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DYNAMIC BEHAVIOUR OF A GUYED TOWER: DESIGN STUDY

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ABSTRACT

The work presented in this paper pretends to detail the main aspects and concepts associated with the design of a guyed tower. In this context, are presented the theoretical foundations and the essential steps for the various methods of analysis. This topic is therefore of major interest for Metalogalva (Trofa, Portugal), with which the authors collaborate and interact, since this industry complex of metallic constructions is also involved in the design and construction of tall telecommunication towers and pole structures.

Two dynamic actions are of paramount concern for the guyed tower design: The gust effect of wind and the action of earthquakes. Still concerning the wind, reference is made to the evaluation of the cross direction of wind response.

Therefore this paper presents various considerations and topics involved in the modeling of a 185 m height guyed tower, that enable to characterize their structural response. Since this is a slender structure it is was necessary to take into account the effects of second-order P-Δ analysis and also the nonlinear behavior of cables.

Keywords: Guyed tower, Design, Dynamic action, P-Δ effect, Geometric non-linearity.

1. INTRODUCTION

Guyed towers are comprised of a very slender mast supported laterally by inclined cables (in tension), which are anchored in concrete foundations. This setting appears as an economical option to self-supporting towers. However, its behavior is generally nonlinear and therefore is complicated to study and describe. The simplifications made and the approximate models used in their design over the years, often unjustified, led to the collapse of many structures.

For the study of the dynamic behavior of the guyed tower it is really important to understand how it reacts to the wind and to an earthquake. Figure 1 shows typical normalized spectral densities of wind and earthquake actions, where average frequencies of concern for wind and earthquake actions are also emphasized. As described, these two dynamic actions excite quite differently any type of tall (and low) structures.

The natural frequencies of truss structures are between 0,5 and 3 Hz. When the masts are very flexible it is essential to study the dynamic behavior in the first mode response due wind action, as well as the contribution of the second order P-Δ effect related to structural instability.

The resonant response of slender structures becomes important when the natural frequency of the structure is below 1 Hz (Carril, 2000). Between the two dynamic actions considered for this study on guyed towers, the wind is generally the controlling design action in this kind of structures.

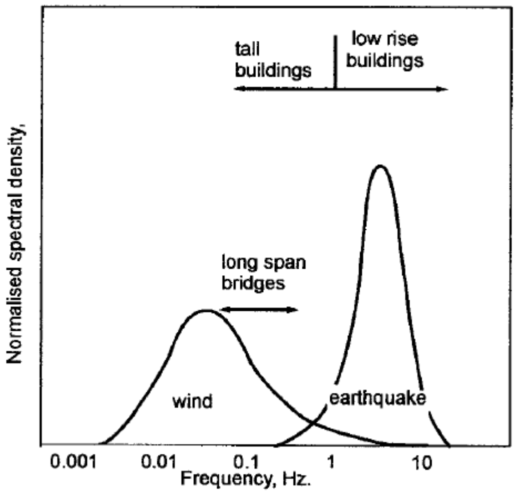


Fig. 1 – Dynamic excitation frequencies of structures by wind and earthquakes (Holmes, 2001)

2. THE GUST EFFECT

2.1. Considerations

The generalized forces of wind on the masts can be characterized by quasi-static and dynamic components. Both forces and the displacements associated depend primarily on the fundamental mode and frequency and its damping.

The quasi-static behavior and the time-varying behavior of these pole masts structures occurs along-the-wind (that is, in the direction of propagation of the wind) and is due to the addition of a constant wind pressure with a non-permanent gust pressure. The purely dynamic vibratory behavior of the pole-masts occurs in the transverse direction of propagation of wind (across-the-wind) and is due to the aerodynamic phenomenon of vortex shedding at the critical wind speed.

Barros (2002) considered the wind pressure P_i (t), at any point i of the metallic mast in the instant t, expressed by adding constant (time-independent) and time-dependent contributions:

$$P_i(t) = \overline{P}_i + p_i(t) \tag{1}$$

$$\overline{P_i} = \frac{1}{2}\rho(c_d)_i\overline{V_i}^2 \tag{2}$$

$$p_i(t) = \frac{1}{2}\rho(c_m)_i\overline{V_i}v_i(t) \tag{3}$$

where ρ is the air density, $\overline{V}_i$ is the average wind speed in section *i* at time *t*, and $v_i(t)$ the fluctuation of wind velocity relative to its average value; c_d and c_m are coefficients of resistance and mass of the mast.

2.2. Standards

The reference standards used in the study of the behavior of lattice towers (fixed or guyed) subjected to wind are: British Standard BS 8100-3 (1999), American Standard ASCE (2000), German Standard DIN V 4131 (2008) and the European Norm EN 1991-1-4 (2005, 2010).

The wind speed varies with height and its variation in time also depends on the type of ground where it is deployed. The wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the effects of turbulent wind (Ferreira et al., 2011).

The characteristic values of the wind actions calculated in EN 1991-1-4 (2010) present an annual probability of being exceeded (2%) equivalent to an average recurrence period of 50 years. The purely dynamic behavior of masts occurs mainly in the transverse direction of the wind (across-the-wind).

The mast structure under analysis and design verifications is a 185 m height tall guyed tower, with structure and cables layout detailed in Fig. 2, located in Lisbon area of soil type D (Mendonça, 2012).

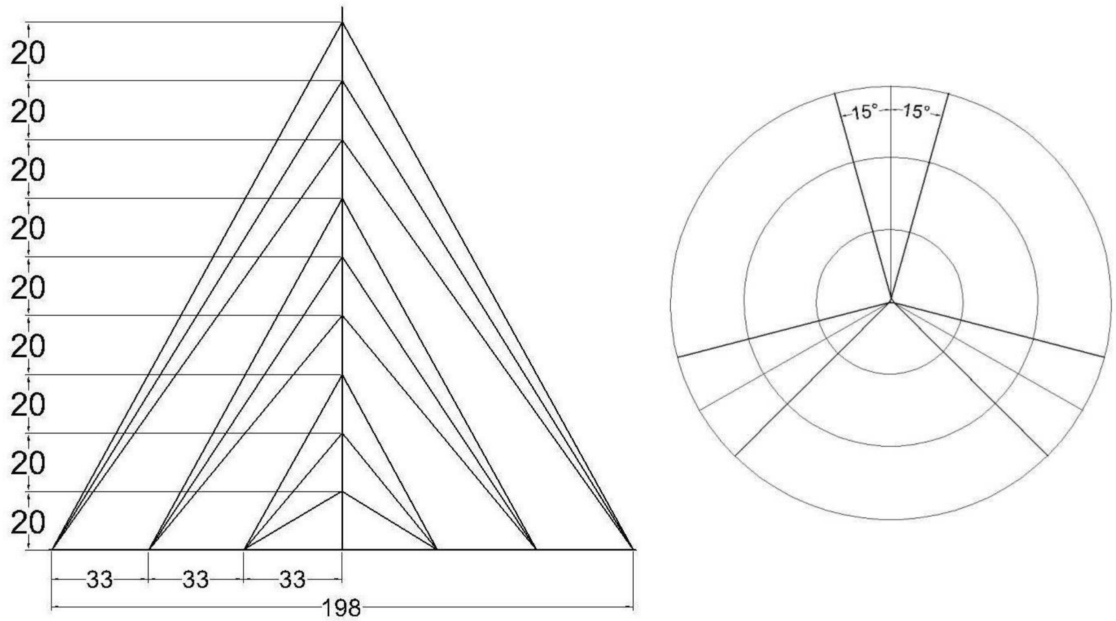


Fig. 2 – Layout (elevation and plan) of the guyed tower 185 m height

Based on Annex E of EN 1991-1-4 (2010), equation (1) and on details in Ferreira et al. (2011), it is possible to evaluate the forces acting on a specific guyed tower model (Fig. 3) and access the mast structural response to vortex shedding as well as ovalization (Zar and Chu, 1979).

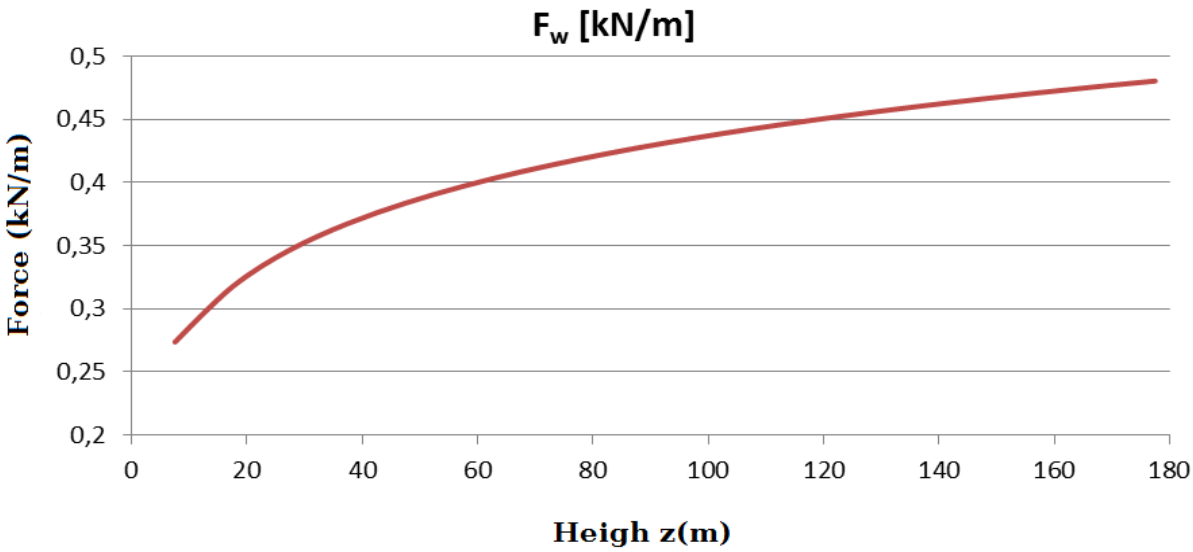


Fig. 3 – Force of the wind o the 185 m guyed tower mast (kN/m)

3. EARTHQUAKES

3.1. General considerations

According to Chopra (2007) when analyzing the response of a general structure to seismic action, the equation of motion of the multi-degree-of-freedom structure is:

$$m\,\ddot{u}_{total} + c\,\dot{u} + k\,u = 0 \tag{4}$$

where *m* is the mass, *c* is damping, *u* is the displacement and *k* is the stiffness of the structure.

The total displacement $u_{total} = u^t$ is equal to the displacement of soil $u_g(t)$, plus the relative displacement of the structure $u(t)$.

$$u^t = u + u_g \tag{5}$$

The seismic movement only causes a dynamic response because the inertia forces depend on the total displacement of the structure (in fact they are expressed in terms of the total acceleration), while the elastic forces and damping depend only on the relative motion.

$$m\ddot{u}^t + c\dot{u} + ku = 0 \leftrightarrow m(\ddot{u} + \ddot{u}_g) + c\dot{u} + ku = 0 \leftrightarrow m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \tag{6}$$

From equation (6), it may be inferred that the structure is acted upon by the seismic force that is defined as $F(t) = m\,\ddot{u}_g(t)$.

3.2. Spectral analyses – Standards

To design a structure subjected to seismic actions, two regulatory norms were used: the Portuguese regulation RSA (1983) and Eurocode 8 (EN 1998-1-1, 2010 ; EN 1998-6, 2003). There are 2 types of spectral analyses: using spectral density functions (for frequency domain analyses) associated with specific probabilities of occurrence of the seismic action; or using (acceleration) response spectra to evaluate expected maximum values of modal contributions.

3.2.1. RSA method

RSA is based upon 4 zones of seismicity (A-B-C-D) defined for Portugal, to help quantifying the action. The seismic action is quantified using seismic coefficients that reflect the influence of the different seismicity zones and of structural characteristics. It also gives consideration to the soil type and the kind of earthquake (types I or II). The spectral density functions, for the 3 soil types, under RSA standards are given in Table 1.

Table 1 – Spectral density functions for zone A

Soil Type I			Soil Type II			Soil Type III		
f [Hz]	S(f) [(cm/s^2)]		f [Hz]	S(f) [(cm/s^2)]		f [Hz]	S(f) [(cm/s^2)]	
	Earthq. type 1	Earthq. type 2		Earthq. type 1	Earthq. type 2		Earthq. type 1	Earthq. type 2
0,04	0	0	0,03	0	0	0,02	0	0
1,05	250	220	0,9	220	220	0,75	190	220
2,1	360	300	1,8	300	400	1,5	240	500
4,2	360	150	3,6	300	160	3	240	200
8,4	160	65	7,2	130	65	6	100	80
16,8	50	20	14,4	40	25	12	35	30
20	20	0	20	16	0	20	12	0

3.2.2. Eurocode 8 (EN 1998) method

The seismicity in Eurocode 8 (EC8) also has a possible double scenario: Earthquake caused far away (seismic action type 1) of Magnitude > 5.5; Earthquake caused by near-fault (seismic action type 2) of Magnitude < 5.5 .

The safe design of a structure to resist seismic actions is related to performance requirements:

- Requirement of no collapse (NCR) – For current structures the return period is 475 years; for special structures that period increases.
- Requirement of damage limitation (DLR) under service conditions – For current structures the return period is 95 years.

The action depends on construction's class of seismic importance, γ_I. The importance classes are defined in the National Annex and its values vary between 0.6 and 1.95. For current buildings it assumes a value of 1, for buildings of less importance, such as agricultural buildings, values less than one; and to buildings essential to post-earthquake's aid can take values up to 1.95. The value of γ_I varies with the type of earthquake and values can be found in Table II of National Annex.

The seismic action zoning is different for each type (1 or 2) of earthquake. As can be seen in EC8, the areas of Lisbon are 1.3 and 2.3. The maximum reference accelerations in all seismic zones can be found in Table 2.

Table 2 – Maximum reference ground accelerations

Earthquake Type 1		Earthquake Type 2	
Seismic Zone	$a_{gr}(m/s^2)$	Seismic Zone	$a_{gr}(m/s^2)$
1.1	2,50	2.1	2,50
1.2	2,00	2.2	2,00
1.3	**1,50**	**2.3**	**1,70**
1.4	1,00	2.4	1,10
1.5	0,60	2.5	0,80
1.6	0.35		

The seismic action can be defined upon the development of the acceleration response spectra $S_e(T)$ – equation (7) – at a given site for given structural characteristics and conditions.

$$\begin{cases} S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot \beta_0 - 1)\right] & 0 \leq T \leq T_B \\ S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 & T_B \leq T \leq T_C \\ S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \left[\frac{T_C}{T}\right] & T_C \leq T \leq T_D \\ S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \left[\frac{T_C T_D}{T^2}\right] & T_D \leq T \end{cases} \quad (7)$$

where

- T is the period of vibration of the structure
- $S_e(T)$ is the value of the acceleration response spectrum
- a_g is the design acceleration
- S is the ground or site factor, defined from S_{max} values given in Table 3
- $\eta = \sqrt{10/(5+\xi)} > 0.55$ is the damping correction factor

β_0 is the dynamic amplification factor (2.5 for horizontal earthquake components, and 3 for the vertical component a_{gv})
TB is the lower limit of the period of the constant acceleration branch
TC is the upper limit of the period of the constant acceleration branch
TD is the upper limit of the period of the constant displacement branch

Table 3 – Values of S_{max} and reference periods for determining the acceleration response spectrum

Soil Type	Earthquake Type 1				Earthquake Type 2			
	S_{max}	TB(s)	TC(s)	TD(s)	S_{max}	TB(s)	TC(s)	TD(s)
A	1	0,1	0,6	2	1	0,1	0,25	2
B	1,35	0,1	0,6	2	1,35	0,1	0,25	2
C	1,6	0,1	0,6	2	1,6	0,1	0,25	2
D	2	0,1	0,8	2	2	0,1	0,3	2
E	1,8	0,1	0,6	2	1,8	0,1	0,25	2

Fig. 4 taken from EN 1998-1-1 (2010) shows the normalized acceleration response spectrum. The description of soil type D, of the location site of this design study of guyed tower, can be obtained from Table 3.1 of EN 1998-1-1 (2010). Applying the values given for soil type D, the two following spectra were obtained for the two types (1 or 2) of earthquakes (Fig. 5).

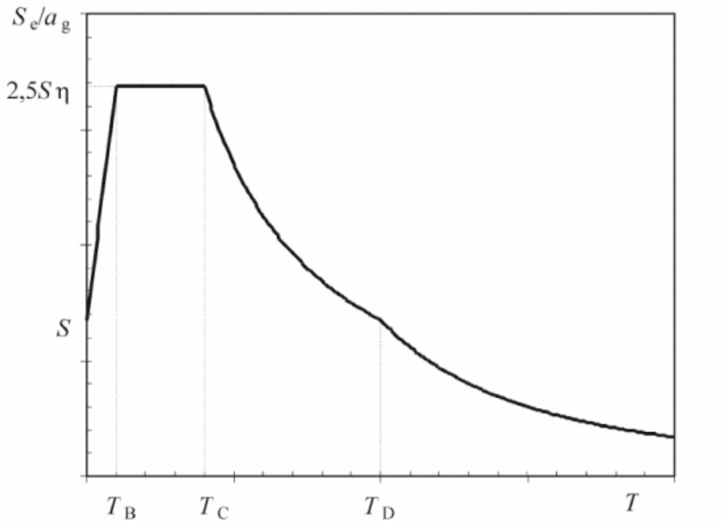


Fig. 4 – Normalized acceleration response spectrum S_e (T)

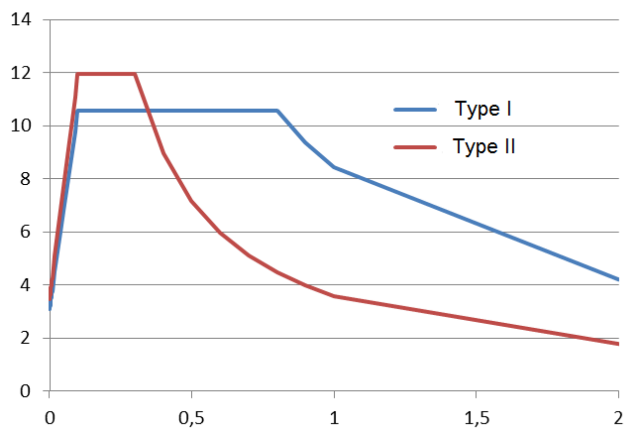


Fig. 5 – Acceleration response spectra for ξ =0.05 (Lisbon area, soil type D)

3.3. Time domain analysis for a given earthquake

The use of a time domain analysis is interesting since it allows the input of scaled ground accelerations based upon time-histories of several earthquakes (recorded or synthetically generated). Since earthquakes occur with different frequencies, it is possible that a given earthquake has the ability to excite a given structure much more than others.

When using the information of past recorded earthquakes of certain frequency content, these accelerations can be applied at the structure ground-site after normalizing the information of these earthquakes by the PGA (Peak Ground Acceleration) at the site; the later is associated with a certain recurrence period and statistical distribution of earthquakes.

The time histories and associated data of earthquakes used in the time domain analyses of the guyed tower were taken from the site of the Pacific Earthquake Engineering Research Center (PEERC, 2012). The present design study used data from four earthquakes: Caldiran (Turkey, 1976), Loma Prieta (USA, 1989), Kobe (Japan, 1995) and Chi-Chi (Taiwan, 1999). It is also noticeable that the frequency band of these earthquakes is mainly 1 Hz to 10 Hz (Fig. 6).

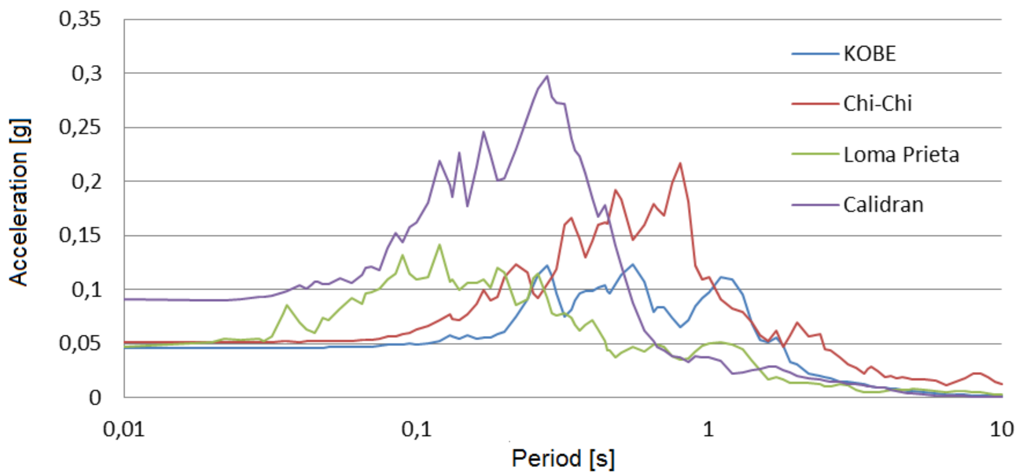


Fig. 6 – Band of frequencies of the four earthquakes considered

4. MODELING

As a first approach to the design of a guyed tower, Gantes et al. (1993) suggest a simplified equivalent beam model with springs to simulate the cables that constitute an interesting initial assessment. It should be used to frame the initial structural characteristics of the guyed tower so that it is possible to model it in any finite element program.

4.1. Requirements

The guyed towers designed for telecommunication have to meet service requirements of the telecommunication devices themselves, since a slight misalignment of the satellites may result in loss of signal, which may lead to poor quality of service for thousands of customers.

Section 3.8.2 of the American standard TIA 222 (1996, 2006) specifies a maximum horizontal displacement of 3% of the height of the guyed tower structure; wherefore for lattice structures the limiting value of the horizontal displacement is only 1.5% of the tower height.

TIA 222 also specifies a maximum value for the rotation of the antennas of 4º 00' 00", which is also the limit imposed by *Telebrás* for VHF antennas. When it comes to broadcasting on UHF antennas *Telebrás* is more restrictive and imposes a maximum rotation of 1º 40' 00".

4.2. Mast

The analysis of more traditional tower masts proposes the modeling as a simple truss structure. As the links are not rigid, the structure appears more flexible than it actually is. To partially solve this problem, it resorts to the use of dummy bars. These prevent the occurrence of undesired degrees of freedom leading to the occurrence of mechanisms. The use of these bars, with very little axial stiffness, allows the structure to be stable nevertheless flexible and therefore enables analyses of the tower under design study using some software based upon the finite element method. Many manufacturers still rely on full-scale tests to verify that the results are as expected using a simpler truss model for design of the guyed tower.

Oliveira et al. (2007) proposes a less conservative analysis method which combines three-dimensional framed members with horizontal and diagonal lattice members, so it is not necessary to use dummy bars. It constitutes a better and more real approximation of the structural behavior.

4.3. Cables

The cables were modeled by existing cable element in SAP 2000 v15 (2012). The program models the cable as a catenary to represent the elastic behavior of a cable subjected to its own weight. Its behavior is nonlinear and takes into account the P-Δ effects as large displacements and large deformations are accounted for.

A cable without tension is not stable and has not an unique position, so all cables should be loaded. The Canadian standard CSA S37-01 (2011) requires that the values of the initial tension in the cables should be between 8% and 15% of the final cable capacity.

The environmental temperature and the applied loads (namely due to wind and earthquake) can change the cable length. The effect of these changes is similar to changes of length of the undeformed cable with the exception that there is no change in self-weight.

An alternative model that can be programmed for the dynamic study of the cables was given by Desai and Punde (2001) which obtained values very close to the analytical values available, and is very quick to apply. Also Bertero (1959) and Naguib and El-Saad (2006) state that one can not disregard the initial deflection, or the pressure exerted by the wind on the cable itself, which otherwise would reach different values in the order of 10% to 15%.

Menin (2002) also used the initial tension as 10% of the ultimate stress, designing the guyed tower from such hypothesis. As expected, and also as it can be seen by the results obtained by Naguib and El-Saad (2006), the higher the value of the initial tension employed in the cables, the smaller the tower displacements would generally be. Naguib and El-Saad (2006) program allows initial tensions up to 40% of the ultimate strength, which is beyond the control parameters adopted in this work and taken from the Canadian standard CSA S37-01 (2011). In this design study it was used an initial tension close to 10% of the ultimate stress. But if it would be needed an expeditious manner, without recourse to a FEM software or specific program, the one proposed by Bertero (1959) is advised.

4.4. Load cases combinations

The combinations used for the verification of the stresses in the guyed tower structure were:

$$\begin{cases} Q1: \quad S = (1.35 \ ou \ 1) \times G + 1.5 \times Q_1 + 0.9 \times Q_2 \\ Q2: \quad S = (1.35 \ ou \ 1) \times G + 0.9 \times Q_1 + 1.5 \times Q_2 \\ CA: \quad S = G + F_{acidental} + 0.4 \times (Q_1 + Q_2) \end{cases} \tag{8}$$

where Q1 is a variable action based on wind overload, Q2 is the variable action based on temperature variation and CA is the accidental combination.

5. BUCKLING EFFECTS

Guyed towers are very slender structures, so it is very important to avoid the loss of stiffness in the structure when it is subjected to high compressive stresses. This effect, known as (tower global) buckling, leads to sudden failure of the structure by progressive and excessive lateral deformations.

The evaluation of all the Euler critical loads of each individual structural member (member buckling) is not a good approach to access the buckling capacity of the tower, when trying to describe what happens with more complex structures instead than the behavior a single column. The critical load of the guyed tower – N_{cr} – can be determined by a series of tests, simulating the degradation of the stiffness of the tower mast.

It is known that the general equilibrium equation of a structure analyzed by the displacement method is $F = F_0 + K_T\, D$ where K_T is the total stiffness (elastic stiffness, geometric stiffness contribution – positive for tensile members, negative for compressed members – and decrease in stiffness due to material non-linearity). In the vicinity of a previous state of stable equilibrium, the incremental equilibrium insures that:

$$\Delta F = K_T\, \Delta D \qquad \Rightarrow \qquad K_T = \Delta F / \Delta D \tag{9}$$

Generalizing this tangent total stiffness concept to the tower structure, and labeling F_x as a disturbing horizontal load applied on the top of the tower mast and d_x as the tower resulting horizontal displacement in the same point, then the ratio F_x/d_x is an index or a measure of the transversal stiffness of the tower mast for each axial compressive load N applied to the mast.

A total of 15 individual computational load tests were performed in the tower model, with different vertical and horizontal forces, and the corresponding top lateral displacements were evaluated by tower structural analyses. A linear regression on the computational results obtained permits to determine computationally when the stiffness of the structure would vanish; in fact:

$$\frac{F_x}{d_x} \approx a\,\lambda + b\; = 0 \quad \rightarrow \quad \lambda = -\frac{b}{a} \tag{10}$$

where λ is the buckling load factor insuring null total stiffness of the tower at the onset of elastic instability.

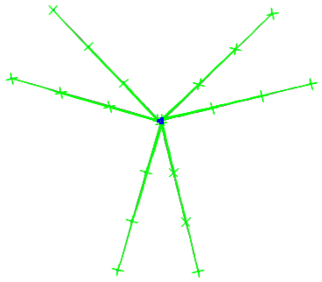
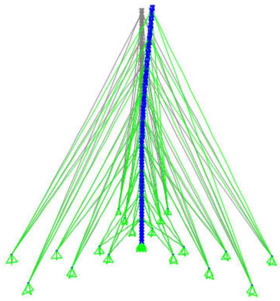
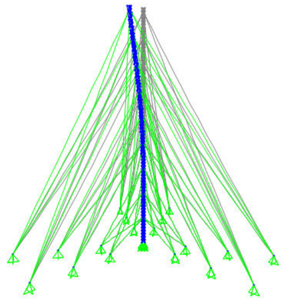
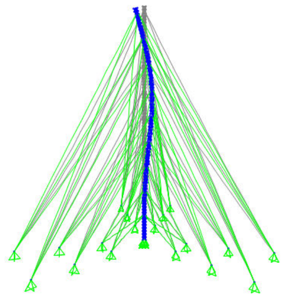
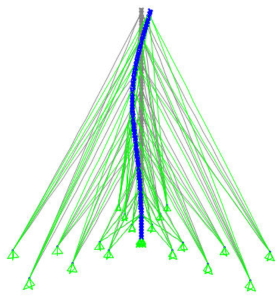
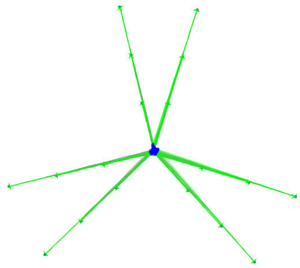
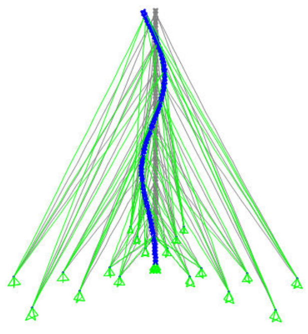
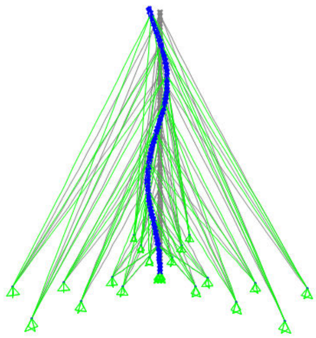
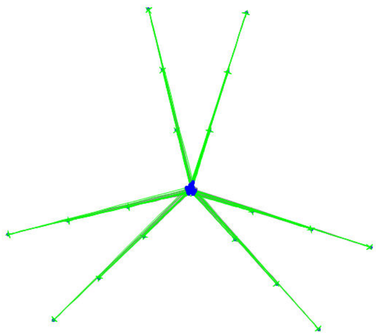
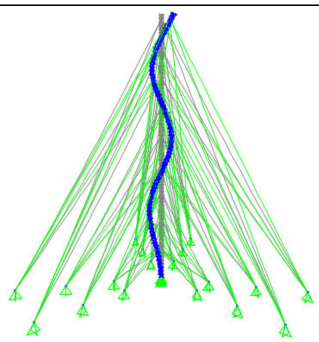
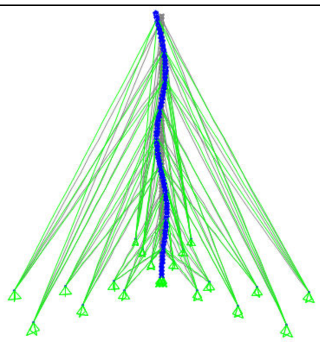
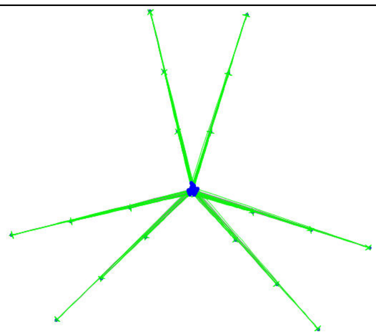
6. RESULTS

6.1. Natural vibration frequencies and vibrations modes

The analyses of the natural vibration frequencies and mode shapes are very important for understanding the dynamic behavior of the structure, as well as for evaluating the effective modal masses and the percentage of these needed to access the response with desired accuracy. Even though the first mode of the 185 m height guyed tower under analysis has a very low frequency (0.625 Hz), it has almost no mass associated with that torsion mode. So it is expected that the next two natural frequencies of the guyed tower (2nd and 3rd, equal by symmetry) of 1st (and 2nd) longitudinal bending mode along 2 perpendicular directions, would have significant effective modal masses vibrating close to 1 Hz natural frequencies.

Table 4 indicates the values and shapes of the natural frequencies (Hz) and vibration mode shapes of the considered design case study of the guyed tower.

Table 4 – Vibration frequencies and mode shapes

		
mode 1 – Torsion - f=0.625 Hz	mode 2 - f=1.011 Hz	mode 3 - Perpendicular to 2 - f=1.011 Hz
		
mode 4 - f=1.358 Hz	mode 5 – Perpendicular to 4 - f=1.358 Hz	mode 6 – Torsion - f=1.498 Hz
		
mode 7 - f=1.983 Hz	mode 8 – Perpendicular to 7 - f=1.983 Hz	mode 9 – Torsion - f=2.379 Hz
		
mode 10 - f=2.840 Hz	mode 11 – Perpendicular to 10 - f=2.840 Hz	mode 12 – Torsion - f=3.261 Hz

6.2. Load Cases Combinations

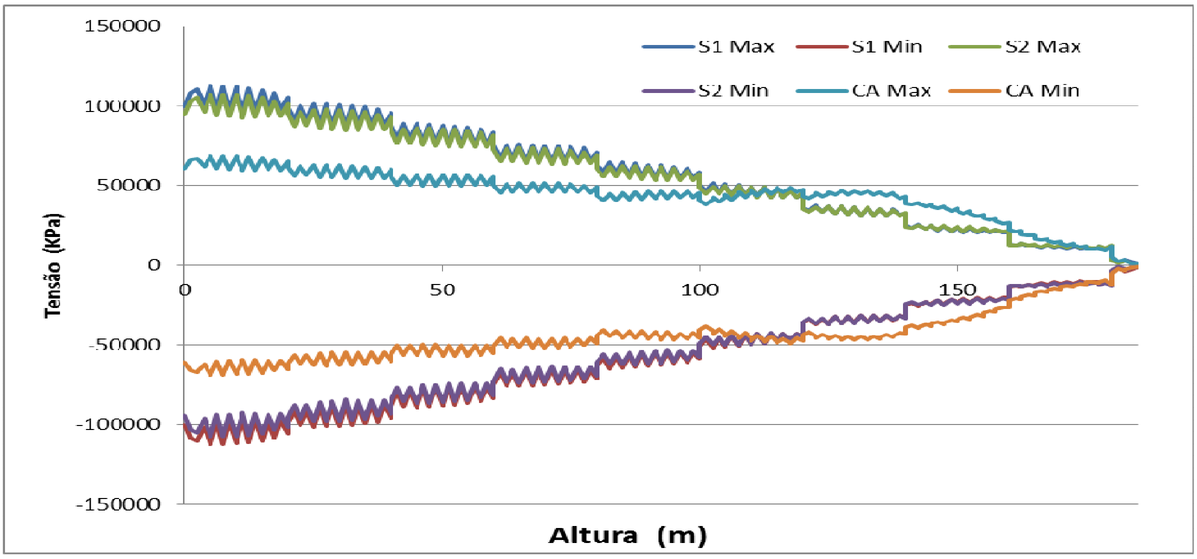


Fig. 7 – Load case combinations

One can see from this analysis that the most onerous load case combination is that of accidental earthquake load using spectrum given by Eurocode 8 for earthquake type 1; followed by the case of earthquake type 2. The load case S1 appears in 3rd place.

However when individually examined (without load combinations) the wind load would undoubtedly be more demanding for the structure than the earthquake load alone.

6.3. Overall Global Buckling

The 15 load cases applied and the displacements resulting from such load disturbances are detailed in Table 5. As it can be seen, the stiffness index F_x/d_x is getting closer and closer to zero. It can never hit zero, since the analysis program would return an error for having become unstable.

From a linear regression on the computational data (Fig. 8) the following equation is obtained:

$$\frac{Fx}{dx} = -12.789\lambda + 75.618 \tag{11}$$

For vanishing stiffness $F_x/d_x \rightarrow 0$, from which $\lambda_{cr} = 75.618 / 12.789 = 5.912738$.

Multiplying the initial axial force (for axial load factor λ=1) by λ_{cr} , the value of N_{cr} would be:

$$N_{cr} = 5.912738 \times 600 \text{ kN} = 3547.643 \text{ kN} \tag{12}$$

Table 5 – Determination of N_{cr} of the guyed tower

	1 Column		3 Columns				
Load Case	F_x'	P'	F_x	P	d_x	F_x/d_x	λ
-	kN	kN	kN	kN	M	kN/m	-
1	4	200	12	600	0,1952	61,48	1,00
2	2	200	6	600	0,0945	63,49	1,33
3	4	250	12	750	0,2139	56,10	1,67
4	4	270	12	810	0,2223	53,98	1,80
5	4	300	6	900	0,2361	25,41	2,00
6	2	300	6	900	0,1145	52,40	2,00
7	4	350	12	1050	0,263	45,63	2,33
8	4	400	12	1200	0,1903	63,06	2,67
9	2	400	6	1200	0,1441	41,64	2,67
10	4	500	12	1500	0,3883	30,90	3,33
11	4	600	12	1800	0,5434	22,08	4,00
12	4	700	12	2100	0,8306	14,45	4,67
13	4	800	12	2400	1,5091	7,95	5,33
14	2	800	6	2400	0,9853	6,09	5,33
15	4	820	12	2460	1,8657	6,43	5,47

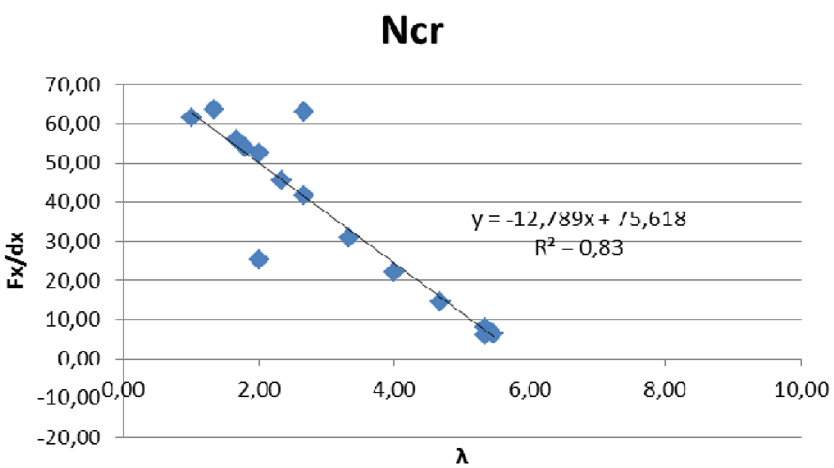


Fig.7 – Determination of N_{cr} through linear regression of stiffness indexes from a numerical simulation

CONCLUSIONS

Some relevant concepts and steps to design tall guyed towers were addressed, particularly those related to wind and seismic actions. The wind pressures were evaluated according to Eurocode 1 (EN 1991-1-4). The seismic actions were evaluated using RSA and also by Eurocode 8 (EN 1998-1-1 , EN 1998-6); for the design study case of the guyed tower mast, the seismic actions evaluated by EC8 are more severe than those evaluated by RSA, or even those evaluated by a time domain analysis of four historic earthquake records scaled by the PGA at the site (Lisbon area). The natural vibration frequencies and mode shapes were obtained using SAP 2000 v15. It was also applied a general methodology for calculating the tower mast buckling load, also applicable to general complex structures.

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