Assessment of the dynamic structural behaviour of steel wind towers when subjected to wind loadings

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Abstract. This research work proposes an analysis methodology to assess the structural behaviour of steel wind towers. This way, the structural model associated to a wind tower supported by an octagonal reinforced concrete foundation designed to accommodate a 2 MW wind turbine was investigated. The tower finite element model was developed based on the use of the Finite Element Method (FEM), utilizing the ANSYS computational program, and considering the wind loadings on the rotor and tower, and the effect of the geometric nonlinearities and soil-structure interaction, aiming to obtain a realistic representation of the structure dynamic behaviour. The stochastic nature of the wind loadings was considered, and a statistical analysis of the structure's dynamic response was conducted. After that, an extensive parametric study was performed, considering several basic wind velocities, to assess the steel tower dynamic structural behaviour, based on the horizontal displacements, von Mises stresses, and the fatigue service life. The results revealed that within the operational limit of the turbine, the investigated tower attends the recommended limits specified in current wind tower design standards. However, for higher basic wind velocities, the wind tower structural design no longer meets these requirements.

Keywords: dynamic structural analysis, finite element modelling, geometrical nonlinearities, steel wind towers.

1 Introduction

The growing need for electricity, coupled with concerns about climate change and environmental sustainability, has placed a greater emphasis on developing and expanding renewable energy sources [1]. Governments, businesses, and investors worldwide are recognizing the advantages of renewable energy, such as its ability to reduce greenhouse gas emissions, enhance energy security, and stimulate economic growth. Among the various renewable energy sources, wind energy assumes a crucial role in the global shift towards a decarbonized energy matrix. It has become a central focus for substantial investments on a global scale, resulting in a remarkable surge in its productive capacity.

The construction of wind towers represents a significant part of the total cost of new wind farms. In this context, an analysis methodology is proposed to assess the structural behavior of wind steel towers for a 2 MW wind turbine, through a finite element model developed based on ANSYS software, considering the wind loadings on the rotor and tower, and the effect of the geometric nonlinearities and soil-structure interaction. Furthermore, an extensive parametric analysis is conducted to assess the impact of basic wind velocities on the structural dynamic response of the investigated wind tower, focusing on the mean maximum values of the translational horizontal displacements, von Mises stresses, and the fatigue service life.

2 Steel wind tower finite element modelling

The investigated steel tower is designed to support a 2 MW wind turbine, whose dimensions are taken from Rebelo *et al*., [2]. It consists of a 76.15 m high conical tower modelled as a freestanding steel tube with varying diameter and thickness by height. The tower is divided into three parts to allow for transportation and assembly on site. The assumed thicknesses at the bottom, middle, and top of the tower vary from 30 mm to 21 mm, 21 mm to 16 mm, and 16 mm to 12 mm, respectively. In the lower part of the tower, there are two elliptical openings, one for ventilation and the other for internal access and maintenance. The doors for maintenance and ventilation are located centrally at 2.33 m and 7.96 m above the lowest part of the steel tower. The entire structure is supported by an octagonal concrete foundation inscribed in a 17 m diameter circle. Figure 1 presents the finite element model of the investigated steel wind tower, developed with the finite element software ANSYS. For the steel tower and nacelle, the four-node-thick SHELL 181 [3] element is chosen. The reinforced concrete foundation is modelled with the four-node tetrahedral solid element SOLID 72 [3]. The soil-structure interaction is modelled as an elastic support to represent soil stiffness through a linear spring element COMBIN14 [3]. The steel tower is constructed of steel S355, with yield strength of 355 MPa and a Young's modulus of 205 GPa. The foundation is modelled in reinforced concrete with yield strength of 16 MPa and a Young's modulus of 30 GPa. To consider the self-weight of the turbine blades and the wind's energy converter equipment at the top of the tower, shell elements with different mass densities are incorporated into the model. In the front part of the nacelle, shell elements with a mass density of 3199 kg/m³ utilized. In the rear part, shell elements with a mass density of 2324 kg/m³ are considered.

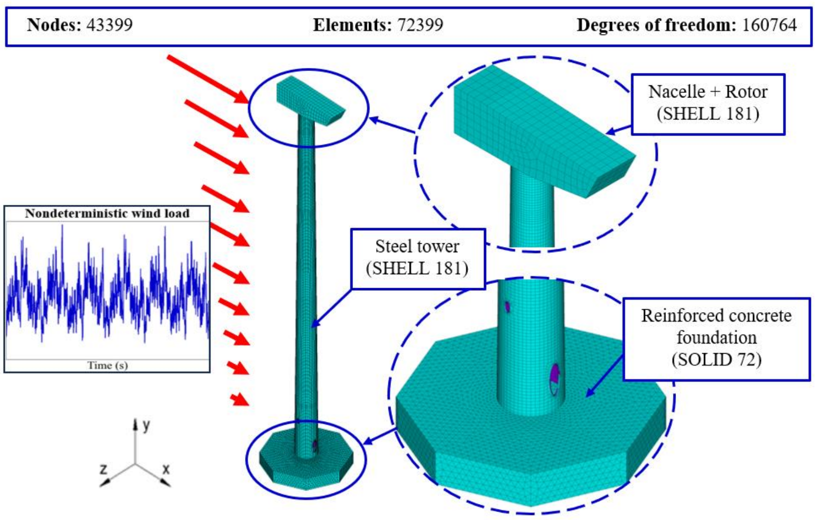
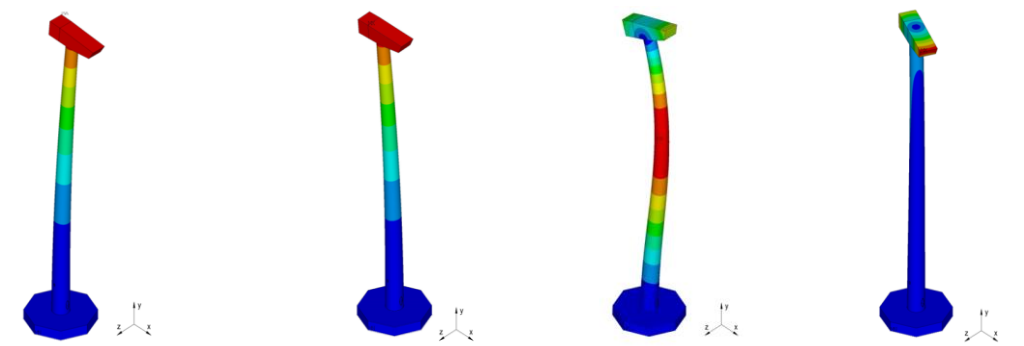


Figure 1. Finite element model of the investigated structure



$f_{01} = 0.338\,Hz$ $f_{02} = 0.34\,Hz$ $f_{03} = 2.461\,Hz$ $f_{04} = 2.498\,Hz$

Figure 2. First four vibration modes of the investigated wind tower

The natural frequencies (eigenvalues) and vibration modes (eigenvectors) of the steel tower were determined considering a free vibration analysis through ANSYS software. Figure 2 shows the results for the steel tower's first four vibration modes and their respective natural frequencies. The first and third vibration modes correspond to bending around the global X-axis, while the second mode represents bending around the Z-axis. The fourth vibration mode corresponds to torsion around the Y-axis. These vibration modes are consistent with the numerical results reported Castilho et al., [1]. Sørensen and Sørensen [4] claim that the fundamental frequency f_{01} of the investigated wind turbine should be in the range of 0.281 Hz to 0.341 Hz. This way, the results obtained from the free vibration analysis demonstrate a satisfactory calibration of the finite element model developed in this work.

3 Wind loadings modelling

The nondeterministic wind velocity along the tower V(t) (m/s) represented as a time-varying function composed of a static $\overline{V}$ (m/s) and a fluctuating part v(t) (m/s), as shown in eq. (1). The static component [5] is determined as a constant and it depends on the height and represents the mean wind velocity in the horizontal direction.

$$V(t) = \overline{V} + v(t) \tag{1}$$

The mathematical formulation for the constant mean wind velocity is presented in eq. (2), where $S_1 = 1$ is a topographic factor related to flat ground; $S_3 = 1.1$ is a statistical factor related to the risk factor and required life in service; and $v_0 = 35$ m/s is the basic wind velocity related to the Rio de Janeiro/RJ, Brazil. Equation (3) describes the ground roughness parameter S_2, where $b = 1$ is a meteorological factor, $p = 1.15$ is a roughness factor, y is the considered height and $f_g = 0.69$ is a gust parameter related to 600 s of wind action.

$$\overline{V} = v_0 S_1 S_2 S_3 \tag{2}$$

$$S_2 = b f_g \left(\frac{y}{10}\right)^p \tag{3}$$

The time-dependent component of the wind velocity in eq. (1) is decomposed into a finite number of harmonic functions, whose amplitudes are obtained through the Kaimal power spectrum density and random phase angles. The power spectrum density (PSD) $S^V(f, y)$ is presented in eqs. (4) and (5), where f (Hz) is the frequency, $x(f, y)$ is the dimensionless frequency, and V_y (m/s) is the wind velocity at height y (m). u_* is the friction velocity, which is given by a logarithmic law for describing the distribution of the longitudinal velocity in the wall-normal direction of a turbulent flow, near a boundary with no-slip condition, where $k = 0.4$ is the von Kaman's coefficient and y_0 (m) is the roughness length.

$$\frac{fS^V}{u_*^2} = \frac{200x}{(1+50x)^{5/3}} \tag{4}$$

$$x(f, y) = \frac{fy}{V_y} \tag{5}$$

$$u_* = \frac{kV_y}{\ln\left(\frac{y}{y_0}\right)} \tag{6}$$

The time-varying component of the wind velocity is formulated as a weakly stationary second-order ergodic process with a mean value of zero and a superposition of harmonics, as depicted in eq. (7). N is the number of harmonics considered in the power spectrum, θ_i is a random phase angle uniformly distributed in the interval $[0, 2\pi]$, f_i (Hz) is the i-th frequency, and Δf (Hz) is the frequency increment.

$$v(t) = \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f} \cos(2\pi f_i t + \theta_i) \tag{7}$$

The wind aerodynamic dynamic pressure $Q(t)$ over the steel tower is obtained based on the classical Davenport method, as presented in eq. (8). This way, the wind load acting on a certain height of the structure F_W (N) is described in eq. (9), where A_i (m²) is the effective area, C_{Di} is the drag coefficient of the i-th area.

$$Q(t) = 0.613[\overline{V} + v(t)]^2 \tag{8}$$

$$F_W(t) = A_i C_{Di} Q(t) \tag{9}$$

The vortex shedding loads considers a formulation through an harmonic function, as presented in eq. (10), where its amplitude is given by the air density $\rho_{air} = 1.225$ kg/m³ and the wind critical velocity v_{crit} (m/s) that is given as a function of the structure first bending mode natural frequency ω_{01} (rad/s), the tower's average cross-section diameter $d = 3.63$ m and the Strouhal number St assumed as 0.18 for weakly variable conical section, as presented in eq. (11).

$$F_v(t) = \frac{1}{2}\rho_{air}v_{crit}^2 \sin(\omega_{01}t) \tag{10}$$

$$v_{crit} = \frac{b\omega_{01}}{St} \tag{11}$$

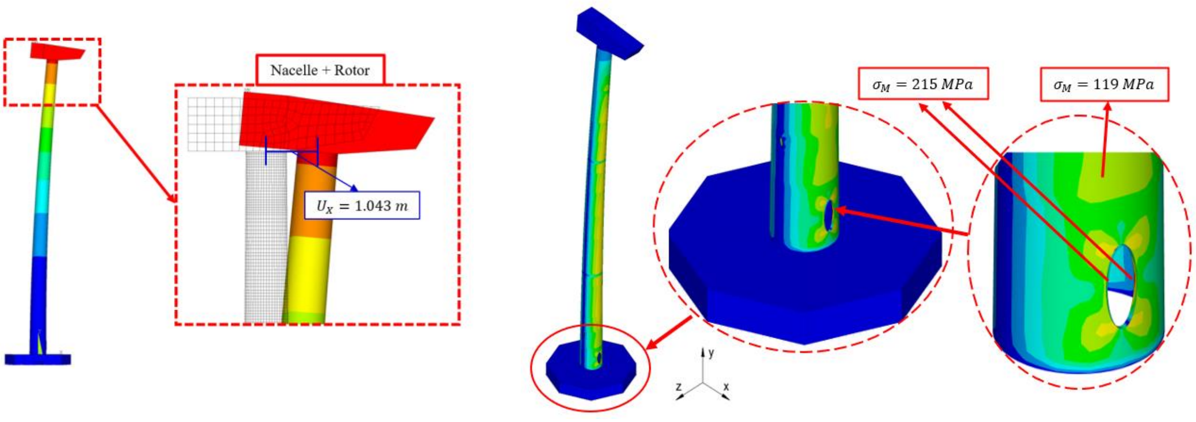
The rotor wind loads can be described as forces and moments that arises from the wind effect over the turbine blades and the rotating machines used in the wind's energy conversion process. Table 1 presents the wind loads on rotor obtained through linear interpolation of the load values provided in Umut *et al.*, [6], considering the global coordinate system presented in Fig. 1b, for a MM92 Repower Systems wind turbine [7] on the survival and operational conditions.

Table 1. Rotor wind loads for a 2 MW wind turbine

Loads	Operational	Survival
$F_X\ (kN)$	181.7	510.3
$F_Y\ (kN)$	554.6	109.8
$F_Z\ (kN)$	0.1	0
$M_X(kNm)$	367.2	220
$M_Y(kNm)$	14.1	14.3
$M_Z(kNm)$	219.8	184.5

4 Results discussion: static and dynamic analyses

Initially, a nonlinear static structural analysis is performed based on the simplified continuum model from NBR 6123 [5], considering geometrical nonlinearities. Figure 3a represents the results for horizontal displacement and von Mises stress distribution on the investigated structure. The maximum horizontal displacement of 1.043 m shows that the structure meets the limit of H/50 = 1.52 m of EUROCODE 3 [8]. If the geometrical nonlinearities are not considered, a maximum displacement of 0.99 m was determined, indicating that the inclusion of the geometric nonlinearities results in a 5.3% increase in the structural displacement response associated to the static loading. The lower opening for internal access presented a maximum stress of 215 MPa, while in its proximity, a maximum value of 119 MPa was observed. When considering a linear static analysis, a maximum stress of 199 MPa was observed in the access area, and a value of 116 MPa in its proximity. When comparing the maximum von Mises stress values between the linear and nonlinear analyses, it is evident that incorporating geometric nonlinearities resulted in an increase of 8% on the structure response.



a) Horizontal displacements b) von Mises stress distribution

Figure 3. Results for the static analysis considering geometrical nonlinearities

Considering the nonlinear dynamic analysis, a generic random load series is adopted. Figure 4 illustrates the horizontal translational displacement at the tower top, and the maximum von Mises stress, in time and frequency domain. It is worth to mention that the value of 1.049 m in the steady state response is slightly lower than the horizontal displacement obtained in the nonlinear static analysis and related to the displacement limit proposed by EUROCODE 3 [8]. For the von Mises stress, a stress peak of 206.5 MPa is observed, representing a value that is 4.1% lower than the maximum stress obtained in the static nonlinear analysis. Comparing this result with a transient analysis neglecting the geometrical nonlinearities, an increase of 4.8% on the maximum von Mises stress is found. In both cases, the tower attends to the limit proposed by IEC 61400-2 [9].

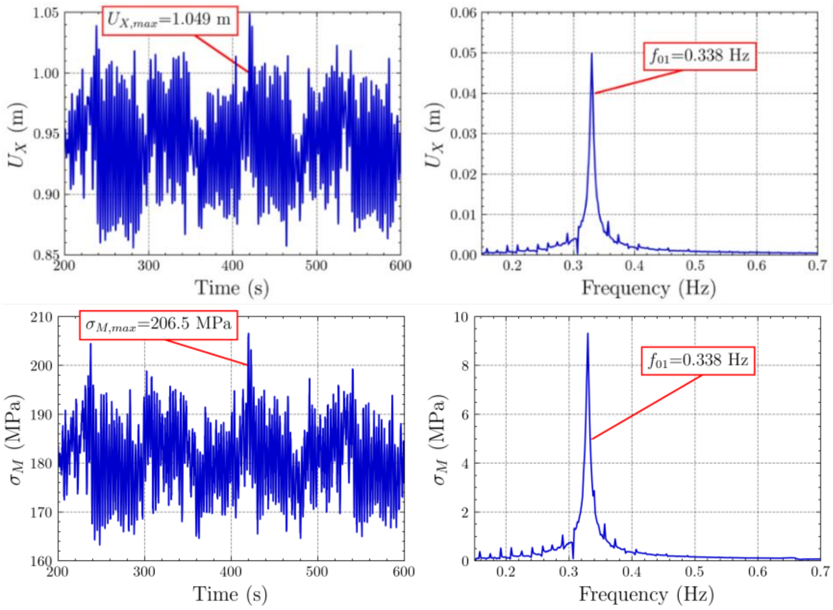


Figure 4. Time history and frequency response for the maximum horizontal displacement and von Mises stress

The steel wind tower nondeterministic dynamic analysis is performed considering now 30 random nondeterministic wind loading series, where the maximum response for horizontal translational displacement U_X and von Mises stress σ_M related to the steady state response are determined. The results the mean (μ), standard deviation (s), and maximum values with a confidence level of 95% (CL_{95}) are presented in Tab. 2. Considering the statistical analysis based on the translational horizontal displacements and von Mises stresses, when geometric nonlinearities are not considered, the displacement values exhibit mean value of 1.026 m and a maximum response of 1.043 m. In contrast, the von Mises stress presents mean and maximum values of 193.4 MPa and 203.5 MPa, respectively. Analysing the nondeterministic response of the structure considering geometric nonlinearities, it was observed that the mean displacement value is equals to 1.051 m with maximum value of 1.070 m. Additionally, the von Mises stress exhibits a mean value of 205.5 MPa with maximum value of 209.6 MPa. When comparing the results obtained from the linear and nonlinear analyses, it was verified that the inclusion of geometric nonlinearities leads to a 2.43% increase in the mean value of the horizontal displacement and a 2.58% increase in the maximum displacement response. In terms of the maximum von Mises stress, there is a variation of 6.26% in the mean stress and 3% in the maximum stress value.

Table 2. Nondeterministic dynamic structural response of the steel tower: linear and nonlinear analysis

Wind	Linear		Nonlinear	
series	U_X (m)	σ_M (MPa)	U_X (m)	σ_M (MPa)
1	1.012	188.4	1.047	206.6
2	1.024	197	1.032	202.3
3	1.023	181.9	1.044	204.8
4	1.026	197	1.05	204
5	1.019	192.2	1.061	207.3
6	1.011	184.7	1.034	207.9

7	1.05	205.4	1.08	209.8
8	1.042	188.9	1.07	208.2
9	1.019	198.4	1.057	208.3
10	1.021	193.7	1.056	204.7
11	1.004	181.2	1.043	202.4
12	1.023	199.2	1.044	202
13	1.041	199.2	1.06	204
14	1.021	197.2	1.039	202.8
15	1.025	196.1	1.032	200.8
16	1.022	195.1	1.047	204.6
17	1.029	182.3	1.045	203.9
18	1.023	192.4	1.042	204.1
19	1.041	199.9	1.059	206.9
20	1.032	198.9	1.055	206.5
21	1.028	188.8	1.054	208.9
22	1.012	192.6	1.053	204.8
23	1.032	189.9	1.057	206.4
24	1.031	198.5	1.059	207.2
25	1.027	195.1	1.062	209.2
26	1.031	197.9	1.067	211.3
27	1.032	195.4	1.049	208.7
28	1.032	197.8	1.06	206.8
29	1.034	195.2	1.058	205.3
30	1.035	199.7	1.052	204
μ	1,026	193,4	1,051	205,5
s	0,010	6,17	0,011	2,44
CL 95%	1,043	203,5	1,070	209,6

Aiming to assess the structure dynamic response, an extensive parametric analysis is conducted, considering several basic wind velocities ($v_0 = 10$ m/s to $v_0 = 70$ m/s) for the computation of the nondeterministic wind loadings. As observed in the previous results (see Tab. 2), the incorporation of geometric nonlinearities does not present a significant effect on the investigated steel wind tower dynamic response. Furthermore, it's well known that nonlinear dynamic analyses induce a considerable computational cost. In this context, the parametric analysis will be conducted without taking geometric nonlinearities into account.

The parametric analysis related to the mean maximum values of the translational horizontal displacements, von Mises stresses, and also the investigated steel tower service life [10], based on a confidence level of 95% $\overline{U}_{X,95}$ and $\overline{\sigma}_{M,95}$ are presented in Fig. 5. The horizontal displacements show that the steel tower complies with the limit state set by EUROCODE 3 [8] for wind velocity up to 58 m/s. When the von Mises stress values are considered, it is evident that the structure complies with the 239 MPa limit set by IEC 61400-2 [9] for wind velocities up to 41 m/s. Regarding the life service, for wind velocities lower than and just above the turbine operational limit equal to 24 m/s, the structure attends the minimum service life requirement of 20 years according to DNV-GL-ST-0262 [11]. On the other hand, the tower begins to exhibit a potential scenario of fatigue failure when the basic wind velocities are higher and exceed 38 m/s.

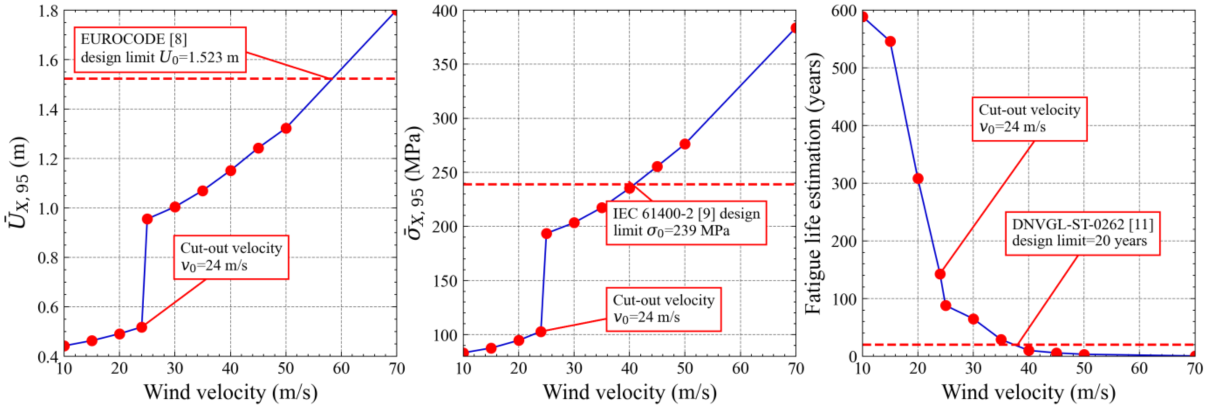


Figure 5. Results for the developed parametric analysis

5 Conclusions

This research work proposes an analysis methodology to assess the structural behaviour wind steel towers and investigates the effects of the geometrical nonlinearities over the dynamic response of a steel tower designed to support a 2 MW wind turbine. In view of the nature of the wind loading modelling, a statistical analysis of the results (displacements and stresses) related to the linear and nonlinear forced vibration analysis, performed based on the use of 30 series of nondeterministic wind loads. This way, the following conclusions can be drawn from the results presented in this work:

1. The static analyses revealed the importance of incorporating geometric nonlinearities on the static structural analyses, particularly when stress concentration effects are present, such as in the region of the opening for internal access.
2. Considering the nondeterministic dynamics investigation, a statistical analysis of these results reveals that the mean and maximum variations were less substantial than in the static response, when comparing the linear and nonlinear analyses.
3. The parametric analysis (v_0 = 10 m/s to v_0 = 70 m/s) demonstrated that for the horizontal displacements, the steel tower does not attend the stated limit present in EUROCODE 3 [8] for basic wind velocities higher than 58 m/s. Regarding the von Mises stress, it was observed that for the design limit proposed by IEC 61400-2 [9], the steel tower does not attend the recommended value for velocities of 41 m/s and above, while the fatigue limit state from DNV-GL-ST-0262 [11] is overcome for wind velocities greater than 38 m/s.
4. However, it must be emphasized that the calculated displacement, stress values and fatigue life service attend all the investigated project limits to wind velocities of 24 m/s and 25 m/s, associated to the transition from operational to survival mode of the studied MM92 [7] wind turbine.

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