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ANALYSIS AND DESIGN OF THE 1 MW STEEL WIND TURBINE TOWERS AT MOUNT KALOGEROVOUNI LACONIA

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1. SUMMARY

In the present paper, some basic features of the design and the construction of a group of steel 1 MW wind turbine towers are presented. The towers are comprising the first group of the "Aeolian park" which is to be constructed on mount Kalogerovouni of Laconia at an altitude of 800-1050 m. Owner of the project is the company «ΑΛΦΑ ΑΙΟΛΙΚΗ ΜΟΛΑΩΝ ΛΑΚΩΝΙΑΣ Α.Ε.» and the project contractor is «ΕΚΜΕ Α.Ε.».

2. INTRODUCTION

The structural design of the project concerns the steel towers and their foundation (Fig. 1a). The manufacturer company «BONUS Energy A/S» has designed the rotor, the blades and all the equipment included in the nacelle, as well as the various ancillaries [2].
All units of the group are identical, having a three-bladed cantilevered rotor of 1 MW capacity, mounted upwind of the tower. The blades are made of fiberglass reinforced polyester and are fixed on pitched bearings that can be feathered 90° for shutdown purposes, in order to allow fine-tuning of the maximum power and reduce the rotor loads under extreme wind conditions. The main shaft is long and has the reaction supports symmetrically located around the tower axis for an optimum transfer of bending moments to the yaw system and the steel under-structure.
The tubular tower has a total height of 44,075m and is formed as a truncated cone, with an external diameter of 3,30m at the base and 2,10m at the top. The tower is divided for transport purposes in two parts, erected on site in succession. The sections are connected each other by means of double flanges with fully preloaded bolts. The flanges are put in the inner side of the shell, permitting easy access and maintenance of the bolts. A similar configuration is performed at the joint between the top flange and the yaw ring, while the bottom flange is fixed to the foundation by partially prestressed anchors (Fig. 7), arranged in two concentric circles on both sides of the shell (according to [3]).

The shell thickness ranges from 18mm at the base to 10mm at the top. All welds are full penetration butt welds of high quality, in order to meet the strict requirements of the fatigue assessment. The allowable fabrication tolerances of the plates are of the excellent quality class (A), due to the critical role of local buckling for the determination of the shell thickness. For the same reason, internal stiffening rings have been foreseen every 3,025m (Fig. 8).

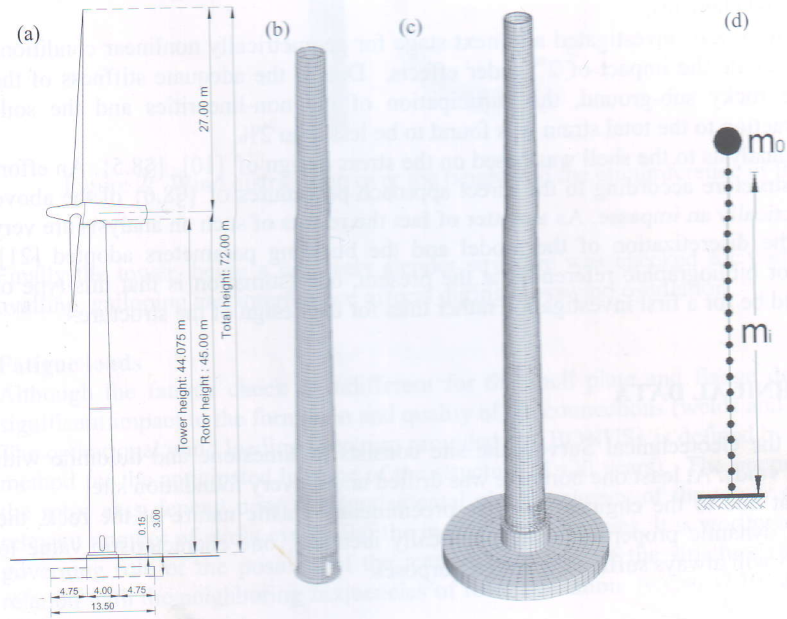

Figure 1: Structure configuration (a) and computer models (b,c,d)

The opening of the door has fully rounded corners and is reinforced by the frame and extra vertical stiffeners in order to counterbalance the effect of the local concentration of stresses. The ascent to the tower is internal by means of aluminum ladders, which are interrupted every 6,05m by platforms, made of wood panels bolted on every second stiffening ring.
The foundation consists of a circular slab footing of diameter 13.50m, and thickness 1,30m. The cylindrical pedestal at the center on which the tower is anchored, has a diameter of 4,00m and height 1,64m (Fig. 1a).

3. MODELING AND ANALYSIS OF THE STRUCTURE

The structural design was based on Finite Element Analysis by application of linear or non-linear elastic material and geometrical laws as appropriate. For this purpose the FEA software Strand7 was used, in parallel with equivalent simplified models computed by the program STATIK-3 (Fig. 1d) in order to confirm the reliability and accuracy of the results. Two different FE models were examined, for the purpose of evaluating the soil-structure interaction to the static and dynamic behavior of the tower. Both models describe

thoroughly the tower geometry (stiffening rings, flanges, door stiffeners etc) in order to take account of the local stress concentrations.
The first model where the Linear Elastic Analysis was used, considers the tower clamped on its base and it is consisting of 5208 4-node shell elements (Fig. 1b).
In the second case, an overall model for the shell and the foundation was formed, in which the foundation was described by 3270 hexahedral and tetrahedral brick elements, elastically supported on the ground by means of unilateral contact (Fig. 1c). The solution of this model demanded the use of material non-linearity algorithms due to the existence of the above special elements.
Both models were also investigated at a next stage for geometrically nonlinear conditions in order to evaluate the impact of 2nd order effects. Due to the adequate stiffness of the shell and the rocky sub-ground, the participation of the non-linearities and the soil-structure interaction to the total strain was found to be less than 2%.
The buckling analysis to the shell was based on the stress design of [10] {§8.5}. An effort to check the structure according to the direct approach procedures of {§8.6} of the above code was practically an impasse. As a matter of fact the results of such an analysis are very sensitive to the discretization of the model and the buckling parameters adopted [21]. Given the poor bibliographic references at the present, our estimation is that this type of analysis should be for a first investigation rather than for the design of the structure.

4. GEOTECHNICAL DATA

According to the Geotechnical Survey, the site consists of limestone and dolomite with random carsic voids. At least one borehole was drilled under every foundation site.
It is noted that due to the engineering wise preeminently elastic nature of the rock, the static and the dynamic properties being practically identical, one characteristic value to each parameter will always suffice for design purposes.

5. LOADINGS – LOAD COMBINATIONS

The basic loads considered for the design of the tower were the self-weight of the elements, the wind and the earthquake.

Wind loads
For the site of the project, the reference wind speed was taken as v_{ref}=36,00 m/s, as stipulated by [6] for the islands and a coastal zone of 10 km from the sea. The effects of the wind actions on the blades and the nacelle of the rotor were provided by the supplier company (BONUS) based on valid European Codes ([6] & IEO 61400) and special aeroelastic tests. The values are related to the actual natural frequencies of the tower and the blades v_1=0,60 Hz and v_2=1,35 Hz respectively.
The wind load cases for the design of the structure are determined by the combination of specific external and operating conditions. A number of situations is investigated, such as wind gust (annual or maximum), changes in wind direction, extreme influence from the consumer, grid failure and/or loss of load, fault in the control or the safety system etc. All these factors adequately associated together according to the Codes, produce three fundamental loading groups the most unfavorable of which was proved to be a semi-persistent/accidental case, having the following characteristics:

- Operating conditions: Faulty – incidents of short period recurrence
- External conditions: v_{ref}=36,0 m/s, Rotor loads: $F_{wr,o}$=282 kN / $M_{wr,o}$ =997 kNm

Along with the above loads which are applied to the rotor center, a distributed wind pressure $p_w(z,\theta)$ is performed over the height and the circumference of tower, according to [6] as shown in Fig. 2.

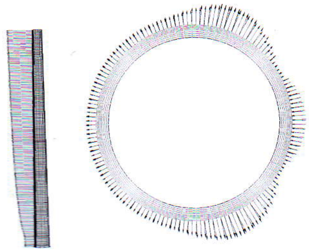

Figure 2: Wind distribution over the height and the circumference of the shell

Finally the tower, being a relatively slender structure was checked for vortex excitation, ovalling, galloping and interference effects during crosswind oscillation.

Fatigue loads

Although the fatigue check is indifferent for the shell plate and flange design, it has a significant impact to the formation and quality of the connections (welds and bolting).
The operational wind loading spectrum provided by {BONUS} is defined by the Rain-flow method for the anticipated lifetime of the structure (T=20 years). The operational loads at the rotor axis depend upon the fundamental eigenfrequency of the tower along with the relevant number of stress cycles and the mean wind velocities. It is worthy to point out the governing role of the position of the natural frequency of the structure (ν_1=0,60 Hz) in relation with the neighboring frequencies of rotor excitation ($\nu_{r,2p}$=0,37 Hz & $\nu_{r,3p}$=0,73 Hz) [2] (Fig. 3).

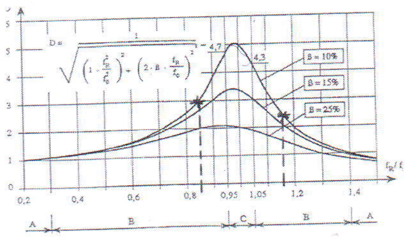

Figure 3: Evaluation of operational loads [15]

Seismic loads

The seismic analysis of the structure was based on a Spectral Response Analysis which was performed according to the Codes [4]. For the specific structure, the magnitude of the maximum stresses were about 60% compared with the ones developed by the wind loading, mainly because seismic data are propitious for the region (Seismic zone II, rocky soil) in contrast with the wind data for the same region.

6. DESIGN

The tower was designed against the following limit states [10]:

1: LS1: Plastic Limit State
2: LS3: Buckling Limit State
3: LS4: Fatigue Limit State
4: Serviceability Limit State

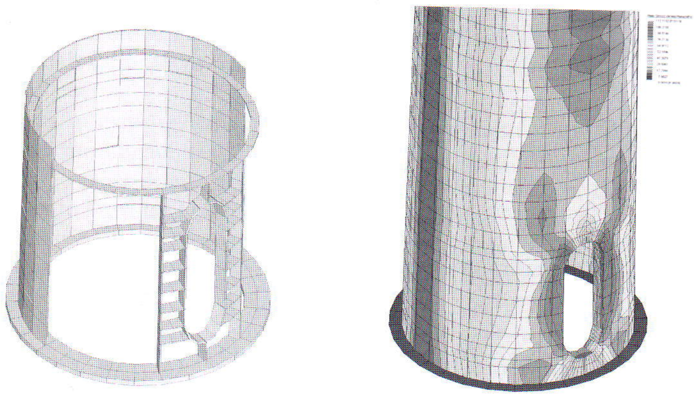

Figure 4: Von Mises stress distribution on door position

A Fortran program was developed by the authors based on Strand7 results (Fig. 4), performing the verification of the structure according to the Codes [5-20]. Shell thickness was optimized by several runs in order to obtain the best design/resistance ratio (Fig. 8). For the lower part of the tower LS1 is dominant, while in the upper part LS3 is the most significant with remarkable participation of compressive circumferencial stress near stiffening rings.
For the design of the flanged connections (rings, bolts and anchors), special detailed models were used (Fig. 5), where the two connected parts were described by finite elements connected to each other by unilateral contact. Bolts and anchors were modeled as cable elements subjected to the prestress forces. Two failure mechanisms were investigated, with and without the preloading forces and the results were used for LS1 and LS4.

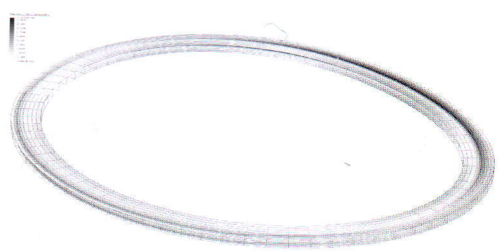

Figure 5: Middle flange model

7. FOUNDATION

As expected, a complex stress state is developed in the foundation slab and pedestal resulting from the loads coming from the tower, and the prestressed anchor bolts. For this purpose, a detailed brick element model was developed (Fig. 6), in which the anchor system was fully represented. Due to the symmetry of the loading and the structure, only the half body of the foundation is described, consisted of 3360 brick elements that is, the concrete, the flanges and the non-shrink mortar (Emaco). The connection between the elements and the support to the ground was realized via unilateral contact elements. The prestressed anchors were modeled as cable elements connecting the steel flanges.

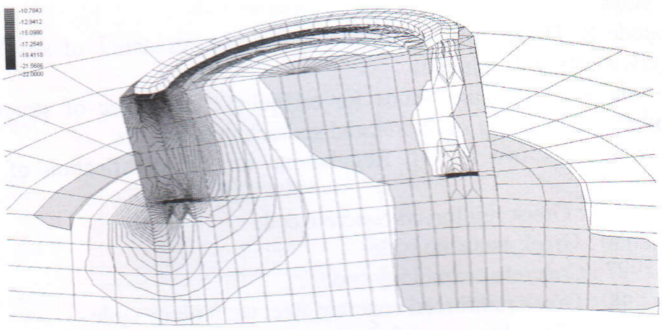

Figure 6: σ_{zz} stress distribution over the foundation model

8. CONCLUSIONS

The main conclusions of the performed analysis are the following:

a) The use of a simplified linear static model is sufficient for the calculation of the basic response and the eigenvalues of the structure, but not for the ULS design, because it neglects the local stress concentrations.
b) The calculation regarding buckling analysis of the shell inserts a high level of ambiguity in the results, given the present status of the relevant Eurocodes.
c) The extreme wind is the main loading combination for the design of the structure in this specific case. The seismic design could be critical when constructing in a seismically hazardous area (zone III or IV), away from the coastal zone, in a medium or soft soil.
d) For the fatigue design, the dynamic characteristics of the structure are critical.

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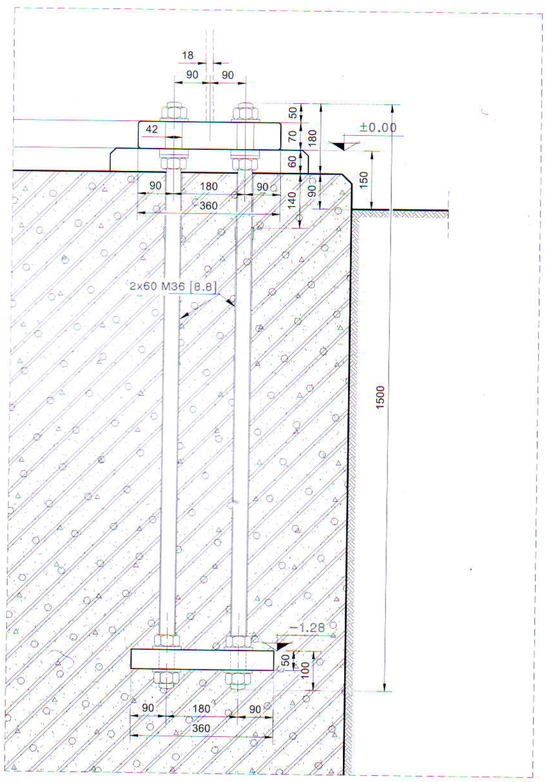
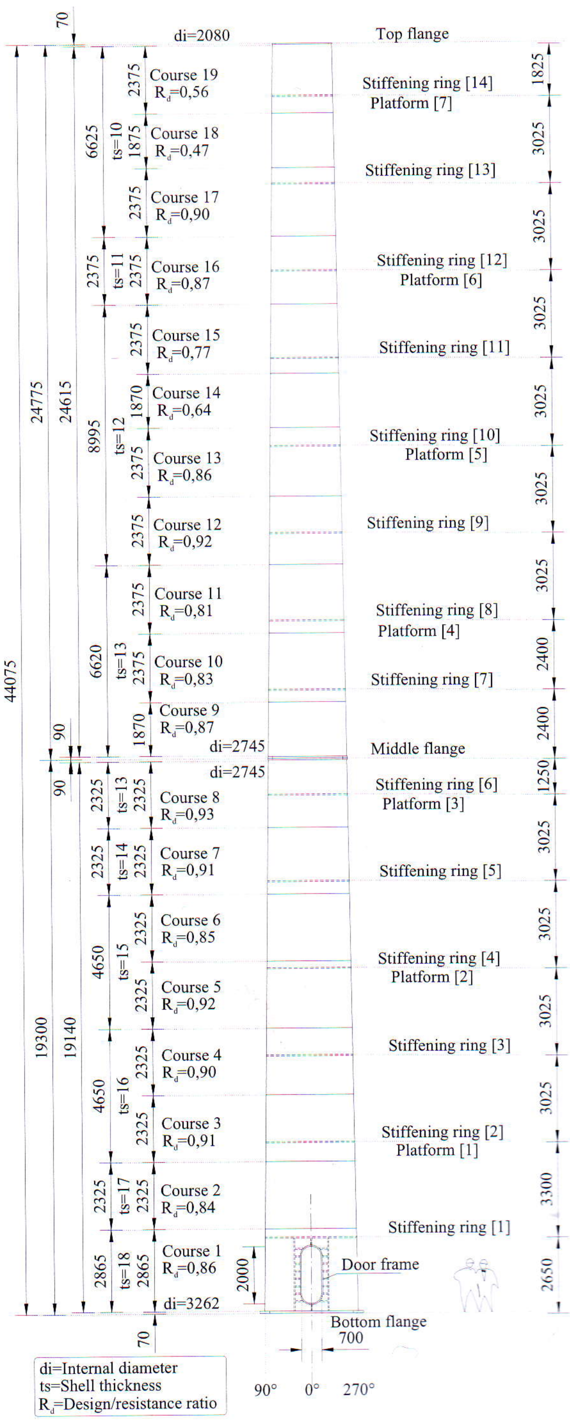
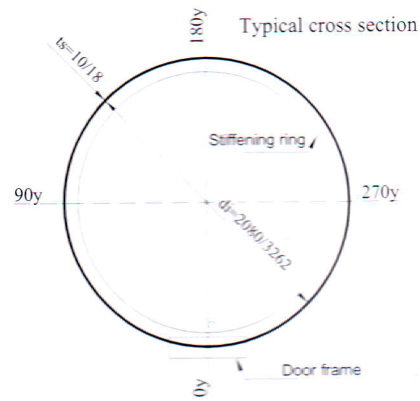


Figure 7: Anchor detailing





MATERIAL LIST			
No	Description	Quality	Code
1	Steel plates	S 355 J2G3	EN 10025
2	Bolts	10.9	ISO 7411
3	Anchors	8.8	ISO 7411
4	Nuts	—	ISO 4775
5	Washers	—	ISO 7415

PART LIST			
No	Description	Weight (kg)	
1	Shell: t=10mm	3.561	
2	Shell: t=11mm	1.483	
3	Shell: t=12mm	6.538	
4	Shell: t=13mm	7.741	
5	Shell: t=14mm	2.290	
6	Shell: t=15mm	5.070	
7	Shell: t=16mm	5.640	
8	Shell: t=17mm	3.089	
9	Shell: t=18mm	3.921	39.333
7	Stiffening rings	1.891	
8	Door stiffeners & frame	397	2.288
9	Topflange	238	
10	Middleflange	1.281	
11	Bottomflange	2.038	
12	Anchor ring	1.456	5.013
13	Bolts	343	
14	Anchors	1.623	1.966
15	Miscellaneous	1.400	1.400
		Total	50.000

Figure 8: Tower elevation

ΑΝΑΛΥΣΗ ΚΑΙ ΣΧΕΔΙΑΣΜΟΣ ΧΑΛΥΒΔΙΝΩΝ ΠΥΡΓΩΝ ΥΨΟΥΣ 44m ΑΝΕΜΟΓΕΝΝΗΤΡΙΩΝ 1 MW ΣΤΟ ΚΑΛΟΓΕΡΟΒΟΥΝΙ ΛΑΚΩΝΙΑΣ

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ΠΕΡΙΛΗΨΗ

Στην εργασία αυτή παρουσιάζονται ορισμένα βασικά στοιχεία τον σχεδιασμού τυποποιημένης σειράς μεταλλικών πύργων ύψους 44 m ανεμογεννήτριας ισχύος 1 MW, οι οποίοι αποτελούν την πρώτη ομάδα στο Αιολικού Πάρκου που προβλέπεται να κατασκευαστεί στο Καλογεροβούνι Λακωνίας σε υψόμετρο 800-1050 m.

Ο πύργος έχει κυλινδρική μορφή με μεταβλητή διατομή (3.30-2.00 m) και μεταβλητό πάχος τοιχώματος καθ' ύψος, είναι δε κατασκευασμένος από χάλυβα [S355 J2G3]. Το συνολικό τον ύψος είναι 44.075 m. Η συνολική μάζα τον συστήματος ρότορα -πτερυγίων είναι 69.1 t με εκκεντρότητα 0.5 m ως προς τον κατακόρυφο άξονα του πύργου. Για λόγους μεταφοράς και ανέγερσης, ο πύργος αποτελείται από δυο τμήματα 19.30 και 24.775 m συναρμολογούμενα στο εργοτάξιο με διπλό δακτύλιο (φλάντζα) και πλήρως προεντεταμένους κοχλίες. Ο πύργος αγκυρώνεται στο φορέα της θεμελίωσης με διπλή σειρά μερικώς προεντεταμένων αγκυρίων, πακτωμένων με την βοήθεια ενός εγκιβωτισμένου χαλύβδινου δακτυλίου. Για την αντιμετώπιση του τοπικού λυγισμού έχουν προβλεφθεί εσωτερικοί δακτύλιοι ανά 3.025 m καθ' ύψος. Η κύρια επιπόνηση της κατασκευής προέρχεται από τη φόρτιση τον ανέμου που αντιστοιχεί σε μέγιστη ταχύτητα αναφοράς Vreεε =36.00 m/s (EC 1-2-4).

Ο αντισεισμικός υπολογισμός έγινε με την δυναμική φασματική μέθοδο. Ο έλεγχος σε κόπωση πραγματοποιήθηκε για σειρά ισοδυνάμων στατικών φορτίων W_0 που αντιστοιχούν σε ταχύτητες ανέμου 6 - 26 m/s (μέθοδος Rain-flow). Για την προσομοίωση και στατική ανάλυση τον πύργου χρησιμοποιήθηκαν δύο μοντέλα πεπερασμένων στοιχείων (με 5208 επιφανειακά 4-κομβα στοιχεία κελύφους και με 3270 εξάεδρα και τετράεδρα brick elements με ελατηριακή στήριξη επί τον εδάφους με συνθήκες μονόπλευρης επαφής αντίστοιχα). Η διαστασιολόγηση της κατασκευής έγινε σύμφωνα με τον EC3-1-6. Ως κυρίαρχη φόρτιση στην διαστασιολόγηση τον κελύφους αναδείχθηκε η ανεμοφόρτιση, ενώ η ένταση του σεισμού προέκυψε περίπου κατά 40% μικρότερη αυτής τον ανέμου. Η εργασία ολοκληρώνεται με την παρουσίαση ορισμένων κατασκευαστικών λεπτομερειών, ενώ στην τελευταία παράγραφο αναπτύσσονται συνοπτικά τα συμπεράσματα που προέκυψαν από την εκπόνηση της παρούσας μελέτης.