Analysis of the Nondeterministic Dynamic Structural Behavior of a Steel Wind Tower when Subjected to Wind Loadings

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ABSTRACT

This study presents an in-depth investigation into the structural dynamics response of a wind tower designed to support a 2 MW onshore wind turbine. The tower's finite element model was developed using the Finite Element Method (FEM), utilizing the ANSYS software 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's dynamic behavior. The stochastic nature of the wind loadings was considered, and a statistical analysis was carried out on the structure's dynamic responses. Then, an extensive parametric study was performed, considering several basic wind velocities to assess the steel tower's dynamic structural behavior based on horizontal displacements, von Mises stresses, and the fatigue service life. The results showed that within the operational limit of the turbine, the investigated tower complies with the recommended limits specified in the current wind tower design standards. However, for higher basic wind speeds, the wind tower's structural design does not meet these requirements.

Keywords-dynamic analysis; finite element modeling; nondeterministic wind loadings; steel wind towers

I. INTRODUCTION

The growing need for electricity, associated with concerns about climate change and environmental sustainability, has placed greater emphasis on the development and expansion of renewable energy sources. In this way, governments, businesses, and investors around the world are recognizing the advantages of renewable energy, such as its ability to reduce greenhouse gas emissions, enhance energy security, and stimulate economic growth [1]. In this context, clean energy sources, such as wind, play a fundamental role in achieving a sustainable energy future. However, the construction of wind towers represents a significant part of the total cost of new wind farms. According to [2], these structures account for 20% to 30% of the wind turbines.

Many researchers have devoted their efforts to investigating the mechanical characteristics of these structures to achieve improved designs. Although several studies have been carried out regarding support structures for wind turbines, most of the approaches consider the linear response of the structure, as well as deterministic modeling for the wind loads. Additionally, many wind tower guidelines provide an accurate description of analytical methods for registering the effect of random wind loads on wind towers. However, more detailed investigations are needed to address nonlinear interactions.

This study presents a structural dynamic analysis of a steel wind tower supported by an octagonal reinforced concrete foundation designed to accommodate a 2 MW wind turbine. The wind tower finite element model was developed based on the use of the Finite Element Method (FEM), utilizing the ANSYS software and considering the wind loading on the rotor and tower and the effect of the geometric nonlinearities and soil-structure interaction, to obtain a realistic representation of the structure's dynamic behavior. The formulation utilized for the wind loading modeling considered a probabilistic formulation based on the Monte Carlo method. In this context, a statistical analysis of the dynamic structural response of the steel tower was carried out to determine the displacement and stress values. Furthermore, an extensive parametric analysis was performed 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 translational horizontal displacements, von Mises stresses, and fatigue service life.

The statistical analysis of the displacement and stress values was based on a forced vibration analysis that considered the nondeterministic wind loads, indicating that despite the variations of the dynamic response observed between the linear and nonlinear analyses, for isolated wind series, the mean and maximum variations of the displacements and stresses were similar to those found in the static analysis. Based on the parametric study developed to assess the steel wind tower's dynamic structural response, it was observed that within the operational velocity limit of 24 m/s, the criteria for displacements, stresses, and fatigue service life were satisfied. However, for higher basic wind speeds, the steel tower design no longer complies with the recommended limits for displacements, stresses, and fatigue service life.

II. MODELING THE NONDETERMINISTIC WIND LOADS

To model the wind effects on the investigated structure, this study incorporated the wind action on both the steel tower and the rotor. The wind action on the steel tower was modeled considering its nondeterministic nature in the direction of the turbine axis along with loads induced by vortex shedding, which act perpendicular to the turbine axis. The wind loads on the tower rotor were obtained based on the method proposed in [3], considering a 2 MW wind turbine. Modeling the wind loads on structures can be a challenging task. To address these challenges, structural engineers utilize a combination of empirical methods, analytical calculations, wind tunnel testing, and computational simulations. Thus, an alternative approach can be pursued based on a mathematical model that considers the uncertainties associated with its formulation [4, 5]. This work introduces a nondeterministic wind load formulation based on the Monte Carlo method. This approach was used accurately to represent the uncertainties associated with wind loads on the structures.

The nondeterministic wind velocity along the tower, denoted as $V(t)$ (m/s), is represented as a time-varying function composed of a static $\bar{V}$ (m/s) and a fluctuating part $v(t)$ (m/s), as shown in (1). Based on the NBR 6123 [6] guidelines, the static component is determined as a constant, depending on the height, and represents the mean wind velocity in the horizontal direction. On the other hand, the fluctuating part is associated with turbulence and is derived from the average wind velocity and the height above the ground level.

$$V(t) = \bar{V} + v(t) \qquad (1)$$

The mathematical formulation for the constant mean wind velocity is presented in (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 state of Rio de Janeiro/RJ, Brazil [6]. 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 seconds of wind action.

$$\bar{V} = v_0 S_1 S_2 S_3 \qquad (2)$$

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

The time-dependent component of the wind velocity in (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 PSD $S^V(f, y)$ is presented in (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).

$$\frac{f S^V(f,y)}{u_*^2} = \frac{200x}{(1+50x)^{5/3}} \qquad (4)$$

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

In (6), u_* is the friction velocity, related to the logarithmic law describing the distribution of the longitudinal velocity on the normal direction of turbulent flow, near a boundary with no-slip condition. $k = 0.4$ is the von Kaman's coefficient, and y_0 (m) is the roughness length.

$$u_* = \frac{kV_y}{ln(y/y_0)} \qquad (6)$$

The time-varying component of the wind velocity is formulated as a weak stationary second-order ergodic process with a mean value of zero and a superposition of harmonics, as shown in (7), N represents the number of harmonics considered in the power spectrum, θ_i is a random phase angle uniformly distributed in the interval [0, 2π] considering a normal distribution, f_i (Hz) is the i^{th} frequency, and Δf (Hz) is the frequency increment.

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

The i^{th} frequency and frequency increment are described in (8) and (9), where f_{min} and f_{max} are the minimum and maximum frequency values within the considered spectrum. According to [7], these values should be taken in such a way that one of the considered frequencies coincides with the structure's fundamental frequency, while the others are its multiples and submultiples.

$$f_i = f_{min} + \Delta f(i + 1) \quad (8)$$

$$\Delta f = \frac{f_{max} - f_{min}}{N} \quad (9)$$

The wind aerodynamic dynamic pressure $Q(t)$ over the steel tower was obtained based on the classical Davenport method, as presented in (10). The wind load that acts on a certain height of the structure F_W (N) is described in (11), where A_i (m²) is the effective area and C_{Di} is the drag coefficient of the i^{th} area.

$$Q(t) = 0.613[\overline{V} + v(t)]^2 \quad (10)$$

$$F_W(t) = A_i C_{Di} Q(t) \quad (11)$$

The nondeterministic aerodynamic wind load (12) was determined considering (1)-(11). It is composed of the superposition of a static component whose intensity increases with height, as described in NBR 6123 [6], and a fluctuation part, which is intended to represent the uncertainties related to the mathematical formulation of wind loads over the structures.

$$F_W(t) = 0.613\, A_i\, C_{Di}\, [v_0 \left(\frac{y}{10}\right)^p + \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f}\, cos(2\pi f_i t + \theta_i)]^2 \quad (12)$$

Based on the geometric characteristics of the steel tower, EUROCODE [8] specifies that the effects of vortex shedding must be considered. Accordingly, this study models these loads using a harmonic function (13), where its amplitude is given by the air density ρ_{air}= 1.225 kg/m³ and the wind critical velocity v_{crit} (m/s) taken as a function of the structure's 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 a weak variable conical section, as presented in (14).

$$F_v(t) = \frac{1}{2}\rho_{air} v_{crit}^2\, sin(\omega_{01} t) \quad (13)$$

$$v_{crit} = \frac{d\omega_{01}}{St} \quad (14)$$

The rotor wind loads can be described as forces and moments that arise from the wind effect over the turbine blades and the rotating machines used in the wind's energy conversion process. The wind loads acting on the rotor were determined using the method proposed in [3], and the corresponding values are presented in Table I, considering a 2MW MM92 Repower Systems [10] wind turbine, where F_X, F_Y, and F_Z are the forces acting on the global X, Y, and Z directions, and M_X, M_Y and M_Z are the bending moments around axes X, Y and Z, considering the global coordinate system presented in Figure 1.

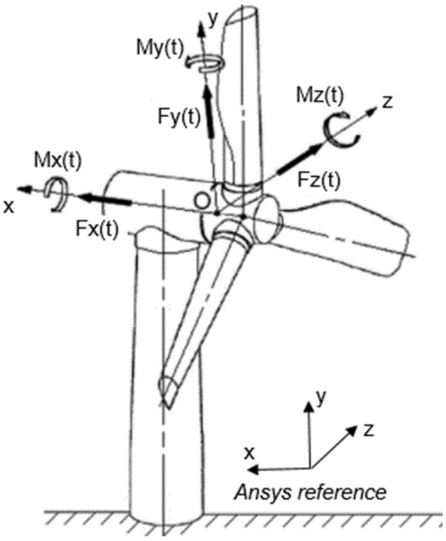

Fig. 1. Global coordinate system.

TABLE I. ROTOR WIND LOADINGS 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

III. FINITE ELEMENT MODELING OF THE STEEL WIND TOWER

The investigated steel tower is designed to support a 2MW Repower Systems [10] wind turbine, whose dimensions were taken from [11]. Figure 2 illustrates the dimensions of the main parts considered in the structure's finite element modeling. As can be seen, the tower was divided into three parts to enable transportation and assembly on site, and the whole structure was supported by an octagonal reinforced concrete foundation inscribed in a 17 m diameter circle. In the lower part of the tower, there are two elliptical openings, one for ventilation (0.825×0.385 m) and the other for internal access and maintenance purposes (2.4×0.85 m). The maintenance and ventilation doors are positioned centrally at a height of 2.33 m and 7.96 m, respectively.

The wind tower finite element model was developed using ANSYS. The entire structure was discretized considering the adequate mesh refinement techniques, resulting in well-proportioned elements to avoid numerical problems. Figure 3 presents the developed finite element model of the investigated structure. For the steel tower and nacelle, the four-node-thick SHELL 181 [12] element was considered. The reinforced concrete foundation was modeled with the four-node tetrahedral solid element SOLID 72 [12]. The connection between the three conical sections of the tower, as well as the connections between the lower section and the concrete foundation and between the upper section and the nacelle, were performed through a 75 mm bolted flanged connection, also modeled with the four-node tetrahedral SOLID 72 [12] element, whereas the bolts were modeled as rigid elements considering different diameters. To connect the flanges from the lower section to the shoe and the intermediate section, 45 mm diameter bolts were considered. The connection between the intermediate part and the upper section was established with 39 mm diameter bolts, while 30 mm diameter screws were used to secure the upper part to the nacelle. For all connections, 116 bolts are used, evenly distributed along the flange's circumference.

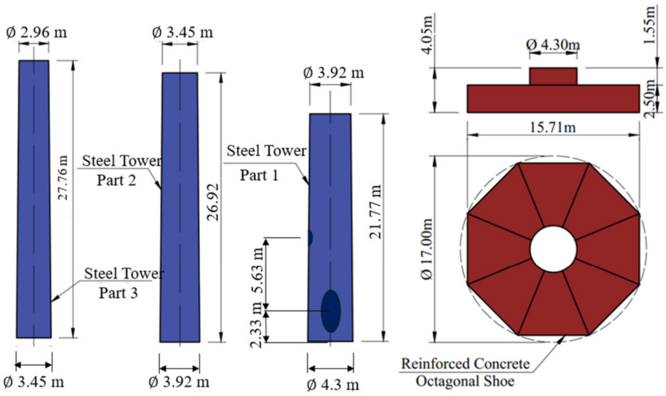

Fig. 2. Investigated steel wind tower and reinforced concrete foundation.

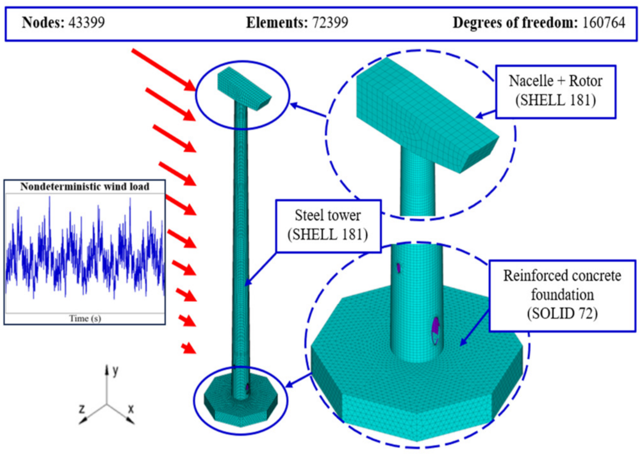

Fig. 3. Finite element model of the investigated structure.

The investigated steel wind tower was constructed of steel S355, with a yield strength of 355 MPa and Young's modulus of 205 GPa. The structure's foundation was modeled in reinforced concrete with a yield strength of 16 MPa and Young's modulus of 30 GPa. To consider the self-weight of the turbine blades and the wind energy converter equipment at the top of the tower, shell elements with different mass densities were incorporated into the model. In the front part of the nacelle, shell elements with a mass density of 3199 kg/m³ were utilized, corresponding to a total volume that results in a mass of 40.7 tons. In the rear part, shell elements with a mass density of 2324 kg/m³ were considered, accounting for a total mass of 71 tons for the generators. This approach in structural modeling leads to a non-uniform distributed load at the top of the tower, considering the specific mass distribution of the components. The lower section of the tower had a height of 27.76 m, with thicknesses ranging from 30 mm at the base to 21 mm at the top. The middle section was 26.92 m high, with thicknesses varying between 21 mm and 16 mm. The upper section had a height of 21.77 m, with thicknesses ranging from 16 mm at the base to 12 mm at the top.

The soil-structure interaction effect was considered using the unidirectional two-node spring element COMBIN39 [12], whose stiffness is determined based on the soil elasticity, typically represented by the subgrade reaction coefficient k_z according to [13], see (15), where B = 17 m is the reinforced concrete diameter, v_s = 0.3 is the Poisson's ratio, and E_S = 3×10^5 kN/m^2 is the Young modulus.

$$k_s = \frac{E_s}{B(1-v_s)} \quad (15)$$

IV. MODAL ANALYSIS: EIGENVALUES AND EIGENVECTORS

The natural frequencies (eigenvalues) and vibration modes (eigenvectors) of the steel tower were determined using a free vibration analysis through the ANSYS software. Figure 4 shows the first four vibration modes and the respective natural frequencies of the tower. 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.

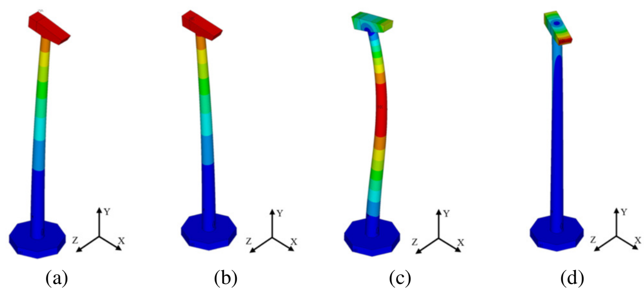

Fig. 4. The first four vibration modes of the investigated steel wind tower: (a) f_{01} = 0.338 Hz, (b) f_{02} = 0.34 Hz, (c) f_{03} = 2.461 Hz, (d) f_{04} = 2.498 Hz.

The natural frequencies and vibration modes obtained from the developed finite element model showed a high level of accuracy compared to the experimental results in [11], highlighting a satisfactory calibration. Specifically, the first mode had an accuracy of 97.6%, the second mode had 97.4%, the third mode had 88.5%, and the fourth mode had 89.2%.

V. RESULTS AND DISCUSSION: STATIC AND DYNAMIC ANALYSIS

In the context of nonlinear structural analysis, geometrical nonlinearities arise from nonlinear relationships of forces and displacements with stresses and forces. Considering the non-linear static analysis, the Newton-Raphson method was typically used to compute variations in the stiffness matrix. On the other hand, for dynamic analyses, the Newton-Raphson method was applied in conjunction with the Newmark method to integrate the equations of motion over time.

Initially, a static analysis was carried out considering the wind loads derived from the simplified continuous model outlined in NBR 6123 [6], assuming a basic wind speed of 35 m/s. Subsequently, a non-deterministic dynamic structural analysis was performed, incorporating random wind loading and vortex shedding effects. In both analyses, the rotor loads were determined based on the values provided in Table I. Throughout this results section, the steel wind tower finite element model is divided into 30 conical sections of equal height with a drag coefficient C_{Di} and an area of influence A_i for calculating static and dynamic wind loads [see (1)-(14)].

Regarding the nonlinear dynamic analysis, the horizontal translational displacements of the investigated steel wind tower considering the static wind loadings are shown in Figure 5(a). The maximum displacement value of 1.043 m was calculated on the nacelle-rotor assembly when considering geometrical nonlinearities. This value meets the design limit of H/50 = 1.52 m recommended by EUROCODE 3 [14]. When not considering the geometrical nonlinearities, a maximum displacement of 0.99 m was determined, indicating a 5.3% increase attributed to geometric nonlinearities. The von Mises stress distribution on the investigated structure, considering the presence of geometrical nonlinearities, is presented in Figure 5(b). 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. The significant difference between these values highlights the stress concentration effect occurring in this specific region. 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's response. On the other hand, for the values obtained near the opening for internal access, the increase was only equal to 2.5%. These results demonstrate that the inclusion of geometric nonlinearities presented a significant impact on stress concentration effects. It is worth noting that the stress results for both linear and nonlinear static analysis show that the steel tower design meets the limit presented in IEC 61400-2 [15].

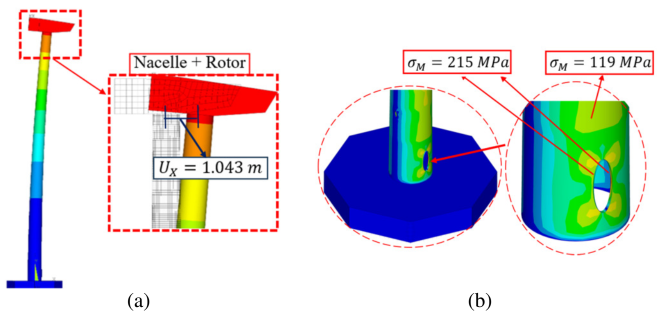

Fig. 5. Nonlinear static structural analysis results: (a) Horizontal displacement, (b) von Mises stress.

To assess the structure dynamic response considering the geometrical nonlinearities, its behavior was first examined using a generic random wind loading series. Figure 6 illustrates the horizontal translational displacement at the top of the tower in both the time and frequency domains, considering the geometrical nonlinearities. The steady-state response revealed a displacement peak of 1.049 m, along with a noticeable energy transfer peak associated with the structure's first vibration mode. A transient linear analysis using this same loading series yielded a maximum horizontal displacement value of 1.032 m, indicating that the geometrical nonlinearities increased only 1.7% on the displacement response.

As in the static analysis, the maximum stress values for the dynamic analysis were observed at the opening for internal access in the lower part of the tower. Figure 7 illustrates von Mises stress in the steady state response for the same wind loading series in time and frequency domains. A stress peak of 206.5 MPa was 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 geometrical nonlinearities, an increase of 4.8% in the maximum von Mises stress was found. Considering both linear and nonlinear analyses, a peak of energy transfer for the von Mises stress was found for the first bending mode natural frequency. It should be noted that for both linear and nonlinear analyses, the maximum von Mises stress attended the limit recommended IEC 61400-2 [15], while the horizontal displacement on the top of the tower attended the design limit of 1.523 m according to EUROCODE 3 [16].

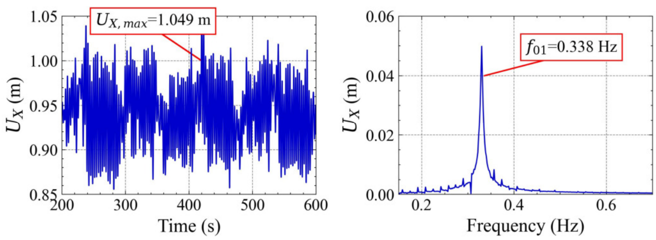

Fig. 6. Horizontal displacement at the top of the steel wind tower: time and frequency domains.

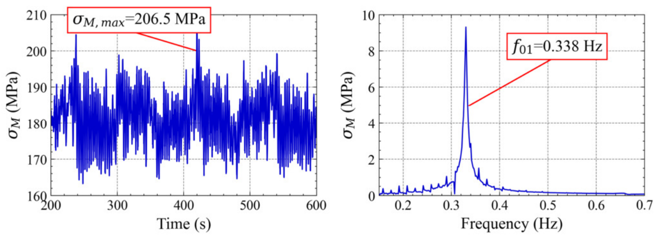

Fig. 7. Von Mises stress at the lower opening for internal access: time and frequency domains.

From this point onward, the steel tower dynamic structural response was analyzed considering 30 sets of nondeterministic loads to ensure numerical convergence of the results [1, 4, 5]. For each wind series, the maximum response for horizontal translational displacement and von Mises stress related to the steady state response were determined. After that, a statistical analysis was carrie out to obtain the mean (μ), standard deviation (s), and maximum values with a confidence level of 95% (CL_{95}). This investigation considered linear and nonlinear dynamic analyses, aiming to verify the influence of geometric nonlinearities on the steel wind tower's structural response. Table II presents the maximum values for the horizontal translational displacement U_X at the top of the structure and the maximum von Mises stress σ_M based on the use of 30 series of nondeterministic wind loads for the linear and nonlinear transient dynamic structural analysis.

The results of the statistical analysis based on the translational horizontal displacements and von Mises stresses reveal that, when considering geometric nonlinearities, the displacement values exhibit a 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. Analyzing the tower's nondeterministic response considering geometric nonlinearities, it was observed that the mean displacement value was 1.051 m with a maximum value of 1.070 m. In addition, the von Mises stress had a mean value of 205.5 MPa with a 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 led 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.

An extensive parametric analysis was performed to assess the structure's dynamic response, considering several basic wind velocities (v_0 = 10 m/s to v_0 = 70 m/s) to calculate nondeterministic wind loads. As observed in Table II, the incorporation of geometric nonlinearities does not present a significant effect on the investigated steel wind tower's dynamic structural response. Furthermore, it is well known that nonlinear dynamic analyses induce considerable computational costs. In this way, the parametric analysis was carried out without considering geometric nonlinearities. For the analysis with a basic wind velocity under the operational limit according to Repower Systems [10], the operational loads on the rotor were considered (see Table I), and for wind velocities above this limit, the survival wind loads of the rotor were taken (see Table I).

Figure 8 presents the results of the parametric analysis related to the mean maximum translational horizontal displacements, and von Mises stresses determined in the steady state response, based on a confidence level of 95% $\bar{U}_{X,95}$ and $\bar{\sigma}_{M,95}$, considering 30 random wind load series for each basic wind velocity (v_0 = 10 m/s to v_0 = 70 m/s). When considering the horizontal displacements, it can be observed that the tower complies with the limit state set by EUROCODE [14] for wind velocity up to 58 m/s. When considering the von Mises stress values, it is evident that the structure complies with the 239 MPa limit set by IEC 61400-2 [15] for wind speeds up to 41 m/s. It can also be noted that there is a discontinuity in displacement and stress response values between wind velocities of 24 m/s and 25 m/s, associated with the transition from operational to survival turbine mode of the MM92 wind turbine.

TABLE II. NONDETERMINISTIC DYNAMIC STRUCTURAL RESPONSE OF THE WIND STEEL TOWER: LINEAR AND NONLINEAR ANALYSIS

Wind series	Linear Analysis		Nonlinear Analysis	
	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

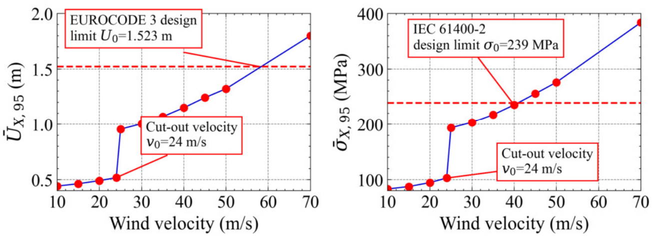

Fig. 8. Horizontal translational displacements and von Mises stresses for different investigated basic wind velocities.

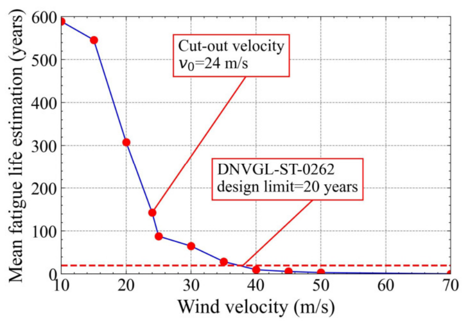

Fig. 9. Steel tower service life considering different wind basic velocities.

The stress history associated with the steel wind tower dynamic response, when subjected to nondeterministic wind loadings, as shown in Figure 7, and the results presented in Figure 8 suggest a scenario that induces a potential structure failure due to fatigue. In this context, the parametric study was expanded to include a fatigue assessment, focusing on the steady-state response of the stress histories induced by nondeterministic wind actions and considering several basic wind velocities. The Nominal Stress Method (NSM) was used based on current design codes prescribed in the recommendations or structure fatigue assessment. However, future research will aim to include also the Hot-spot Stress Method (HSM) for a more accurate definition of stresses. In this context, this research performed the fatigue assessment of the investigated steel wind tower through the Palmgren-Miner rule, based on the Rainflow cycle counting algorithm, considering all guidelines presented in EUROCODE 3 [16] related to the investigated structural detail 40, based on the steady-state stress response associated to the 30 nondeterministic wind load series having in mind different basic wind velocities (v_0 = 10 m/s to v_0 = 70 m/s). In this way, Figure 9 presents the results for the mean service life of the structure, considering the different basic wind speeds. It can be observed that the steel tower meets the service life requirements from DNG-GL-ST [17] for a basic wind speed of up to 38 m/s. However, winds above this threshold could lead the structure to potential fatigue failure.

VI. CONCLUSION

This study presented an in-depth investigation of the structural dynamic response of a steel wind tower designed to support a 2 MW turbine under nondeterministic dynamic wind loading, considering the influence of geometric nonlinearities. Dynamic structural analysis was performed using a three-dimensional finite element model, considering rotor loads, wind action on the structure, and the soil interaction effect considering the modeling of a reinforced concrete foundation. A statistical analysis of the results (displacements and stresses), related to the linear and nonlinear forced vibration analysis, was performed based on the use of 30 series of nondeterministic wind loads.

Considering the wind loadings calculated utilizing the NBR 6123 [6] recommendations, based on a basic design wind velocity of 35 m/s, both linear and nonlinear static analyses were performed to provide an initial assessment of the structural behavior of the steel wind tower. In this way, based on the linear analysis, a displacement of 0.99 m and a maximum stress value of 199 MPa were determined. On the other hand, in the nonlinear analysis, the values increased to 1.043 m for displacement and 215 MPa for maximum stress. This initial analysis revealed that incorporating geometric nonlinearities led to a 5.3% increase in displacement and an 8% increase in maximum von Mises stress. These findings highlight the significance of incorporating geometric nonlinearities in the structural response of the investigated steel tower, particularly when stress concentration effects are present, such as in the region of the opening for internal access.

The tower's nondeterministic dynamic response was assessed based on a statistical analysis of the maximum displacements and von Mises stresses determined in the steady state response, having in mind 30 random series of wind loads, also considering the same basic design wind velocity of 35 m/s. Thus, when geometric nonlinearities (linear dynamic analysis) were neglected, the calculated mean and maximum displacement values were 1.026m and 1.043m, respectively. Regarding von Mises stress, the mean and maximum values obtained were 193.4MPa and 203.5MPa, respectively. On the other hand, when geometric nonlinearities were considered (nonlinear dynamic analysis), the mean displacement was 1.051 m with a maximum of 1.070 m, while the mean von Mises stress was 205.5 MPa with a maximum of 209.6 MPa. Based on these results, it was evident that incorporating the geometric nonlinearities into the dynamic analysis resulted in a 2.43% increase in the mean horizontal displacement and a 2.58% increase in the maximum displacement of the structure. In contrast, the inclusion of geometric nonlinearities led to a 6.26% increase in the mean von Mises stress and a 3% increase in the maximum von Mises stress.

Based on the developed parametric analysis (v_0 = 10 m/s to v_0 = 70 m/s), concerning the recommended project limit related to the horizontal translational displacements according to EUROCODE 3 [14], the investigated wind tower does not attend this proposed limit for basic wind speeds higher than 58 m/s (v_0 = 58 m/s). Regarding the von Mises stress, the parametric analysis revealed that the wind tower does not meet the limits specified by IEC 61400-2 [14] for basic wind speeds above 41 m/s (v_0 = 41 m/s). When fatigue was investigated, it was observed that the tower could reach potential fatigue problems for wind velocities higher than 38 m/s, considering the recommended limit outlined in DNVGL-ST-0262 [16]. However, it is important to note that the structure meets the displacement, stress, and fatigue life requirements for the initially assumed basic wind speed of 35 m/s.

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