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# DESIGN OF LATTICE WIND TOWERS AND COMPARISON WITH THE TYPICAL SELF-SUPPORTED TUBULAR TOWERS

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## ABSTRACT

The following work aims to model and design a latticed wind tower 150 meters height, and afterwards establish a comparison with the typical self-supported tubular towers.

The actions considered in this kind of tower design were the wind actions, combined with the action of the ice, and the seismic actions; taking into account all dynamic effects. For the design process it was developed an Excel application using Visual Basic programming, which calculates the wind action according to the disposals established by the Eurocodes (EN 1991-1-4 and EN 1993-3-1).

A lattice tall tower geometry was defined by the authors interacting with the metallic company Metalogalva (Trofa, Portugal), satisfying the needed characteristics to support a wind turbine. The metallic calculations were achieved with the Autodesk Robot Structural Analysis Professional 2012, iteratively with a calculation program of the wind actions developed by the authors. As expected in this kind of towers, the actions that conditioned the design were the wind action combined with ice.

**Keywords:** wind turbine tower, latticed towers, tubular towers, Eurocodes, dynamic effects

## INTRODUCTION

With fossil fuels becoming increasingly scarce and increasingly expensive [1], the world seeks a solution that serves the interest of economic development and the preservation of nature. This concern of the World is leading to an increasing demand for renewable energy sources such as wind energy [2]. Wind energy assumes a very important role in the global panorama of energy, as it is a source of renewable energy that has the least impact on nature [3], as shown in Figure 1 below. The rising demand of wind energy has lead to a huge development of the related technologies, for example on the type of tower to be used.

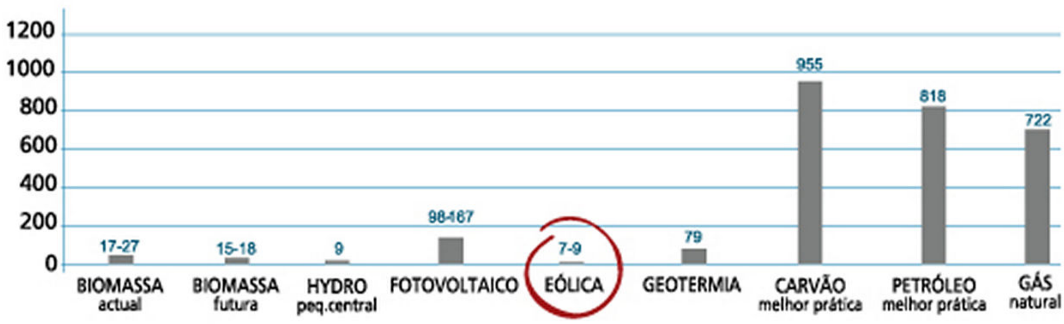


Fig. 1 - Environmental impact of the different sources of electrical energy [3]

## GENERAL CONSIDERATIONS

The geometry of the tower (Fig. 2) was developed and used by the first co-author of this paper, in his master of science thesis [4]. The geometric and technical requisites have been: 150 m of height and dimensions of turbine (FL2500 of 2,5 MW and rotor diameter of 100m).

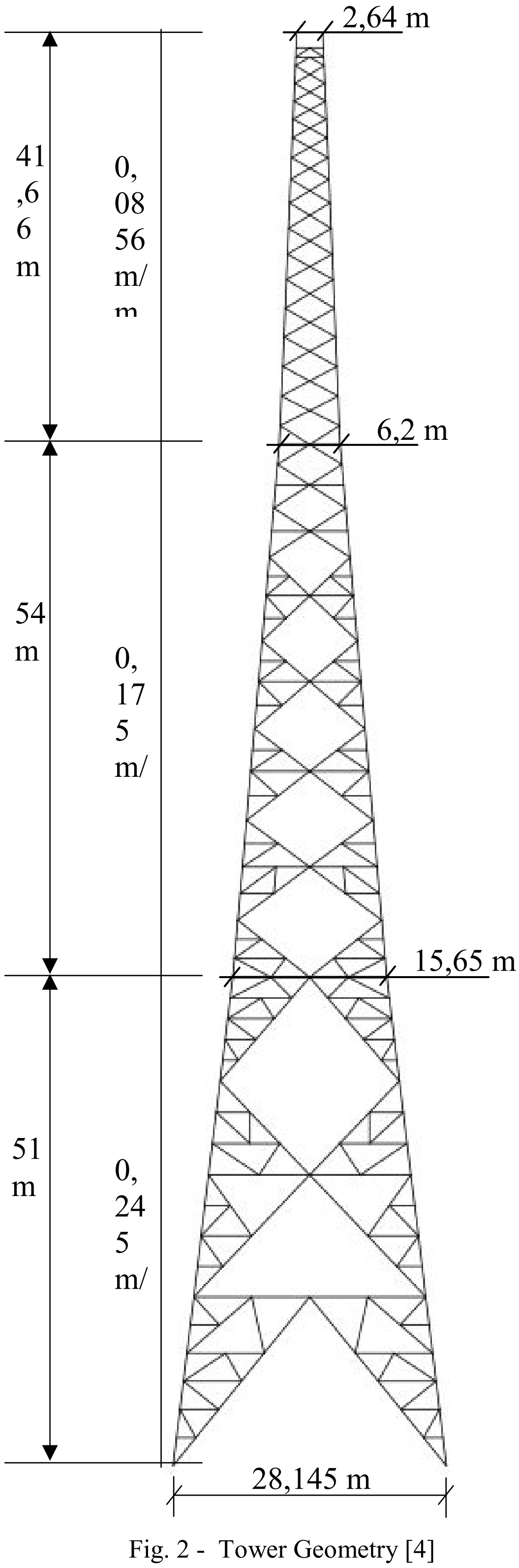


Fig. 2 - Tower Geometry [4]

The elements of the structure were disposed based on rules of triangles so as to shorten the buckling lengths of the structural system members. The construction of the geometry of the beams and diagonals was based on the Eurocode EN 1993-3-1 [5] and on dispositions of existing towers.

The sections of the bars used in the tower are angles and association of angles (Fig. 3). The steel used in the design is S235 and S355.

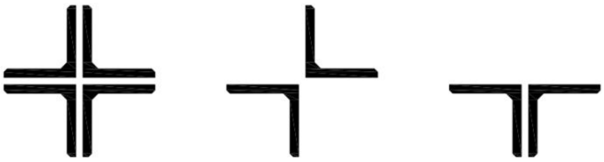

Fig. 3 - Association of angles [4]

The model of the tower was introduced in Autodesk Robot Structural Analysis Professional 2012, using model bars linked through rigid connections; the foundations were modeled with supports that restrict all displacements and rotations.

The non-structural elements (stairs, cables) were not modeled, however were introduced two additional nodes in the bar elements of support to non-structural elements, so as to introduce the forces resulting from the actions on the non-structural elements.

The modeling of the wind turbine (by itself) was not performed. However were introduced bars with great rigidity and null weight to simulate the rigidity of the wind turbine on the top of the latticed tower structure. The weight of the wind turbine was considered at the top the tower by adding four vertical forces in the top of the tower with 362.60 kN each.

For definition of wind action this tower was divided into 14 panels, as seen in Fig. 4.

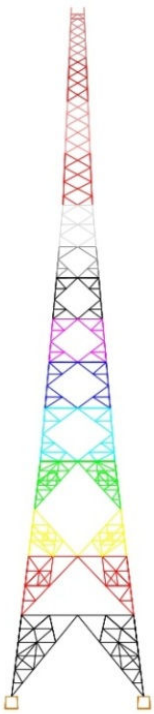

Fig. 4 – Definition of 14 panels [4]

# ACTIONS

## Wind Action

Wind is an air movement initiated by the transport of air masses in the atmosphere, and its development is related with the variations of air pressures which, in turn, are originated thermally by means of radiation. The movement of the wind when it encounters an obstacle causes a loading in the obstacle. It is therefore necessary, in some structures, consideration of the wind for their analysis and design [6].

The study of the wind characteristics is in the field of meteorology. It is therefore this science that provides to the designer the information of the wind flow characteristics, which are necessary for the determination of the wind action [7].

At the same time, the problem of a given load due to wind action also is a subject in aerodynamics. This discipline is used for many purposes in aeronautics and in the car industry, and ever since was applied in structural engineering by helping the designer in the determination of loading from wind actions [8] [9].

Joining the information supplied by the meteorology and aerodynamics, the designer can begin the resolution of the problem.

However it will be still necessary to take into account the vibrations due to wind actions. This additional effect will be achieved through the application of laws of aeroelasticity.

Therefore calculation of the dynamic effect of wind action into slender structures, which includes the case of this study, is composed by three stages: description of the wind, description of physical and aerodynamics properties and the combination of these factors for determination of the structural response. These procedures are the basics of Eurocodes EN 1991-1-4 [9] and EN 1993-3-1 [5], which were applied in this study as well as in [9].

The wind force in the direction of the wind on the tower, according to the Eurocodes, is determined by of the following expressions.

For the mean wind load:

$$F_{m,W}(z) = \frac{q_p(z_e)}{1+7.I_v(z_e)}\sum c_f.A_{ref} \tag{1}$$

For equivalent gust wind load:

$$F_{T,W}(z) = F_{m,W}(z).\left[1+\left[1+0{,}2.\left(\frac{z_m}{h}\right)^2\right].\frac{[1+7.I_v(z)].c_sc_d-1}{c_o(z_m)}\right] \tag{2}$$

where

| | |
|---|---|
| $I_v(z_e)$ | is the turbulence intensity at height $z_e$ and is defined as the standard deviation of the turbulence divided by the mean wind velocity. The $I_v$ depends of the basic wind velocity, terrain factor, turbulence factor, orography factor and roughness length |
| $q_p(z_e)$ | is peak velocity pressure at height $z_e$ and includes mean and short-term velocity fluctuations. Depends of the air density, turbulence intensity and mean wind velocity |
| $A_{ref}$ | is the reference area of the structure or structural element. |
| $c_sc_d$ | is the structural factor |
| $c_f$ | is the force coefficient for the structure or structural element. |

The structural factor ($c_s c_d$) determines the dynamic response of structures in the fundamental mode of vibration, and can be divided into its components: the size factor ($c_s$) and the dynamic factor ($c_d$). The structural factor combines effects of non-simultaneous action of peak wind pressures over faces of the structure (generally called the 'size effect') and vibration of the structure in its fundamental mode due to the action of turbulence (generally called the "dynamic response") [9].

The calculation procedure assesses the dynamic response of a structure in the along-wind direction as the root-sum-square of a "background" and a "resonant" component. The background component represents the quasi-steady (i.e. not amplified) response of the structure to the atmospheric turbulence, while the resonant part represents the dynamic oscillation of the structure at its natural frequencies. This is usually called the Davenport method [9].

The structural factor is calculated by the following expression:

$$c_s c_d = \frac{1 + 2.k_p.I_v(z_s).\sqrt{B^2 + R^2}}{1 + 7.I_v(z_s)} \tag{3}$$

where

$z_s$ is the reference height for determining the structural factor

$k_p$ is the peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation

$B^2$ is the background factor, allowing for the lack of full correlation of the pressure on the structure surface

$R^2$ is the resonance response factor, allowing for turbulence in resonance with the vibration mode.

In Table 2 are represented the required parameters to define the wind action on the tower. The tower under study will be deployed in a zone type B, with the terrain category 2.

Table 1 - Parameters for the definition of wind action

| Designation | | | |
|---|---|---|---|
| Basic wind velocity | $v_b =$ | 30 | m/s |
| The reference height for determining the structural factor | $z_s =$ | 87,996 | m |
| Mean wind velocity at a height $z_s$ | $v_m(zs) =$ | 42,60 | m/s |
| Turbulence intensity at height $z_s$ | $I_v(zs) =$ | 0,134 | |
| Turbulent length scale | $L(zs) =$ | 195,72 | m |
| Non-dimensional power spectral density | $S_L(zs, n) =$ | 0,115 | |
| Non-dimensional frequency | $f_L(zs, n) =$ | 1,103 | |
| Background factor | $B^2 =$ | 0,544 | |
| Resonance response factor | $R^2 =$ | 0,103 | |
| | $\eta_h =$ | 3,801 | |
| | $\eta_b =$ | 0,720 | |
| Aerodynamic admittance | $R_h =$ | 0,228 | |
| Aerodynamic admittance | $R_b =$ | 0,653 | |
| Peak factor | $k_p =$ | 3,057 | |
| Up-crossing frequency | $v =$ | 0,096 | |
| Logarithmic decrement of damping | $\delta =$ | 0,827 | |
| Logarithmic decrement of aerodynamic damping | $\delta_a =$ | 0,777 | |
| Logarithmic decrement of structural damping | $\delta_s =$ | 0,050 | |
| Equivalent mass per unit length | $m_e =$ | 12271,4 | Kg/m |
| Force coefficient for wind action in the wind direction | $c_f =$ | 3,093 | |
| Structural factor | $c_s c_d =$ | 0,856 | |

Taking into account the constitution and the symmetry of the tower, were analysed two directions of wind: 0º and 45º (Fig. 5).

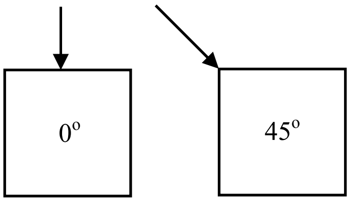


Fig. 5 - Wind orientations [4]

**Ice Action and Combination of Wind Action with Ice Action**

The ice action and the combination of the wind action with the ice action are determined as described in EN 1993-3-1 [5] and ISO 12494 [8] R.F. Almeida, R.C. Barros. Análise do efeito dinâmico do vento em torres metálicas. V Congresso de Construção Metálica e Mista, Portugal, 2005.

[9] N.A. Ferreira, R.C. Barros, R. Delgado. Comparisons of a tall building wind response with and without a TMD. 3rd International Conf on Computational Methods in Structural Dynamics and Earthquake Engng (CompDyn 2011: # 654), Corfu, 2011.

], and as were used in [8]. For exposed locations, the atmospheric ice on towers can grow considerably in thickness; when combined with the wind action, this total action on the structure can increase considerably conditioning the design.

The magnitude, the density, the placement and shape of the ice on the towers depend on the local weather conditions, topography and structure shape. For engineering design it is usually considered that all members of a tower are covered with uniform thickness and uniform density of ice. To be able to express the amount of ice that may be formed at a given site, the term Ice Class (IC) was introduced in ISO 12494 [8] R.F. Almeida, R.C. Barros. Análise do efeito dinâmico do vento em torres metálicas. V Congresso de Construção Metálica e Mista, Portugal, 2005.

[9] N.A. Ferreira, R.C. Barros, R. Delgado. Comparisons of a tall building wind response with and without a TMD. 3rd International Conf on Computational Methods in Structural Dynamics and Earthquake Engng (CompDyn 2011: # 654), Corfu, 2011.

]. In the design of this tower was chosen the glaze ice type, with 900 kg/m$^3$ of density. Within the glaze ice type was chosen IG2 class, where the thickness of ice to be used is 20 mm and the *k* parameter is *0.45*.

The combinations of ice and wind to be taken into consideration have been:

$$\gamma_G \, G_k + \gamma_{ice} \, Q_{k,ice} + \gamma_w \, k \, \psi_w \, Q_{k,w} \tag{4}$$

$$\gamma_G \, G_k + \gamma_w \, k \, Q_{k,w} + \gamma_{ice} \, \psi_{ice} \, Q_{k,ice} \tag{5}$$

The factors $\gamma$ are given in EN 1993-3-1 [5] and additional information in EN 1993-1-1 [12].

Table 2 shows the forces of the wind action on the final structure, in the direction at 0º.

Table 2 – Wind forces and wind+ice combined forces

| | | | | | Wind | | | | Wind with Ice | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Panels** | **$z_e$ (m)** | **$v_m(z_e)$** | **$q_p$ ($z_e$)** | **$I_v$ ($z_e$)** | **$A_{ref}$** | **$c_f$** | **$F_{m,W}$** | **$F_{T,W}$** | **$A_{ref}$** | **$c_f$** | **$F_{m,W}$** | **$F_{T,W}$** |

| | | (m/s) | | | ($m^2$) | | (kN) | (kN) | ($m^2$) | | (kN) | (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| panel 1 | 18,60 | 33,74 | 1,553 | 0,169 | 48,28 | 3,355 | 115,26 | 215,36 | 56,99 | 3,257 | 132,02 | 246,68 |
| panel 2 | 30,95 | 36,64 | 1,753 | 0,156 | 28,28 | 3,386 | 80,35 | 143,69 | 33,56 | 3,289 | 92,59 | 165,59 |
| panel 3 | 40,98 | 38,24 | 1,868 | 0,149 | 20,74 | 3,370 | 63,87 | 111,72 | 24,87 | 3,263 | 74,18 | 129,76 |
| panel 4 | 51,00 | 39,49 | 1,959 | 0,144 | 26,63 | 3,123 | 81,05 | 139,50 | 32,00 | 2,978 | 92,86 | 159,82 |
| panel 5 | 62,20 | 40,62 | 2,044 | 0,140 | 24,60 | 3,160 | 80,16 | 136,03 | 29,57 | 3,020 | 92,07 | 156,25 |
| panel 6 | 71,90 | 41,44 | 2,107 | 0,138 | 20,48 | 3,091 | 67,96 | 114,19 | 24,19 | 2,956 | 76,78 | 129,01 |
| panel 7 | 81,00 | 42,12 | 2,159 | 0,135 | 17,69 | 3,048 | 59,81 | 99,71 | 21,03 | 2,903 | 67,71 | 112,86 |
| panel 8 | 89,70 | 42,71 | 2,205 | 0,133 | 15,96 | 2,973 | 54,10 | 89,58 | 18,97 | 2,819 | 60,96 | 100,94 |
| panel 9 | 96,23 | 43,11 | 2,236 | 0,132 | 11,89 | 2,849 | 39,33 | 64,83 | 13,99 | 2,692 | 43,73 | 72,09 |
| panel 10 | 105,00 | 43,60 | 2,276 | 0,131 | 17,53 | 2,582 | 53,78 | 88,17 | 20,07 | 2,437 | 58,11 | 95,27 |
| panel 11 | 115,93 | 44,17 | 2,321 | 0,129 | 19,22 | 2,510 | 58,81 | 95,83 | 21,91 | 2,366 | 63,23 | 103,02 |
| panel 12 | 126,93 | 44,68 | 2,362 | 0,128 | 18,96 | 2,335 | 55,26 | 89,55 | 21,61 | 2,190 | 59,06 | 95,70 |
| panel 13 | 136,13 | 45,08 | 2,395 | 0,126 | 15,73 | 2,125 | 42,47 | 68,53 | 17,92 | 1,989 | 45,27 | 73,06 |
| panel 14 | 146,66 | 45,51 | 2,429 | 0,125 | 16,95 | 1,941 | 42,56 | 68,38 | 19,47 | 1,823 | 45,96 | 73,84 |

## Other Actions

The self-weight of the superstructure was calculated from the specific weight of steel of 78.5kN/$m^3$. Was even considered a sel-weight increase of 15%, to simulate the weight of the connecting elements and galvanization.

For the seismic action the tower was considered installed in zone A and having a behavior factor of 2.5, satisfying the regulatory conditions of the Eurocodes EN 1998-1-1 [11] and EN 1998-6 [12]. This action was not the controlling conditioning action for the design of the tall latticed wind turbine tower [4] [14] [16].

It was still considered the wind action on the non-structural elements and on the turbine in the most unfavourable position (Table 3).

Table 3 - Wind forces in the turbine

| Blade | cf | z (m) | $A_{ref}$ ($m^2$) | $v_m(z)$(m/s) | $F_{T,W}$ (kN) | $\Sigma F_{T,W}$ (kN) | $M_X$ (kN.m) | $\Sigma M_X$ (kN.m) | $M_Z$ (kN.m) | $\Sigma M_z$ (kN.m) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0,5 | 137,5 | 138,43 | 45,14 | 141,86 | | -1773,30 | | -3071,35 | |
| 2 | 0,5 | 137,5 | 138,43 | 45,14 | 141,86 | 432,27 | -1773,30 | 167,08 | 3071,35 | 0,00 |
| 3 | 0,5 | 175,0 | 138,43 | 46,51 | 148,55 | | 3713,67 | | 0,00 | |

## DESIGN

The regulatory provisions needed to design this type of structures – steel lattice towers – are given in Eurocodes EN 1993-1-1 [10] and EN 1993-3-1 [5]. In these references more specific aspects discussed are related to: classification of the structure, analysis method, incorporation of imperfections and safety verification of the structure (sections and bars).

According to EN 1993-3-1 [5], lattice towers may be analysed using the initial geometry (first order theory). Due to their structure, the lattice towers mainly present compressive forces on the bars. The elements used in the construction of lattice towers are slender thus when subjected to big compressive forces, characterized by the occurrence of large transversal deformations, are susceptible to instability. This phenomenon is named buckling and, more simply, consists in the appearance of secondary bending moments (and succesive larger deformations) due only to axial compressive forces.

The critical load, i.e., the load for which the structural element starts to develop undetermined lateral deformations when only axially loaded, is defined by the equation (6). This formulation is based on several hypothesis, which are: material with linear elastic behavior, bars without initial geometric imperfections and residual stresses, and load perfectly centered applied at the nodal points.

$$N_{cr} = \frac{\pi^2.E.I}{L^2} \tag{6}$$

Thus buckling resistance of a given element depends on the bending stiffness of the cross section, of its length and of the supporting conditions. According to the Eurocode EC3 (EN 1993-1-1 [10]) the design of elements subjected to simple compression is based on the buckling curves. The use of the curves lets reproduce the effect of imperfections of real bar members (lack-of-straightness, eccentricity of the loads, residual stresses, etc) replacing these by a deformed equivalent initial configuration.

In the Eurocode EN1993-1-1 [10], the resistance of the cross sections of elements axially compressed is given by the following condition (7):

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1{,}0 \tag{7}$$

where $N_{ed}$ is the design normal force and $N_{c,Rd}$ is the design resistance to normal forces of the cross-section for uniform compression and is given by the expression (8).

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} \tag{8}$$

where $A$ is the total area of the section and $f_y$ is yield strength.

The compression members shall additionally verify the condition $N_{Ed} \leq N_{b,Rd}$ which in general is more conditioning. The design buckling resistance of a compression member ($N_{b,Rd}$) is calculated by:

$$N_{b,Rd} = \frac{\chi \times A \times f_y}{\gamma_{M1}} \tag{9}$$

In expression (9) $\chi$ is the reduction factor for the relevant buckling mode and is calculated by the following expression

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but such that} \quad \chi \leq 1 \tag{10}$$

with $\phi = 0.5\,[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2]$ where $\bar{\lambda}$ is a non-dimensional slenderness given by the following expression

$$\bar{\lambda} = \sqrt{\frac{A.f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \tag{11}$$

where

$A$ is an imperfection factor

$N_{cr}$ is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

$L_{cr}$ is the buckling length in the buckling plane considered

$I$ is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section

$\lambda_1$ is given by $\lambda_1 = \pi\sqrt{E/f_y} = 93{,}9\,\varepsilon$ and $\varepsilon = \sqrt{235/f_y}$

Some existing compound members are designed according to the EN 1993-1-1 [10] as built-up compression members with: (a) chords in contact or closely spaced and connected through packing plates; (b) star battened angle members connected by pairs of battens in two perpendicular planes. These built-up compression members should be checked for buckling as a single integral member, ignoring the effect of shear stiffness (SV = ∞) provided that connections are performed along its length with a maximum spacing of 15 $i_{min}$ (or 70 $i_{min}$ , in the case of bars connected by pairs of battens, as shown in Fig. 6).

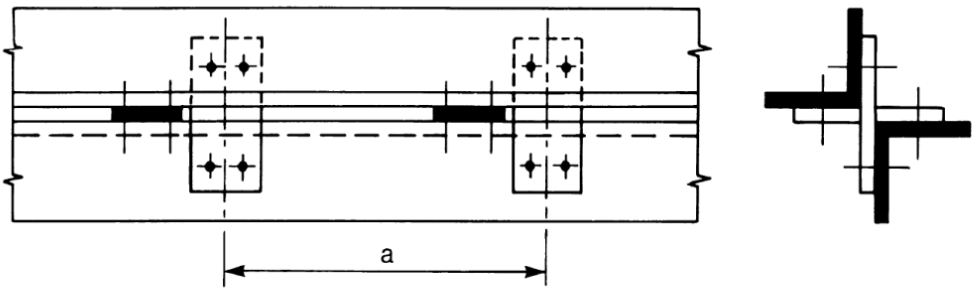


Fig. 6 - Star-battened angle members [10]

In this work were defined several buckling lengths for the different bars and different directions. Table 4 indicates the buckling lengths used.

Table 4 - Buckling lengths

| **Buckling length** |
|---|
| 1 L |
| 0,5 L |
| 0,33 L |
| 0,25 L |
| 0,20 L |

The rational for selecting and using buckling lengths is explained herein. The elements which only are locked at the ends have a buckling length equal to the real length of the bar. The elements which are locked by one, two, three or four lockups, beyond the ends locked, have a buckling length of 0.5L, 0.33L, 0.25L and 0.20L, respectively.

The metallic calculation was achieved with the Autodesk Robot Structural Analysis Professional 2012, being adequately verified by calculating the most significant bars.

For the design process an Excel application was developed, using Visual Basic programming, which calculates the wind action and the ice action according to the disposals established by the Eurocodes (EN 1991-1-4 [10] and EN 1993-3-1 [5]). This application works jointly with Robot Structural Analysis 2012; so with the geometric properties of the tower and with the mechanical properties of the bars available from the Robot, the program develops the force value to be applied in each of the structural panels.

The design process of lattice structures subjected to wind action (and of wind action combined with ice action) is an iterative process, because for each structural design the exposition area to wind is changed. The iterative process is detailed in the following Fig. 7.

A preliminary design of the tower foundations was also performed, following the Eurocode EN 1997-1-1 [15] and the DNV rules [16]. Since it is beyond the scope of the present contribution, no more reference to such analysis and design will be mentioned herein.

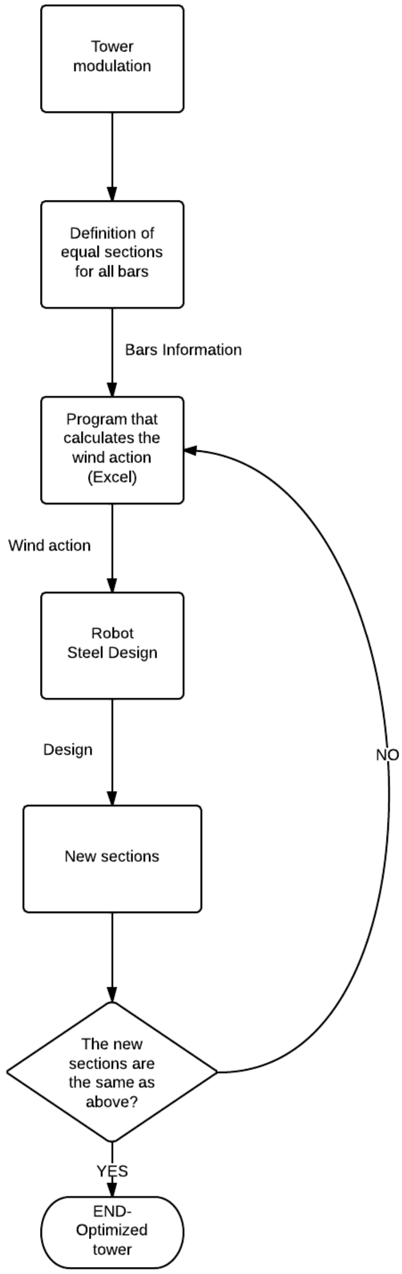


Fig. 7 - Flowchart of the design process [4]

## RESULTS AND COMPARATIVE STUDY

The weight results from the design process are presented in summary on Table 5.

Table 5 - Design Summary

| | kg | t |
|---|---|---|
| Tower (with links and galvanization) | 350422 | 350,422 |
| Foundations | 1028750 | 1028,75 |

Table 6 presents a comparison of the costs of the lattice tower designed with the one's of an equivalent tubular tower. The information of the tubular tower was taken from a study conducted in Sweden [13].

Table 6 - Comparative table of costs of the two solutions

| **1 - Tower** | | **Tubular Tower (150 m)** | **Lattice tower (150m)** |
|---|---|---|---|
| Diameter/Width (top/base) | m | 3,0/4,5 | 2,64/28,15 |
| Plate thickness (min/max) | mm | 15/75 | - |
| Weight | t | 610 | 350,422 |
| Tower total | 1.000 € | 1404 | 806 |
| | | | |
| **2 - Transportation** | | | |
| Blades, hub, nacelle | 1.000 € | 29 | 29 |
| Tower | 1.000 € | 92 | 30 |
| Transportation total | 1.000 € | 121 | 59 |
| | | | |
| **3 - Lifting** | | | |
| Heaviest lift | t | 120 | 120 |
| No crane hours (6 hours/lift) | h | 46 | 46 |
| Lifting total | 1.000 € | 124 | 124 |
| | | | |
| **4 - Foundation** | | | |
| Weight | t | 3181 | 1028,75 |
| Foundation total | 1.000 € | 350 | 114 |
| | | | |
| 5 – Power cable in tower | 1.000 € | 74 | 74 |
| | | | |
| 6 – Wind turbine price | 1.000 € | 2783 | 2783 |
| | | | |
| **7 – Total price** | 1.000 € | 4855 | 3960 |

## CONCLUSIONS

The final results for the lattice tower solution were somehow expected: in the body of the towers it is possible to save steel; as well as saving concrete in the foundations. The concrete amount used in a lattice tower foundation represents only 35% of the concrete used in a tubular tower with the same height (Table 6). In relation to the tower's weight it is possible to save almost 40%, when compared to a tubular tower with the same height (150 m).

In conclusion, the initial cost of a tall lattice wind tower is cheaper and the capacity to elevate the rotors is higher, which allows them to produce more energy in relation to the self-supported tubular towers.

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