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COMPARISONS OF A TALL LATTICE WIND TOWER RESPONSE WITH AND WITHOUT A TMD

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Abstract. *This paper addresses basic concepts regarding the wind effects on a lattice wind tower with 150 m of height and the subsequent wind responses. The methodology presented for simulating series of natural wind is the method of Shinozuka. 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. It was also accomplished a comparative study of response of lattice wind tower when subjected to natural winds without and with a TMD.*

1 INTRODUCTION

With fossil fuels becoming increasingly scarce and expensive, the world seeks solutions to serve the interest of economic development and the preservation of nature. Wind energy plays a very important role in the global panorama of energy, as it is a source of renewable energy that has the least impact on nature [1]. Therefore the rising demand on wind energy caused the development of related technologies, for example on the type of tower.

This new idea of developing lattice towers of great height, since they have significantly lower construction costs [2][3], poses new challenges for the structural engineers with regard to dynamic effects. Lattice towers are sensitive to the dynamic environments generated by wind, ice, earthquakes, impact, blast, explosions and mechanical failures of some of their components. The vibrations induced in the tall lattice tower structures by these environmental and mechanical causes cover an ample spectrum of frequencies, which affect the towers in different ways ranging from serviceability problems to fatigue and collapse [4].

To mitigate the dynamic effects can be installed several types of damping devices, one of which is a tuned mass damper (TMD). A TMD consisting of a mass, damping and a spring, is an effective and reliable structural vibration control device commonly attached to a vibrating primary system for suppressing undesirable vibrations induced by machinery as well as by wind and earthquake loads. The natural frequency of the TMD is tuned in resonance with the fundamental mode of the primary structure, so that a large amount of the structural vibrating energy is transferred to the TMD and then dissipated by the damping as the primary structure is subjected to external disturbances. Consequently, the safety and comfort characteristics of the structure are greatly enhanced. The TMD system has been successfully installed in slender skyscrapers and slender towers to suppress the wind-induced structural dynamic responses [5]. In the following Figure 1 can be seen the scheme of a TMD

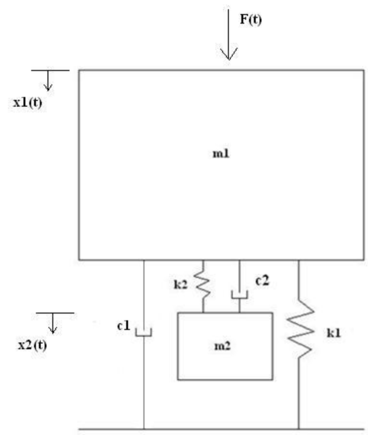

Figure 1: Theoretical scheme of a TMD [6]

2 NUMERICAL MODELING OF A TOWER AND ITS DYNAMIC WIND ACTION

This chapter begins by describing the structure and how it was modeled computationally. The program used for structural calculations associated with analysis and design was the "*Autodesk Robot Structural Analysis Professional 2012*".

The dynamic wind action was modeled to obtain the structural response of the tower. Mathematical modeling of turbulent flow is rather complex and the possibility of interaction between the flow and the tower may lead to changes in dynamic pressure and in the response of the tower along time. This chapter also addresses the simplifications used to consider this dynamic action [7].

2.1 Structural Description of the Lattice Wind Tower

The structure chosen was a tall lattice wind tower, whose design resulted from academic studies of the authors of this paper [2] [3] and is represented in Figure 2. The tower has 150 m of height and the turbine used is FL2500 of 2,5 MW with rotor diameter of 100m.

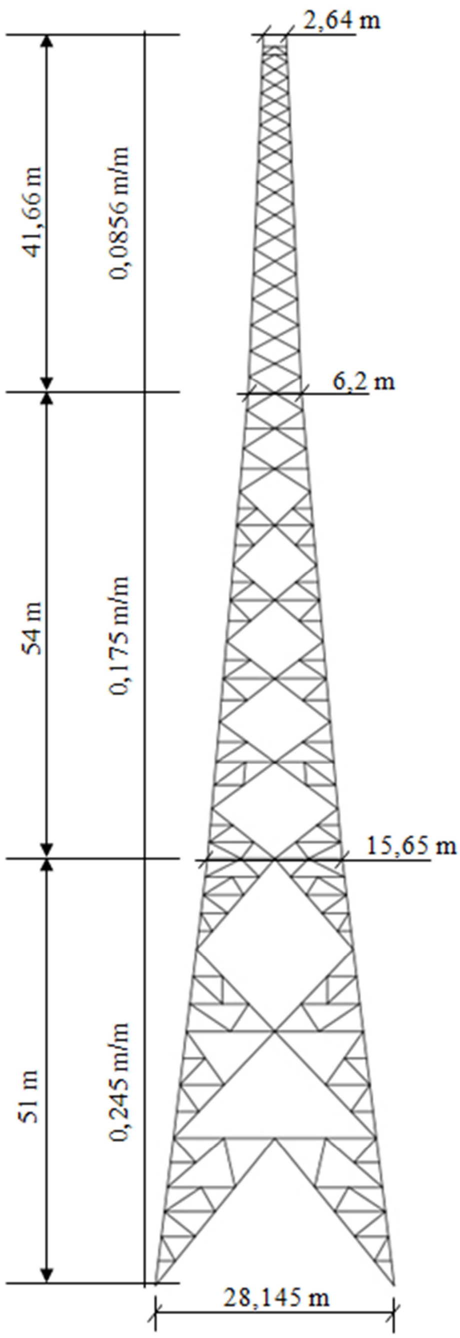

Figure 2: Tower Geometry [3]

The elements of the structure were disposed based on rules of triangles so as to shorten the lengths of buckling of the structural system. The construction of the geometry of the beams and diagonals was based on the Eurocode 3 (EN 1993-3-1) [8] and on dispositions of existing towers [3]. The sections of the bars used in the tower are angles and association of angles (Figure 3). The steel used in the design is S235 and S355.

Figure 3: Association of angles

The design follows the rules disposed in the Eurocode 1 (EC 1) EN 1991-1-4 [9] and also Eurocode 3 (EC 3) given in EN 1993-3-1 [8] and EN 1993-1-1 [10].

2.2 Modeling a Latticed Wind Tower

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

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

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

In this work it was considered that during the dynamic action the rotor is stopped in its most unfavorable position.

The mass of the structure is lumped at the structure nodal points; the masses are assumed to have only one degree of freedom (translation in X-direction).

One final aspect of the analytical model development that requires discussion is damping. Damping is the property that allows decreasing the amplitude of free vibration and is always present in structural systems. Several damping mechanisms may act simultaneously allowing dissipation of vibration energy. Due to the difficulty in identifying the damping mechanisms that may be present at a particular time, damping is cumbersome to evaluate [4]. The damping of similar towers has been calculated by other authors and researchers [3] by means of the logarithmic decrement, and the value of the viscous damping ratio used is $\xi = 5\%\xi = 5$ %.

Autodesk Robot Structural Analysis includes damping in its dynamic time history analysis application, through the classic formulation of Rayleigh damping with user-defined quantities. A target damping ratio equal to $\xi = 5$ % was used for every mode of vibration.

In the modal analysis only the response from the first three modes were considered. The vibration modes are shown in Figure 4 and the values of the natural frequencies and of the percentage of modal mass (from the total) for each vibration mode are presented in Table 1.

Mode	f (Hz)	M_x=450161,80 kg
1	0,47	53,57 %
2	2,25	25,44 %
3	3,95	14,62 %

Table 1: Natural frequencies and percentage of modal mass, from the modal analysis

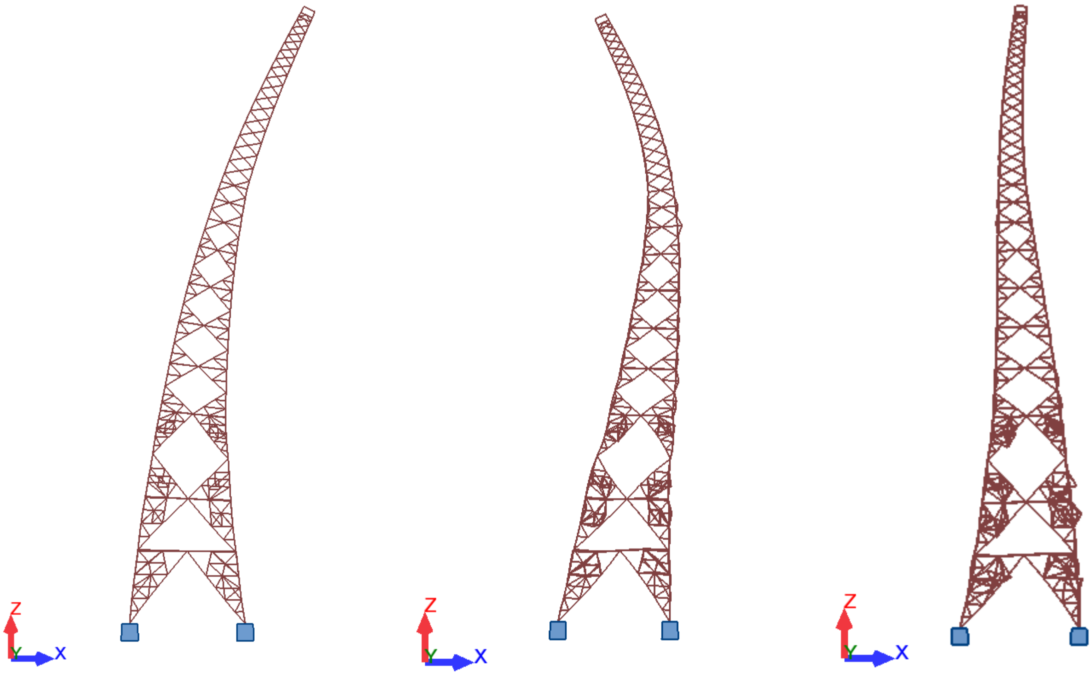

Figure 4: First, second and third modes of vibration (from left to right) from the numerical model

2.3 Modeling Wind Dynamic Action

Research has been done in the past in order to develop a spectrum that would accurately predict the dynamic characteristic of wind. Although it is recognized the great complexity in modeling turbulent flow around lattice towers, even with scaled physical models in a wind tunnel, some simplifications will be considered herein with regards to the quantification of dynamic pressures and generalized forces due to wind action along the time. For that, the fluid structure interaction (FSI) is considered negligible and also the correlations of the velocity fluctuations along the height of tower 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 tower (diaphragms of the tower).

The methodology used to generate synthetic time series is usually referred as the Method of Shinozuka, which bases the generation of time series in calculating 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) [7].

Such generation of synthetic series of wind occurs in the range of wavelengths corresponding to fluctuations of wind velocity with an approximate Gaussian distribution of the atmospheric wind flow [11]. The purpose of such 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 of the random process [7].

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 a 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 [11] [12] by the equations (1)-(2). This method has been evaluated by Ianuzzi and Spinelli [13], and it has been found to generate accurate fluctuations of longitudinal wind velocity histories *u(t)* when compared to measured wind records.

$$u(t) = \sqrt{2}\sum_{k=1}^{N} A_k.\cos(2\pi n_k t + \phi_k) \tag{1}$$

with

$$A_k = \sqrt{S_v(z, n_k)\Delta n} \tag{2}$$

and

$$\Delta n = \frac{n_{max} - n_{min}}{N} \tag{3}$$

In the previous expression *N* is the number of frequencies of the discretization of the spectrum, and *n* is 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 wind spectral density function S_L given in Eurocode 1 (EN 1991-1-4) [9] is used herein in the general dimensionless form of equation:

$$S_L(z, n_k) = \frac{n_k S_v(z, n_k)}{\sigma_v^2} = \frac{6{,}8 f_L(z, n_k)}{(1 + 10{,}2 f_L(z, n_k))^{5/3}} \tag{4}$$

with

$$f_L(z, n_k) = \frac{n_k L(z)}{v_m(z)} \ \ and\ \ L(z) = L_t\left(\frac{z}{z_t}\right)^{\alpha} \tag{5}$$

where:

L(z) is turbulent length scale represents the average gust size for natural winds. With a reference height of $z_t = 200\ m$ and a reference length scale of $L_t = 300\ m$, the power $\alpha = 0{,}67 + 0{,}05\ ln(z_0)$ where z_0 is the roughness length.

$S_v(z,n_k)$ is the one-sided variance spectrum.

$f_L(z,n_k)$ is a non-dimensional frequency.

σ_v is standard deviation of the turbulence.

For the generation of the synthetic series to be considered an ergodic process, according to [11] 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 wind spectrum into different number of frequency ranges. Herein four processes, corresponding to four frequency intervals chosen, were studied for the division of the wind spectral density function in 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*=90 m, and considering that the tower was located on a terrain of class I according to EC 1. The base velocity assumed for a return period of 50 years was 30 m/s (Zone B in EC1) and under these conditions, according to EC1, standard deviation of the turbulent component of wind velocity was $\sigma_v = 5{,}7$.

The frequencies n_{min} and n_{max} must be determined accordingly with the f_L limits of power spectral density function of the EN 1991-1-4 [9], that finally allow the wind turbulence effect to be clearly characterized. Figure 5 represents the spectral density functions S_L (given in a dimensionless form) and S_v, for the previously mentioned conditions.

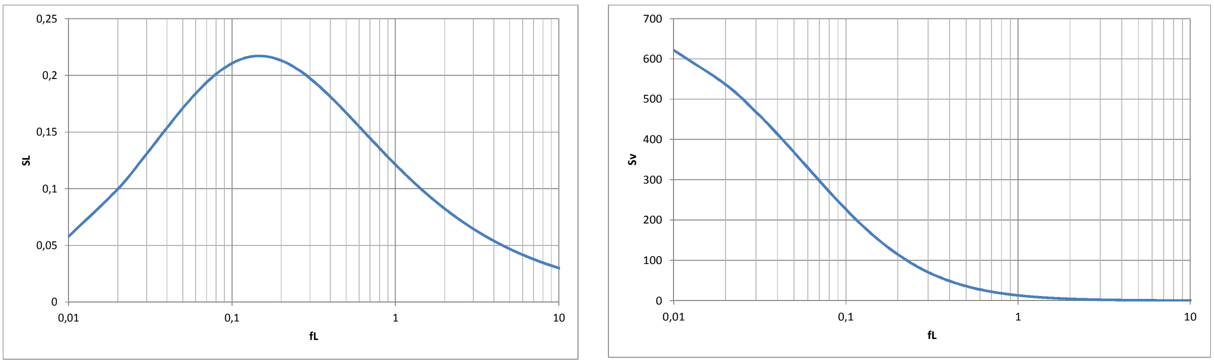

Figure 5: Wind spectral density functions

Figure 6 represents the time series of the fluctuations of wind velocity, evaluated with the previous data, for the four successively higher discretizations.

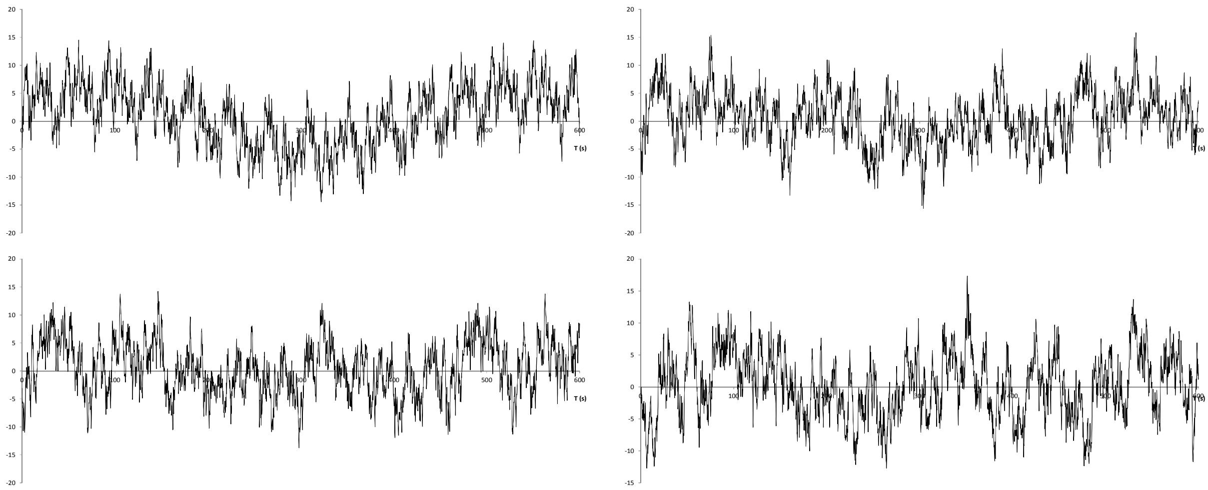

Figure 6: Time series of fluctuations of wind velocity for wind spectrum with 100-500-1000-5000 intervals

The spectral density function has high values for low frequencies but reduces rapidly with increasing frequency (Figure 5). Thus, from the analysis of the 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 the low frequency components (where the spectrum has more energy) resulting in a value numerically higher.

The series chosen for resolving the problem was the one of N=1000, for which it is no longer noticeable the influence of low frequency components. The results in [6] [7] also show that the value of N=1000 is a good compromise.

For the instantaneous wind velocity *U(t)* at any height given as the sum of a constant mean component $\bar{U}$ with a dynamic fluctuation component *u(t)* (Barros [14]), the instantaneous wind force *F(t)* on any surface *A* is given by equations (6) and (7).

$$F(t) = \frac{1}{2}.\rho.c_f.A.\left[(\overline{U} + u(t)\right]^2 \tag{6}$$

$$F(t) = \frac{1}{2}.\rho.c_f.A.\overline{U}^2 + \rho.c_f.A.\overline{U}^2.u(t) + \frac{1}{2}.\rho.c_f.A.u(t)^2 \tag{7}$$

The fluctuations of wind velocity along time also have a spatial variability, which for a first approximation is herein neglected. For the case tall slender tower under study, whereas the response is majorly due to the contribution of the first mode of vibration (which is also a condition imposed by EC 1 for the calculation of the structural factor), 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:

$$S_X(n) = \frac{4\overline{X}}{\overline{U}^2}.[H(n)]^2.\chi^2(n).S_u(n) \tag{8}$$

where $[H(n)]^2$ represents the mechanical admittance and $\chi^2(n)$ represents an aerodynamic admittance function [15] given approximately by equation (9).

$$\chi(n) = \frac{1}{1+\left(\frac{2.n.\sqrt{A}}{\overline{U}}\right)^{4/3}} \tag{9}$$

According to Holmes [16], in a frequency domain analysis for very tall tower structures, it is this latter function that takes into account the non-simultaneous occurrence of 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 $\chi^2(n)$ is introduced*".

According to EC 1 for tall tower 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 of the height of the structure. Given this indication, for generating different sets of time series the height chosen was 90 meters that is about 60% of the height of lattice wind tower.

The applied wind generated forces were obtained through equation (6) taking into account the acting dynamic pressures and the influence area for each floor, considering the mean wind velocity depending on the height (given by the equation (10) below, taken from EC 1) and the fluctuation velocities given by the random series generated [6] [7]. Supporting the procedure used in Ferreira [6] and Ferreira *et al.* [7], appropriate simplifications were performed so that the wind power spectrum was multiplied by the aerodynamic admittance; it was with this new spectrum that the turbulent velocities were calculated.

$$v_m(z) = c_r(z).c_0(z).v_b \tag{10}$$

As an example, Figure 7 presents one series for the fluctuations of wind velocity generated in these conditions at a height of 90 m and using wind power spectrum of EC 1. Figure 8 shows the same series, that is adopting the same phase angles for the harmonics, but generated from the wind power spectrum multiplied by the mentioned aerodynamic admittance function.

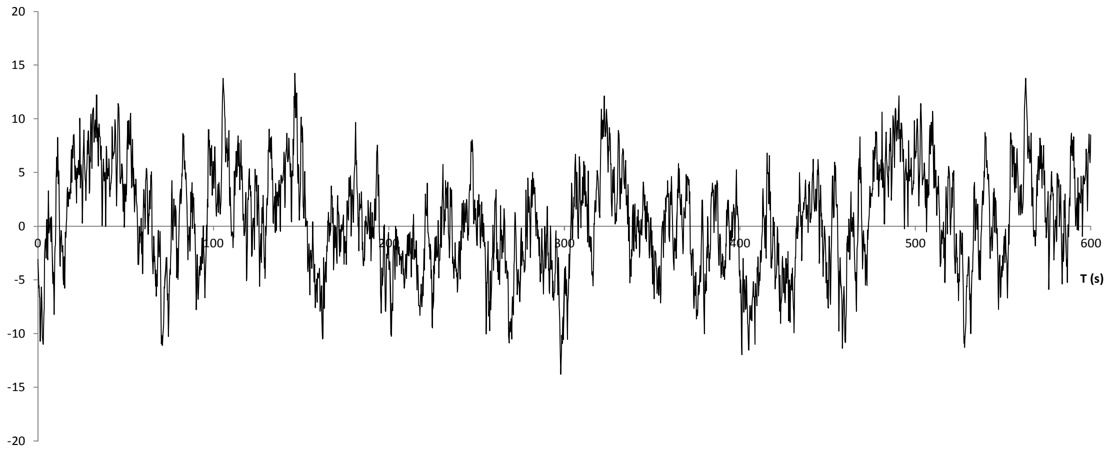

Figure 7: Fluctuation velocity time series (for height of 90 m, base velocity 30 m/s, terrain category I) using EC1 wind power spectrum (wind power spectrum not multiplied by the aerodynamic admittance function)

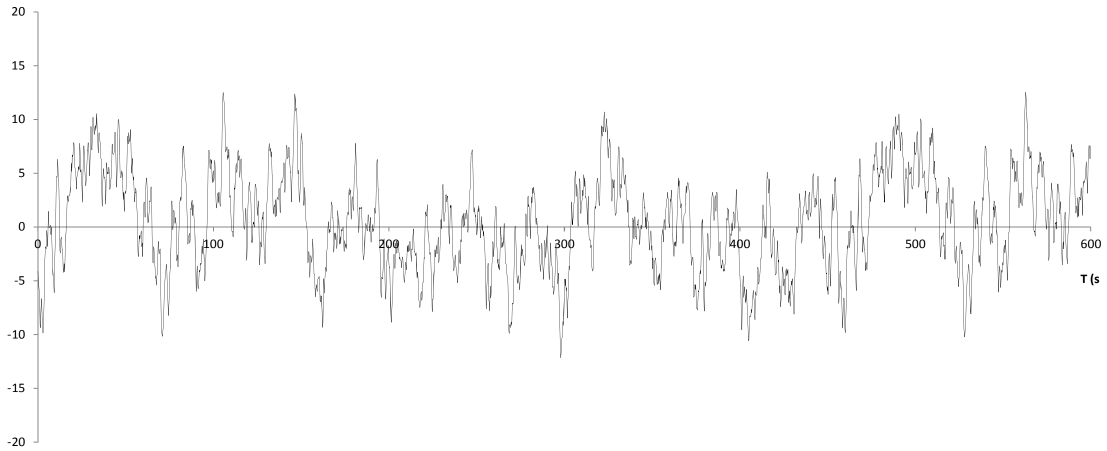

Figure 8: Fluctuation velocity time series (for height of 90 m, base velocity 30 m/s, terrain category I) using EC1 wind power spectrum multiplied by the aerodynamic admittance function

3 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 [7].

The design of a TMD for application to structures without damping is universally based on two parameters – mass ratio μ and frequency ratio q – as detailed in Kelly [17]. The optimum frequency ratio q_{opt} (corresponding to locating 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 by the set of equations (11).

$$q_{opt} = \frac{1}{1+\mu} \quad , \quad \frac{X_1}{X_{1,static}} = \sqrt{\frac{2+\mu}{\mu}} \quad , \text{ and } \quad \xi_{2,opt} = \sqrt{\frac{3\mu}{8(1+\mu)^3}} \tag{11}$$

For the design of a TMD tuned for the application to structures with damping, it is still possible to use these equations provided that the damping of the principal primary system is less or equal to 1%. For higher damping of the primary system, 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 (Figure 9) associated with the numerical solution of the expression giving the maximum amplitude of the controlled principal system (as used successfully in [6] [18] [19]).

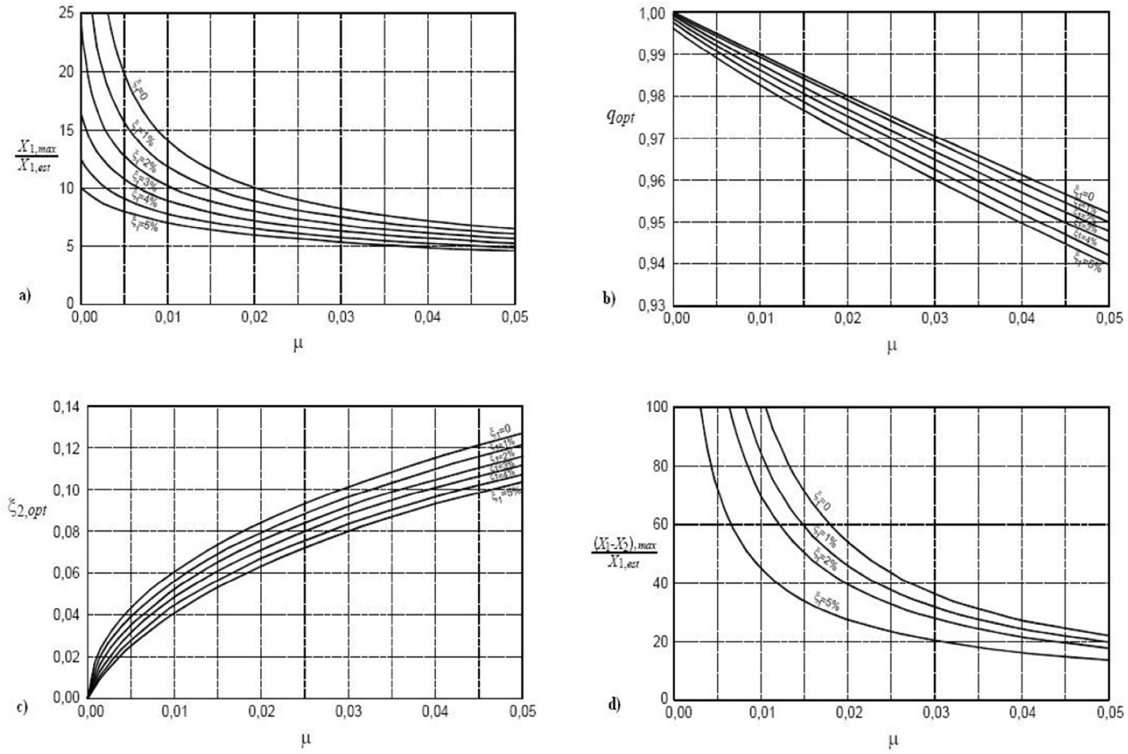

Figure 9: Design graphs of TMD ($\boldsymbol{\xi_1} \neq \mathbf{0}$) [6] [18] [19]

Kwok and Samali [20] also studied the behavior of a TMD in tall buildings subjected to the action of wind and, according to them, the considerations presented about the effectiveness of a TMD in response of a system of one degree of freedom can also be extended to other solid structures – such as in the case of tall buildings – leading to a modal analysis. Kwok and Samali [20] indicated that occurred large decreases in response for the modes controlled by the TMD's installed, while the higher order modes were not affected. For such higher modes to be also 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 to control, it is necessary to determine the corresponding values of stiffness, of mass and of modal damping [7].

Since the fundamental frequency of the tall wind tower under study is very low (0.47 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. For control of vibrations purposes it is assumed herein 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 the 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 as 126,25 ton. For the case study wind tower structure with the deployment of a TMD, only one mass ratio is herein considered $\mu=0.01$, for which with the design charts (Figure 9) it was possible to determine the optimal parameters to be adopted for each TMD situation. In Table 2 the values adopted are systematized.

TMD mass ratio μ	q_{opt}	$\xi_{TMD,opt}$	m_{TMD} (ton)	ω_{TMD} (rad/s)	k_{TMD} (kN/m)	Size (cm) of square section steel bar, L=2 m
0.01	0,987	0,046	1,7626	2,915	14,974	3,89

Table 2: Optimal parameters of a TMD for the tall wind tower

Since the structural software used does not have an intrinsic function that allows the direct introduction of dampers, herein for the simulation of a TMD 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 [7].

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 2 also indicates the dimensions required for such bar, for the mass ratio considered in the design of the TMD. Table 3 shows the first four natural frequencies of the vibration modes of the case-study wind tower structure incorporating the TMD solution.

Mode	1	2	3	4
f (Hz)	0,45	0,49	2,25	3,95

Table 3: Natural Frequencies of the first four modes of wind tower with TMD

4 ANALYSIS OF RESULTS FOR THE TOWER WITHOUT AND WITH TMD

Based on the methodology adopted for consideration of the dynamic wind action (using a set of 4 time series and for frequencies in the wind spectral density function evaluated with 1000 frequency intervals), the results in terms of top displacements and accelerations were evaluated and compared for the computational structural model, without and with an installed TMD vibrating bar (with an hypothetical vibrating mass with the appropriate stiffness and damping properties). Using the mentioned structural software with modal superposition, a damping ratio of 5 % and an integration time step of $\Delta t = 0.2$ seconds, the four series of wind dynamic loads were applied and their average results obtained in terms of top displacements and accelerations. As an example, Figure 10 and Figure 11 show the time variations of top acceleration and displacement of the wind tower for the wind loads evaluated using equation (6), with velocity fluctuations corresponding to wind series 1. Table 4 presents a summary of maximum values of tower top displacements and accelerations, for each of the time series.

Series	1	2	3	4	Average
Maximum displacement (cm)	70,39	66,07	68,99	65,77	67,80
Maximum acceleration (cm/s^2)	184,15	179,49	179,89	222,36	191,47

Table 4: Maximum displacements and accelerations on node 3, for each of the wind time series (without TMD)

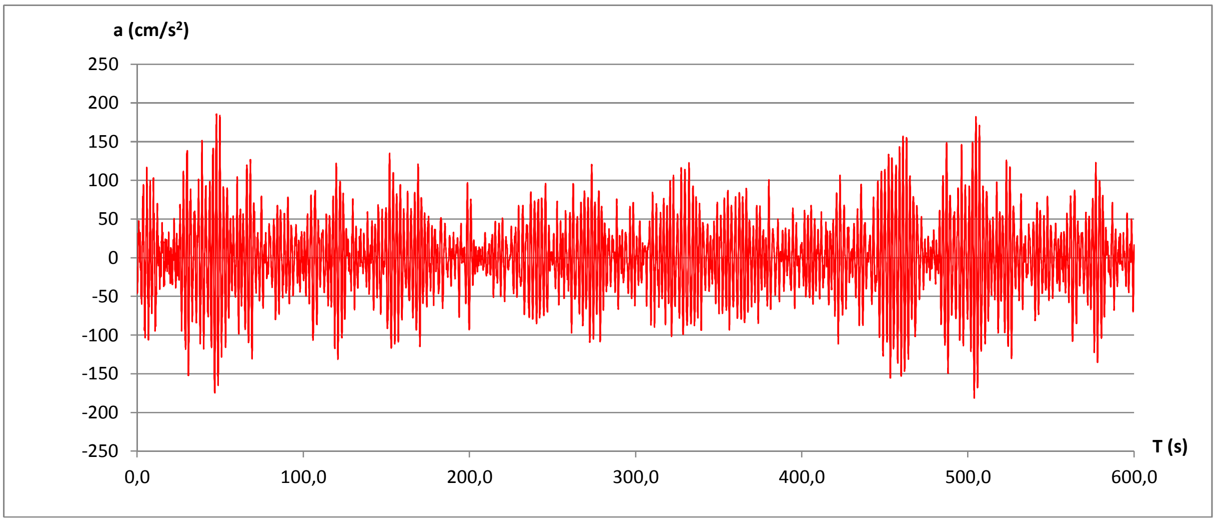

Figure 10: Accelerations on top of tower, for wind loads corresponding to wind series 1

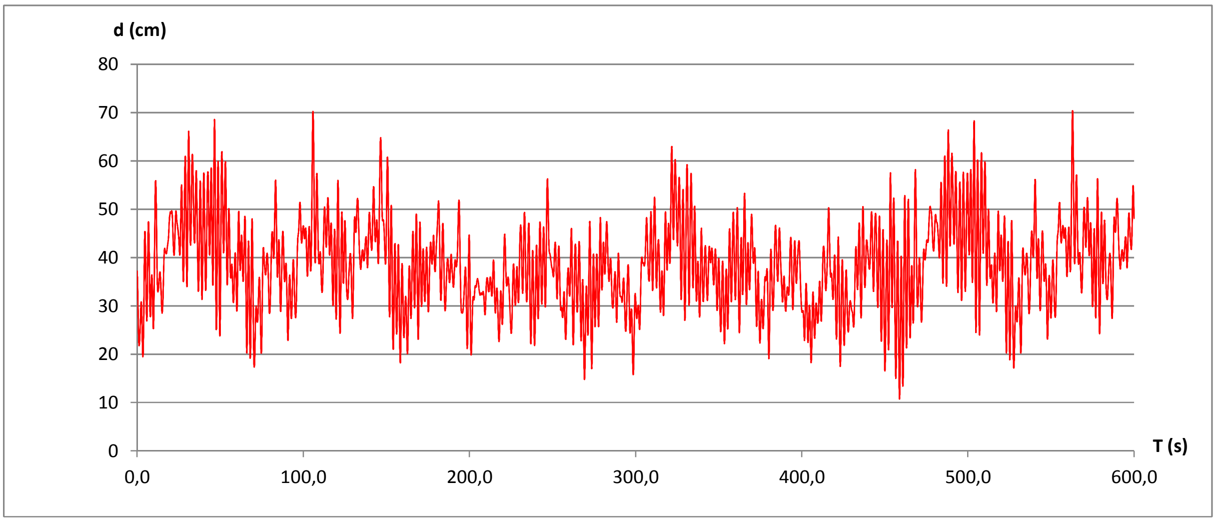

Figure 11: Displacements on top of tower, for wind loads corresponding to wind series 1

As regards to the use of a TMD on the top of the wind tower, as used in [6] [7] in an earlier comparison associated with a tall structure subjected to harmonic excitation in resonance with the fundamental frequency, the Figures 12 and 13 show such comparison of top displacements and accelerations of the given tall wind tower along the time, without and with TMD with mass ratio of 1 %. As can be seen in Figure 12 and 13, if the structure is acted upon by a harmonic action in resonance with fundamental frequency, the implementation of the TMD can considerably attenuate the response of the wind tower structure.

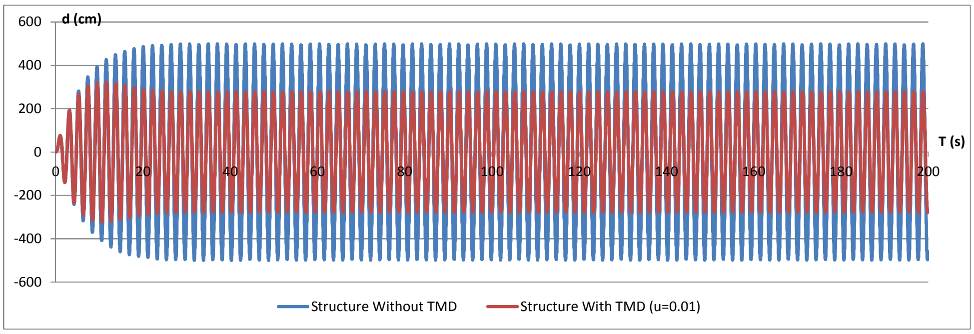

Figure 12: Top displacements, under a harmonic fundamental resonant excitation, without and with TMD

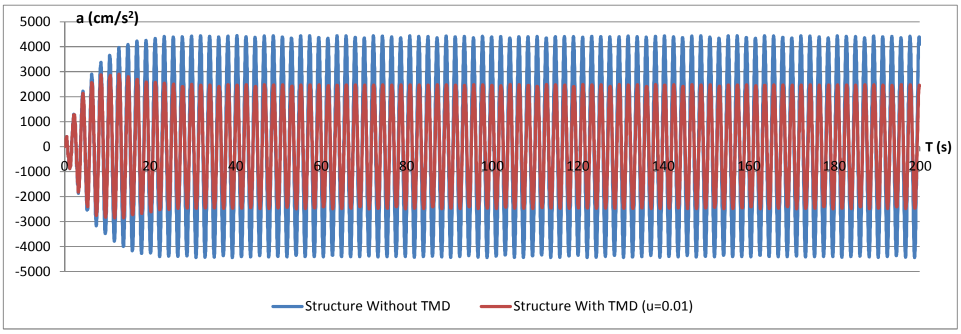

Figure 13: Top accelerations, under a harmonic fundamental resonant excitation, without and with TMD

Figure 14 and Figure 15 show the time variations of top acceleration and displacement of the wind tower, equipped with the TMD modeled before with a 1% mass ratio, for the wind loads evaluated using equation (7), with velocity fluctuations corresponding to wind series 1. Table 5 presents a summary of maximum values of top displacements and accelerations of the tower, for each of the time series. It also presents the average of such maximum values.

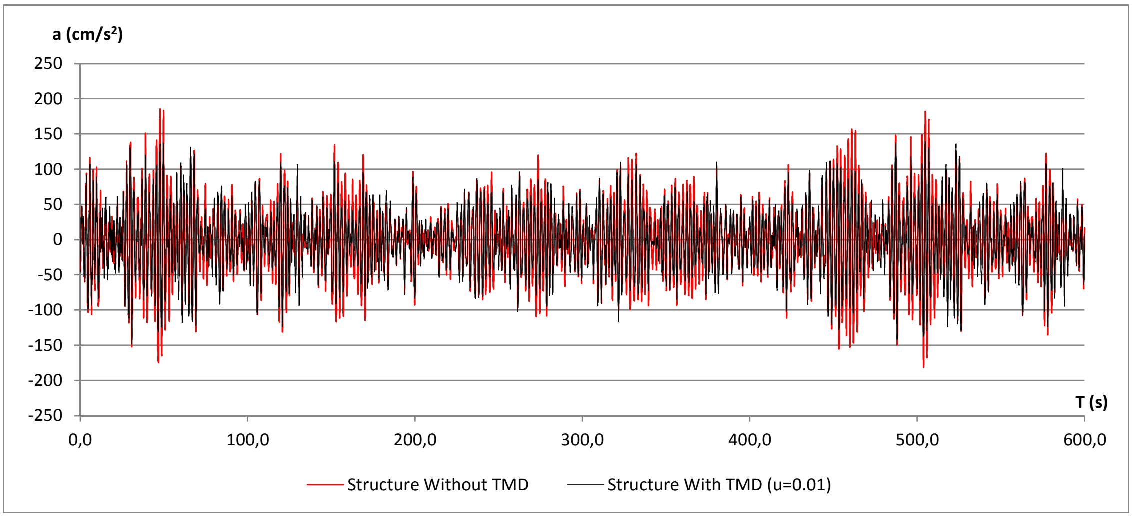

Figure 14: Acceleration on the top of tower, equipped with the TMD modeled with mass ratio of 1 % for the wind loads corresponding to wind series 1

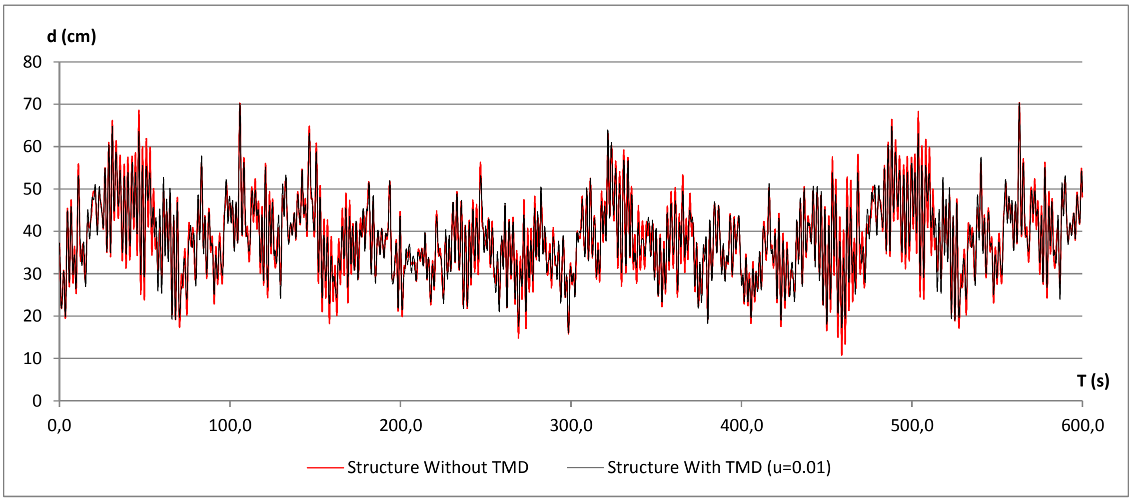

Figure 15: Displacement on the top of tower, equipped with the TMD modeled with mass ratio of 1 % for the wind loads corresponding to wind series 1

Series	1	2	3	4	Average
Maximum displacement (cm)	70,30	63,42	70,62	63,96	67,07
Maximum acceleration (cm/s^2)	139,35	151,72	156,57	180,69	157,08

Table 5: Maximum displacements and accelerations on node 3, for each of the wind time series (with TMD)

The efficiency on the use of the modeled TMD in the tower structure can be interpreted by the results of Table 6, here associated with a mass ratio of 1% : reduction of top maximum displacements and accelerations on the order of 1% and 18%, respectively.

	Structure without TMD	Structure with TMD (u=0.01)	Reduction (%) in relation to the structure without TMD
Maximum displacement (cm)	67,80	67,07	1 %
Maximum acceleration (cm/s^2)	191,47	157,08	18 %

Table 6: Efficiency of using the modeled top TMD for mass ratio of 1%

5 CONCLUSIONS

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 frequency 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 stiffening floor, following the same law of variation. This law is obtained, for each generated time series, from the Eurocode 1 wind power spectrum multiplied by the aerodynamic admittance function.

As regards to the implementation of the TMD in the tall wind tower structure under study, it was concluded that it proved to be very effective in terms of both top displacements and top accelerations, when the tower is subject to a harmonic action in resonance with fundamental frequency of vibration of the tower.

The application of this TMD passive device on top of the tower for vibration control of the designed wind tower, subjected to natural wind actions based on the generated wind series, is not as effective as for controlling harmonic resonant phenomena. With the implementation of the TMD it was concluded that this device is considerably more effective in controlling top accelerations (rather than top displacements), when the structure is subjected to the artificially generated natural wind. For the TMD modeled with the parameters calculated, were observed maximum accelerations reductions of the order of 18%, while the achieved reduction of maximum displacements was only of the order of 1%.

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