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Proceedings of the Tenth International Conference
on Civil, Structural and Environmental
Engineering Computing,
B.H.V. Topping (Editor),
Civil-Comp Press, Stirling, Scotland.

Paper 102

Analysis and Design of the Foundations of Wind-Turbo Generators using the Finite Element Method

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Abstract

The FEM analysis and the design of the reinforced concrete foundations of a 65 m high wind-turbo generator tower, to be constructed at the wind energy farm in 'Serra do Barroso' in northern Portugal, are detailed. On such trunk-conic towers are mounted wind turbines with three 37,5 m blades. The analysis quantifies the permanent actions as well as the variable actions due to wind on tower, on blades and on rotor – for two wind scenarios – associated with 90º and 45º wind incidences. Although the wind energy generator towers are located at the least seismic risk zone of the country, a seismic verification under standard principles of spectral modal analysis is still performed to ascertain the anticipations. The analysis is supported by efficient interactive FEM modeling software already developed, that permitted detailed description of the pressure at tower footings, settlements under such footing slabs, total tower displacements, among others. According to Eurocode EC2, as well as to Portuguese standards RSA and REBAP, the reinforced concrete foundations were designed for the stress resultants obtained for the worst design combination.

Keywords: wind turbines, wind energy conversion systems, finite elements, preliminary design, seismic analysis.

1 Introduction

Due to its geographical position Portugal presents an extensive coast of around 600 km, under continuous incidence of Atlantic winds. Moreover the country's north and interior-central topography is moderately mountainous, and studies during last decade indicated a few locations with significant potential for energy production through wind farms. Since hydroelectric major sources have already been developed in the past and RCC dams are here at their earlier years [1], the development of wind energy sources is of great national economic importance since it can reduce national fragility with respect to dependency mainly in hydrocarbon-imported sources.

The present technical article details some design aspects and design procedure associated with the structural analysis of the foundation for a wind turbine generator of the type V80 2 MW manufactured by VESTAS (Danish Wind Technology) to be integrated in the wind farm of the mountain Serra do Barroso, located in the district of Vila Real, in northern Portugal.

The foundation consists of a square slab in reinforced concrete, giving support to a metallic circular tower of variable cross-section along the height that holds the wind turbine generator and the blades (Figure 1).

The present study is divided in three phases. The first phase corresponds to the static analysis of the foundation slab, where the structural strength is verified under the actions transmitted by the superstructure, namely the effect of the wind action.

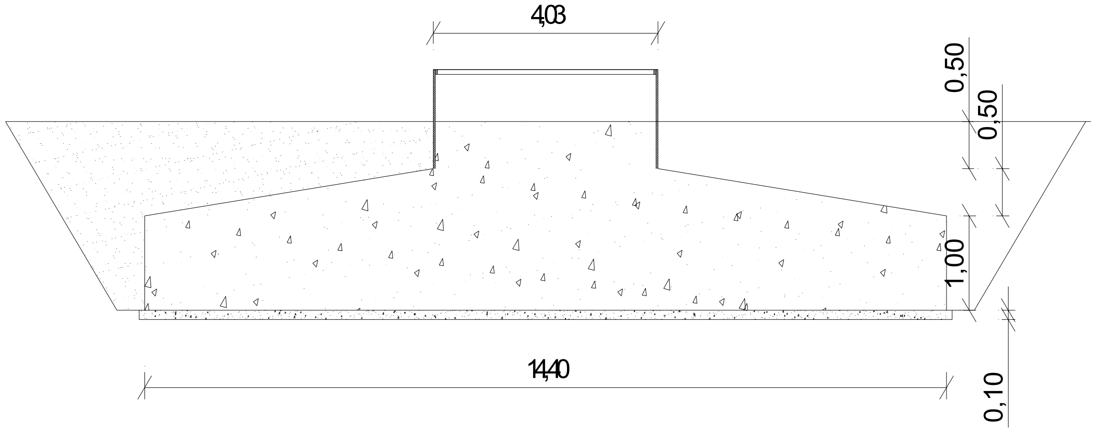

Figure 1: Geometry (in meters) of the foundation of the aeroturbine V80

The second phase corresponds to the study of the dynamic behaviour of the structure of the aeroturbine, through the quantification of the generalized actions transmitted to the foundation by the metallic tower under regulatory seismic actions.

The third phase corresponds to the reinforced concrete design and detailing of the foundation slab, and the verification of the safety of the connection between the metallic tower and the foundation slab.

As an additional final verification, an estimate of the resistant capacity of the foundation soil is determined.

2 Design Codes and Regulations

The structural analysis and the structural design of the wind turbine tower is made in accordance with Portuguese design code, internationally recognized design codes, and other bibliography associated with wind energy conversion systems. Among Portuguese and international design codes and regulations, the following have been extensively used: *Regulamento de Segurança e Acções em Estruturas de Edifícios e Pontes* (RSA), *Regulamento de Estruturas de Betão Armado e Pré-Esforçado* (REBAP), CEB-FIP Model Code 1990, Eurocode 2, Eurocode 7 and ENV-206.

3 Static Analysis

3.1 Introduction

The simulation of the static behaviour of the foundation slab is achieved with a formulation of finite elements for thick slabs, commonly designated as formulation of Mindlin. In fact for flat plates an approach widely used in the finite element literature invokes the so-called Reissner-Mindlin plate theory, wherein the flexural behaviour is expressed in terms of rotations of the normals to the midplane; additionally terms are included that account for transverse shear deformations (written in terms of the transverse displacements) and rotatory inertia, as detailed by Shames and Dym [2] and by Kardestuncer and Norrie [3].

The foundation slab was discretized in a finite element mesh with 480 parabolic finite elements of 8 nodes (Figure 2), above elastic soil media.

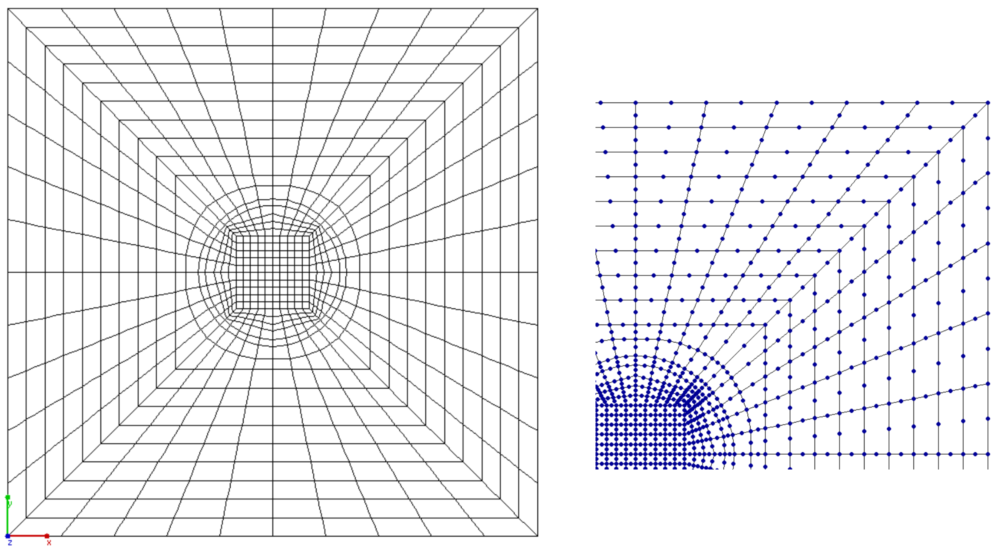

Figure 2: Finite element mesh

The zone of the foundation slab with variable thickness was divided into four sub-zones of equivalent constant thickness, according to the material properties mentioned in Table 1, as represented in Figure 3. For soil stiffness the value of 40000 kPa/m was used, corresponding to an altered rock.

Material	E [GPa]	ν	h [m]
1	30.5	0.22	1.10
2	30.5	0.22	1.29
3	30.5	0.22	1.44
4	30.5	0.22	2.00

Table 1: Material properties of the slab zoning

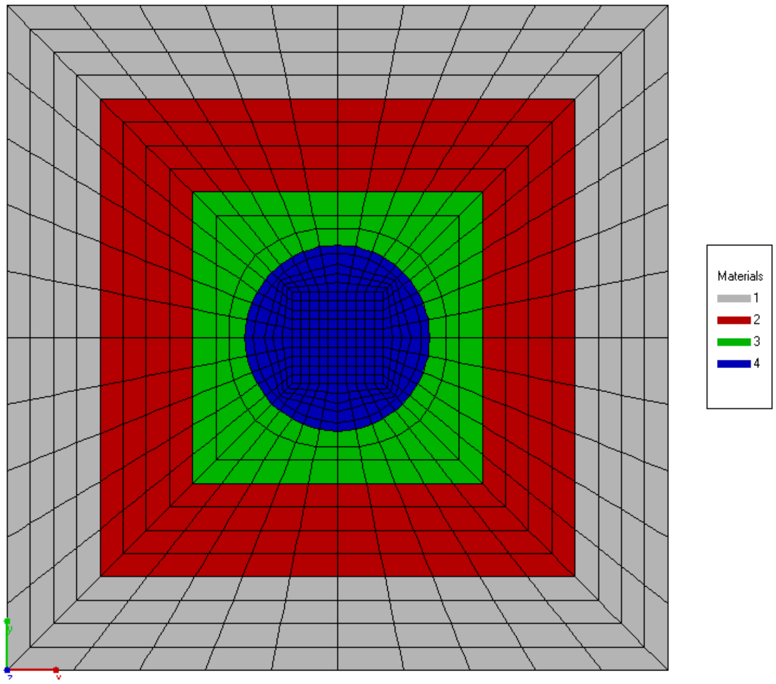

Figure 3: Zoning of the slab for equivalent constant thicknesses

The calculation model is based on a technical iterative process that cancels the soil stiffness at the nodes where negative pressures would occur, proceeding to a new calculation of the slab with the corrected stiffness. This process is repeated until the final iteration, where no more mesh nodes under negative pressure (nodes stretched under hypothetical tensile forces) would exist; this means that for all slab nodes in contact with foundation soil, positive reactive pressures (at most null in some of them) would occur.

3.2 Actions

The static actions in the base of the foundation are of two types:
(i) Permanent actions

Self-weight of foundation soil	18 kN/m^3 => $q_{,Gk} = 18 \times h_{\text{average of soils above slab}}$
Self-weight of slab	25.0 kN/m^3 => $q_{,Gk} = 25 \times h_{\text{average of slab}}$
Self-weight generator + blades	965.3 kN
Self-weight of tower	1200 kN

(ii) Variable wind actions

In the quantification of the wind actions the following parameters were used: soil roughness of type II (corresponding to a placement land of low roughness at an elevation above 600 m); zone B.

The effect of wind action on the structure was subdivided in two components: wind actions on the metallic conic tower; wind action on the blades and rotor.

3.2.1 Wind actions on the metallic conic tower

For the quantification of the wind actions on the conic tower, the tower was subdivided into 2 m height segments. In each, the total area blown by the wind (frontal area) is the 2 m segment length multiplied by the average between the lower and the upper diameters at the segment length height. The characteristic dynamic pressure corresponds to the average between the lower dynamic pressure and the upper dynamic pressure at the segment length height.

The value of the equivalent static wind force results from the product between the frontal area, the average characteristic pressure and the force coefficient specified in RSA for closed smooth cylindrical structures, as calculated in Table 2.

h [m]	1.3 x w,IIB(h) [kN/m²]	d(h) [m]	δf	Fwi (h) [kN]	zi [m]	Mwi [kN.m]
0.000	1.45	4.038	0.6			
2.000	1.45	3.987	0.6	7.00	1.0	7.00
4.000	1.45	3.935	0.6	6.91	3.0	20.74
6.000	1.45	3.884	0.6	6.82	5.0	34.12
8.000	1.45	3.832	0.6	6.73	7.0	47.13
10.000	1.45	3.781	0.6	6.64	9.0	59.79
12.000	1.52	3.729	0.6	6.71	11.0	73.83
14.000	1.59	3.678	0.6	6.91	13.0	89.89
16.000	1.64	3.626	0.6	7.08	15.0	106.21
18.000	1.70	3.575	0.6	7.22	17.0	122.68
20.000	1.75	3.523	0.6	7.33	19.0	139.24
22.000	1.79	3.472	0.6	7.42	21.0	155.81
24.000	1.83	3.420	0.6	7.49	23.0	172.33
26.000	1.87	3.369	0.6	7.55	25.0	188.76
28.000	1.91	3.318	0.6	7.59	27.0	205.04
30.000	1.95	3.266	0.6	7.63	29.0	221.14
32.000	1.98	3.215	0.6	7.65	31.0	237.03
34.000	2.02	3.163	0.6	7.66	33.0	252.65
36.000	2.05	3.112	0.6	7.66	35.0	267.99
38.000	2.08	3.060	0.6	7.65	37.0	283.01
40.000	2.11	3.009	0.6	7.63	39.0	297.69
42.000	2.14	2.957	0.6	7.61	41.0	312.01
44.000	2.17	2.906	0.6	7.58	43.0	325.93
46.000	2.20	2.854	0.6	7.54	45.0	339.43
48.000	2.22	2.803	0.6	7.50	47.0	352.49
50.000	2.25	2.751	0.6	7.45	49.0	365.10
52.000	2.27	2.700	0.6	7.40	51.0	377.23
54.000	2.30	2.649	0.6	7.34	53.0	388.87
56.000	2.32	2.597	0.6	7.27	55.0	399.99
58.000	2.35	2.546	0.6	7.20	57.0	410.58
60.000	2.37	2.494	0.6	7.13	59.0	420.62
62.000	2.39	2.443	0.6	7.05	61.0	430.10
64.000	2.41	2.391	0.6	6.97	63.0	439.00
65.000	2.42	2.365	0.6	3.45	64.5	222.65
			Hw,total [kN] =	236.79	Mw,total [kN.m] =	7766.11
					dHw [m] =	32.80

Table 2: Equivalent static wind actions on the tower

The bending moment contribution at the foundation level resulting from a static force on a segment at certain height above foundation is the product of the equivalent static wind force by the average segment height above foundation.

The total bending moment at the foundation level (basal moment) is the cumulative sum of the bending moment contributions induced by the equivalent static wind forces on all the tower segments of the subdivision. Also, the horizontal force at the foundation level (basal shear) is the cumulative sum of the segmental contributions to the static forces.

The abovementioned Table 2 summarizes the quantification of the equivalent static wind actions on the metallic conic tower, as well as of the basal shear and basal bending moment. Figure 4 depicts both the wind tower dimensions and the wind forces on the tower.

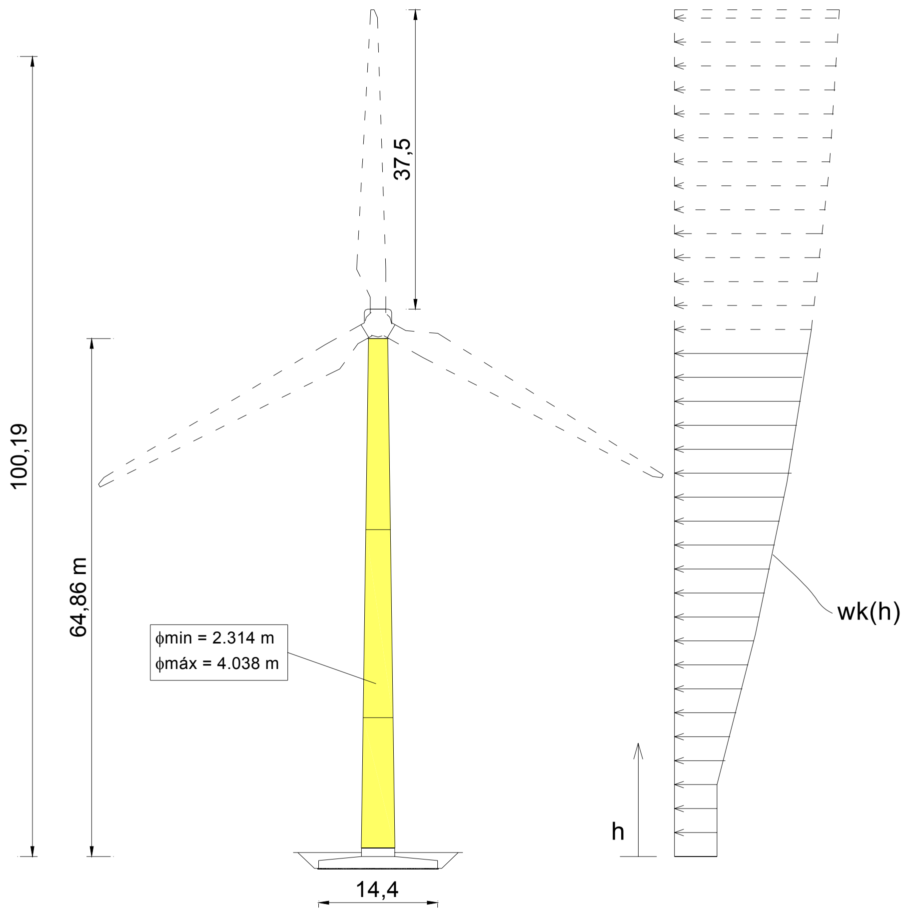

Figure 4: Wind tower and wind forces on the metallic tower

3.2.2 Wind actions on the rotor and blades

The wind actions on the generator rotor and blades were calculated multiplying the area of its frontal surface by the average dynamic characteristic pressure at the height of its centre of gravity. This value of dynamic pressure was corrected multiplying it by the aerodynamic coefficient (α) of the surface corresponding to its orientation relatively to the wind. The most unfavourable configuration [4] corresponds to the situation in which one of the blades is coincident with the axis of the tower while the remaining two are positioned symmetrically in relation to the same axis (Figure 4).

Two different scenarios were considered for orientation of the blades with respect to the direction of wind gust. In the first scenario, the most unfavourable, it is assumed that in the presence of strong winds the rotation device of the blades is not functional in one of the blades [4]. In this scenario the most stressed blade is the vertical blade, to which correspond the wind actions presented in Table 3.

	Orientation	1.3 x wk,méd [kN/m^2]	A [m^2]	α	Fk [kN]	z [m]	Mk [kN.m]
Blade 1(vertical)	Perpendicular	2.58	82.0	1.0	211.2	83.8	17695.3
Blade 2(oblique)	Parallel	2.35	34.0	0.3	23.9	58.0	1388.0
Blade 3(oblique)	Parallel	2.35	34.0	0.3	23.9	58.0	1388.0
Rotor	Perpendicular	2.46	12.6	1.0	30.9	68.0	2099.5
Total					289.9		22570.9

Table 3: Wind actions on the rotor and blades (Scenario 1)

The second scenario corresponds to the situation in which the operation of the orientation device (of all the blades) allows that these are positioned parallel to the wind for high wind speeds, to which correspond the wind actions in Table 4.

		1.3 x wk,méd [kN/m^2]	A [m^2]	α	Fk [kN]	z [m]	Mk [kN.m]
Blade 1(vertical)	Parallel	2.58	34.0	0.3	26.3	83.8	2201.1
Blade 2(oblique)	Parallel	2.35	34.0	0.3	23.9	58.0	1388.0
Blade 3(oblique)	Parallel	2.35	34.0	0.3	23.9	58.0	1388.0
Rotor	Perpendicular	2.46	12.6	1.0	30.9	68.0	2099.5
Total					105.0		7076.7

Table 4: Wind actions on the rotor and blades (Scenario 2)

3.2.3 Design actions

							Sd = 1.0Gk + 1.5wk		
	N_{GK}	H_{GK}	M_{GK}	N_{WK}	H_{WK}	M_{WK}	N_{sd} [kN]	H_{sd} [kN]	M_{sd} [kN.m]
Scenario 1	2070.0	0.0	0.0	0.0	526.7	30337.0	2070.0	790.0	45505.5
Scenario 2	2070.0	0.0	0.0	0.0	105.0	14842.8	2070.0	157.5	22264.2

Table 5: Design wind actions (base action: Wind) for the 2 previous wind scenarios

The concentrated actions, M_{Sd} and N_{Sd}, are transformed into equivalent nodal vertical forces, applied to the nodes of the foundation mesh in the periphery of the interface zone with the metallic tower, through the following equilibrium conditions:

$$\begin{cases} \sum_{i=1}^{\#\,nodes} F_i \times z_i = M_{Sd} \\ \\ \sum_{i=1}^{\#\,nodes} F_i = N_{Sd} \end{cases} \tag{1}$$

where z_i is the value of the distance of the equivalent force F_i to the axis of the tower, F_i is the vertical force applied in the periphery at the distance z_i of the axis of the tower, and '# nodes' is the number of periphery nodes of the mesh for which the concentrated actions are distributed.

In Figure 5 such equivalent periphery forces at the interface zone with the metallic tower are shown in perspective view, for the wind energy conversion system used.

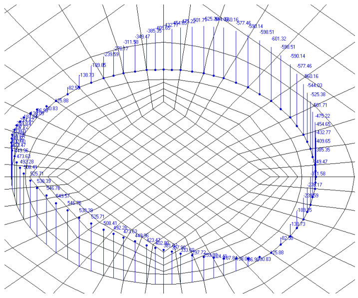

Figure 5: Periphery vertical forces equivalent to N_{Sd} and M_{Sd}

3.2.4 Results (wind at 90º)

On the basis of the already defined design actions, the following figures (Figures 6 through 9) give some of the results calculated for the wind direction at 90º.

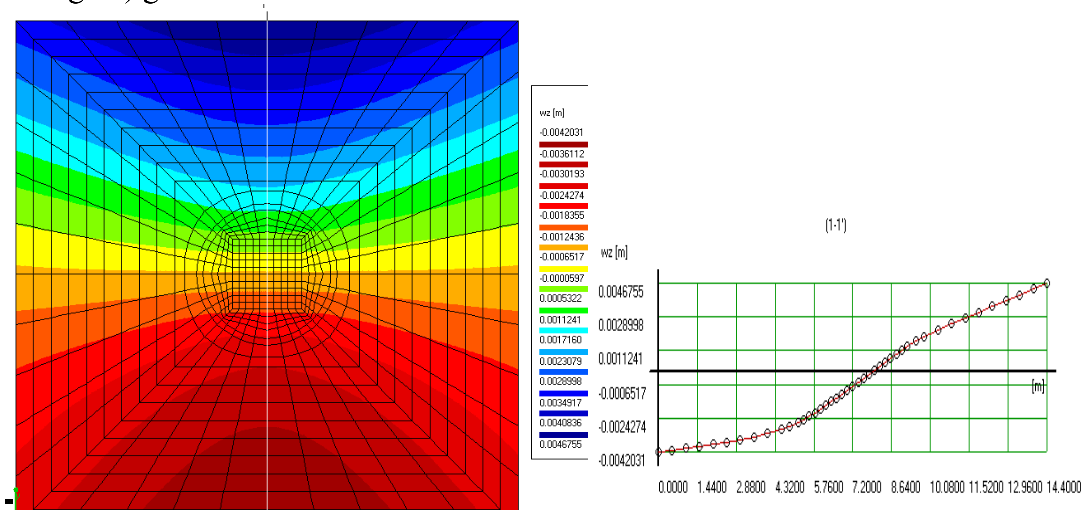

Figure 6: Scenario 1 (wind at 90^o) – Slab settlements

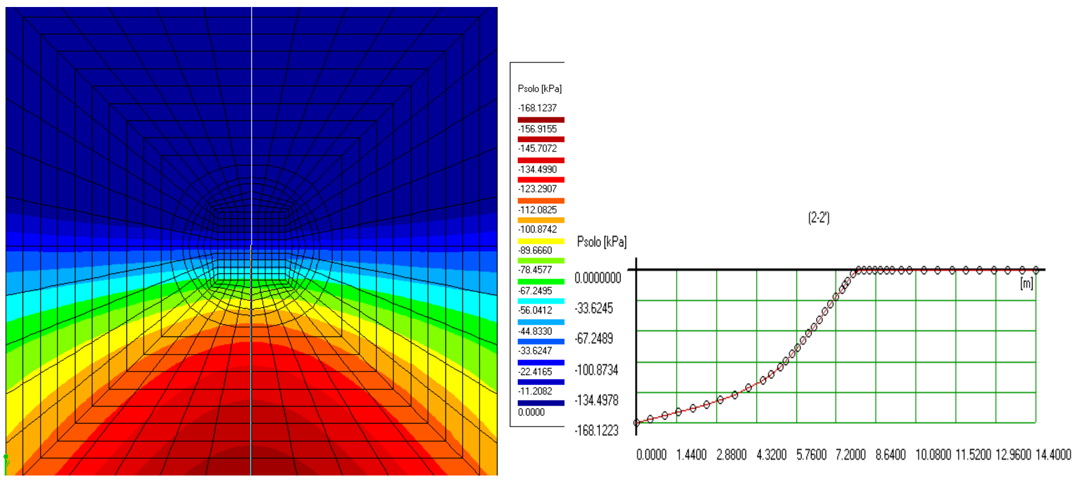

Figure 7: Scenario 1 (wind at 90^o) – Pressure on the foundation soil

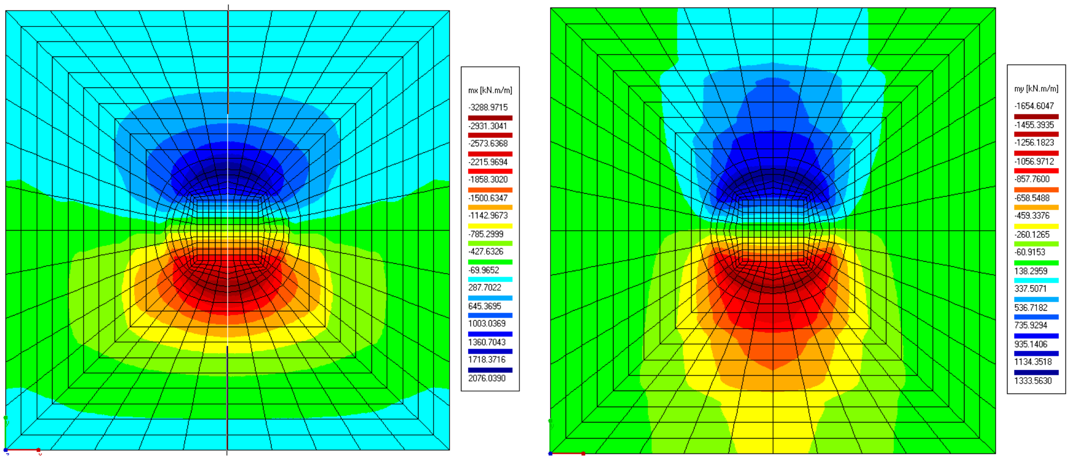

Figure 8: Scenario 1 (wind at 90°) – Moments m_x and m_y

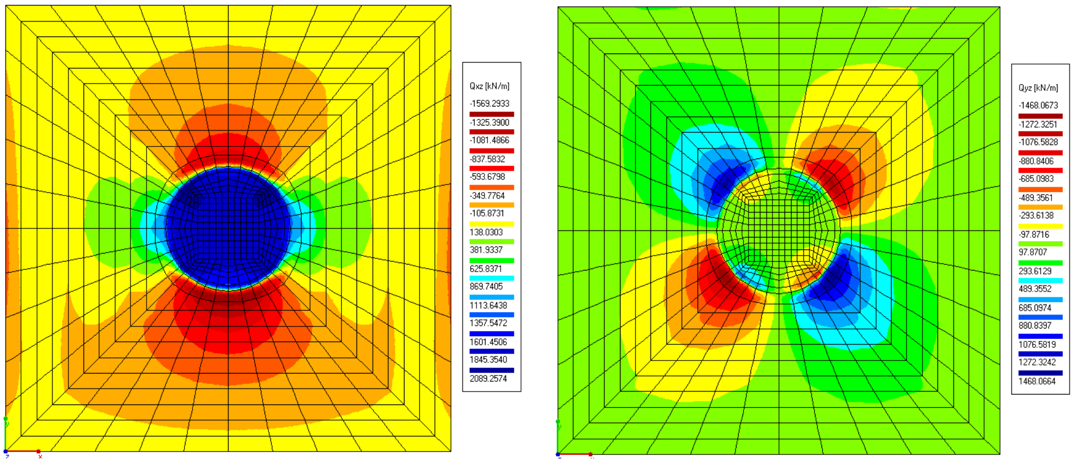

Figure 9: Scenario 1 (wind at 90°) – Shear forces Q_{xz} and Q_{yz}

On the basis of the already defined design actions, similar figures could be obtained corresponding to the results calculated for the wind direction at 45º.

4 Seismic Analysis

4.1 Analysis According to Portuguese Design Code (RSA)

The wind turbine is to be located in northern Portugal (seismic zone code C, to which corresponds a seismic coefficient of 0.3) in a foundation soil media of Type I. A seismic action of Type 1 is considered, acting on this wind tower steel structure of behavioural coefficient η=1, with assumed critical damping factor ξ=0.02 .

The study of the effect of the seismic action at the foundation of the wind generator was done through a standard modal dynamic analysis (Clough and Penzien [5]), in agreement with the parameters defined earlier, following the outline described below:

- Quantification of the stiffness matrix [K] corresponding to the lateral horizontal displacements;
- Quantification of the mass matrix [M];
- Resolution of the linear Eigenvalue-Eigenvector problem, determining the natural frequencies ω_i by solving the characteristic equation:

$$\det\left([K]-\omega^2 [M]\right)=0 \tag{2}$$

and determining the mode shapes φ_i by solving:

$$\left([K]-\omega_i^2 [M]\right)\{\varphi\}_i = \{0\} \tag{3}$$

- Obtaining the generalized variables K_i, M_i and L_i through the relationships:

$$K_i = \{\varphi\}_i^T \cdot [K] \cdot \{\varphi\}_i \tag{4}$$

$$M_i = \{\varphi\}_i^T \cdot [M] \cdot \{\varphi\}_i \tag{5}$$

$$L_i = \{\varphi\}_i^T \cdot [M] \cdot \{1\} \tag{6}$$

- Quantification of the maximum seismic response of uncoupled oscillators, evaluated with the acceleration seismic response spectra of RSA for the site:

$$Y_i = \frac{\alpha\, S_a(\xi_i, f_i)}{\omega_i^2} \cdot \frac{L_i}{M_i} \tag{7}$$

- Quantification of the total displacements of the degrees of freedom, calculated through a root mean square quadratic combination:

$$\{u_{seismic}\} = \sqrt{\sum_{i=1}^{n}([u_i])^2} = \sqrt{\sum_{i=1}^{n}(\{\varphi\}_i \cdot Y_i)^2} \tag{8}$$

- Quantification of the seismic forces through the relationship:

$$\{F_{seismic}\} = [K] \cdot \{u_{seismic}\} \tag{9}$$

Thus, the trunk-conic tower was discretized in 10 bar elements and 11 nodes. The mass of the tower was distributed in 10 concentrated masses: m_1 to m_9, correspond to the average mass of each segment; additionally was applied a concentrated mass in the top, m_{10}, corresponding to the blades and to the rotor (Figure 10).

Also in Table 6 are synthesised the determined mechanical characteristic needed to quantify the seismic response of the wind generator.

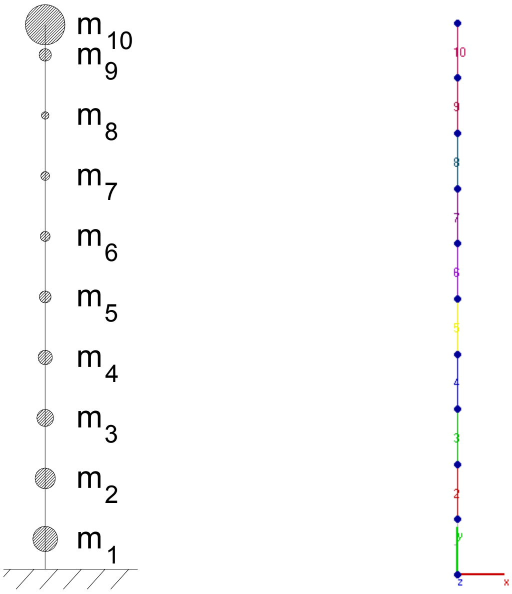

Figure 10: Model and mesh for analysis

h [m]	φ(h) [m]	e(h) [m]	re(h) [m]	ri(h) [m]	A [m2]	Barra	I [m4]	A,méd [m2]	I,méd [m4]	m_i [t]
0.000	4.038	0.034	2.019	1.985	0.43		0.85714	0.00000		
3.722	3.942	0.033	1.971	1.938	0.41	1.00	0.77935	0.41784	0.81824	12.21
11.167	3.751	0.032	1.875	1.844	0.37	2.00	0.63755	0.38836	0.70845	22.69
18.611	3.559	0.030	1.780	1.750	0.33	3.00	0.51352	0.34925	0.57554	20.41
26.056	3.368	0.028	1.684	1.656	0.29	4.00	0.40639	0.31062	0.45996	18.15
33.500	3.176	0.026	1.588	1.562	0.25	5.00	0.31512	0.27273	0.36076	15.94
40.944	2.984	0.023	1.492	1.469	0.22	6.00	0.23858	0.23584	0.27685	13.78
48.389	2.793	0.021	1.396	1.375	0.18	7.00	0.17554	0.20022	0.20706	11.70
55.833	2.601	0.018	1.301	1.282	0.15	8.00	0.12468	0.16613	0.15011	9.71
63.278	2.410	0.016	1.205	1.189	0.12	9.00	0.08467	0.13384	0.10468	7.82
67.000	2.314	0.014	1.157	1.143	0.10	10.00	0.06830	0.11074	0.07648	83.24

Table 6: Mechanical characteristics of the wind tower V80

Mode	K_i [kN/m]	M_i [t]	L_i	w_i [rad/s]	f_i [Hz]	L_i / M_i	$L_i / (M_i \times w_i^2)$	α	Sa_i [cm/s^2]	Y_i [m]
1	796.1	94.1	107.9	2.9	0.46	1.14590	0.13551336	0.30000	150.0	0.0609810
2	20334.0	48.6	42.4	20.5	3.26	0.87315	0.00208703	0.30000	720.0	0.0045080
3	126191.0	36.6	-23.9	58.7	9.34	-0.65349	-0.00018962	0.30000	750.0	-0.0004267
4	493932.5	35.1	16.6	118.7	18.88	0.47226	0.00003355	0.30000	750.0	0.0000755
5	1921475.1	48.1	-14.4	199.9	31.82	-0.29932	-0.00000749	0.30000	750.0	-0.0000169
6	2822205.7	31.3	8.7	300.1	47.76	0.27880	0.00000310	0.30000	750.0	0.0000070
7	3388538.6	20.5	-5.0	406.6	64.71	-0.24360	-0.00000147	0.30000	750.0	-0.0000033
8	9719402.8	40.6	5.3	489.4	77.88	0.13087	0.00000055	0.30000	750.0	0.0000012
9	20921761.8	56.4	6.7	609.1	96.94	0.11865	0.00000032	0.30000	750.0	0.0000007
10	26315244.9	12.6	9.6	1444.1	229.84	0.75830	0.00000036	0.30000	750.0	0.0000008

Table 7: Maximum seismic response of the first 10 uncoupled oscillators

The tower seismic response in displacements was calculated solely with the first three modal contributions. Table 8 quantifies spectral responses from such modal contributions. Total displacements are evaluated with a root mean square quadratic combination, from which seismic forces are also calculated. Notice that the cumulative sum of the last two columns in Table 8, are respectively the spectral basal shear and spectral basal moment of the wind tower at the site.

	Φ_1 x Y_1	Φ_2 x Y_2	Φ_3 x Y_3			
h[m]	u_1 [m]	u_2 [m]	u_3 [m]	u_{tot}[m]	F_{sis} [kN]	$M_{i,base}$ [kN.m]
0.0	0.000000	0.000000	0.000000	0.000000	0.00	0.00
3.7	0.000148	0.000074	0.000017	0.000166	-6.30	-23.44
11.2	0.001382	0.000630	0.000125	0.001524	6.92	77.24
18.6	0.003959	0.001606	0.000253	0.004280	5.84	108.76
26.1	0.008024	0.002788	0.000298	0.008500	8.03	209.24
33.5	0.013724	0.003878	0.000186	0.014262	5.18	173.38
40.9	0.021189	0.004508	-0.000071	0.021664	0.70	28.73
48.4	0.030505	0.004297	-0.000344	0.030808	-5.80	-280.61
55.8	0.041640	0.002958	-0.000427	0.041747	-8.33	-465.03
63.3	0.054311	0.000489	-0.000174	0.054314	-20.13	-1273.88
67.0	0.060981	-0.001011	0.000048	0.060989	81.47	5458.29
					67.58	4012.69

Table 8: Spectral responses of the wind tower V80 at the ‘Serra do Barroso’ site

It should be noticed that the seismic response of the structure is not solely influenced by the first vibration mode of the fundamental frequency. Due to the fact that the fundamental frequency is very small and as such is little affected by the seismic action, there is also a contribution of the second vibration mode subjected to the maximum seismic acceleration.

Figures 11 and 12 show respectively the total lateral displacements of the tower and the variation of seismic forces on the tower, along the height.

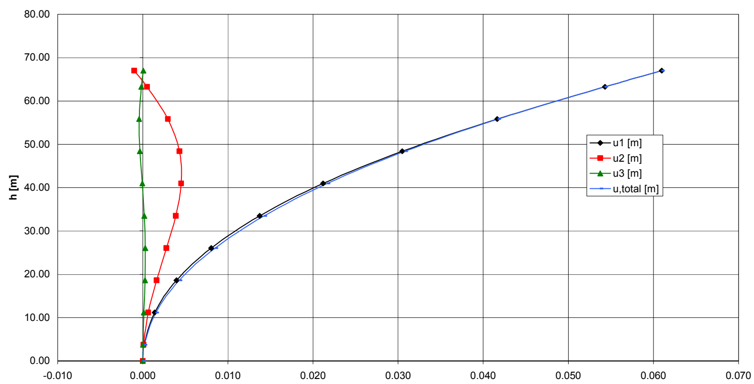

Figure 11: Total lateral seismic displacements for wind tower V80 at the site

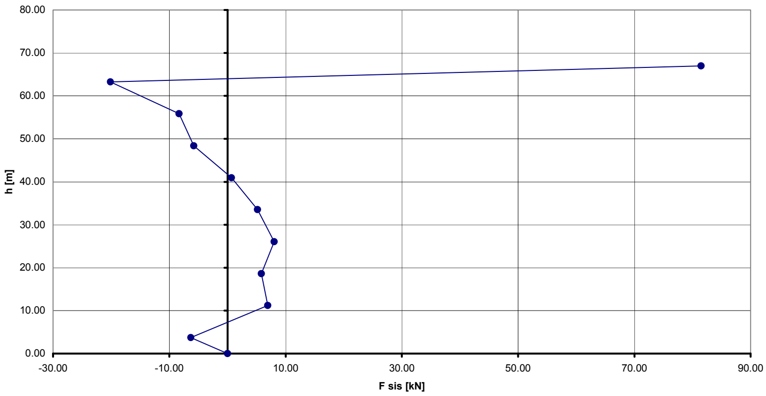

Figure 12: Spectral seismic forces for wind tower V80 at the site

4.2 Design Load Combination and Associated Results

The seismic action was combined with the remaining actions in accordance with design specifications in Portuguese design code RSA. The following design actions (Table 9) are obtained for wind scenario 1:

N_{GK}	H_{GK}	M_{GK}	N_{WK}	H_{WK}	M_{WK}	N_{EK}	H_{EK}	M_{EK}
2070.0	0.0	0.0	0.0	394.2	20861.3	0.0	67.6	4012.7

Sd = 1.0Gk + 0.40Wk + 1.5Ek		
N_{sd} [kN]	H_{sd} [kN]	M_{sd} [kN.m]
2070.0	259.0	14363.6

Table 9: Design table for wind scenario 1 (base action: Earthquake)

Although the load case combination in which the seismic action (Earthquake) is taken as 'base action' does not control the design – (the load case combination that controls design has 'wind' as base action) – the results associated with the load case of Table 9 are presented herein for comparison with the previous ones.

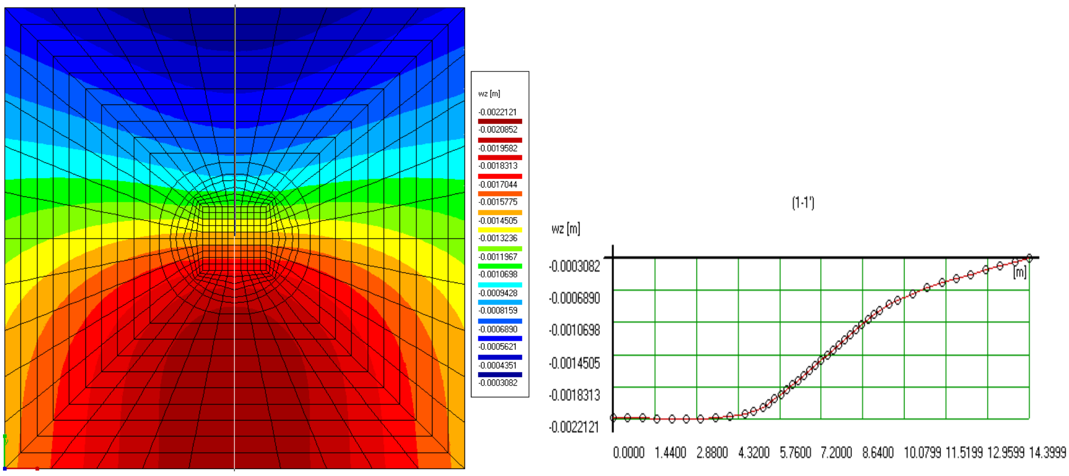

Figure 13: Seismic action (wind at 90^{o}) – Slab settlements

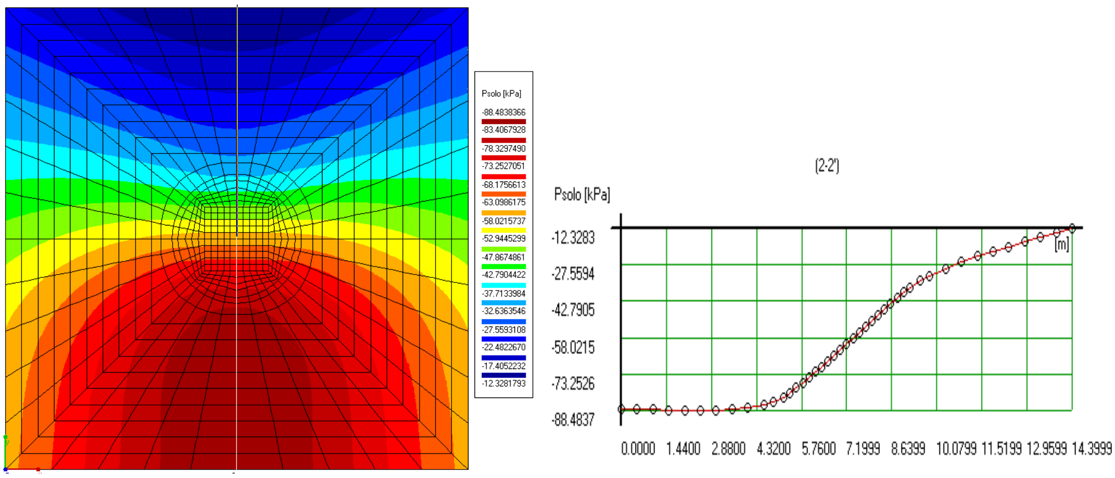

Figure 14: Seismic action (wind at 90°) – Pressure on the foundation soil

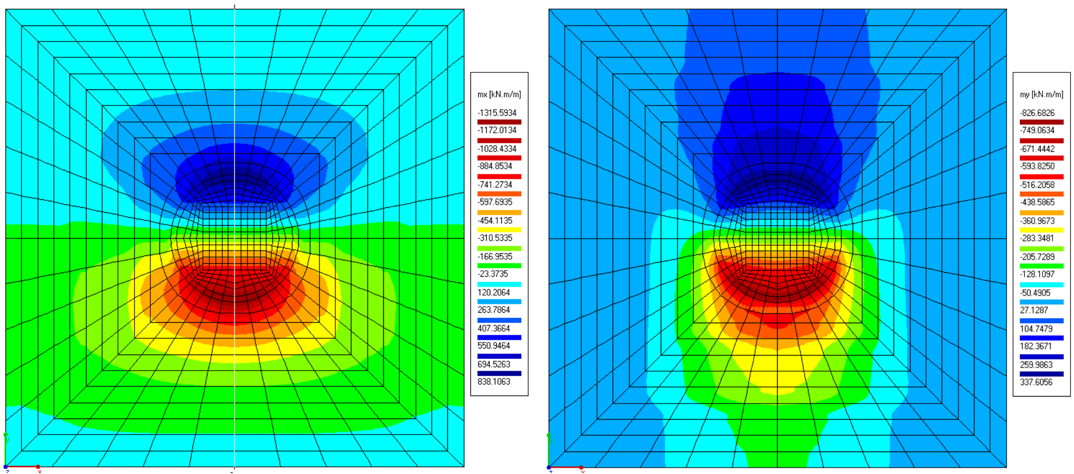

Figure 15: Seismic action (wind at 90°) – Moments m_x and m_y

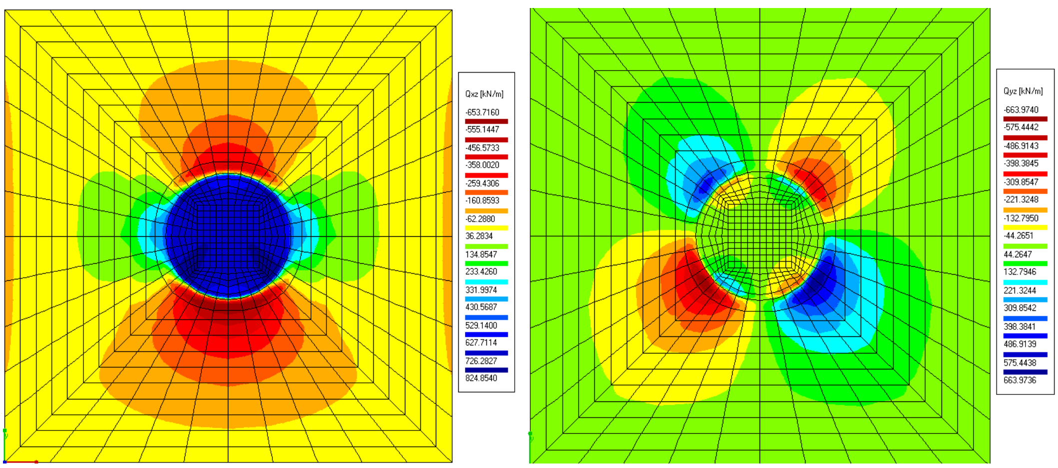

Figure 16: Seismic action (wind at 90°) – Shear Forces Q_{xz} and Q_{yz}

5 Design of the Slab in Reinforced Concrete

With the load case combinations envelopes of forces, moments and stress resultants can be determined, with which the verification of the tower design is achieved with respect to regulatory strength, stiffness and stability requirements.

On the base of the obtained forces and stresses the reinforced concrete slab was designed for the control load case combination, corresponding to the scenario 1 of the base action 'Wind' with a 90º incidence towards the structure.

The following properties were used for the materials: concrete B30 (C25/30), with a compressive strength f_{cd} = 16.67 MPa and a shear strength such that τ_1 = 0.75 MPa ; Steel A500 (S500) with yield strength fsyd = 435 MPa.

Using standard practice for the concrete design of beams and of one-way or two-way slabs [6], one would find the following Table 10 and Table 11 related to the reinforcement characteristics required to withstand the flexure and shear actions in the foundation slab. Additionally, considerations should be taken for the deflection control in the slabs, assessing flexural cracking limited by tolerable crack widths [6].

				Reinforcement	
Msd,x$^+$ [kN.m/m]	Msd,x$^-$ [kN.m/m]	Asy$^+$min [cm^2/m]	Asy$^-$min [cm^2/m]	Asy$^+$eff [cm^2/m]	Asy$^-$eff [cm^2/m]
3380.0	-2100.0	55.6	34.5	78.4	49.6

				Reinforcement	
Msd,y$^+$ [kN.m/m]	Msd,y$^-$ [kN.m/m]	Asx$^+$min [cm^2/m]	Asx$^-$min [cm^2/m]	Asx$^+$eff [cm^2/m]	Asx$^-$eff [cm^2/m]
3380.0	-2100.0	55.6	34.5	78.4	49.6

Table 10: Verification of bending reinforcement (average height h=1.50 m)

	Vsd [kN/m]	Vcd [kN/m]	Vrd [kN/m]	Verif.
h=2.00 m	1850.0	1911.0	1911.0	Ok !

Table 11: Verification capacity for shear

In view of the geotechnical properties and characteristics at the wind tower site at 'Serra do Barroso', it can be said that the stresses underneath the proposed foundation – of the order of 180 kPa – are well below the bearing capacity of the foundation media of around 300 kPa. Moreover, footings uplift would not occur beyond about 40% of foundation area.

6 Conclusions

From this technical study it is concluded that the global safety of the wind tower generator foundation is guaranteed in accordance with the actual Portuguese regulatory design codes (RSA. and REBAP) as well as with Eurocode EC2.

Similarly, geotechnical safety is equally guaranteed since the contact pressures on the foundation interface are well below the bearing capacity at the site.

The static balance was also verified in the static equilibrium analysis, even for the worst combination of actions that control the analysis (base action: Wind). Foundation uplift would not occur beyond about 40% of foundation area.

Although important, seismic action effects do not control design for this tower at this implantation site. Wind tower design is controlled by a load case with base action ‘Wind’. This is due to the fact that the Wind Farm at Serra do Barroso is located in a very low seismicity zone.

Acknowledgements

This research was partially developed within the scientific and technical framework of *Centro de Engenharia Civil* at FEUP, associated with the activities of *Laboratório de Engenharia Sísmica e Estrutural* (LESE).

The authors wish to thank the office of consultants “*Ferreira Lemos – engenharia*”, for the permission and for the opportunity to publish part of this analysis and design technical note.

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