COMPDYN 2011
3rd ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

COMPARISONS OF A TALL BUILDING WIND RESPONSE WITH AND WITHOUT A TMD

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Keywords: Wind response, Tall buildings, Vibration control, TMD, Eurocode 1.

Abstract. *This work addresses basic concepts regarding the wind effects on tall buildings and their subsequent wind responses. The methodology proposed in Eurocode 1 for quantification of the wind actions, is compared with the methodology still in force in the Portuguese legislation, to assess the pros and cons of each towards a more realistic building design. Using commercial software for the structural calculations, a specific study of a given tall building is performed by the actions associated with the two regulations, with the intention of comparing stress resultants and generalized displacements. A simplified method for quantifying the dynamic action on these structures is adopted, with the purpose of studying techniques for vibration control of the along-wind response in terms of displacements and accelerations. For the tall building case under study, a realistic comparison is made between the building's response under dynamic wind action with and without a TMD.*

1 INTRODUCTION

The wind action on towers (Almeida and Barros [1]) and masts (Barros [5]) and on a tall building (Ferreira [11], Taranath [21]) is of paramount importance to their design, which can influence the structural system to be adopted or even lead to the needed implementation of control systems to reduce vibrations caused by wind dynamic phenomena.

After a comparative analysis between the Portuguese Regulation on Safety and Actions for Building Structures and Bridges (RSAEEP) [18] and Eurocode 1 (EC1) [10] it can be concluded that there are differences in these methods here considered to quantify the wind actions, with an improvement for design inherent to the European regulation.

Using the World Trade Center (WTC) as reference tall building type, a parametric study was performed on the variation of shear forces and bending moments as a function of height, as evaluated by the RSAEEP and by EC1. An equivalent frame structure to the reference World Trade Center was modeled, based upon matching fundamental mode contribution, leading to a structural behavior very close to the three-dimensional modeled behavior found in available references.

For modeling the dynamic wind action reference is made to a method of generating sets of synthetic wind, called the method of Shinozuka. The dynamic response of the reference building, in the longitudinal along-the-wind direction, permits to assess occupants comfort level with regards to accelerations. A theoretical implementation of TMD will be studied and proven to be effective for this tall building under harmonic or under wind loads. The reductions of the responses were found to be more effective on accelerations than on displacements.

2 WIND ACTIONS UNDER TWO DESIGN CODES

For better understanding of the case of study that will be analyzed, some considerations and synthesis of the methodologies are given concerning the two used regulations, in this paper, for wind effects: Portuguese Regulation on Safety and Actions for Building Structures and Bridges (RSAEEP) [18] (still in use) and Eurocode 1 (EC1) [10] (in view of its implementation at European level).

2.1 Wind actions according to Portuguese Regulation RSAEEP

2.1.1 Zoning of territory

RSAEEP [18] admits the differentiation of Portuguese territory in two zones (A and B) based on the analyses of existing meteorological records that, for the same occurrence probability of occurrence, attributes wind intensities sufficiently differentiated.

	Zoning of territory
Zone A	Practically all Portugal except regions belonging to zone B
Zone B	Azores and Madeira; Coastal strip 5 km width; Places of altitude above 600 m; Places on zone A but subjected to particularly unfavorable wind conditions

Table 1 Zoning of Portuguese territory

2.1.2 Aerodynamic roughness of the terrain profile

RSAEEP [18] proposes the differentiation of the terrain profile roughness in two types: type I and type II. The variation of the wind speed with height depends strongly on the dimensions and distribution of existing obstacles in the terrain that affect the air flow in the neighborhood. Notice that the consideration of just two roughness types is a bit schematic and it results from the difficulty of characterizing objectively the multiplicity of the wind situations.

	Aerodynamic roughness of the terrain profile
Type I	Places inside urban zones where medium and tall buildings prevail
Type II	Remaining places: rural zones and periphery of urban zones

Table 2 Distinction between the two types of roughness at RSAEEP

Since roughness of type I is greater, to it will correspond lower wind speed and therefore lower dynamic wind pressures.

2.1.3 Quantification of wind speeds

The wind action on structures depends on the greatness and distribution of the wind speed and of its characteristics; therefore, it is necessary to define the characteristic values and the reduced values of the wind speed in function of the height above the soil.

The average wind speed is defined in function of the height above the soil and is referred to time intervals of 10 minutes. The variation with height h of the characteristic value of the mean wind velocity v (m/s) is given by equation (1). In this formula, h_0 is the height above the soil for which the roughness of the soil is no longer felt; v_0 is the average wind speed at the height h_0; and α is a parameter that depends on the soil roughness. For terrain roughness of type I, v_0 is equal to 18 m/s and $1/\alpha$ takes the value 0,28 ; for terrain roughness of type II, v_0 is equal to 25 m/s and $1/\alpha$ takes the value 0,20.

$$v = v_0 \left(\frac{h}{h_0} \right)^{1/\alpha} \tag{1}$$

Given the imprecision of the definition of the wind speeds in the immediate neighborhood of the terrain, it is advisable to admit a constant value of the wind speed: (i) of 20 m/s up to a height of 15 m, for terrain roughness of type I; (ii) of 25 m/s up to a height of 10 m, for terrain roughness of type II.

For the case of structures identically loaded by the wind in any direction (as is the case of structures with symmetry of revolution), the values of the wind speed to consider should be multiplied by $\sqrt{3}$.

To take into account the fluctuations of wind velocity resulting from the flow turbulence, RSAEEP [18] contemplates a constant value of 14 m/s to be added to equation (1) regardless of the terrain roughness type. RSAEEP [18] also contemplates that the characteristic value of the average wind velocity for zone B, are 10% higher than those of zone A. Table 3 synthesizes all the previous considerations. Additionally RSAEEP [18] contemplates situations on how to consider the height of ground in inclined terrains.

		Zone A		Zone B	
		Roughness Type I	Roughness Type II	Roughness Type I	Roughness Type II
Height (m)	≤ 10	$v_A = 20+14$	$v_A = 25+14$	$v_B = 1.1\ v_A$	$v_B = 1.1\ v_A$
	$10 \leq h \leq 15$	$v_A = 20+14$	$v_A = 25\left(\frac{h}{10}\right)^{0.20} + 14$		
	>15	$v_A = 18\left(\frac{h}{10}\right)^{0.28} + 14$			

Table 3 Characteristic values of wind velocity (m/s)

2.1.4 Determination of Wind Actions on Buildings

For determining the wind actions on buildings, RSAEEP [18] is based on a simplified method that consists on the application of a static pressure on the surface of the structure. The static pressure p is obtained multiplying the dynamic wind pressure w_k (dependent on the wind speed and therefore also on height or elevation along the structure) by shape coefficients δ that are characteristic of the aerodynamic shape of the structure, as given by equation (2).

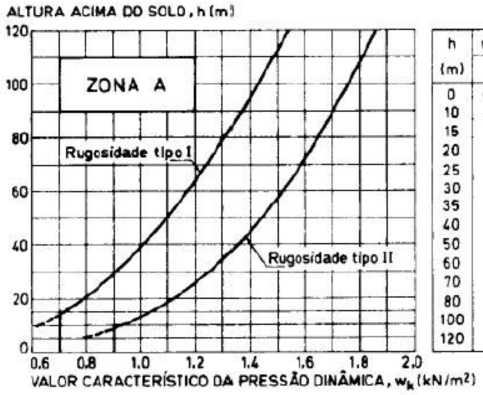
$$p = w_k\, \delta \tag{2}$$

It should be enhanced that this process does not lead to correct results for flexible structures, since they are dynamically excited by the flow and therefore generating a fluid-structure interaction (FSI) difficult to quantify. Nevertheless, on this section, wind actions are evaluated according to RSAEEP [18] methodology, for comparison with the results that will be obtained through the EC1. Although the latter also calculates the actions based in equivalent static loads, it presents however a structural factor contemplating dynamic effects.

The dynamic wind pressure w_k (N/m^2 or Pa), corresponding to a specific or volumetric air mass density of 1,225 kg/m^3, is given by equation (3) with the wind speed v in m/s.

$$w_k = 0.613\ v^2 \tag{3}$$

For determining the characteristic values of the wind dynamic pressure along height above ground, RSAEEP [18] presents a graph for zone A (Figure 1) coherent with Table 3.



h (m)	w_k (kN/m^2) I	II
0	0.70	0.90
10	0.70	0.90
15	0.70	1.04
20	0.79	1.12
25	0.85	1.19
30	0.91	1,25
35	0,96	1,30
40	1,01	1,35
50	1,09	1,44
60	1,17	1,52
70	1,24	1,59
80	1,31	1,65
100	1,43	1.76
120	1,54	1.86

Figure 1 Characteristic values of the dynamic pressure w_k (kN/m^2) for zone A

The values corresponding to zone B are obtained multiplying by 1.2 the values indicated for zone A (since wind speed for zone B is 10% higher than that of zone A, and in the equation for dynamic pressure the speed is raised to the power exponent 2). According to regulation RSAEEP [18], the reduced values of the wind dynamic pressures should be obtained through the following coefficients: $\Psi_0 = 0.4$, $\Psi_1 = 0.4$ and $\Psi_2 = 0.4$. Also, depending on the building typology, Ψ_0 can reach the value of 0.6.

The shape coefficients can be: (exterior and interior) pressure coefficients and force coefficients. The pressure coefficients permit to determine the wind pressure perpendicularly to the building surfaces, in agreement with equation (2). The acting force on a surface will be given by the product of the pressure by the respective area. The resulting force of the wind action is obtained through the summation of the (interior and exterior) forces applied to each considered surface. Figure 2, taken literally from RSAEEP [18], gives the exterior pressure coefficients for building facades.

Relações geométricas do edifício (*)		Planta	Direcção do vento α (graus)	Acções globais sobre as superfícies				Acções locais na faixa referenciada na figura
$\frac{h}{b}$	$\frac{a}{b}$			A	B	C	D	0,25 b
$\frac{h}{b} \le \frac{1}{2}$	$1 < \frac{a}{b} \le \frac{3}{2}$		0	+0,7	−0,2	−0,5	−0,5	−0,8
			90	−0,5	−0,5	+0,7	−0,2	
	$\frac{3}{2} < \frac{a}{b} \le 4$		0	+0,7	−0,25	−0,6	−0,6	−1,0
			90	−0,5	−0,5	+0,7	−0,1	
$\frac{1}{2} < \frac{h}{b} \le \frac{3}{2}$	$1 < \frac{a}{b} \le \frac{3}{2}$		0	+0,7	−0,25	−0,6	−0,6	−1,1
			90	−0,6	−0,6	+0,7	−0,25	
	$\frac{3}{2} < \frac{a}{b} \le 4$		0	+0,7	−0,3	−0,7	−0,7	−1,1
			90	−0,5	−0,5	+0,7	−0,1	
$\frac{3}{2} < \frac{a}{b} \le 4$	$1 < \frac{a}{b} \le \frac{3}{2}$		0	+0,8	−0,25	−0,8	−0,8	−1,2
			90	−0,8	−0,8	+0,8	−0,25	
	$\frac{3}{2} < \frac{a}{b} \le 4$		0	+0,7	−0,4	−0,7	−0,7	−1,2
			90	−0,5	−0,5	+0,8	−0,1	

Figure 2 Values of the external pressure coefficients for the facades of buildings

The force coefficients allow to determine directly the resulting force F of the wind pressures on a certain structure (or on part of it) according to the equation (4), where A is the area of the surface (or portion) of the structure and d_f is the force coefficient.

$$F = w_k \, d_f \, A \tag{4}$$

2.2 Wind actions according to Eurocode EC1

The Eurocode 1 Part 1-4 (EC1) [10] gives characteristic values of wind actions for the global structure, and for parts of it (for example, walls and roofs) or for elements linked to the structure (for example, chimneys).

The calculation is based in a model using a peak factor. The basic idea of this model is that the maximum wind action in a static analysis or the dynamic response of the structure, can be described by the sum of an average component (constant part of the action) with a turbulent component (not constant). The development of this calculation model and its promotion is attributed to Davenport [8].

The wind action calculated according to the EC1 [10] gives characteristic values, that are obtained from base values of wind speed and wind pressure, corresponding to a probability of annual exceedence of 2% (ie, 0.02) that is equivalent to a return period of 50 years. The return period (R) for a set of generic value of a random variable (U) is the reciprocal or inverse of the probability of exceedence, and it corresponds to the number of samples that it is necessary to consider, on the average, so that a sample is registered with a value higher than U.

The effect of the wind in the structure depends on the size, shape and dynamic properties of the structure. The response of the structure should be calculated from the peak velocity pressure q_p at a reference height and in the undisturbed wind field. Also the coefficients of pressure and/or of force should be considered, as well as the contribution of a structural factor $c_s\, c_d$. Peak velocity pressure depends on the wind climate, the terrain roughness and orography, and the reference height; such pressure is equal to the mean velocity pressure plus a contribution from short-term pressure fluctuations.

2.2.1 Zoning of territory by categories

EC1 characterizes and distinguishes between five categories of terrain roughness according to Table 4, which also gives terrain parameters (roughness length z_0 and minimum height $z_{\min}$).

Terrain category	z_0 (m)	$z_{\min}$ (m)
0 Sea or coastal area exposed to the open sea	0,003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1	10

Table 4 Terrain categories and terrain parameters

2.2.2 Basic values

The fundamental value of the basic wind velocity $v_{b,0}$ is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights (terrain category II).

The basic wind velocity v_b (defined as a function of wind direction and time of year at 10m above ground of terrain category II) shall be calculated from the following equation

$$v_b = c_{dir}\; c_{season}\; v_{b,0} \tag{5}$$

where c_{dir} is the directional factor, c_{season} is a season factor and $v_{b,0}$ is the fundamental value of the basic wind velocity. In some situations it is advantageous to consider a fundamental velocity with a probability of annual exceedence (*p*) different from 0,02 (or 2%). The method used by EC1 is based in the statistical treatment of European wind pressure data. The basic velocity is then obtained multiplying the previous equation (5) by a probably factor c_{prob} (given below) and dependent of the desired or intended probability (*p*).

$$c_{prob} = \left(\frac{1 - K \ \ln(-\ln(1-p))}{1 - K \ \ln(-\ln(0.98))} \right)^n \tag{6}$$

The shape parameter K, depending on the coefficient of variation of the extreme-value distribution, and the exponent n, are parameters that should be quantified in the respective national annexes and whose recommended values are 0,2 and 0,5, respectively.

2.2.3 Mean wind

The mean wind velocity varies with height and is defined as the average value of wind speed for a 10 minutes period at an appropriate height above ground. The variation of the mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness (Houghton and Carruthers [13]) and orography and on the basic wind velocity v_b and should be determined according to:

$$v_m(z) = c_r(z) \ c_o(z) \ v_b \tag{7}$$

where $c_r(z)$ is the roughness factor and $c_o(z)$ is the orography factor (usually taken as 1). The roughness factor accounts for the influence of the ground roughness of terrain (upwind of the structure in the wind direction considered) in the vertical distribution of the mean wind velocity at the site (profile along the height above ground level). It is given by:

$$c_r(z) = \begin{cases} k_r \ \ln(\dfrac{z}{z_0}) \ , \ for \ \ z_{\min} \le z \le z_{\max} \\ \\ c_r(z_{\min}) \ , \ for \ \ z \le z_{\min} \end{cases} \tag{8}$$

where the roughness length z_0 and the minimum height $z_{\min}$ depend on the terrain categories and assume the recommended values given in Table 4. The terrain factor k_r depends on the roughness length z_0 and is calculated using

$$k_r = 0.19 \ (\frac{z_0}{z_{0,II}})^{0.07} \tag{9}$$

where $z_{0,II}$ = 0.05 m (terrain category II), $z_{\min}$ is the minimum height (Table 4, according to terrain categories) and $z_{\max}$ is to be taken as 200 m.

The category of roughness which should be used for a given wind direction depends on the distance at which the roughness is constant, called the reference distance (small areas presenting different roughness of the terrain can be ignored), and within a sector angle of 15 degrees to each side of the fixed wind direction. The reference distance must be specified in the National Annex and in case of choice between two or more terrain categories it should be chosen the area corresponding to the lower roughness. The orographic factor takes into account the existence of mountains, hills or cliffs, where their existence results in an increase in wind speed by more than 5%. The regulation indicates that the procedure for determining this factor should be given in the national annex of each country.

Nevertheless, the effects of orography can be overlooked when the theoretical slope (average slope of terrain) upstream is less than 3. Although this is not usually a problem for tall buildings, it should be noted that for considering the wind effects on lower building structures, it must be taken into account that these may be affected by strong winds led to the near-ground due to the existence of higher neighboring structures. Annex A.4 of the Eurocode EC1 proposes a methodology for taking into account these effects on structures.

2.2.4 Wind turbulence

According to Cook [7], because the standard deviation of the turbulence near the ground is relatively constant with height, EC 1 [10] adopts a simplified model where the turbulence intensity decreases with height in inverse proportion to the growth of average wind speed between heights $z_{\min}$ and $z_{\max}$, and taking a constant value for heights below $z_{\min}$.

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence σ_V divided by the mean wind velocity, as given in equation (10).

$$\begin{cases} I_v(z) = \dfrac{\sigma_v}{v_m(z)} = \dfrac{k_l}{c_0(z)\ln(\dfrac{z}{z_0})} & for \;\; z_{\min} \le z \le z_{\max} \\ I_v(z) = I_v(z_{\min}) & for \;\; z \le z_{\min} \end{cases} \tag{10}$$

The turbulent component of wind velocity has a mean value of 0 and a standard deviation σ_v determined - in terms of the terrain factor k_r, basic wind velocity v_b and for the turbulence factor k_I - by:

$$\sigma_v = k_r \; v_b \; k_l \tag{11}$$

The value of the turbulence factor k_I may be given in the National Annex of each country. The recommended value for k_I is 1. Cook [7] proposes a better balance of turbulence factor values, as given in Figure 3 where k_I values are also a function of the height. Figure 4 shows the evolution of turbulence intensity as a function of height for different terrain categories, using the values recommended by the Eurocode EC1 EC 1 [10] for k_I and c_o.

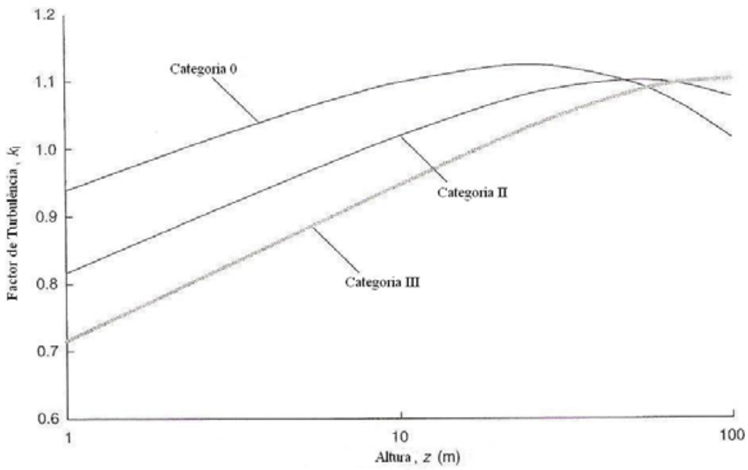


Figure 3 Alternative values for turbulence factor k_I

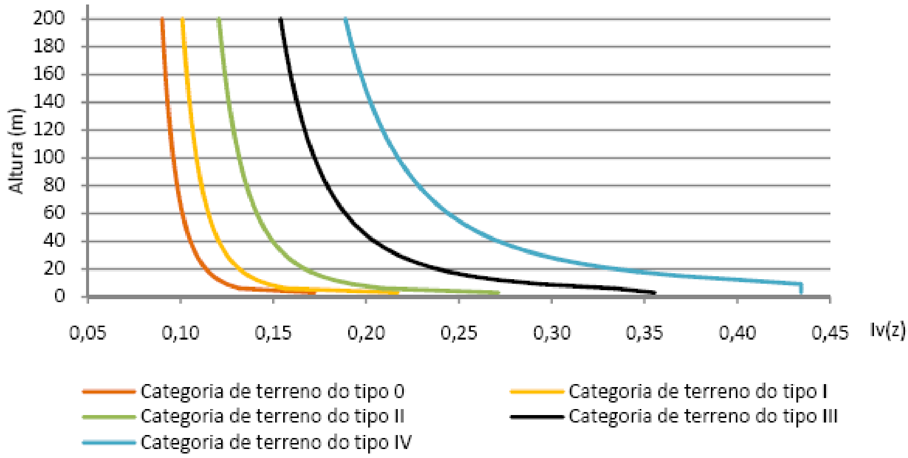


Figure 4 Profiles of turbulence intensities according to EC1

2.2.5 Wind dynamic pressure for peak velocity

The calculation of wind dynamic pressure by EC1 is based on the method of the “gust loading factor”, assuming that the response of the structure can be obtained through the sum of an average (constant) component with a fluctuating (dynamic) component; the latter component is expressed through a peak gust factor *g(t)* (Kappos [15], Almeida and Barros [1], Cook [7]). In this situation the peak velocity can be given by

$$\hat{v} = v_m(z) + g(t)\sigma_v(z) = v_m(z)\left[1 + g(t)I_v(z)\right] \tag{12}$$

As the dynamic wind pressure is $\frac{1}{2}\rho\, v(z)^2$, the peak pressure is given by:

$$q_p(z) = \frac{1}{2}\rho\left(\hat{v}(z)\right)^2 = \frac{1}{2}\rho\left(v_m(z)\right)^2\left[1 + 2\,g(t)\,I_v(z) + \left(g(t)\right)^2\left(I_v(z)\right)^2\right] \tag{13}$$

As last term in the previous equation is negligible with comparison with the others, EC1 suggests the following equation for determining the wind dynamic pressure for peak velocity:

$$q_p(z) = \frac{1}{2}\rho\left(\hat{v}(z)\right)^2 \approx \left[1 + 7\,I_v(z)\right]\frac{1}{2}\rho\left(v_m(z)\right)^2 = c_e(z)\,q_b \tag{14}$$

where ρ is the specific mass air density varying with height, temperature and barometric pressure (recommended value is 1.25 kg/m^3), q_b is the basic velocity pressure given by $\frac{1}{2}\rho\, v_b^{\,2}$ and $c_e(z)$ is the exposure factor interpreted as the ratio

$$c_e(z) = q_p(z)/q_b = 1 + 7\,I_v(z) \tag{15}$$

The value 7 in the previous equation corresponds to a gust peak factor of 3.5 for an average period of 10 minutes.

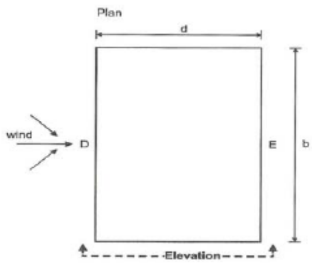
2.2.6 Actions due to wind

The wind pressures on external surfaces w_e are obtained by multiplying a pressure coefficient of external pressure c_{pe} by the dynamic pressure for peak velocity $q_p(z_e)$ evaluated at a certain reference height for the external pressure z_e (section 7 of EC1), according to the equation:

$$w_e = q_p(z_e)\ .\ c_{pe} \tag{17}$$

Similarly, the wind pressures on internal surfaces w_i are obtained by multiplying a pressure coefficient of external pressure c_{pi} by the dynamic pressure for peak velocity $q_p(z_i)$ evaluated at a certain reference height for the external pressure z_i, according to a similar equation. The actuating pressure on a given area will be given by the difference pressure between the exterior and interior, taking into account the signs. The reference heights to adopt as well as the coefficients of external and internal pressure for different structures are recommended in Section 7 of EC1.

According to EC1 and with respect to buildings, the pressure coefficient of external pressure depends on the size of the area where wind action is applied. The coefficient of external pressure for loaded areas smaller than 1 m^2 ($c_{pe,1}$) and greater than 10 m^2 ($c_{pe,10}$) are provided in tables for appropriate settings of buildings. Table V presents the recommended values for pressure coefficients of external pressure on the windward and leeward sides of the vertical walls of buildings rectangular in plan. Figure illustrates the reference height z_e to consider for the same type of structures on the windward side.



Zona	D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	+0,8	+1,0	-0,7	
1	+0,8	+1,0	-0,5	
≤0,25	+0,7	+1,0	-0,3	

Table 5 Coefficients of external pressure on the windward and leeward sides of the vertical walls of buildings rectangular in plan

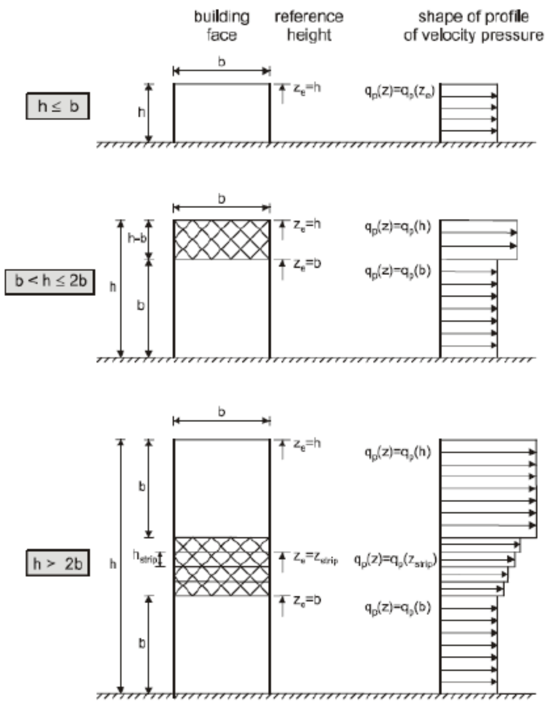


Figure 5 Reference height for the external pressure and corresponding pressure profile in the windward side

According with EC1, the wind forces acting in the entire structure (or on a structural component) can be determined directly through the: (i) appropriate force coefficients applied to the whole structure or on the structural component; (ii) vectorial sum of the components of pressure (external and internal forces in structural members evaluated from surface external and internal pressures) and frictional stresses acting on the structure resulting from the friction of the wind parallel to the external surfaces.

When using force coefficients, the wind force F_w acting on a structure or a structural component may be determined directly by

$$F_w = c_s \ c_d \ c_f \ q_p(z_e) \ A_{ref} \tag{18}$$

or by vectorial summation over the individual structural elements using the expression

$$F_w = c_s \ c_d \sum_{elements} c_f \ q_p(z_e) \ A_{ref} \tag{19}$$

where the product $c_s \, c_d$ is the structural factor, c_f is the force coefficient for the structure or structural element (Section 7 of EC1), $q_p(z_e)$ is the peak velocity pressure evaluated at the reference height z_e, and A_{ref} is the reference area of the structure or structural element (Section 7 of EC1).

Wind forces obtained by the sum of the components of external and internal pressure and frictional forces are given by:

$$F_w = c_s \ c_d \sum_{surfaces} w_e \ A_{ref} + \sum_{surfaces} w_i \ A_{ref} + c_{fr} \ q_p(z_e) \ A_{fr} \tag{20}$$

where w_e is the external pressure on the individual surface at height z_e, w_i is the internal pressure on the individual surface at height z_i, c_{fr} is the friction coefficient and A_{fr} is the area of external surface parallel to the wind.

The frictional forces act only on the surfaces parallel to the wind and are small when compared with the forces of pressure, so that only become significant when the area of surfaces parallel to the flow is considerable. The EC 1 [10] states that the effects of friction surfaces may be waived when the area of all surfaces parallel to the wind flow (or that do the least angle with this) is equal to or less than four times the area of all exterior surfaces perpendicular to the flow.

It is important to also emphasize that the dynamic effects and the effects related to the size of the structure, due to the structural factor $c_s \, c_d$, are restricted to the external components because the code standard assumes that the internal pressures and the frictional forces are static values that are fully distributed along the surfaces.

According to Cook [7] this assumption is correct with respect to the internal pressures, which depend on the volume inside, but it is not quite correct with respect to the friction forces. If in fact the structural factor takes into account fluctuations of pressure normal to the surfaces (due to wind gusts not acting simultaneously along a large surface), then the friction effects caused by these blasts also do not act simultaneously.

2.2.7 Structural factor

The structural factor $c_s\, c_d$ takes into account the combined effect of: (i) No simultaneous occurrence of the peak pressure of wind on the facades of the structure, often called the size effect (size factor c_s); (ii) Vibration of the structure in its fundamental mode due to the turbulence action, commonly called dynamic response (dynamic factor c_d).

The Eurocode defines a number of situations for which the value of the structural factor can be used as 1, avoiding the detailed process of calculation. These situations correspond to small structures or structural elements, for which the effect of size and the dynamic effect are both small. In the case of structures which are not suitable for the direct consideration of this factor as a unit value, this can be obtained using two different procedures specified in the code standard. In Annex D, EC1 provides indicative figures of the structural factor for different types of structures and their characteristics. Figure shows the values of the structural factors (taken from Annex D), with respect to reinforced concrete buildings.

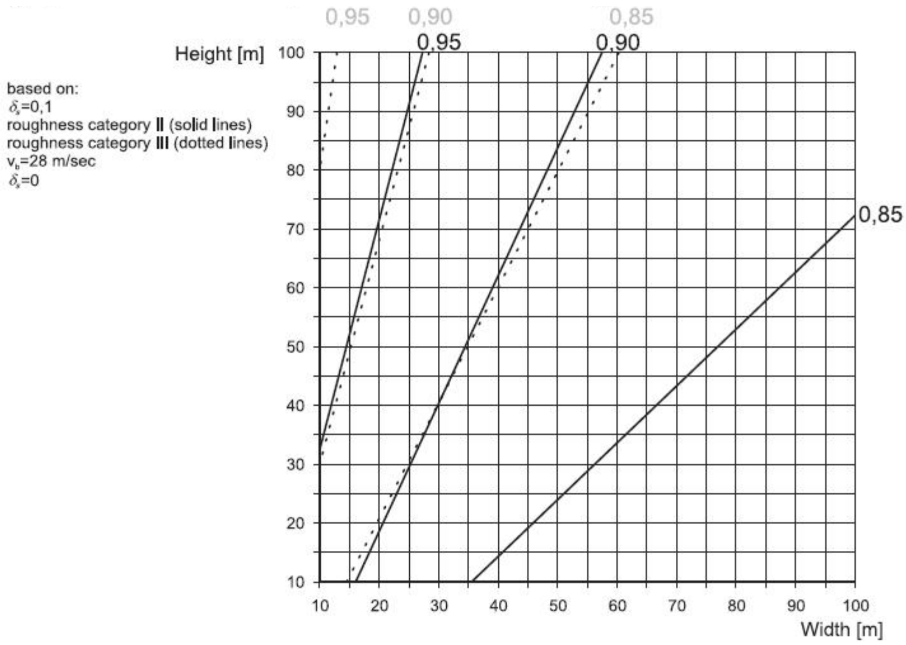


Figure 6 Structural factor for multistory concrete buildings

For the case of vertical structures such as buildings, or horizontal structures such as bar members or beams or point-like structures as signboards, and for the case where the vibration of the structure in wind direction occurs only in its fundamental mode of vibration, the structural factor is given by equation (21):

$$c_s\, c_d = \frac{1 + 2\, K_p\, I_v(z_s)\, \sqrt{B^2 + R^2}}{1 + 7\, I_v(z_s)} \tag{21}$$

where z_s is the reference height, 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, $I_v(z_s)$ is the turbulence intensity, 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.

This procedure then estimates the dynamic response of the structure as the square root of the sum of one resonance component and another background component. According to EC1 the evaluation of component parcels (K_p, B^2, R^2) can be made according to two procedures suggested in separate annexes B and C of the code standard. Both annexes indicate that it is conservative to take $B^2 = 1$. As an indication it is suggested that the values obtained by these two procedures should not differ by more than 5%.

In the following paragraph a comparative study of these procedures will be presented, here labeled *Procedure 1* and *Procedure 2* and evaluated respectively in agreement with Annexes B and C of EC1. A synthetic description of these is given in Table 6, outlining the calculation the main parameters.

Procedure 1 – Annex B (EC1)	**Procedure 2 – Annex C (EC1)**
$B^2 = \dfrac{1}{1 + 0{,}9 \left(\dfrac{b+h}{L(z_s)}\right)^{0{,}63}}$	$B^2 = \dfrac{1}{1 + \dfrac{3}{2} \cdot \sqrt{\left(\dfrac{b}{L(z_s)}\right)^2 + \left(\dfrac{h}{L(z_s)}\right)^2 + \left(\dfrac{b}{L(z_s)} \cdot \dfrac{h}{L(z_s)}\right)^2}}$
$R^2 = \dfrac{\pi^2}{2.\,\delta} . S_L(z_s, n_{1,x}) . R_h(\eta_h) . R_b(\eta_b)$	$R^2 = \dfrac{\pi^2}{2.\,\delta} . S_L(z_s, n_{1,x}) . K_s(n_{1,x})$
$K_p = \sqrt{2. \ln(v.T)} + \dfrac{0{,}6}{\sqrt{2. \ln(v.T)}}$	$K_p = \sqrt{2. \ln(v.T)} + \dfrac{0{,}6}{\sqrt{2. \ln(v.T)}}$
$\sigma_{a,x}(z) = \dfrac{c_f . \rho . b . I_v(z_s) . v_m{}^2(z_s)}{m_{1,x}} . R . K_x . \phi_{1,x}(z)$	$\sigma_{a,x}(z) = c_f . \rho . I_v(z_s) . v_m{}^2(z_s) . R . \dfrac{K_y . K_z . \phi(y,z)}{\mu_e . \phi_{max}}$

Table 6 Two procedures for calculating parameters of the structural factor

In both procedures the turbulence length scale *L(z)*, representing the average gust size for natural winds, is given by equation (22); the non-dimensional power spectral density function $S_L(z,n)$ given by equation (23) is illustrated in Figure 7.

$$\begin{cases} L(z) = L_t \left(\dfrac{z}{z_t}\right)^{\alpha} & , for \quad z \geq z_{\min} \\ L(z) = L(z_{\min}) & , for \quad z \leq z_{\min} \end{cases} \tag{22}$$

$$S_L(z,n) = \frac{6{,}8\ f_L(z,n)}{\left(1 + 10{,}2\ f_L(z,n)\right)^{5/3}} \qquad \text{with} \quad f_L(z,n) = \frac{n\ L(z)}{v_m(z)} \tag{23}$$

In these equations the reference height z_t = 200 m, the reference length scale L_t =300 m, the exponent α = 0,67 + 0,05 ln(z_0), where the roughness length z_0 (m) and the minimum height $z_{\min}$ (m) are given in Table IV.

Also the non-dimensional frequency f_L is determined by the natural fundamental frequency *n* of the structure (in *Hz*) by the mean velocity $v_m(z)$ and by the turbulence length scale $L(z)$, as detailed in equation (23) above.

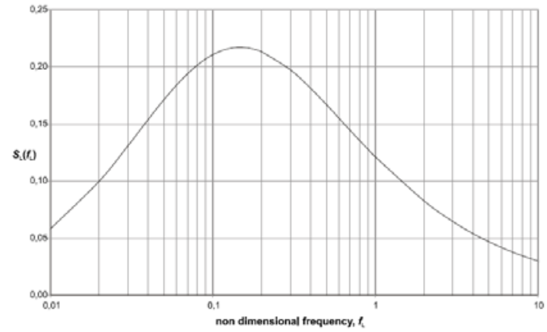


Figure 7 Power spectral density function $S_L(f_L)$

Annex F of EC1 presents the considerations for calculating the dynamic characteristics of some structures and for the case of buildings, with height *h* above 50 m, the fundamental frequency can be approximately calculated according to equation (24):

$$n = \frac{46}{h} \qquad (Hz, \text{with height in } m) \tag{24}$$

For the use of Table VI: *b* and *h* are the width and height of the building, respectively; δ is the logarithmic decrement of the damping, given by the sum of the structural logarithmic decrement δ_s with the aerodynamic logarithmic decrement δ_a and the logarithmic decrement due to the existence of special damping systems (tuned mass dampers TMD, sloshing tanks TLD, etc) δ_d; the calculation of δ is specified in Annex F of the code standard EC1; the variables $R_h(\eta_h)$ and $R_b(\eta_b)$ are aerodynamic admittance functions and $K_s(n_{1x})$ is a size reduction function (expressed in terms of the natural frequency of the structure n_{1x}, which may be determined using Annex F), given in Annexes B and C of EC1; T is the averaging time for the mean wind velocity (T = 600 seconds); the up-crossing frequency ν should be obtained from equation (25); $\sigma_{a,x}(z)$ is the standard deviation of the characteristic along-wind acceleration of the structural point at height *z* (Annex B of EC1); $\Phi_{1,x}(z)$ is the fundamental along wind modal shape (first approximation expressions are given in Annex F); $m_{1,x}$ is the along wind fundamental equivalent mass; K_x K_y and K_z are coefficients defined in Annexes B and C.

$$\nu = n_{1,x}\sqrt{\frac{R^2}{B^2+R^2}} \qquad (\nu \geq 0,08\,Hz) \tag{25}$$

3 COMPARATIVE ANALYSIS BETWEEN RSAEEP AND EC1

As summarized in previous chapters, it can be seen that although both code regulations RSAEEP [18] and EC 1 [10] detail calculation methods for the actions of wind on buildings through consideration of equivalent static loads, these two regulations are quite distinct.

In short, EC 1 [10] departs from a basic wind velocity from which the mean wind velocity is calculated, and thereafter the turbulence intensity. The dynamic pressure for peak velocity is obtained from the turbulence intensity and from the mean wind velocity. The wind pressure exerted on the surfaces is then calculated by multiplying the pressure peak velocity by their pressure coefficients, specific to each zone. The forces are finally obtained multiplying the wind pressure by the reference area, introducing corrections taken into account by the structural factor.

In RSAEEP [18] the quantification of the wind forces results from the multiplication of the dynamic pressures (function of the gust velocity, at the height of the floor in analysis) by the coefficient of external pressure (depending on wind direction and dimensions of the building) and by the area of influence corresponding to the application of force on each floor.

In order to understand and compare the procedures adopted by both code regulations for evaluating the shear forces and the bending moments at the base of a building due to wind actions, the results obtained for these generalized actions on a reference tall building are given herein; the considered reference building has a square cross section of 20 m x 20 m and a variable height ranging from 3 m to 200 m. It is also intended with this analysis to have an idea of the variation of wind forces along the height of the reference buildings, as evaluated by these two independent design code regulations.

The EC1 refers to the national annex defining the reference point for calculating the pressures at the leeward side. Since RSAEEP allows the distribution of pressure on this face equal to the face of windward, it is adopted here this same assumption; in this way taking the coefficients of internal pressure the same value, the corresponding forces in opposite directions due to internal pressure shall be cancelled. The wind was considered acting solely in one direction, the topography of the terrain is considered flat and horizontal and were not considered the effects of the possible interferences in the wind flow due to existing structures in the vicinity of the building under review.

The building was considered as being situated in zone A and all the types of terrain roughness available in each design code were considered (as a way to interpret its influence in the results).

To analyze this problem under RSAEEP, equation (1) was used to calculate the mean wind velocity with the appropriate parameters for terrain roughness of type I or type II.

To calculate the mean wind velocity according to EC1 equation (5) was used with $V_{b,0}$ equal to 28 m/s, since up to now there is no recommendation for this value in the national annex. The choice of this basic value is in accordance with the established for the zone A in an earlier version of EC1.

The velocity profiles for the two types of roughness considered in RSAEEP and for the five different categories of terrain considered in EC1 are represented in Figure 8.

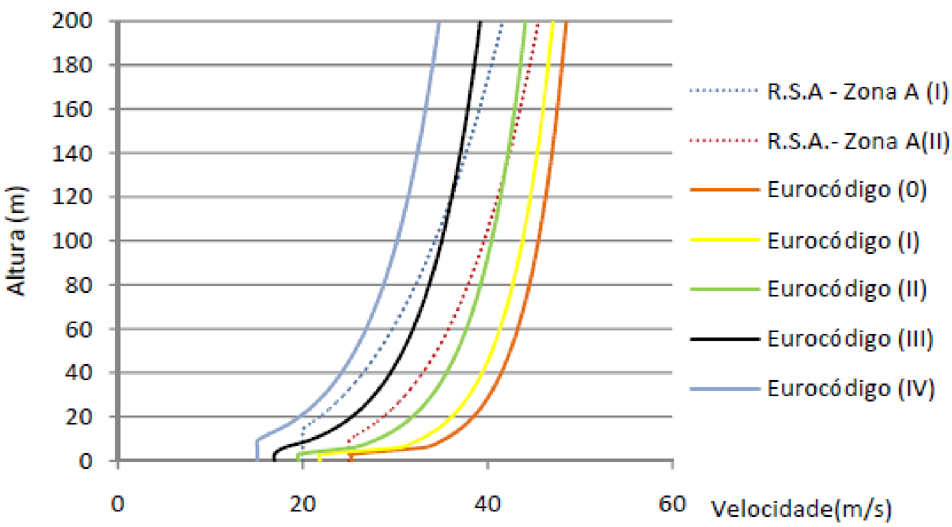


Figure 8 Profiles of mean wind velocities for two design codes and different types of terrain roughness

From the analysis of Figure 8 it is seen that the growth of wind velocity with height is more enhanced when calculated by RSAEEP than when it is calculated according to EC1: the gradient or slope of the curve is bigger. This can be justified and understood because the law used to calculate the wind velocity by the Portuguese code RSAEEP is a power law, while the one of EC1 is a logarithmic law. It was refered by Cook [7] that the logarithmic law, in the EC1 design code, for a height near the ground reveals to be a good approximation while for bigger heights it becomes non-conservative compared with the power law used by older design code regulations (such as RSAEEP). It should be noted however that considering a larger number of terrain categories (as in EC1) leads to a better differentiation of the ground types reflected in the velocity profile. According to the same Figure 8, it appears that as the terrain category evolves from the category 0 to the category IV (EC1) or from type II to type I (RSAEEP), the velocity values decrease for a given height. This would be expected since the terrain category IV or the terrain profile type I corresponds to a bigger terrain roughness, therefore lower wind speed.

Based on the calculation method proposed in the design code RSAEEP previously outlined, the variations with the height of the buildings of the shear forces and bending moments at the base of the buildings were determined for the two terrain profile types (Figures 9 and 10, for RSAEEP).

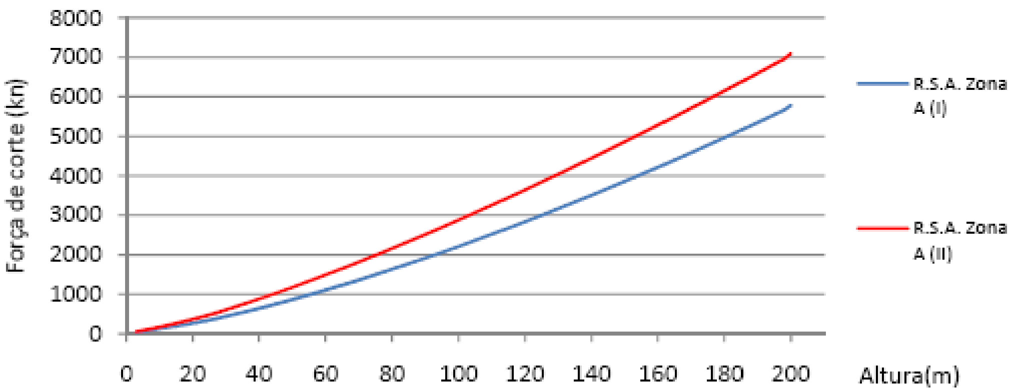


Figure 9 Total shear force at the base of buildings according to RSAEEP

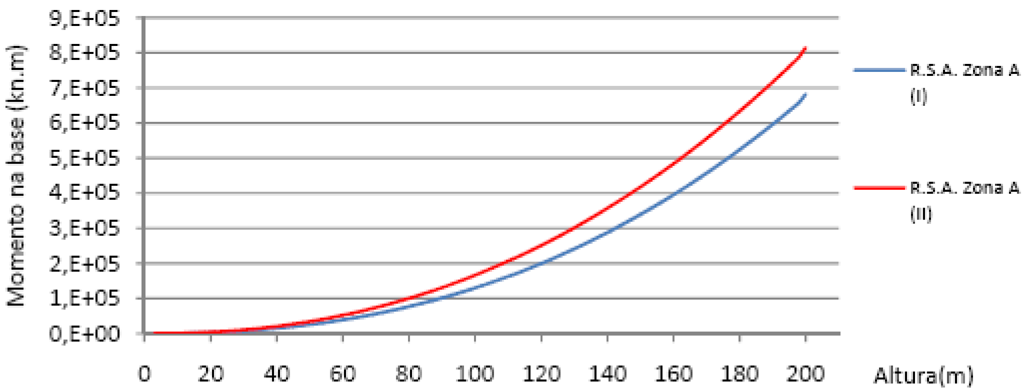


Figure 10 Total bending moment at the base of buildings according to RSAEEP

As shown in Figures 9 and 10 (RSAEEP), a terrain profile corresponding to roughness of type II leads to higher shear forces and bending moments at the base. This situation can be understood because a profile of type II is associated with a roughness smaller than that of a terrain profile of Type I; therefore, type II corresponds to higher mean wind velocity and consequently higher dynamic pressure and greater generalized forces at the base of the buildings.

For a building 200 m height and with the square cross section considered earlier, the shear forces and the bending moments at the base are about 13% higher when calculated for a terrain profile with roughness of type II than when calculated for a terrain profile with roughness of the type I.

Now by using the calculation method proposed in EC1, the wind forces on a structure can be determined by the sum of forces acting on each one of the facades multiplied by a structural factor $c_s c_d$ that can be calculated by two procedures (Procedure 1 and Procedure 2, in Table 6), for the five different categories of terrain roughness.

The left part of Figures 11 and 12 detail the variations with the height of the buildings of the shear forces and bending moments at the base of the buildings determined for the categories of terrain roughness, using EC1 – Procedure 1. Seemingly, the right part of Figures 11 and 12 detail the variations of the shear forces and bending moments at the base of the buildings determined for the categories of terrain roughness, using EC1 – Procedure 2.

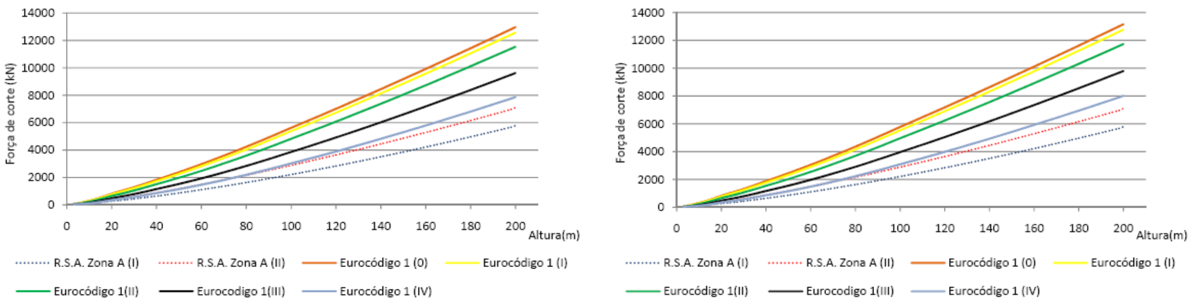


Figure 11 Base shear force as a function of height and terrain category, for structural factor calculated by Procedure 1 (left) and Procedure 2 (right)

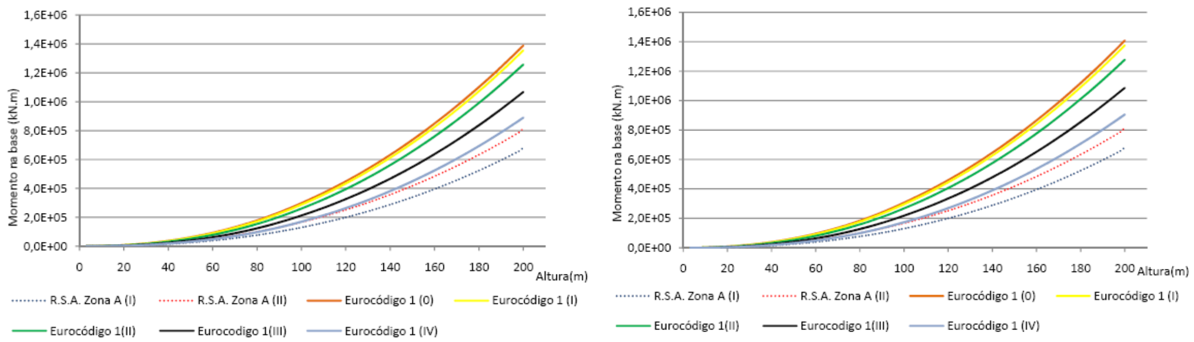


Figure 12 Base bending moment as a function of height and terrain category, for structural factor calculated by Procedure 1 (left) and Procedure 2 (right)

Figures 11-12 show that for the considered basic velocity (28 m/s) the values obtained by the calculation Procedures (1 and 2) described in EC1 are generally higher than those obtained by applying the current Portuguese design code RSAEEP. The difference in the results is due to the pressure coefficients adopted by each design code for the leeward facade of the building. While for the conditions analyzed Portuguese design code allows for the coefficient of external pressure a value of -0.25 (Figure 2), the EC1 admits this coefficient varies as a function of building height and may take values of -0.7 for h/d >5 (Table 5).

By this study and for the building situations examined, it is understood that EC1 is based in the method of the Gust Loading Factor, to quantify the wind actions. Also EC1 considers the non-simultaneity of occurrence of pressures over the building as well as possible dynamic phenomena that may occur.

As regards to this aspect it is seen that the values obtained by each of the proposed procedures (Procedure 1 and Procedure 2) do not differ considerably; also, for lower building heights it seems conservative to use this factor with the unit value.

With EC1 it is possible to estimate the acceleration of the building along the wind longitudinal direction (Table 6), while the Portuguese design code RSAEEP has no indication to that effect. It is also noted that EC1 considers coefficients that take into account the wind direction, season of the year, presence of structures in the vicinity of the building, which constitute factors that are not properly contemplated in RSAEEP. The use and definition in EC1 of a higher number of terrain categories, also allows quantification of more realistic wind actions.

Therefore it seems to exist a considerable evolution in the way of handling the wind actions on buildings by the Eurocode EC1, as compared with RSAEEP especially when dealing with larger tall buildings.

4 NUMERICAL MODELING OF A REAL STRUCTURE UNDER WIND ACTION

This chapter begins by describing the structure of the tall building chosen for case study and how it was modeled computationally. The building chosen was tower-1 of World Trade Center (WTC) and the program used for structural calculations was the "ROBOT MILLENNIUM v16.5" available at FEUP. [It was so chosen WTC because of its significant structural importance, but also majorly *as of public homage to all the innocent victims of the ignominious attack and consequent destruction of this structural icon*].

For use of this program a frame was modeled with specific characteristics that reproduce the structural behavior of the chosen building. The wind loads were calculated according to RSAEEP and to EC1, and its application in the equivalent frame permitted to compare generalized displacements and stresses.

It is also addressed the way the dynamic wind action was modeled to obtain the structural response of the building. Mathematical modeling of turbulent flow is complex and the possibility of interaction between the flow and the tall building structure (FSI) may lead to changes in dynamic pressure and in the response of the building along time. This chapter also addresses the simplifications used to consider this dynamic action.

4.1 Structural description of the building case-study (Tower 1 of WTC)

Each of the twin towers of WTC had 110 floor level above ground zero plaza and 7 underground sub-caves. With a 417 m height and a 64 m x 64 m base, tower-1 owned then the title of tallest building in the world.

The building weighed an average of about 420 ton/m [39] but despite the huge gravitational weight the wind action was the most significant action affecting its structural system (Eagar and Musco [9]). Rigidly connected to each other, the walls of the facades that formed the structure consisted of 240 tubular square hollow columns (tubular pillars) spaced about 1 meter apart (Figure 13) and with varying thickness every 22 floors (Santos [19]). With square section 36 cm x 36 cm, the pillars played a key role in the building structural strength to resist the wind action. The adjacent pillars were connected by a deep beam 1.3 m high (Figure 13).

The wall acted by the incident wind behaved as the tensile flange of the tube, and the opposite wall behaved as the compression flange. The side walls behaved like the web of the tube, and transferred the wind actions between the windward and the leeward walls.

The stiffness of the deep beams, created by the combined effect of reduced span and significant beam height, created a structural system that was rigid laterally and vertically.

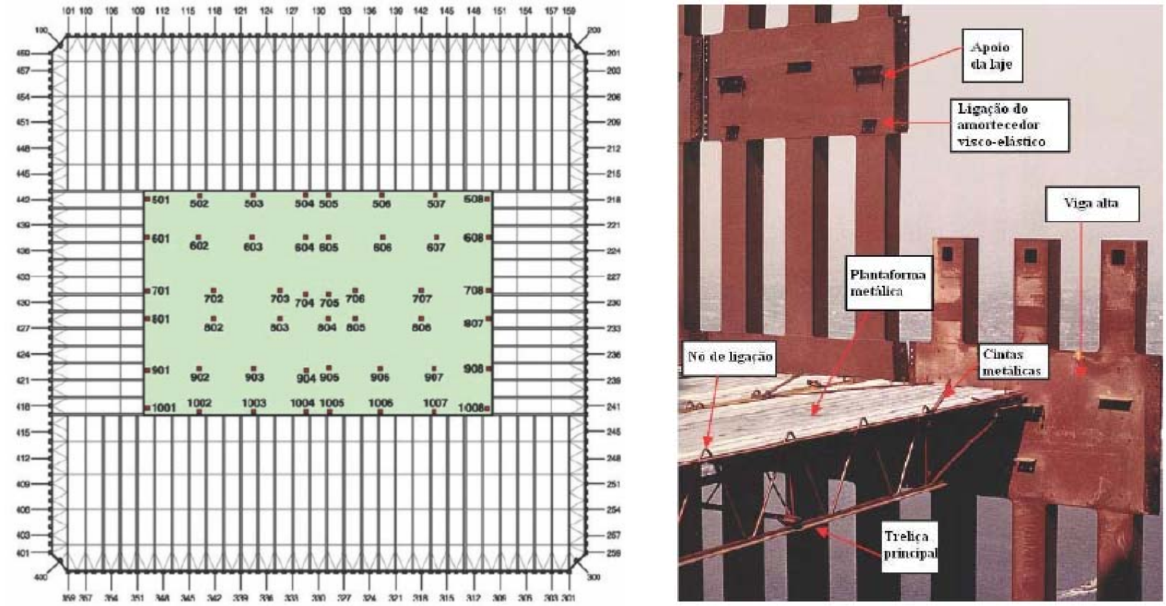


Figure 13 Structural plan of WTC tower-1 and detail of facade pillars with deep beam and metallic truss

The construction of the exterior walls was achieved using prefabricated modules, each consisting of three pillars with a height of three floors, connected by deep beams, and welding all the elements. At the base of the building each set of three adjacent pillars were gathered to form a bigger pillar in a formation like that in a “fork”. According to Eagar and Musco [9] this building was designed to withstand a wind lateral pressure of about 2 kPa.

In the center of the building there was a core that supported most of the building weight. Some of the constitutive pillars were very thick, with sections of 356 mm by 915 mm, converting themselves at the upper floors into large laminated profiles. The building slabs had the behavior of a mixed composite structure, consisting of 10 cm lightweight concrete slabs (13 cm in the core zone) and a metal lattice platform (Wilkinson [24]). Outside the core the platform floor was supported by a series of trusses which were placed between the outer wall and core (Figure 14) and which gave great torsional stiffness to the building.

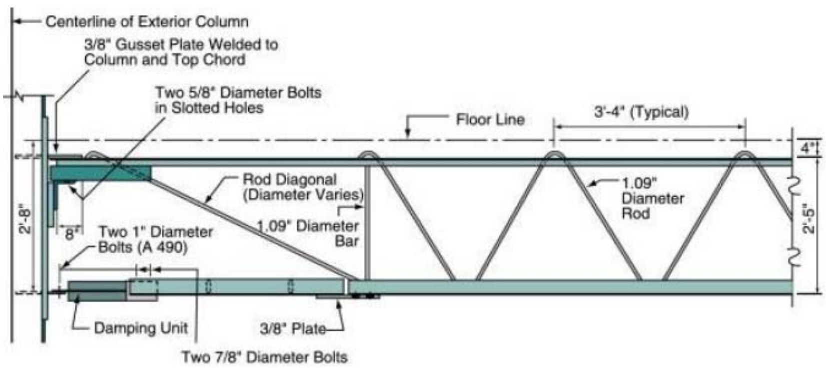


Figure 14 Metallic truss supporting the floor slabs (detail of connection floor-outer facade column)

Between floors 106 and 110 there were a series of diagonal bars in the frames of the building. These diagonals, together with the columns and slabs, formed a space truss extending between the exterior walls and passing through the core of the building. This system reinforced the framework for wind resistance, mobilizing some of the weight supported by the core itself to ensure stability against overturning.

The proximity of the two towers caused that the wind action in each one was conditioned by change in flow caused by such proximity. The quantification of wind action was made through a wind tunnel test, developed at the University of Colorado using a scale model of 1:500 [39]. About 10000 visco-elastic dampers were used, placed between the bottom of the main truss of the slab and the columns of the outer wall, from the 7th to the 107th floor. These dampers, designed by Mahmoodi (Santos [19]), were first applied in skyscrapers with the purpose of reducing building movement induced by wind and earthquakes. Figure 14 also details their location and connection with the outer columns.

4.2 Modeling a frame equivalent to WTC

In the software ROBOT MILLENNIUM v.16.5 was computationally modeled a frame that would reproduce the behavior of the structural system of the WTC tower-1, having stiffness and mass that would approximately describe the behavior of the building under the horizontal wind action. Each of the side columns of the equivalent frame has inertia equivalent to the total inertia of the columns on a plane façade perpendicular to the wind direction. The central column of the frame has an inertia equivalent to the sum of inertias of the two plane facades along lateral wind direction added with the inertia due to the core contribution. A schematic illustration is given in Figure 15.

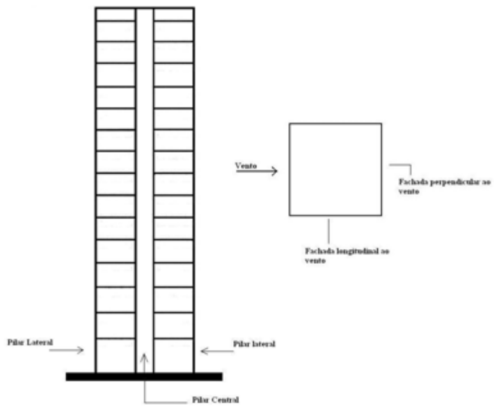


Figure 15 Schematic equivalent frame to WTC

According to Santos [19], the contribution of inertia of the facades along with core is given approximately by the values in Table 7, taking into account the variation in thickness of the columns every 22 floors. The average building mass is 420 ton/m, and the considered steel elasticity modulus and damping ratio were 200 GPa and 2% respectively.

Adopting a rectangular section 1 m wide, the dimensions of the central column were calculated so that the equivalent inertia of inertia would correspond to the inertia of each group. The equivalent sections adopted are presented in Table 5.2. The contribution of the side columns in terms of inertia is small as compared with that the central column.

Structural Group	***WTC structural data***				***Modeled equivalent frame***	
	M (ton/m)	I_{FAC} (m^4)	(%) Core	I_{Total} (m^4)	Lateral Columns	Central Column
Until 22nd floor	454,16	2831,57	34	3794,30	1 m x 1 m	1 m x 36 m
Until 44nd floor	436,32	2165,73	30	2815,44	1 m x 1 m	1 m x 33 m
Until 66nd floor	413,52	1666,18	27	2116,04	1 m x 1 m	1 m x 30 m
Until 88nd floor	392,19	1333,08	21	1613,02	1 m x 1 m	1 m x 27 m
Until 110nd floor	368,80	999,90	15	1149,88	1 m x 1 m	1 m x 24 m

Table 7 Structural data of WTC tower-1 and of the modeled equivalent frame

Regarding to the simulation of the floors, an equivalent beam was simulated equivalent to the metallic lattice truss that supports the slab of lightweight concrete. The center of gravity of the truss and the inertia of the metallic truss bars to that center were determined, corresponding to a value of 3.86 x10^{-2} m^4 for such total inertia of floor. Choosing a beam of rectangular area with an area approximately equal to the area of the main members of the truss, and with the calculated inertia given above, it was found an equivalent beam size with dimensions b=0.55 m and h=0.95 m.

Table 8 shows the values of the natural frequencies and natural periods found for the first six modes of vibration of the equivalent structure. Figure 16 depicts the first two modes of vibration of the structure modeled by the equivalent frame to WTC tower-1.

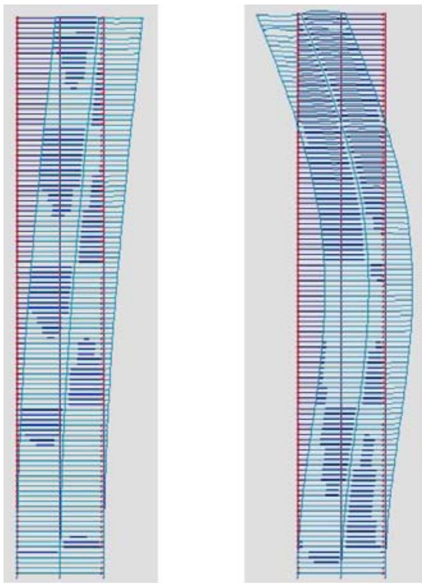


Figure 16 First two modes of vibration of the equivalent frame modeled

The vibration frequency of the first mode was evaluated as f_1=0.132 Hz, quite close to the fundamental frequency given by Santos [19] and relative to a three-dimensional modeling of the building in the program SAP2000. It is seen that the structure had two modes whose frequencies of vibration were less than 1 Hz, but the first mode clearly displays a lower frequency corresponding to a very flexible structure.

Mode	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
Frequency (Hz)	0,132	0,692	1,831	3,496	5,550	8,104
Period (s)	7,577	1,446	0,546	0,286	0,180	0,123

Table 8 Natural frequencies and natural periods of the first six vibration modes of the equivalent frame

4.3 Modeling wind dynamic action

Although it is recognized the great complexity in modeling turbulent flow around buildings, even with scaled physical models in wind tunnels, some simplifications will be considered herein with regards to the quantification of dynamic pressures and generalized forces due to wind action in a tall building along the time. For that, the FSI is considered negligible and the correlations of the velocity fluctuations in height are considered in a simplified manner. Firstly it is addressed the methodology for generating time series of wind to be used latter in the calculation of the instantaneous dynamic pressures and therefore in the quantification of the generalized wind forces acting at every floor level of the equivalent frame.

The methodology used to generate synthetic time series is commonly referred as the Method of Shinozuka, which bases the generation of time series in the calculation of the inverse function of the Fourier Transform of the amplitude of the random process (given by a spectral density function of the energy of a process). Such generation of synthetic series of wind occurs in the range of wavelengths corresponding to fluctuations of wind velocity with an approximately Gaussian distribution of the atmospheric wind flow (Saraiva and Silva [20]).

The purpose of the method is to obtain a realization of a stochastic process (for example: a time series of the fluctuations of the longitudinal component of wind velocity) from the spectral density function that characterizes the process.

The method uses this function to perform a weighted sum of sinusoidal functions (in this case of cosines). The contribution of each of the N waves is given by the amplitude of the spectrum ($S_L(z,n)$, real function) for each corresponding natural frequency (*n*). The phases are obtained (for the case of one-dimensional spectrum of simple non-correlated series) by pseudo-random number generation in the interval [0, 2π].

According to the Method of Shinozuka, in the simplest case of one-dimensional univariate stochastic processes, a realization of the random process may be obtained (Barbat and Canet [4], Saraiva and Silva [20]) by equation (26).

$$u(t)=\sqrt{2}\sum_{k=1}^{n} A_k \cos(\omega_k\, t+\phi_k) \quad \text{with } A_k=\sqrt{S(\omega_k)\,\Delta\omega} \text{ and } \Delta\omega=\frac{\omega_{\max}-\omega_{\min}}{N} \tag{26}$$

In the previous expression N is the number of frequencies of the discretization of the spectrum, and $\omega = 2\pi n$ is the angular frequency. To generate the synthetic time series of wind velocity it is necessary to define a spectral density function of the fluctuations of longitudinal velocity of the wind; the spectral density function given in EC1 is used herein in the dimensionless form of equation (23) and the description of Figure 7.

For the generation of the synthetic series to be considered an ergodic process, according to [40] the number N of frequencies for discretization of the spectrum should be sufficiently high. However, not having an indication of the number of discrete frequencies to use, a study of the generation of wind series was made discretizing the spectrum into a different number of frequency ranges. Herein five processes for five frequency intervals were studied, corresponding to the division of the spectral density function in 50, 100, 500, 1000 and 5000 intervals respectively; for each case, fluctuations of longitudinal wind velocity were obtained according with the process described.

These fluctuations were calculated for an elevation *H*=100 m, and considering that the WTC was located on a terrain of class IV according to EC1 to which corresponds a roughness length z_0=1 m. The basic velocity assumed for a return period of 50 years was 30 m/s (based on the recorded data in 3 meteorological stations close to New York city) and in these conditions, according to EC1, standard deviation of the turbulent component of wind velocity was $\sigma_v = 7.03$ (Santos [19]). Figure 17 represents the time series of the fluctuations of wind velocity, evaluated with the previous data, for the four higher discretizations.

As the spectral density function has high values for low frequencies but reduces rapidly with increasing frequency (Figure 7), from the analysis of previous figures it is noted that when choosing a lower number of frequencies of discretization (lower N) for generation of the synthetic time series of wind, these are clearly more affected by low frequency components (where the spectrum has more energy) resulting in a value numerically higher.

Herein was adopted a reference number of discretization frequencies of around 500, for which it is no longer noticeable the influence of low frequency components; also, although for discretizations with 1000 and 5000 intervals the greater number of sinusoids would induce a better defined process, there seems not to exist large differences between the time series associated with these divisions of the wind spectrum.

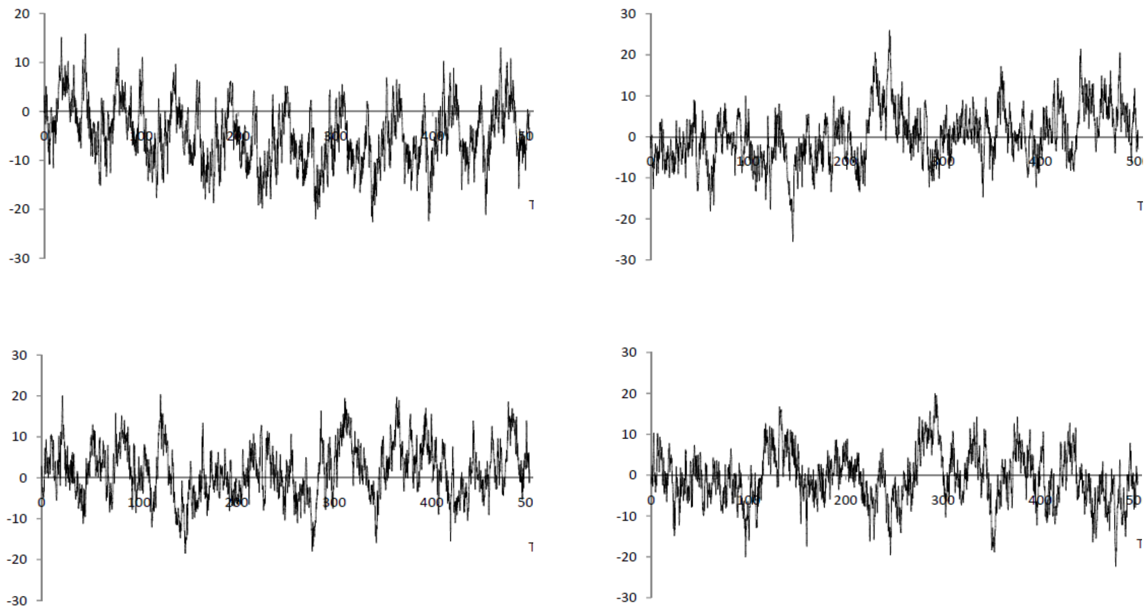


Figure 17 Time series of fluctuations of wind velocity for wind spectrum with 100-500-1000-5000 intervals

For the instantaneous wind velocity *U(t)* at any height given by the sum of a constant mean component $\overline{U}$ with a dynamic fluctuation component *u(t)*, the instantaneous wind force *F(t)* on any surface *A* is given by:

$$F(t)=\tfrac{1}{2}\rho c_f A\left[\overline{U}+u(t)\right]^2=\tfrac{1}{2}\rho c_f A\overline{U}^2+\rho c_f A\overline{U}\,u(t)+\tfrac{1}{2}\rho c_f A\left(u(t)\right)^2 \qquad (27)$$

where c_f is the force coefficient for the structure or structural element.

The fluctuations of wind velocity along time also have a spatial variability, which for a first approximation is herein neglected. For the case of tall buildings whereas the response is majorly due to the contribution of the first mode of vibration (which is also a condition imposed for the calculation of the structural factor $c_s c_d$ by EC1), modeled as a structural system with one degree of freedom, the passage or conversion of the power spectrum of the wind velocity fluctuations into structural response spectrum is given by:

$$S_X(n)=\frac{4\overline{X}}{\overline{U}^2}\left|H(n)\right|^2\left(\chi(n)\right)^2 S_u(n) \qquad (28)$$

where $\left|H(n)\right|^2$ represents the mechanical admittance function given by equation (29) and $\left(\chi(n)\right)^2$ represents an aerodynamic admittance function (Vickery and Kao [22]) given approximately by equation (30).

According to Holmes [12], in a frequency domain analysis for tall large structures, it is this latter function that takes into account the non-simultaneous occurrence of the fluctuations of wind velocity. It is explicitly stated that: "*For larger structures, the velocity fluctuations do not occur simultaneously over the windward face and their correlation over the whole area must be considered. To allow for this effect, an aerodynamic admittance* $(\chi(n))^2$ *is introduced*".

$$|H(n)|^2 = \frac{1}{\left(1-\left(\frac{n}{n_1}\right)^2\right)^2 + \left(2\xi\frac{n}{n_1}\right)^2} \qquad (29) \qquad\qquad \chi(n) = \frac{1}{1+\left(\frac{2n\sqrt{A}}{\overline{U}}\right)^{4/3}} \qquad (30)$$

According to EC1 for tall structures with the shape and conditions equivalent to the case-study under consideration, the parameters of the spectral density function for calculating the structural factor should be determined for a reference height of approximately 0.6 times the height of the building. Given this indication, for generating sets of time series, the height chosen was 250 meters that is about 60% of the height of tower-1 of the World Trade Center.

So, for the dynamic analysis of this case-study, ten random sets of fluctuation velocities were generated. The applied wind generated forces were obtained according to equation (27) taking account the acting dynamic pressures and the influence area for each floor, considering the mean wind velocity depending on the height (given by the expression of EC1) and the fluctuation velocities given by the random series generated.

As an example, Figure 18 (left) presents one series (called Series 1) for the fluctuations of wind velocity generated in these conditions at a height of 250m and using wind power spectrum of EC1. Figure 18 (right) shows the same series, that is adopting the same phase angles for the harmonics, but generated from the wind power spectrum multiplied by the previously mentioned aerodynamic admittance function $\chi^2(n)$.

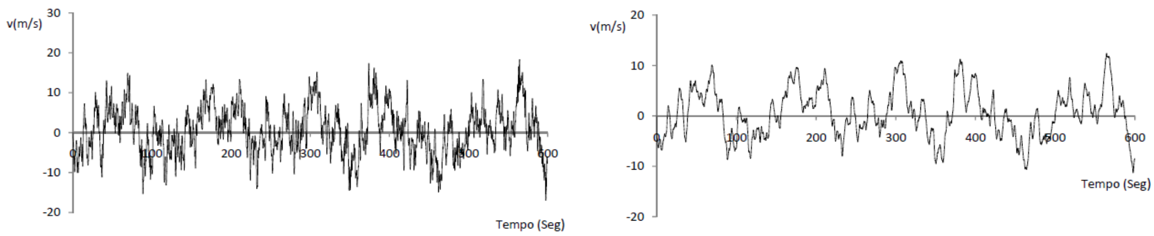


Figure 18 Fluctuation velocity time series for: height of 250 m, basic velocity 30 m/s, terrain roughness of category IV, using EC1 wind power spectrum (at left, not multiplied by the aerodynamic admittance function; at right, already multiplied by aerodynamic admittance function)

4.4 Modeling a TMD for passive control of vibrations

The tuned mass dampers (TMD) can be used to control one or more vibration modes of structures excited by external actions. However, in many cases, control of the first mode is sufficient to reduce significantly the level of vibrations recorded. Except for cases in which it is intended to simultaneously monitor the contribution of more than one mode of vibration, the use of a single TMD may be satisfactory. Examples of application of this system in tall buildings are listed in the literature, namely: John Hancock Tower Building (with 60 floors) in Boston (USA) and the building Chiba Port Tower (with 125 m) in Japan, where large-scale measurements were performed to evaluate the efficiency these systems (Holmes [12]).

The building tower Taipei 101 (with 508 m) in Taiwan, has the largest (so far) TMD mass (about 730 ton mass) placed on top of the building to control excessive vibrations due to wind and earthquakes; according to [14], the accelerations would be reduced by about 40% to 45%.

The behavior of TMD in tall buildings under wind action was studied by Kwok and Samali [17]. The design of a TMD for application to structures without damping is based on two parameters – mass ratio μ and frequency ratio q – as detailed in Kelly [16]. The optimum frequency ratio q_{opt} (corresponding to the fixed points at the same level or with the same displacement amplitude), the maximum amplitude of the controlled principal system, and the inherent optimal damping $\xi_{2,opt}$ of the TMD, are given in the set of equations (31).

$$q_{opt} = \frac{1}{1+\mu} \;, \qquad \frac{X_1}{X_{1,static}} = \sqrt{\frac{2+\mu}{\mu}} \;, \qquad \text{and} \qquad \xi_{2,opt} = \sqrt{\frac{3\mu}{8\left(1+\mu\right)^3}} \tag{31}$$

For the design of a TMD tuned for application to structures with damping, it is still possible to use these equations provided the damping is less or equal to 1%. For higher damping, the use of such equations will lead to a non optimized tuning of the TMD. For such cases, the design of the TMD can be done with design graphs associated with the numerical solution of the expression of maximum amplitude of the controlled principal system (Barros, Moutinho and Barros [6]).

The considerations made previously focused its application to a structure with a single degree of freedom which was associated with a second mass, thus resulting in a system of two degrees of freedom. However, in the case of real structures with various degrees of freedom, it becomes essential to consider a system of one degree of freedom that translates roughly the dynamic behavior of the structure under analysis.

The reduction of structural vibrations using the vibration control theory can be addressed either through modal control or through optimal control (Yao [25]). In the former, a predetermined structural mode of vibration is controlled; in the case of long slender metallic towers under wind actions, tuned mass dampers (TMDs) are frequently the choice but tuned liquid dampers (TLDs) or tuned liquid column dampers (TLCDs) can also be used with equivalent efficiency. In the latter, structural performance criteria are controlled such as the minimization of the structural deflections (top tip displacement and/or top section rotation).

Kwok and Samali [17] studied the behavior of TMD's in tall buildings subjected to the action of wind and, according to the authors, the considerations presented here about the effectiveness of a TMD in response of a system of one degree of freedom can be extended to solid structures such as in the case of tall buildings, leading to a modal analysis. Kwok and Samali [17] indicated that while there were large decreases in response for the modes controlled by the TMD's installed, the higher order modes were not affected. For such higher modes to be less contributive to the structural response would require implementing new TMD's tuned according to their frequency.

Thus, using modal analysis, for each vibration mode whose contribution to the overall response of the structure is important, and that one wishes control, it is necessary to determine the corresponding values of stiffness, of mass and of modal damping. According to Villaverde [23], for the determination of the dynamic characteristics of the equivalent system, should be adopted a mode normalization criteria based on attributing a unit value to the mode component associated to the degree of freedom where TMD would be applied.

Since the fundamental frequency of the WTC tower-1 is very low (0.132 Hz) and because the wind action has a spectral density function with strong content for low frequencies, it is possible that the response is conditioned by the harmonic of the fundamental frequency. It is therefore assumed herein, for control of vibrations purposes, that the response is only dependent on the first vibration mode, with which the TMD solutions were designed with the expressions available for harmonic vibration with frequency equal to the first vibration frequency of the overall structure.

Accordingly, the value of the modal mass corresponding to the first mode of vibration was determined (with the structural software used: Robot Millennium) as 41378.414 ton and the corresponding modal stiffness of 28463kN/m, values quite close to the ones found in a three-dimensional modeling of the building (Santos [19]). For the case-study structure with the deployment of TMD's, two different mass ratios of μ=0.01 and μ=0.005 were used, for which with design charts (Barros, Moutinho and Barros [6]) it was possible to determine the optimal parameters to be adopted for each TMD situation. In Table 9 the values adopted are systematized.

TMD mass ratio	q_{opt}	$\xi_{TMD, opt}$	m_{TMD} (ton)	ω_{TMD} (rad/s)	k_{TMD} (kN/m)		Size (cm) of square section steel bar E=210 GPa , L=2 m
μ = 0.01	0,987	0,046	413,78	0,81860	277,2764		8,1 x 8,1
μ = 0.005	0,993	0,036	206,89	0,82357	140,3284		6,8 x 6,8

Table 9 Optimal parameters of TMD in WTC tower-1 case-study, for two mass ratios μ

Since the structural software used does not have an intrinsic function that allows the direct introduction of dampers, herein for the simulation of the TMD's were determined the dimensions of a square section bar with a lateral stiffness equivalent to that required for the damper placed on top. Acting as a vibrating bar (built in end – free end) with a concentrated mass that would give the frequency obtained for the sizing of the TMD with the damping introduced in the material parameters constitutive of the bar.

Assuming a bar length L=2 m, made of steel with elasticity module E=210 GPa, from the bar stiffness $3EI/L^3$ is obtained the equivalent inertia I of the square section bar. Table 10 also indicates the dimensions required for such bar, for the two mass ratios considered in the design of the TMD. Table 10 shows the first six natural frequencies of the vibration modes of the case-study structure (WTC tower-1) incorporating the two TMD solutions; some changes occur as compared with Table 8 of the structure without any TMD.

Mode	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
Frequency (Hz) for WTC using TMD with μ=0.01	0,125	0,138	0,692	1,831	3,496	5,550
Frequency (Hz) for WTC using TMD with μ=0.005	0,127	0,136	0,692	1,831	3,496	5,550

Table 10 Natural frequencies of first six vibration modes of the equivalent frame with TMD vibrating bar

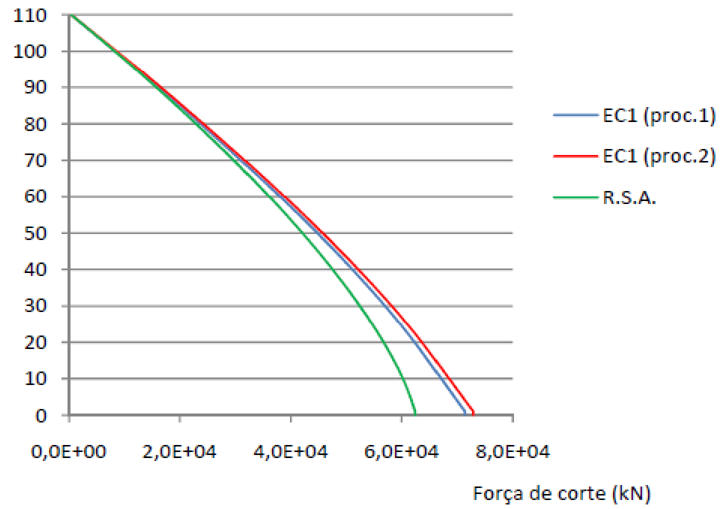
5 ANALYSIS OF RESULTS BY DESIGN CODES

This chapter begins by comparing the results obtained for the forces acting on each floor due to wind action, obtained by applying each of the two regulations addressed in this case-study of the WTC tower-1. The corresponding displacements were also evaluated and compared with the computer software used herein, applied to the computational structural model addressed earlier.

The wind was considered actuating perpendicular to the facade of the building and for the implementation of RSAEEP, the building was considered as implanted in Zone B with terrain roughness of type 1 corresponding to an urban area.

For application of EC1 the building was considered on a terrain category of type IV, with a basic velocity of 30 m/s, with unit orography factor and with frictional forces neglected (since the total area of the facades of the building parallel to the wind direction is less than 4 times the area of the facades perpendicular to the wind direction. Since the ratio between the height of building h and its width d is greater than 5, the EC1 indicates that the forces due to wind on buildings should be based on force coefficients. For the building under study, the value obtained for the force coefficient was 1.428 . The structural factor calculated according to EC1, by the two procedures (Procedure 1 and Procedure 2) contemplated in that design code as outlined in the previous paragraph (2.2.7), was 0.95 (Procedure 1) or 0.97 (Procedure 2).

Table 11 compares the results for the wind forces, along the height of the case-study building, evaluated by the two design codes. Figure 19 compares the results for the shear forces and bending moment due to wind pressures, along the height of the case-study building, evaluated by the two design codes.



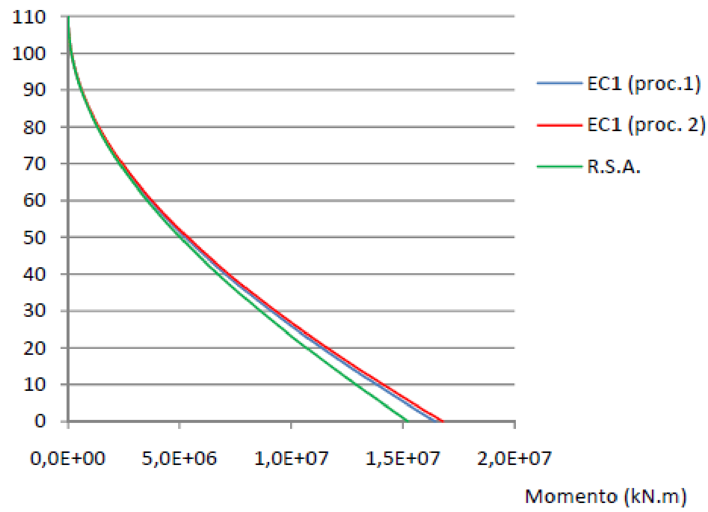


Figure 19 Shear forces (top graph) and bending moments (bottom graph) at every floor of WTC, evaluated by the two design codes: RSAEEP and EC1

Floor	RSAEEP	EC1(Pr.1)	EC1(Pr.2)
1	218,8	472,9	482,9
2	218,8	472,9	482,9
3	218,8	472,9	482,9
4	218,8	472,9	482,9
5	239,1	472,9	482,9
6	254,5	472,9	482,9
7	268,8	472,9	482,9
8	281,6	472,9	482,9
9	293,8	472,9	482,9
10	305,2	472,9	482,9
11	316,1	472,9	482,9
12	326,4	472,9	482,9
13	336,3	472,9	482,9
14	345,8	472,9	482,9
15	354,9	472,9	482,9
16	363,7	472,9	482,9
17	372,2	472,9	482,9
18	380,5	483,3	493,5
19	388,5	491,9	502,2
20	396,3	500,0	510,6
21	403,9	507,9	518,5
22	411,3	515,3	526,2
23	418,6	522,5	533,6
24	425,7	529,5	540,6
25	432,6	536,2	547,5
26	439,4	542,6	554,0
27	446,0	548,9	560,4
28	452,5	554,9	566,6
29	458,9	560,8	572,6
30	465,2	566,4	578,4
31	471,4	571,9	584,0
32	477,5	577,3	589,5
33	483,5	582,5	594,8
34	489,3	587,6	600,0
35,0	495,1	592,5	605,0
36,0	500,8	597,4	609,9
37,0	506,5	602,1	614,8
38,0	512,0	606,7	619,5
39,0	517,5	611,2	624,0
40,0	522,9	615,6	628,5
41,0	528,3	619,9	632,9
42,0	533,5	624,1	637,2
43,0	538,7	628,2	641,4
44,0	543,9	632,2	645,5
45,0	549,0	636,2	649,6
46,0	554,0	640,0	653,5
47,0	559,0	643,8	657,4
48,0	563,9	647,6	661,2
49,0	568,7	651,2	664,9
50,0	573,6	654,8	668,6
51,0	578,3	658,4	672,2
52,0	583,0	661,8	675,8
53,0	587,7	665,3	679,3
54,0	592,3	668,6	682,7
55,0	596,9	671,9	686,1
56,0	601,5	675,2	689,4
57,0	606,0	678,4	692,7
58,0	610,4	681,5	695,9
59,0	614,8	684,6	699,0
60,0	619,2	687,7	702,2
61,0	623,6	690,7	705,2
62,0	627,9	693,7	708,3
63,0	632,2	696,6	711,2
64,0	636,4	699,5	714,2
65,0	640,6	702,3	717,1
66,0	644,8	705,1	720,0
67,0	648,9	707,9	722,8
68,0	653,0	710,6	725,6
69,0	657,1	713,3	728,3
70,0	661,2	716,0	731,0
71,0	665,2	718,6	733,7
72,0	669,2	721,2	736,4
73,0	673	724	739
74,0	677,1	726,3	741,6
75,0	681,0	728,8	744,1
76,0	684,9	731,3	746,7
77,0	688,7	733,7	749,2
78,0	692,6	736,1	751,6
79,0	696,4	738,5	754,1
80,0	700,2	740,9	756,5
81,0	703,9	743,2	758,9
82,0	707,7	745,5	761,2
83,0	711,4	747,8	763,5
84,0	715,1	750,1	765,9
85,0	718,8	752,3	768,1
86,0	722,4	754,5	770,4
87,0	726,0	756,7	772,6
88,0	729,6	758,9	774,8
89,0	733,2	761,0	777,0
90,0	736,8	763,1	779,2
91,0	740,3	765,2	781,4
92,0	743,9	767,3	783,5
93,0	747,4	769,4	785,6
94,0	750,9	801,3	818,1
95,0	754,3	801,3	818,1
96,0	757,8	801,3	818,1
97,0	761,2	801,3	818,1
98,0	764,6	801,3	818,1
99,0	768,0	801,3	818,1
100,0	771,4	801,3	818,1
101,0	774,8	801,3	818,1
102,0	778,1	801,3	818,1
103,0	781,4	801,3	818,1
104,0	784,8	801,3	818,1
105,0	788,1	801,3	818,1
106,0	791,3	801,3	818,1
107,0	794,6	801,3	818,1
108,0	797,9	801,3	818,1
109,0	801,1	801,3	818,1
110,0	401,7	400,6	409,1
TOTAL	**62446**	**71347**	**72849**

Table 11 Forces on every floor due to wind, under the two design codes RSAEEP and EC1

For both generalized forces, the two EC1 procedures gave higher values than the one's obtained by RSAEEP. In fact the basal bending moment by RSAEEP was evaluated as 15231856 kNm while by the procedures in EC1 was 16612740 kNm and 16962482 kNm.

Applying the previous wind design forces on every floor in the computational model of the case-study WTC, the following lateral displacements were obtained (Figure 20) leading to top floor lateral displacements of 0.99 m (with RSAEEP forces) and 1,05 m and 1,07 m (with the two procedures of EC1 for the determination of the structural factor). These lateral displacements, obtained with the simplified equivalent computational model proposed, slightly overpass the recommended maximum value of top displacement given by the building height/500 (0.85 m, for this WTC tower with a height of 417 m) as suggested by the *Subcommittee on Wind Bracing* of ASCE [2].

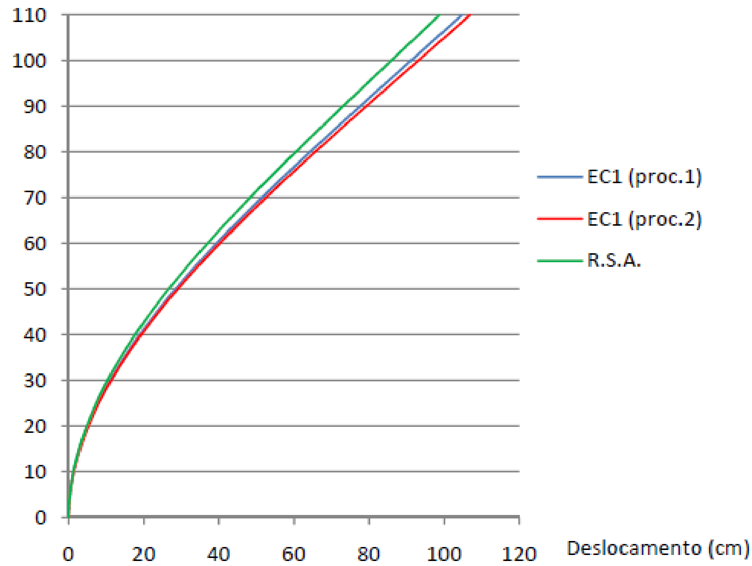


Figure 20 Lateral displacements in every floor by the application of the design forces on the considered computational model

As regards to the determination of the maximum acceleration on the top of this building, according to the two procedures in EC1 described earlier (paragraph 2.2.7) for multiplying the standard deviation of the acceleration by the peak factor, the values obtained were 0.27 m/s2 (Procedure 1) and 0.32 m/s2 (Procedure 2). Comparing these acceleration values due to wind action with the limits of human perception, it was clear that the lateral top acceleration lies between the threshold of perception and discomfort (Bachmann [3]); therefore no problems are expected to the level of human comfort due to the wind horizontal direction analyzed.

It should be noted that if the aerodynamic damping would not have been considered, the acceleration values would be ranging from 0.38 m/s2 to 0.45 m/s2 and that would lead to discomfort for human occupants. This shows once more that, in the longitudinal along the wind direction, the aerodynamic damping plays an important role in vibration control, and therefore its correct evaluation is really important for a good design.

6 ANALYSIS OF RESULTS EVALUATED COMPUTATIONALLY

Based on the methodology adopted for consideration of the dynamic wind action (using a set of 10 time series, for frequencies in the wind spectral density function evaluated with 500 frequency intervals), the results in terms of displacements and accelerations were evaluated and compared for the computational structural model, without and with installed TMD vibrating bar (with an hypothetical vibrating mass with appropriate stiffness and damping properties).

Using the mentioned structural software with modal superposition, a damping ratio of 2% and an integration time step of Δt=0.25 sec, ten series of wind dynamic loads were applied and their average results obtained in terms of displacements and accelerations. As an example Figure 21 shows the time variations of displacement and acceleration on the top of WTC, for the wind loads evaluated using equation (27), with velocity fluctuations corresponding to wind series 1.

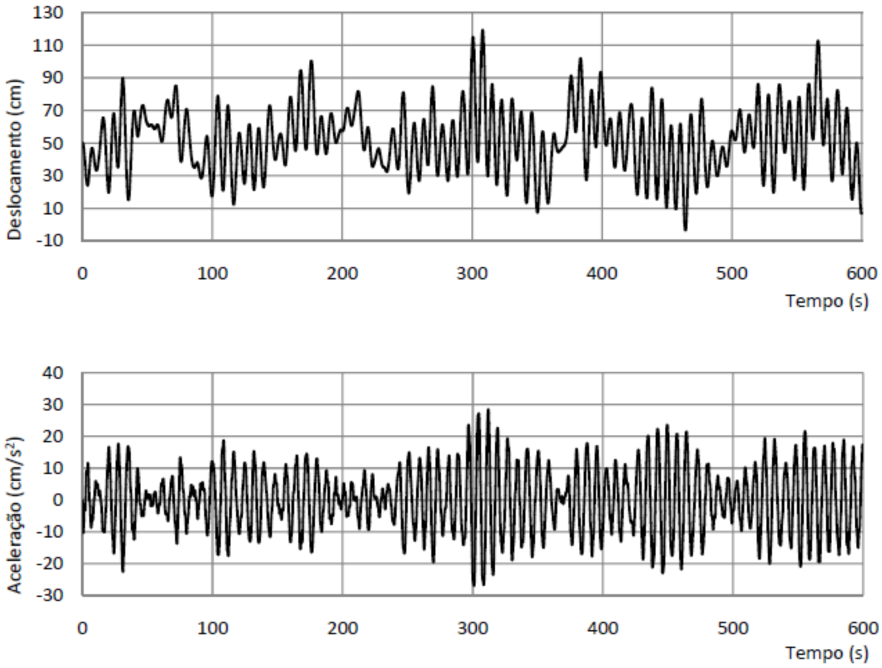


Figure 21 Displacement and acceleration on the top of WTC, for the wind loads corresponding to wind series 1

Table 12 presents a summary of maximum values of displacements and accelerations on top of the building, for each of the time series. It also presents the maximum values of the same variables but which are calculated (Holmes [12]) by multiplying the standard deviation of the responses by the peak factor of 3.16 (evaluated conservatively for the natural frequency of the structure of 0.11 Hz and for a time interval of 600 seconds, during which the maximum value is evaluated (Davenport [8]).

SÉRIE	1	2	3	4	5	6	7	8	9	10	Média
Deslocamento máximo obtido directamente da resposta (m)	1,19	1,32	1,03	1,09	1,09	1,33	1,05	1,15	1,22	1,27	**1,17**
Aceleração máxima obtida directamente da resposta (m/s^2)	0,28	0,35	0,32	0,27	0,36	0,32	0,35	0,29	0,36	0,37	**0,33**
Deslocamento máximo obtido através do factor de pico (m)	1,12	1,16	1,21	1,16	1,18	1,20	1,09	1,10	1,24	1,25	**1,17**
Aceleração máxima através do factor de pico (m/s^2)	0,32	0,34	0,37	0,30	0,36	0,31	0,33	0,30	0,39	0,38	**0,34**

Table 12 Maximum displacements and accelerations on top of WTC, for each of the wind time series (without TMD)

As regards to the use of a TMD on the top floor, and as an earlier comparison with the building subjected to a harmonic excitation in resonance with the fundamental frequency, the Figure 22 shows such comparison of displacements (left) and accelerations (right) of top floor, along the time, without and with TMD with mass ratios of 1% and 0,5%.

Table 13 shows the comparison of the maximum displacements and maximum accelerations of the top floor in such circumstances; to it correspond attenuation of displacements of 58% and 68% respectively.

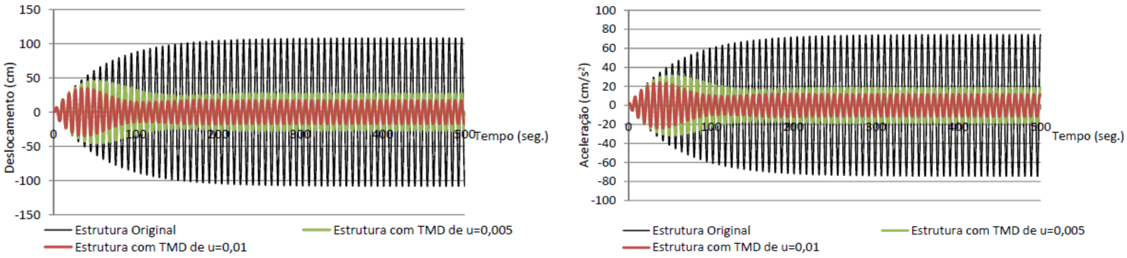


Figure 22 Displacements and accelerations of the top floor, under a harmonic fundamental resonant excitation, without and with TMD's

	Maximum displacement (m)	Maximum acceleration (m/s^2)
Building without TMD	1,08	0,75
Building with TMD (μ=0,005)	0,45	0,31
Building with TMD (μ=0,01)	0,35	0,24

Table 13 Maximum displacements and accelerations of the top floor, under a harmonic fundamental resonant excitation, without and with TMD's

Figure 23 shows the time variations of displacement and acceleration on the top of WTC, equipped with the TMD modeled before with mass ratio of 1%, for the wind loads evaluated using equation (27), with velocity fluctuations corresponding to wind series 1. Table 14 presents a summary of maximum values of displacements and accelerations on top of the building, for each of the time series. It also presents the maximum values of the same variables, multiplying the standard deviation of the responses by the peak factor.

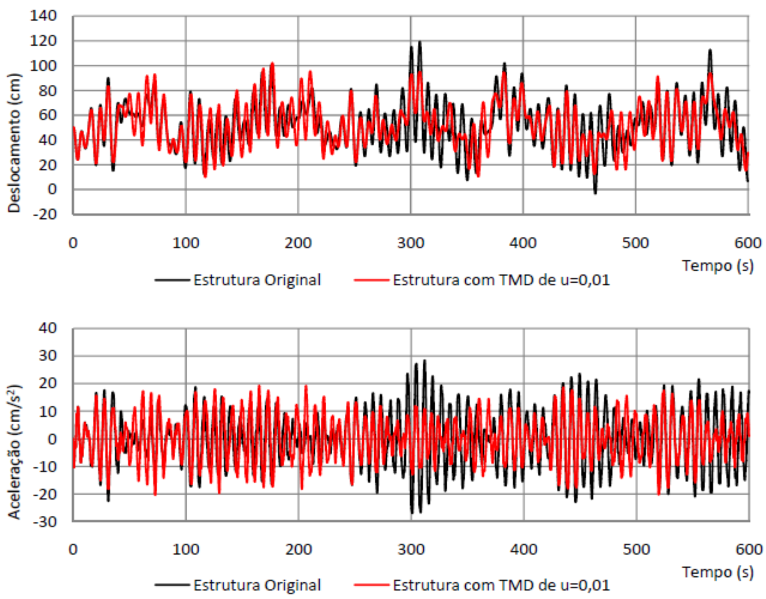


Figure 23 Displacement and acceleration on the top of WTC, equipped with the TMD modeled with mass ratio of 1%, for the wind loads corresponding to wind series 1

SÉRIE	1	2	3	4	5	6	7	8	9	10	Média
Deslocamento máximo obtido directamente da resposta (m)	1,02	1,08	1,08	0,98	0,96	1,28	0,94	1,03	1,11	1,21	**1,07**
Aceleração máxima obtida directamente da resposta (m/s^2)	0,20	0,21	0,22	0,20	0,23	0,28	0,33	0,27	0,29	0,30	**0,25**
Deslocamento máximo obtido através do factor de pico (m)	1,07	1,05	1,05	1,05	1,08	1,17	1,04	1,05	1,11	1,17	**1,08**
Aceleração máxima através do factor de pico (m/s^2)	0,26	0,25	0,22	0,23	0,28	0,28	0,29	0,23	0,33	0,32	**0,27**

Table 14 Maximum displacements and accelerations on top of WTC, for each of the wind time series (using TMD with mass ratio of μ=0,01)

The efficiency on the use of the modeled TMD on top of building can be interpreted by the results of Table 15, here associated with mass ratio of 1%: reduction of maximum displacements and accelerations on the order of 10% and 20%, respectively. Quite similar conclusions, with less efficiency, were obtained for the TMD with mass ratio of 0,5%.

	Building wº/ TMD	Building w/ TMD (μ=0,01)	Reduction relative to building wº/ TMD
Average max displacement evaluated from response	1,17 m	1,07 m	9%
Average max acceleration evaluated from response	0,33 m/s^2	0,25 m/s^2	24%
Average max displacement evaluated from peak factor	1,17 m	1,08 m	8%
Average max acceleration evaluated from peak factor	0,34 m/s^2	0,27 m/s^2	21%

Table 15 Efficiency of using the modeled top TMD for mass ratio of 1%

7 CONCLUSIONS

After a comparative analysis between the Portuguese Regulation on Safety and Actions for Building Structures and Bridges (RSAEEP) [18] and Eurocode 1 (EC1) [10], it can be concluded that there are differences in these methods here considered to quantify the wind actions with an improvement of the European regulation. Eurocode 1 is based on the method of "Gust Loading Factor" for the quantification of wind action and that takes into account a structural factor that considers the non-simultaneity of the occurrence of the pressures along the facade of the building where potential dynamic phenomena may occur.

Comparing the two procedures suggested by EC1 to quantify this structural factor, it can be concluded that the difference in values obtained by each of them is less than 5%; since it is dependent on the frequency of vibration of the structure, a decrease of same frequency leads to an increase in the value of the structural factor. With a variation of vibration frequency of the structure of 25% over the value obtained by the expression recommended for buildings by EC1, the difference in the value of the structural factor as calculated by the two procedures continues to be no more than 5%, thus leading both of the procedures adopted to very similar results.

A parametric study on the variation of the design wind shear forces and design wind bending moments as a function of height, as evaluated by the RSAEEP and by EC1, was performed using the World Trade Center (WTC) as the reference tall building type.

It can be concluded that for the considered base velocity (28m/s), the values obtained for these generalized forces by EC1 are generally higher than those obtained by RSAEEP, a fact that is related to differences in the coefficient of external pressure evaluated by each code standard adopted for the along the wind façade. Also the definition of a greater number of terrain categories inherent to EC1, allows a more realistic quantification of the wind actions as compared with RSAEEP.

Regarding the modeling of an equivalent frame structure equivalent to the reference World Trade Center, it can be concluded that the value of the structural parameters found for the frequency, mass and stiffness corresponding to the first mode of vibration led to a structural behavior very close to the three-dimensional modeling performed earlier by Santos [19].

For modeling the dynamic wind action reference is made to a method of generating sets of synthetic wind – called the method of Shinozuka – and for which the number of discretization intervals to adopt is discussed; the greater the number of intervals to adopt, the better the process, but with divisions over 1000 intervals results are already quite acceptable.

The simplified methodology adopted for the evaluation of the effects of the dynamic wind action, consisted of varying forces over time at each floor, following the same law of variation. This law is obtained, from each generated time series based on the power spectrum previously multiplied by the aerodynamic admittance function.

Through the generation of ten different series, it was concluded that the dynamic results obtained in terms of average values of maximum displacements, were about 10% higher than those results obtained by applying EC1. Also, the dynamic response of the structure is greatly influenced by the wind series of load considered.

The dynamic response of the building, in the longitudinal along-the-wind direction, indicates that the dynamic phenomena for the case study are not significant with regards to the occupants comfort level since the accelerations obtained are within the limits considered acceptable. It should be emphasized that for this situation the aerodynamic damping, added to the structural damping, appears to contribute significantly in controlling the response under service conditions in this direction.

Regarding the implementation of TMD, it was concluded that these are proving very effective in terms of displacements and in terms of acceleration when the structure is subjected to a harmonic action with frequency equal to the fundamental frequency of vibration of the building. The application of these devices for vibration control can therefore be very effective for control resonances in the transverse direction to the flow caused by the formation of vortices.

However, the attenuation found for the structure equipped with a modeled top floor TMD, when subjected to the wind action modeled as representative, depends greatly on the wind series generated; it is not as effective as before, under perfect harmonic action at resonance.

For the TMD modeled with the parameters calculated, it was concluded that in terms of maximum accelerations reductions of the order of 24% can be achieved for a TMD with 1% mass ratio, and reductions of the order of 18% can be achieved for a TMD with a 0.5% mass ratio. With regard to maximum displacements it was concluded that the structural reference system has proved less effective, achieving reductions of only 9% for a TMD with 1% mass ratio, and of the order of 7% for a TMD with 0,5% mass ratio.

ACKNOWLEDGEMENTS

This work is integrated in the thematic and activities of the international collaborative research project COVICOCEPAD approved by the European Science Foundation (ESF) within the Smart Structural Systems Technologies (S3T) Program. It was sponsored in Portugal until last December 2010 by FCT (Fundação para a Ciência e a Tecnologia) project PPPCDT-05-S3T-FP054-COVICOCEPAD, fact that is herein acknowledged.

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