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Analysis and design of the prototype of a steel 1-MW wind turbine tower

I. Lavassas, G. Nikolaidis, P. Zervas, E. Efthimiou, I.N. Doudoumis, C.C. Baniotopoulos *

Department of Civil Engineering, Aristotle University of Thessaloniki, GR-54124 Thessaloniki, Greece

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Abstract

In the present paper, some basic features of the analysis and the design of the prototype of a steel 1 MW wind turbine tower are presented. The structure is 44,075 m high and has a tubular shape with variable cross section and variable thickness of the wall along its height. The steelwork has been manufactured by steel quality S355J2G3. For the simulation of its structural response, two different finite element models have been developed. Based on the results of the latter analyses, the design of the steel tower for gravity, seismic and wind loadings has been performed according to the relevant Eurocodes. In particular, regarding seismic loading, a dynamic phasmatic analysis of the tower has been carried out according to the Greek Antiseismic Code (EAK 2000). The structure has been also checked against fatigue by applying the respective Eurocodes methodology. In the last part of the paper, some points that concern the previous analysis and the respective design procedure are discussed in detail.
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Keywords: Wind turbine tower; Steel tower; Static analysis; Seismic analysis; Fatigue analysis

1. Introduction

During the last decades the demand for sustainable energy production has led to a plethora of innovative technological solutions. The forecast of the fuel shortage in the near future combined with the negative environmental impacts caused by the use of the traditional electricity production methods forced all those involved in the energy production field to start exploring new directions in energy production. The so-called clear energy sources (e.g. the wind and the sun) recently became the basic subjects of these investigations. Among the latter efforts, specialized infrastructure, the so-called Aeolian parks, aiming to produce energy from the wind play a predominant role on the scene of clear energy production. As a matter of fact, Aeolian parks are composed of families of wind turbines supported on steel towers. The enrichment of the theoretical background, the understanding of the technical elements and the development of the necessary software for such technological applications is nowadays the result of a detailed and multiple study of these kinds of structures (cf. e.g. [1,2]). It is worth underlining here, that as a result of the previous investigations, the use of steel for the construction of wind turbine towers increased very rapidly over recent years. In the following paragraphs, selected results from the study of such a steel wind turbine tower prototype are presented in detail.

2. On the geometry of the structure

The steel tower under investigation is the prototype of a group of steel wind turbine towers that is nowadays under construction in the wind park at Mount Kalogerovouni in Laconia, Greece at an altitude of 800–1050 m. The wind turbine manufacturer has designed the rotor, the blades and all the equipment included in the nacelle, as well as the various ancillaries. In particular, a 1 MW capacity three-bladed cantilevered rotor is mounted on the top of the steel tower. The blades are made of

* Corresponding author. Tel.: +302310-995753; fax: +302310-995642.
E-mail address: ccb@civil.auth.gr (C.C. Baniotopoulos).

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reinforced fiberglass polyester material and are fixed on pitched bearings that can be feathered 90° for shutdown purposes, both to allow for fine-tuning of the maximum power and reduce the rotor loads under extreme wind conditions. The main shaft is long enough with supports symmetrically located around the tower axis aiming at an optimum transfer of the bending moments to the yaw system and the steel structure.

The tubular tower has a total height of 44.075 m and is formed as a truncated cone with an external diameter of 3.30 m at the base and 2.10 m at the top (Fig. 1a). For transportation purposes the tower has been divided into two parts that are easily erected on site. The sections are connected to each other by means of double flanges with fully preloaded bolts. The flanges are designed at the inner side of the shell, thus permitting easy access for the maintenance of the bolts. A similar configuration has been used at the joint between the top flange and the yaw ring. The bottom flange has been fixed at the foundation by partially prestressed anchors arranged in two concentric circles on both sides of the shell.

The shell thickness of the steel tower ranges from 18 mm at the base to 10 mm at the top. In order to meet the strict requirements of the fatigue design, all welds have been designed as full penetration butt welds of high quality. Due to the critical role that local buckling plays in the determination of the shell thickness, the allowable fabrication tolerances of the plates are of the excellent quality class (A). For the same reason, internal stiffening rings have been foreseen to be located in close distances and in particular, every 3.025 m.

In order to counterbalance the effect of the local concentration of stresses, the opening of the door has been designed with fully rounded corners and has been reinforced by both a frame and a number of extra vertical stiffeners. The ascent to the tower is internal by means of aluminum ladders, interrupted every 6.05 m by platforms made of wood panels bolted on every second stiffening ring.

The steel tower foundation consists of a circular slab footing with a diameter equal to 13.50 m and a thickness of 1.30 m. The central cylindrical pedestal where the tower is anchored has a 4.00 m diameter and 1.64 m height.

3. Analysis of the steel tower response

The design of the steel tower was based on a detailed Finite Element analysis performed by applying appropriately chosen linearly or non-linearly elastic material and geometrical laws. To this end, the FEA software Strand7 has been used along with equivalent simplified models computed by the STATIK-3 software (Fig. 1b, c, d). The reason for combining both detailed and simplified FE models was the assessment of the reliability and accuracy of the numerical results.

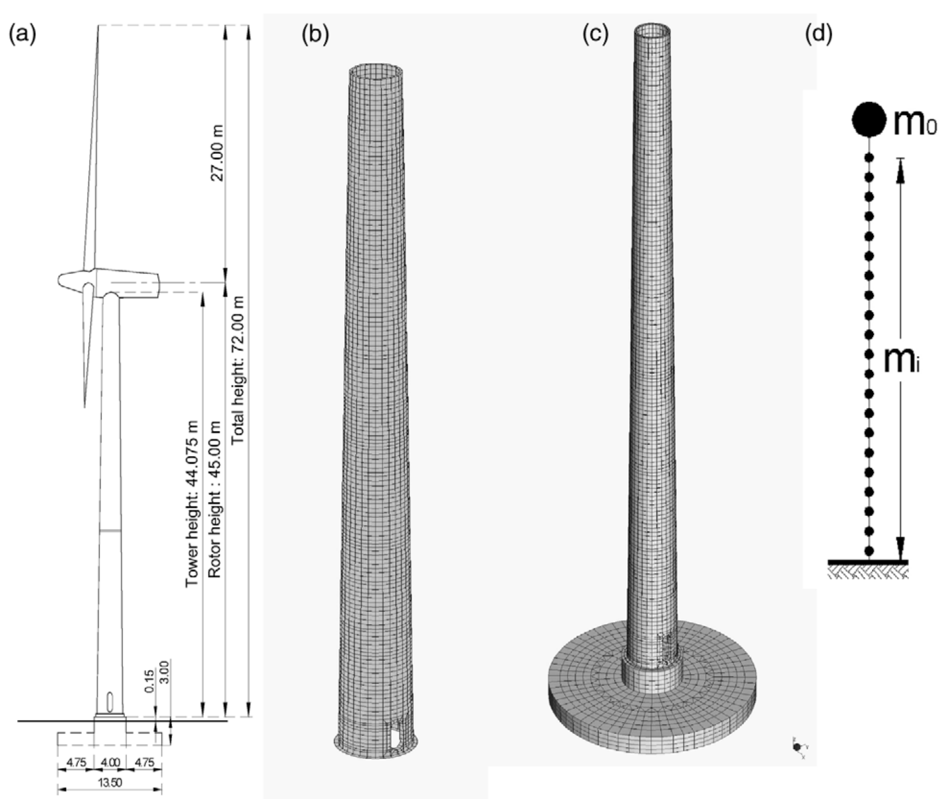


Fig. 1. Structure configuration (a) and F.E. models (b, c, d).

In particular, in order to evaluate the effect of the soil-structure interaction upon the static and dynamic behavior of the steel tower, two different FE models have been investigated. In order to take into account any possible local stress concentration, both models have been developed in such a way that they precisely describe the tower geometry including details e.g. the stiffening rings, the flanges and the door stiffeners.

In the first FE model where Linear Elastic analysis was used, the tower consisting of 5208 4-node shell elements has been considered clamped in its base (Fig. 1b).

In the second case, a complete FE model for both the tower and its foundation was formulated. The tower foundation has been described by 3270 hexahedral and tetrahedral brick elements, elastically supported on the ground by unilateral contact elements (Fig. 1c). Due to the existence of the latter unilateral contact elements, the numerical treatment of this model required the application of non-linear algorithms.

Having scope to evaluate the impact of the second order effects on the structure at hand, both models have also been investigated at the next stage for geometrically nonlinear conditions. Due to the adequate stiffness of the shell and the stiff (rocky) underground, the participation of the aforementioned non-linearities and the soil-structure interaction into the total strain state of the tower was calculated as less than 2%, thus not affecting the overall structural response of the tower. The buckling analysis of the shell structure was based on the stress design method dictated by Eurocode 3, Part {§8.5} [3]. An effort to alternatively check the structure, according to the direct approach procedures of Eurocode 3, Part {§8.5}[3], was practically at an impasse. As a matter of fact, the results of such an analysis appeared to be very sensitive with respect to both the discretization of the model and the adopted buckling parameters. It should, however, be mentioned that this type of analysis should be used for preliminary investigation of the shell response and not for its final design [4].

4. Load combinations

Following what the Eurocodes dictate, the basic loads considered for the design of the steel tower were the gravity loads (i.e. self-weight of the structural elements), the wind pressure and the earthquake loading [5].With respect to wind loading (being critical to the dimensioning of the steel structure), the site where the towers are constructed significantly influences some of the wind load characteristics. For the site of the project, the reference wind speed was taken as v_{ref} = 36.00 m/s (as dictated by Eurocode 1 for islands and a coastal zone of 10 km away from the sea) [5]. The effects of the wind action on the blades and the nacelle of the rotor based on the existing European Codes and ad hoc aeroelastic tests have been provided by the manufacturer. The computed critical values related to the actual natural frequencies of the tower and the blades were $\nu_1 = 0.60$ Hz and $\nu_2 = 1.35$ Hz respectively.

The design wind load cases have been determined by combining specific external and operating conditions. A series of situations have been investigated, as are e.g. the annual or maximum wind gust, the changes of the wind direction, the grid failure and the loss of load and the possibility of fault in the control or the safety system. As a matter of fact, all these factors adequately associated together within a Eurocodes design framework logic, produce three fundamental loading groups; the most unfavorable of them was the 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 applied to the rotor center, a distributed wind pressure $p_w(z,\theta)$ computed by applying the analytical relations given in Eurocode 1 has been applied along the height and around the circumference of the tower (Fig. 2) [5]. The tower being a relatively slender structure, it has in addition been checked for vortex excitation, ovalling, galloping and interference effects during crosswind oscillation.

In particular, in order to design the structure against fatigue, the vortex excited vibrations should be considered. In this case though, the check with the operational wind loading is considered to fully establish the specified safety level and therefore, the assessment for vortex excitation is unnecessary.

With respect to the safety of the structure against ovalling oscillations that normally occur on unstiffened shells, the wind tower has such design characteristics that meet the safety criteria: the top flange is rigidly bolted on the nacelle and the 14 stiffeners along with the middle flange are welded internally around the shell (maximum distance between two adjacent rings of 3.05 m).

The tower is considered safe against phenomena such as galloping as the corresponding criterion $v_{CG} > 1.25\, v_M$ is satisfied.

Finally, the geometrical features of the structure and the distance of each tower from any neighboring one ensure a safe response regarding interference effects during crosswind oscillations.

Despite the fact that the fatigue check was indifferent for the shell plate thickness selection and the flange design, it had a significant impact on the detailing and the quality of both the welds and the bolts. The operational wind-loading spectrum (Table 1) provided by the wind turbine manufacturer, has been defined by means

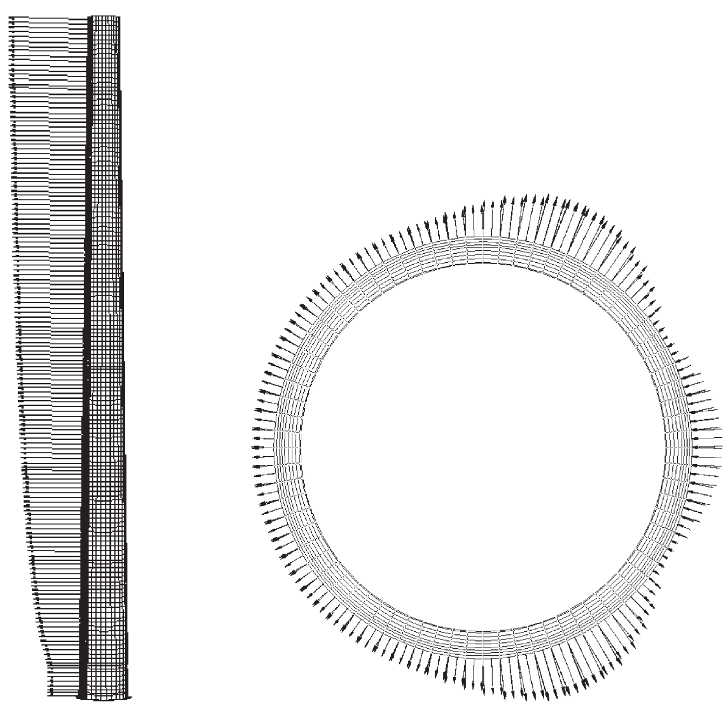


Fig. 2. Wind distribution along the height and over the circumference of the steel tower.

Table 1
Operational wind loads

No.	$v_m(h_r)$ (m/s)	n_i (cycles)	$M_{wr,o}$ (kNm)	$F_{wr,o}$ (kN)	$F_{wt,o}$ (kN)	No.	$v_m(h_r)$ (m/s)	n_i (cycles)	$M_{wr,o}$ (kNm)	$F_{wr,o}$ (kN)	$F_{wt,o}$ (kN)
1	6.30	5,00E+08	80	0.0	4.1	10	12.60	5,00E+05	1.080	70.0	21.7
2	7.00	2,00E+08	120	10.0	5.3	11	13.30	2,00E+05	1.160	75.0	24.6
3	7.70	1,00E+08	200	20.0	6.8	12	14.00	1,00E+05	1.280	80.0	27.9
4	8.40	5,00E+07	440	30.0	8.4	13	14.70	5,00E+04	1.360	85.0	31.3
5	9.10	2,00E+07	640	35.0	10.1	14	15.40	2,00E+04	1.400	90.0	34.9
6	9.80	1,00E+07	760	40.0	12.1	15	16.10	1,00E+04	1.400	180.0	39.0
7	10.50	5,00E+06	840	50.0	14.2	16	16.80	5,00E+03	1.400	180.0	43.2
8	11.20	2,00E+06	920	60.0	16.5	17	17.50	2,00E+03	1.440	185.0	47.8
9	11.90	1,00E+06	960	65.0	19.0	18	18.20	1,00E+03	1.640	220.0	52.5

of the Rain-flow method for the anticipated lifetime of the structure (T = 20 years).

The calculation procedure of the tower fatigue design was based on nominal stress ranges [$\Delta\sigma_i$] and the structure has been analyzed for the 18 load cases of the wind spectrum.

As a matter of fact, the connections are those parts of the tower being prone to fatigue; therefore, the fatigue checking covered all the welds, the bolts and the anchors of the tower.

Some of the characteristics of this procedure are worth mentioning. In particular, only the principal stresses are introduced to the calculations because the contribution of the other components was negligible in all cases. The welds of the same configuration and type were grouped together and each time the most unfavorable one has been examined.

In addition, the minimum value of the preloading force [$F_{p,\,min}$] has been used to the bottom flange structural model instead the maximum one [$F_{p,\,max}$] because the effect of the former was found to be slightly more unfavorable.

It is also worth mentioning that the shell courses have not been welded directly to the top and to the middle flange rings, but to the free end of the 30 mm high neck, that is formed after adequate planning of the original steel plate. The above configuration has been chosen taking into account factors such as the importance of the joints to the overall stability of the tower, the poor accessibility of the weld faces, the location of the shell

Table 2
Fatigue resistance parameters

No.	Connection	Description	Stress component	$\Delta\sigma_c$ (Mpa)	$\Delta\sigma_D$ (Mpa)	$\Delta\sigma_L$ (Mpa)
1	Shell to shell butt welds	Circumferential	Transverse	90	66	36
2	Shell to shell butt welds	Meridional	Longitudinal	125	93	51
3	Shell to stiffening rings butt welds	Circumferential	Longitudinal	90	66	36
4	Shell to top flange butt welds	Circumferential	Transverse	90	66	36
5	Shell to middle flange butt welds	Circumferential	Transverse	90	66	36
6	Shell to bottom flange butt welds	Circumferential	Transverse	90	66	36
7	Top flange bolts	Fully prestressed	Tensile	50	33	21
8	Middle flange bolts	Fully prestressed	Tensile	50	33	21
9	Bottom flange anchors	Partially prestressed	Tensile	50	33	21

around the outer edge of the ring cross-section and the demand of facilitation of the construction procedure in order to reach the required high quality of the specific welds.

The stress range of the prestressed bolts and anchors never exceeds the cut-off limit, reflecting this way their limited contribution into the stiffness of the overall joint. For each group of connections the procedure has been carried out according to Eurocode 3 (the fatigue resistance parameters of each group are shown in Table 2) [3].

As a matter of fact, the operational loads at the rotor axis strongly depend on the fundamental eigenfrequency of the tower along with both the relevant number of stress cycles and the mean wind velocities. Note that the position of the natural frequency of the structure (ν_1 = 0.60 Hz) in relation with the neighboring frequencies of the rotor excitation ($\nu_{r,2p}$ = 0.37 Hz & $\nu_{r,3p}$ = 0.73 Hz) plays a predominant role on the fulfillment of the serviceability criteria for the tower under investigation (Fig. 3).

In order to deal with the seismic analysis of the structure, a Spectral Response Analysis has been applied with reference to the Codes [6,7]. Due to the seismic data used in the analysis that correspond to the region where the tower is to be constructed (Seismic zone II, rocky soil), the maximum stresses have been computed about

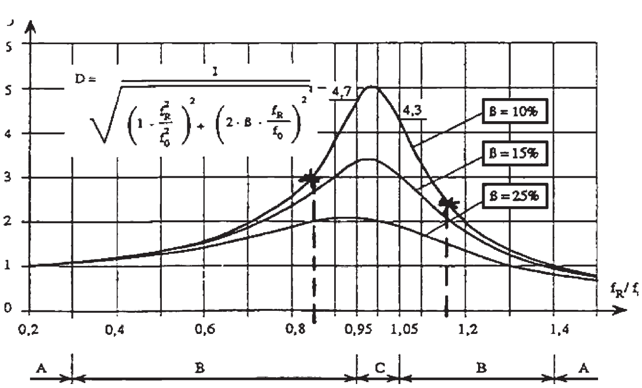


Fig. 3. On the evaluation of operational loads.

60% smaller compared to the ones developed due to wind loading.

In order to design the structure against seismic vibrations, a multimodal Linear Analysis has also been applied in terms of the appropriate design response spectrum. The impact of the torsional excitation of the ground was negligible and has been neglected in the analysis. Therefore, following the methodology dictated by the Codes, the Spectral Analysis came as a result of the combination of the translation and rotation spectra [6,7]. The mathematical expression describing this approach is based on the dynamic similarity of the structure to a single degree of freedom oscillator:

$R\,(T) = R_d^{(T)+R\theta}(T) \bullet h_r$, where

$R_d(T)$ is the transitional spectra, $R_d^{\theta}(T)$ the rocking spectra and h_r = 45.00 m is the hub height.

5. On the design of the structure

The design of the steel tower prototype has been based on a Limit Analysis methodology [3]. The steel wind turbine tower has been designed against the following four limit states:

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

Based on the Strand7 results, a Fortran program (Fig. 4) performing the verification of the structure according to the Codes has been developed by the authors [3,8,9]. Applying a trial-and-error approach, the shell thickness has been optimized by several runs so that the best design/resistance ratio to be obtained. For the lower part of the tower, the LS1 state was dominant, whereas in the upper part, the LS3 was the most significant with a remarkable participation of the compressive circumferential stress near the stiffening rings into the overall stress state configuration.

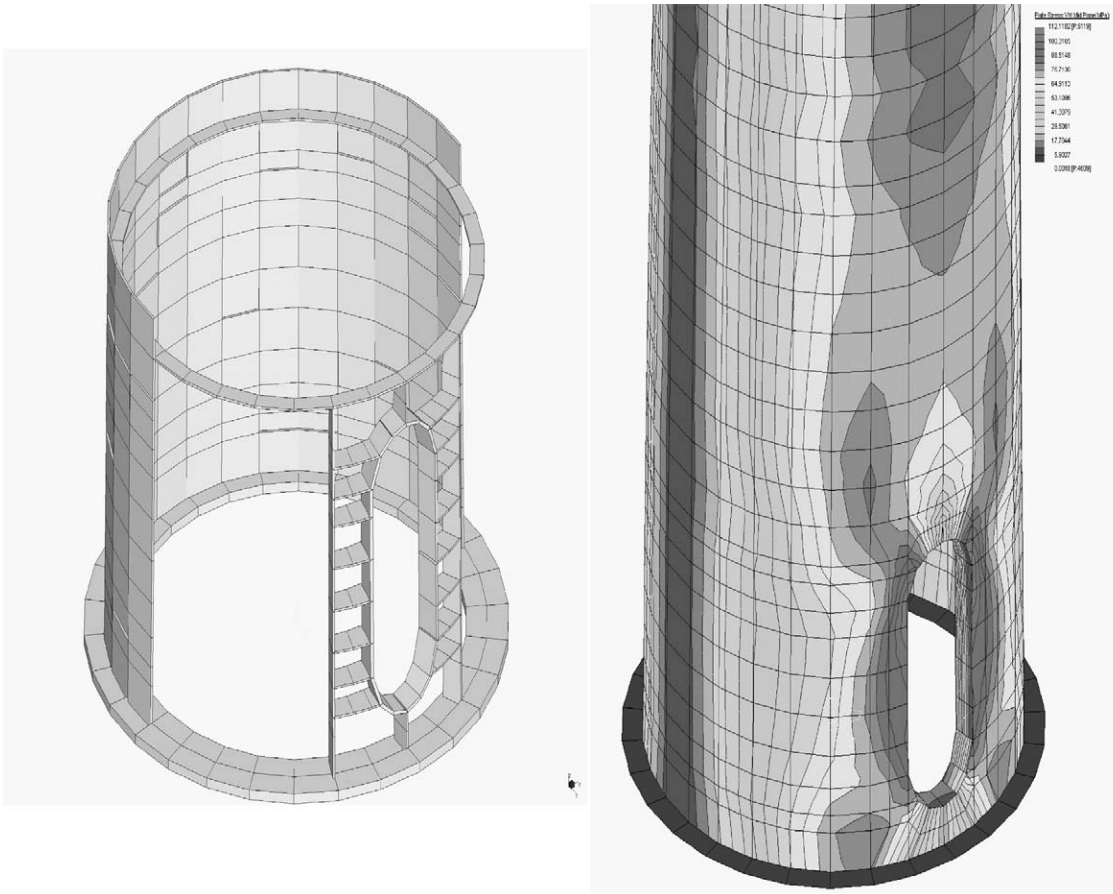


Fig. 4. Tower door opening: F.E. model and von Mises stress distribution.

Concerning the LS4 state, the stiffeners (stiffening rings, door stingers, ribs and frame) and the flanges have not been checked against local buckling, since their width to thickness ratio satisfies the code requirements. The same applies to the parts of the shell around the door bordered by the stringers, the bottom flange and the first stiffening ring. Regarding the rest shell courses, the verification has been done by applying the Eurocode 3 methodology [3]. The maximum design to resistance ratios [R_d] for the most unfavorable load cases and stress components are presented in Table 3.

It is worth presenting here some assumptions about the design against buckling. In particular, the tower has been divided by the stiffening rings and the flanges into 16 buckling-reference sections. According to Eurocode 3, the boundary conditions are the bottom, the top flanges (BC1) and the middle flange and stiffening rings (BC2). In order to simplify the calculations, the stepped cylindrical section has been transformed into an equivalent one with uniform thickness. Seven sections, out of a total of 16, were consisted of two or three courses with a 1 mm difference in the wall thickness. It must be noted

Table 3
Shell courses: maximum design to resistance ratios [R_d]

No.	Element	Stress	Rd	No.	Element	Stress	Rd
1	Course[1] t=18 mm	Meridional	0.86	11	Course[11] t=13 mm	Meridional	0.81
2	Course[2] t=17 mm	Combined	0.84	12	Course[12] t=12 mm	Meridional	0.92
3	Course[3] t=16 mm	Meridional	0.91	13	Course[13] t=12 mm	Meridional	0.86
4	Course[4] t=16 mm	Meridional	0.90	14	Course[14] t=12 mm	Combined	0.64
5	Course[5] t=15 mm	Combined	0.92	15	Course[15] t=12 mm	Meridional	0.77
6	Course[6] t=15 mm	Combined	0.85	16	Course[16] t=11 mm	Meridional	0.87
7	Course[7] t=14 mm	Meridional	0.91	17	Course[17] t=10 mm	Meridional	0.90
8	Course[8] t=13 mm	Combined	0.93	18	Course[18] t=10 mm	Combined	0.47
9	Course[9] t=13 mm	Meridional	0.87	19	Course[19] t=10 mm	Meridional	0.56
10	Course[10] t=13 mm	Meridional	0.83				

that representative sample checks demonstrated the sufficient accuracy of this approach for the specific configuration of the shell approximation less than 2%. The conical section has also been transformed into an equivalent cylinder one. In this case, a specific procedure has been carried out according to Eurocode 3 for each stress type.

Since the maximum circumferential buckling stresses occur along the stiffeners which are the boundary zones of the shell sections, the interaction checks need not be carried out over the adjacent to the stiffening rings stripes, up to a distance equal to 10% of the height of the section.

For the design of the flanged connections (rings, bolts and anchors), special detailed models have been used, where the two connected parts were described by unilateral contact finite elements interconnected with each other (Fig. 5). Bolts and anchors have been modeled as cable elements subjected to the prestressing forces. Two failure mechanisms have been investigated corresponding to those with and without the preloading forces and the respective results have been used for the LS1 and the LS4 state.

According to the Geotechnical Survey, the site where the Aeolian park is under construction consists of limestone and dolomite with random carsic voids. At least one borehole has been drilled under the foundation site of each. Due to the engineering wise preeminently elastic nature of the rock, the static and the dynamic properties being practically identical, only one characteristic value of each parameter always suffices for the purposes of the design.

As expected, a complex stress state resulting from the loads transferred to the foundation from the tower and the prestressed anchor bolts, has been developed in the foundation slab and the pedestal. To overcome this difficulty, a detailed brick element model has been developed, where the anchoring system has been fully represented (Fig. 6). The details of the anchoring are depicted in Fig. 7 where, due to the symmetry of the loading and the structure, only half of the foundation has been described and analysed. The latter consisted of 3360 brick elements corresponding to the concrete, the flanges and the non-shrinking mortar (Emaco). The connection between the elements and the support to the ground has been realized by means of unilateral contact elements, whereas the prestressed anchors have been modeled as cable elements connecting the steel flange with the foundation. The elevation of the designed tower prototype is presented in Fig. 8.

6. Concluding remarks

The previously presented design procedure for a prototype steel wind turbine tower has been based on the rules with respect to serviceability requirements dictated by the Codes [3,5,8,9].

It is noteworthy that the use of a simplified linear static model is sufficient for the calculation of the basic response and the eigenvalues of the structure. However, it does not suffice for the ULS design due to the fact that local stress concentrations are neglected in this model.

It has also to be underlined that, given the present status of the relevant Eurocodes, calculations regarding buckling analysis of the shell of the tower inserts a high level of ambiguity in the results and therefore, it has to be handled with utmost care.

With respect to the design loads, the extreme wind is the dominant load combination for the design of the structure, whereas the seismic design could become critical only for the case of constructing the steel wind tur-

Fig. 5. Middle flange model.

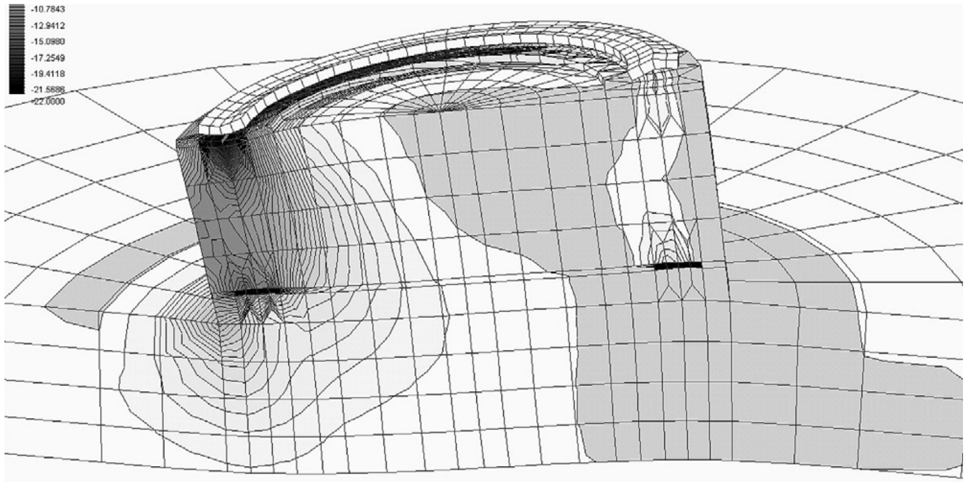


Fig. 6. On the foundation model: σ_{zz} stress distribution.

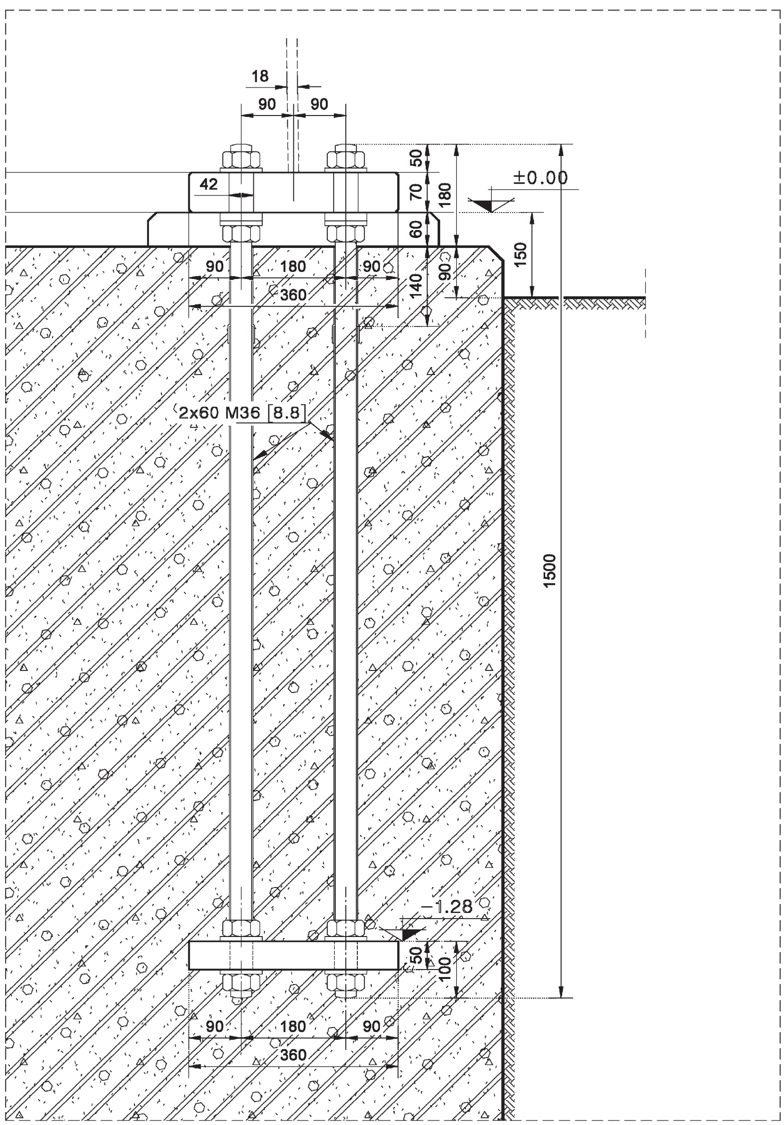
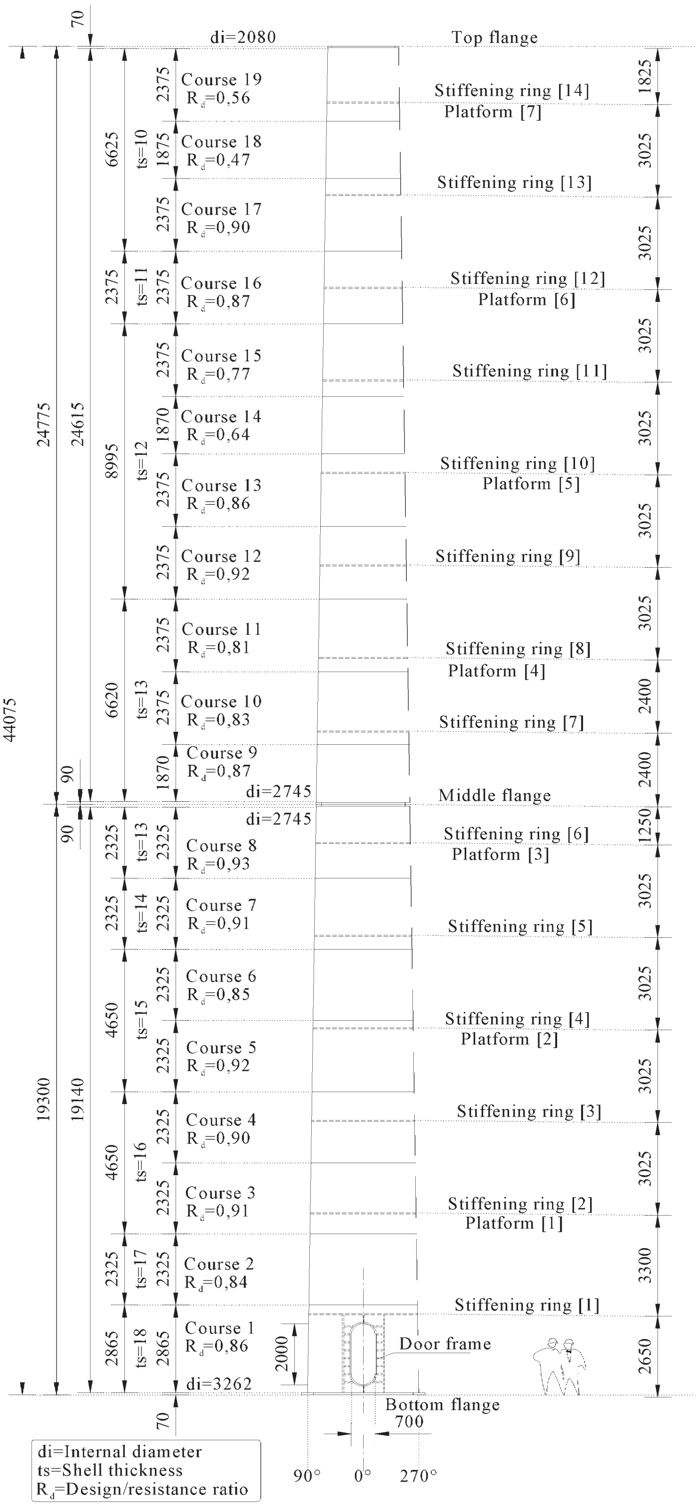
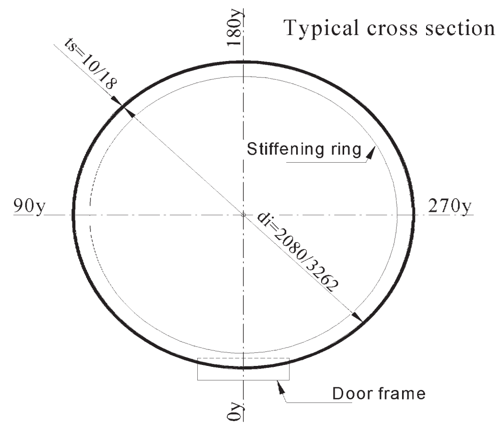


Fig. 7. The anchoring detail.





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

Fig. 8. Tower elevation.

bine towers in a seismically hazardous area (zone III or IV), far away from the coastal zone and underground being a medium or soft soil.

Concerning the fatigue design, it has to be noted that it is the dynamic characteristics of the structure that remain critical for the overall design of the steel tower.

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