.

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2. INTRODUCTION

The wind turbine tower is mainly a simple cantilever beam. However, its section forms a thin-walled cylindrical shell and therefore, several issues arise during the analysis such as the local buckling of the shell structure or the stress concentrations around the door opening which must be thoroughly examined. The prototype tower examined corresponds to a 2 MW wind turbine. The height of the tower is 76.15 m, and the total height of the wind turbine including the rotor and the blades is 123 m. The shell diameter at the base is 4.30 m and the diameter at the tower top is 3.0 m. Shell thicknesses vary from 30 mm at the bottom to 12 mm at the top. The tower is divided into three parts connected together by bolted flanges. The steel quality is S355 and the fabrication Class is B. It is worthy to note that the steel tower is embedded to the reinforced concrete foundation.

3. MODELLING THE TOWER

For the analysis of the tower, a full FE model has been developed for the tower and the foundation with all the structure details included (flange connections, door opening, anchoring detail etc.). A linear model has been also developed for the cross-checking of the results of the aforementioned advanced FE model.

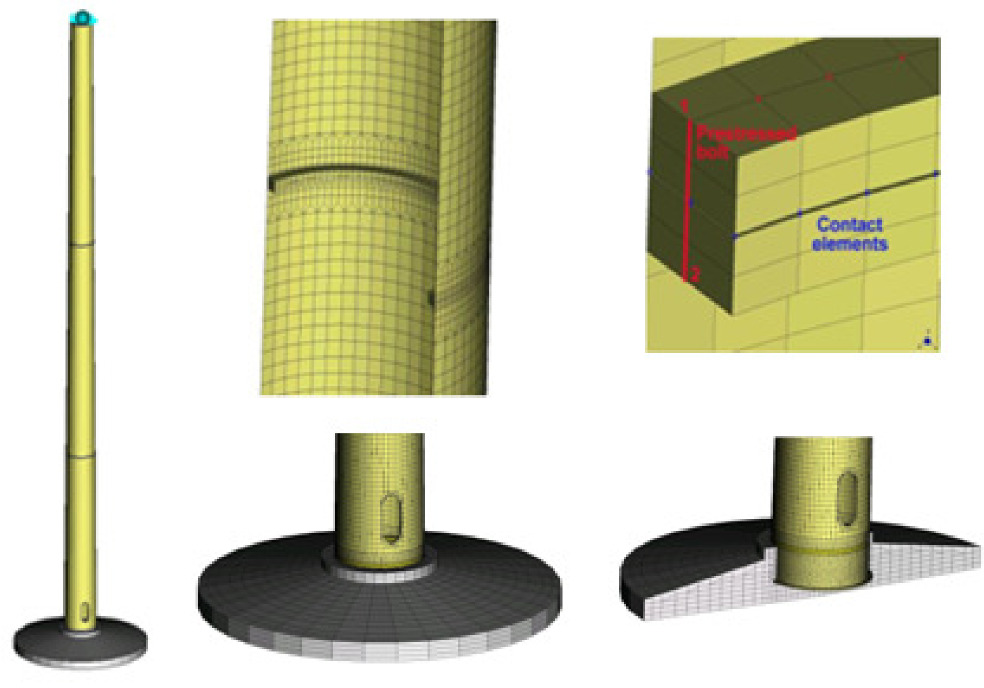

Fig. 1: The complete FE model and the detailing

The FE tower model is composed by the shell, the intermediate flanges and the embedded to the foundation skirt (Fig. 1). The reinforced concrete foundation is also modeled. The tower shell is modeled by the use of shell elements, whereas the intermediate flanges are modeled by brick elements and the foundation by using brick elements. The interfaces of the flanges are connected by means of unilateral contact elements with friction (active only in compression). The whole system is assumed to be elastically supported to the foundation base with unilateral contact conditions taking into account the soil-structure interaction.

4. LOADING

The self-weight of the tower itself is 1422 kN. The weight of the nacelle, including the blades and the rotor as provided by the manufacturer is equal to G_r=1067 kN, having the center of gravity shifted horizontally +0,725 m from the axis of the tower and vertically +0.50 m above the upper flange level. The wind loads of the tower are divided into two parts; the loads at the top of the tower and the loads over the tower stem. The loads on the tower top are provided by the manufacturer for various accidental cases. In the present work, one of the most unfavorable pair of loads will be used (V_{top}=598.74 kN, M_{top}=1665,41 kNm). The loads over the tower stem are calculated according to EC 1-1-4 for a basic wind velocity at 10m above ground of v_b=27,0 m/sec and for a terrain of category II (Fig. 2).

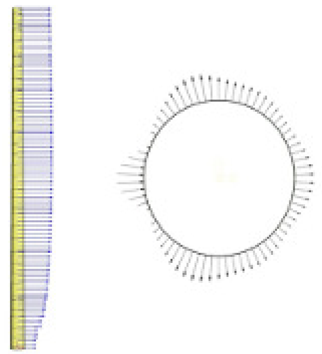

Fig. 2: Stem loading over the height and the circumference of the tower

5. ANALYSIS FOR EXTREME WIND LOADING

The overall FE model for the extreme wind loading has been analyzed using GMNIA (Geometric & Material Non-linear with Imperfections Analysis). To the linear model, an LSA (Linear Static Analysis) has been performed. The section forces for the load combination G+1,50W at the tower base are: N=2453 kN, V=1342 kN, M=88.731 kNm (as expected, almost identical to both models). The displacement at the top of the tower for the load combination G+W is 1,195 m. For the same combination, an uplift of about 20% of the foundation is recorded (Fig. 3).

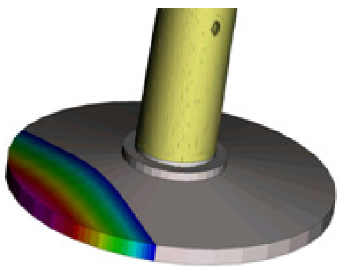

Fig. 3: Uplift of the foundation

The main difference between the linear and the FE model is caused by the wind distribution along the tower circumference, which cannot be described by the linear model. Note that in the FE model it causes on-plane deformation of the tower section. The fact that the section is forced to remain circular on the flange positions due to the stiffness of the flanges, causes concentration of circumferential stresses in the vicinity of the flange positions. Buckling analysis has been also performed to the FE model. Global and local imperfections to the model for the GMNIA and LBA analyses are introduced according to the specifications [3]. The first buckling eigenvalue for imperfection class B has been found equal to 3.44 corresponding to the local shell elastic buckling at about 6/8 of the tower height.

6. SEISMIC ANALYSIS

The wind action is in most cases the dominant loading for the design of a wind turbine tower. However, the turbine as a flexible structure having a big height and the mass of the rotor system and the blades concentrated at the top, it forms an inverse pendulum. Designing the tower for an area with high seismic risk, it must be analyzed for the seismic loads as well, in order to determine whether the latter significant for the design of the structure or not.

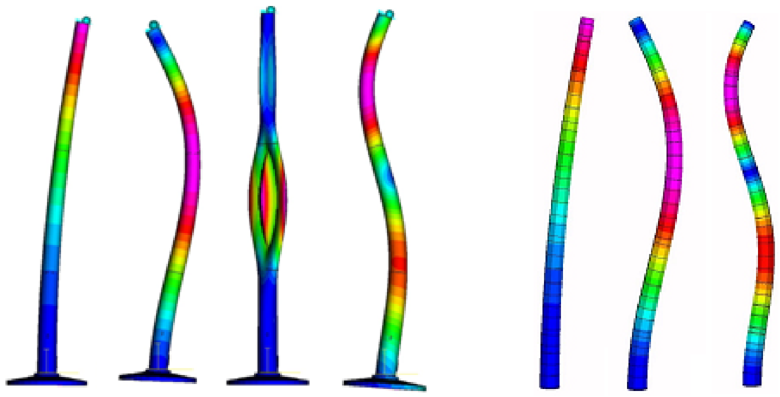

Fig. 4: Eigenshapes: FE mode (left), Linear model (right)

The seismic actions have been determined on the basis of the linear-elastic behavior of the structure with bilateral support to the ground, whereas the reference method is the modal response spectrum analysis. Firstly, an eigenvalue analysis was performed for both models. The total participating mass, not considering the contribution of the foundation is about 93% in both directions. The governing eigenvalues are shown on Table 1.

Eigen-frequency (Hz)	FE model	Linear model	Mass participation
1^{st}	0,357	0,324	61.7%
3^{rd}	2,820	2,626	14.9%
9^{th}	7,520	7,850	4.9%

Table 1: Eigenvalues of the tower

The relevant earthquake data for performing the spectrum analysis are as follows:

$$a=0{,}24,\ \ T_B=0{,}10,\ \ T_C=0{,}40,\ \ \gamma_I=1{,}40,\ \ \zeta=2\%$$

For the assessment of the effects due to the combination of the components of the seismic action, the SRSS procedure was adopted. A comparison of the seismic loading results versus the relevant for extreme wind loading is shown on Table 2.

Description	G+W	G+1.5W	G+E
f,top (mm)	1.195		530
V,Base (kN)		1.342	499
M,Base (kNm)		88.731	25.449

Table 2: Displacements and forces for wind and seismic loading

As shown, the earthquake internal forces are less than 30% of those corresponding to the extreme wind. For this type of seismic data, the critical loading for the design of the tower is the extreme wind. Even after combining the seismic load with the operating wind [6] it gives a stress state of about 75% of the corresponding for the extreme wind for the specific seismic data.

7. LIMIT STATE DESIGN

Having on hand the analysis results, the tower is designed for plastic (LS1) and buckling (LS3) limit states. For the plastic limit state, the maximum von Mises stress at any point of the tower is compared to the yield limit of the steel. According to the Finite Element model results, the maximum von Mises stress on the tower shell has been found equal to 348 MPa, at the vicinity of the door. For the main body of the tower, the maximum von Mises stress is lower, reaching 293 MPa. It is worth-mentioning that there is an almost uniform distribution along the 2/3 of the tower height. Regarding the Linear Model, the corresponding maximum stress is 251 MPa. In this case, special models to the tower details (flange connections, door position) are needed for examining the stress state on those positions.

For the buckling design of the tower shell, three methods are proposed according to [3]:

1. Design by global numerical analysis using GMNIA analysis [3] §8.7
2. Design by global numerical analysis using MNA and LBA analyses [3] §8.6.
3. Stress design to [3] §8.5.

The first two methods require advanced calculations and in particular, a limit load analysis with or without imperfections and a buckling analysis. The third one requires linear elastic calculations. The limit load of the tower is calculated for the perfect shell and for the three types of local imperfections proposed by [3].

a) Out-of-roundness imperfection for the whole shell
b) Dimple imperfection at the position of the 1st buckling eigenmode.
c) Accidental eccentricity imperfection at the position of the 1st buckling eigenmode.

The limit load analysis resulted to a limit load factor: r_{Rk} = 2,05 for the perfect shell and r_{Rk} = 1,95 for the imperfect. The tower reaches the limit load due to shell buckling at the plasticized areas near the door position. The corresponding buckling resistance ratios are 1,42 for Case 1, and 1,32 for Case 2.

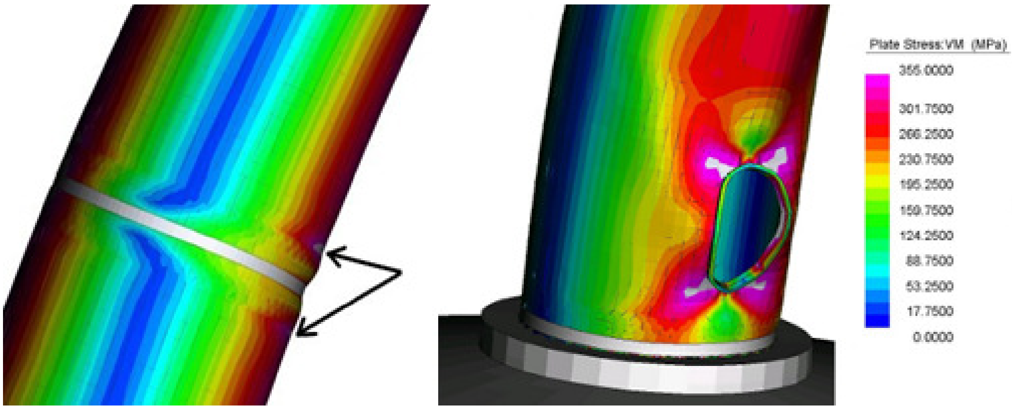

Fig. 5: Shell buckling at the limit load of the tower

Case 3 cannot be applied to an unstiffened shell. Taking into account as stiffened the parts between two connection flanges, the allowable stresses in the circumferential direction are very small comparing to the ones obtained near the flange positions (Table 3).

Courses	**L**	**R_{mean}**	**T_{mean}**	**$\sigma_{\chi,Ed}$ / $\sigma_{\chi,Rd}$**	**$\sigma_{\theta,Ed}$ / $\sigma_{\theta,Rd}$**	**$\sigma_{\tau,Ed}$ / $\sigma_{\tau,Rd}$**
Lower	21,50	2.064	24,7	236 / 327	55,0 / 13,7	103 / 57,0
Middle	26,395	1.85	18,5	208 / 304	62,0 / 7,60	74,0 / 8,0
Upper	27,425	1.603	13,6	186 / 253	68,0 / 4,95	55,0 / 8,0

Table 3: Buckling check according to [3] §8.5

8. CONCLUSIONS

A linear model can be used for the seismic design of the tower, but not for the design for wind actions because of the presence of circumferential stresses near the flange positions.

The dominant load for the design of the specific tower is the extreme wind loading. However it is not evident that this conclusion stands for all cases of wind tower design. An analysis for seismic loads must be done in all cases in order to determine whether it is significant or not for the structural design.

The results from the limit load analyses indicate that the tower is reaching the plastic limit load before approaching the elastic bifurcation point. The collapse is induced by the shell buckling at the material yielding zones, where the plasticizing von Mises stresses are components of the compressive meridional and circumferential stresses.

The preferable method for the design of the tower against buckling seems to be the global numerical analysis using LBA and MNA analyses, according to [3] §8.6. In this case the imperfections are introduced indirectly by the employment of the overall elastic imperfection factor (r_{Rov}).

The global numerical analysis using GMNIA analysis design according to [3] §8.7 is more straightforward, but at the same time it is proved to be more tedious and requires an in-depth knowledge of the applicable imperfections and the calibration factor (k_{GMNIA}).

The stress design procedure of [3] (§8.5) results in rather conservative values, especially considering the circumferential stresses. The use of stiffening rings for this type of analysis is inevitable.

9. ACKNOWLEDGEMENTS

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ΣΧΕΔΙΑΣΜΟΣ ΠΥΡΓΩΝ ΑΝΕΜΟΓΕΝΝΗΤΡΙΩΝ ΜΕΓΑΛΟΥ ΜΕΓΕΘΟΥΣ ΣΕ ΣΕΙΣΜΙΚΕΣ ΠΕΡΙΟΧΕΣ

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ΠΕΡΙΛΗΨΗ

Η παρούσα εργασία αφορά στην ανάλυση και διαστασιολόγηση χαλύβδινων πύργων ανεμογεννητριών μεγάλου μεγέθους σε σεισμικές περιοχές. Για την αριθμητική προσομοίωση αναπτύχθηκαν δύο υπολογιστικά προσομοιώματα, ένα γραμμικό και ένα πλήρες επιφανειακών και χωρικών πεπερασμένων στοιχείων. Ο πύργος διαστασιολογήθηκε για οριακές καταστάσεις LS1 (πλαστική) και LS3 (λυγισμού) για ανεμοφόρτιση και σεισμικά φορτία στο πλαίσιο των Ευρωκωδίκων. Για την ανεμοφόρτιση έχει γίνει (γεωμετρική, υλικού και συνοριακών συνθηκών) μη γραμμική ανάλυση με την εισαγωγή ολικών και τοπικών γεωμετρικών ατελειών. Επιπροσθέτως πραγματοποιήθηκε ανάλυση οριακού φορτίου για το σύνολο της κατασκευής. Όσον αφορά στον σεισμικό σχεδιασμό, αυτός βασίστηκε σε δυναμική φασματική ανάλυση, οπότε παρουσιάζεται η συμπεριφορά του πύργου σε σεισμικά φορτία σε σύγκριση με αυτήν σε φορτία ανέμου. Επίσης γίνεται σύγκριση των τριών μεθόδων που προτείνονται από τους Ευρωκώδικες για τον έλεγχο του τοπικού λυγισμού του κελύφους και εξάγονται σχετικά συμπεράσματα.

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